INVESTIGATION OF PRESTRESSED BOX BEAM PLAZA STRUCTURES

Michigan Department of Transportation
MDOT

Investigation of Prestressed Box Beam Plaza Structures Z01, Z02, and Z03 of 63102 over I-696 in Southfield, Michigan

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16. Abstract: This report investigates the plaza structures (Z01, Z02, and Z03 of 63102) that cross I-696 in Southfield, Michigan. These structures contained numerous diagonal shear cracks in the fascia as well as the interior beams after being in service for 11 to 14 years. Reviewing the design files, we found that loads produced by a 10' tall concrete parapet wall were not completely accounted for in the design of the fascia beams. Therefore, using AASHTO LRFD combined shear and torsion analysis as well as analysis from Arthur H. Nilson, "Design of Prestressed Concrete," we found that the fascia beams on all three structures did not have sufficient vertical reinforcement to account for the loads imposed. However, when the superimposed dead loads were proportioned between the parapet wall and the fascia beam, only Z03 has sufficient vertical reinforcement when analyzed using the measured crack angle. We also found (assuming 5 feet of earth fill) that the east fascia beams of span 1 and span 2 on Z01, the west fascia beams of span 1 and span 2 on Z02, 7 percent of the interior beams on Z01, and 42 percent of the interior beams on Z02 require additional longitudinal reinforcement at the bearing location to assist in controlling diagonal shear cracks. During phase II of this investigation an extensive topographical survey on the top of all three structures was performed and the maximum fill depths were located. These locations did not correlate with the cracked interior beams. Therefore, the interior beam cracks must have been initiated as a result of fabrication errors or during construction and grown due to the lack of longitudinal reinforcement. Eight cost estimates were generated and from these it was decided to replace the first four beams and parapet walls on all three structures as well as fifty-six interior beams on Z03 and place carbon fiber reinforced polymer (CFPR) sheets on the bottom flange of all remaining beams. As part of the repair contract, a total of five prestressed box beams were load tested and analyzed for material properties and reinforcement arrangements.

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EXECUTIVE SUMMARY

As a result of the visible fascia beam cracks in the plaza structures (Z01, Z02, and Z03 of 63102) over I-696 in Southfield, Michigan, the Maintenance Division asked the Structural Research Unit, Construction and Technology Division, to investigate the structural integrity of these structures. To accomplish this, we performed detailed field inspections, extensive file reviews (design data, construction history, and materials' documentation), in-depth calculations, and extensive surveys.

Phase I

Through our inspections, we discovered that all three structures contained numerous diagonal cracks in the fascia beams as well as in the interior beams. Since the majority of the fascia beam cracks were outlined with either water or a leachate, they were visible from the roadway. A major source of water appeared to be the lawn sprinkler systems on top of each of the plaza structures.

Our file review revealed that the structures were built in the order of Z03, then Z01 and then Z02. Through this review, we discovered that the fascia beams on all three structures cracked either during construction or soon after. In an attempt to alleviate the cracks, which appeared to have been caused by the placement of the 10-foot tall concrete parapet wall, the design methodology was modified after the construction the first plaza structure. Unfortunately, the modification was not successful. During our review, we discovered that the parapet wall loads were not completely accounted for in the design of the fascia beams on all three structures. We also did not find any mention nor accounting for the loads produced by the sidewalks, trees, playground equipment, or sprinkler systems that are present. These items appeared to have been placed after the design was completed.

Our analysis showed that the fascia and interior beams on all three structures should have experienced diagonal shear cracking during construction or soon after because the design shear loads were greater than the shear strength of the concrete. However, a cracked beam does not necessarily mean that it is structurally deficient. If the steel stirrups and the concrete compression struts can carry the excess load, then the beam is structurally sufficient.

For the fascia beams, the combined shear and torsional loading produced by the 10-foot tall concrete parapet wall was not completely accounted for in their design. Therefore, there was insufficient vertical reinforcing steel when the entire loads were used to analyze the beam(s). When the superimposed dead loads (snow and earth fill loading) were proportioned between the parapet wall and the concrete beam(s), the fascia beams for Z01 and Z02 still had insufficient vertical reinforcement when analyzed using the measured crack angle obtained from our inspection. We also discovered that the east fascia beams of span 1 and span 2 on Z01 and the west fascia beams of span 1 and span 2 on Z02 had insufficient longitudinal tensile resistance at the bearing. As a result of insufficient longitudinal tensile resistance, once a shear crack develops, the longitudinal steel cannot prevent the crack from opening.

A total of twenty-three interior beams was analyzed for all three structures. Assuming 5 feet of earth fill, all twenty-three beams had sufficient vertical reinforcement steel and sufficient longitudinal tensile resistance at the critical shear location d_v. However, from these analyzed beams we found that 7 percent of the interior beams on Z01 and 42 percent of the interior beams on Z02 had insufficient longitudinal tensile resistance at the bearing location, which allows shear cracks to open wider after the cracks have developed.

While reviewing the reinforcement details for the fascia beams, we found that neither the longitudinal reinforcement nor vertical reinforcement (stirrups) is properly detailed for torsion. For the longitudinal reinforcement to be properly detailed it must be distributed around the perimeter of the closed stirrups. The majority of the longitudinal reinforcement for these structures consists of the prestressing strands, which are concentrated in the bottom flange. A properly detailed stirrup would consist of one bar that wraps around the entire beam and is anchored with a 135-degree hook around the longitudinal reinforcing steel. For each structure, the stirrups are lapped in the beam web. Fortunately, for all three structures, torsional loading on the fascia beams only comprises about 30 percent of the load on the stirrups. Therefore, the beams are subjected to a greater shearing force than torsional force.

Aside from the reinforcing steel improperly detailed for torsion, the lap length for the welded wire fabric (WWF) shear reinforcement in Z03 was not properly detailed. The required lap length is 7-7/8 inches, which is greater than the detailed lap length of 7 inches. To prevent premature failure of the WWF when used for transverse reinforcement, the wires must provide a minimum elongation of 4 percent when measured over a 4-inch gage length and be stress relieved after fabrication according to AASHTO LRFD. Reviewing the specifications cited for WWF, we did not find elongation requirements nor requirements for the WWF to be stress-relieved after fabrication. A lack of shear reinforcement lap length, along with the possibility of premature failure of WWF, may also explain the diagonal shear cracks found on the interior beams of Z03. Since shear failures occurs with little warning, proper steel detailing is required to prevent premature failure.

At the end of Phase I, a list of possible solutions was generated to increase the durability and strength of the beams to account for the added and unaccounted dead loads, i.e., additional earth fill, playground equipment, sidewalks, trees, and water from the sprinkler systems. From this list, it was recommended that the cracks in the fascia beams as well as any crack in the bottom flange of the interior beams with a width greater than 0.010 inches be epoxy injected. It was also recommended that carbon fiber reinforced polymer (CFRP) sheets be attached to all of the exposed fascia beam webs and all bottom flanges of the fascia beams. The CFRP sheets should also be attached to the bottom flanges of the first, second, and third interior beams from the east and west fascia for span 1 and 2 on Z01 and Z02 and on the bottom flanges of any interior beam with a crack width (measured on the bottom flange) greater than 0.040 inches. In all cases, the CFRP should extend a distance not less than twice the beam depth from the end to provide the required shear and torsional reinforcement. The CFRP sheets will also serve to mitigate the improper detailing of the torsional reinforcement. The outside fascia beam web should be coated with a concrete coating in the area of the CFRP sheets for esthetics. For the interior beams, the existing cracks in the webs should be sealed to prevent moisture from contacting the reinforcing and prestressing steel by injecting an expandable urethane foam between the beam gaps. The expandable urethane foam should extend a distance at least three times the beam depth from the end of the beam. Also, a topographical survey of the earth fill should be performed. If the existing fill depths exceed the maximum fill depth as stated in Figures 16, 17, 18 (Z01, Z02, and Z03 summary of results, respectively), then the excess fill should be removed. Note that if the pine trees are removed along the parapet walls to reduce the total dead load applied to the fascia beams, then the affects of wind loading need to be considered in the analysis. If removing the earth fill is not feasible, then attaching CFRP sheets to the bottom flanges for a distance not less than twice the beams depth from the end was recommended. On top of the structures, the sprinkler systems use should be eliminated. Maintenance should continue to inspect these structures on a six-month basis until the repair work is performed.

Phase II

Once Phase I was completed, a meeting was held with various Michigan Department of Transportation (MDOT) personnel to discuss our findings from Phase I and to set the direction for Phase II. It was determined that the next logical step was to determine the maximum fill depths on all three structures. To ascertain the fill depths, we performed an extensive survey. From this, we found that the current loads applied to the interior beams would not cause the magnitude and number of cracks that were found. Therefore, the cracks must have been initiated during construction and grown due to the lack of longitudinal reinforcement in the bottom flange. Through conversations with the construction engineer at the time of construction, there were minimal load restrictions placed on the superstructure during the construction of Z03. However, as the successive bridges were built, the load restrictions increased. This was evidenced by the number of cracked beams found on the three structures. The first structure constructed, Z03, had the widest and largest number of cracked beams and was constructed with minimal load restrictions. The second constructed structure, Z01, had some load restrictions and had fewer and tighter cracks than Z03. The final structure constructed, Z02, had the most load restrictions and had minimal cracks on the interior beams.

Upon the completion of Phase II, there were a total of eight cost estimates generated. From the eight options investigated, it was decided to remove and replace the first four beams and parapet wall on all three structures as well as fifty-six interior beams on Z03 and place carbon fiber reinforced polymer (CFRP) sheets on the bottom flange of all remaining interior beams. The beam gaps of the cracked beams are also to be filled with urethane foam to prevent moisture from entering the beam. This option although the most expensive, $6,900,000, will provide the largest factor of safety and greatly extend the service life of the plaza structures.

As part of the repair contract, the contractor was required to salvage and load test four of the replaced prestressed box beams to shear failure. The load test results were used to verify the adequacy of the remaining prestressed box beams. All of the box beams carried more than the expected load except for beam number 166 (span 1) of Z03 of 63102. Upon removal of this beam we found an obvious cold joint and a layer of Styrofoam between the web and the bottom flange near the beam end. As a result, the ultimate proof load at d_v for this beam was less than the actual factored dead load plus factored pedestrian loading at d_v. Due to the obvious fabrication error, beam number 27 (span 2) from Z03 was salvaged, repaired according to the contract plans, and load tested. For beam number 27, the ultimate proof load at d_v was greater than the actual factored dead load plus factored pedestrian loading at d_v. The remainder of the tested beams demonstrated that the remaining beams have adequate shear strength to carry the applied loads.

INTRODUCTION

In October 1996, a motorist wrote a letter to the MDOT with concerns regarding the visible cracks in the fascia beams of the plaza structures that cross I-696 in Southfield, Michigan, Z01, Z02, and Z03 of 63102. Soon after the department received the letter, the Structural Research Unit, along with the Bridge Maintenance Unit, performed an initial investigation to determine the validity of the motorist's concern. From this, we determined that the condition of these three structures required further investigation. Therefore, under request by the Maintenance Division, the Structural Research Unit opened a technical investigation to determine cause of the cracks and the structural integrity of these structures.

These plaza structures serve as recreational parks and were created for the local community in the late 1980s. The first structure in service, Z03, was constructed in 1985 and followed by Z01 then Z02, both in 1988. All three contain a 10-foot tall concrete parapet wall located on the top edge of the fascia beams, which creates a combined shear and torsional loading, and about 5 feet of earth fill covering the top of the structures. In addition, they also contain playground equipment, sidewalks, sprinkler systems, roadway (Z03 only), and numerous trees. During the investigation, we performed detailed field inspections, extensive file reviews (design data, construction history, and materials documentation), in-depth calculations, and extensive surveys.

PHASE I

Initial Field Inspection

The first step in our technical investigation was to perform a detailed inspection on all three structures. During these inspections, in May and June of 1998, we closed the fast lane of traffic and concentrated on the pier end of the beams. Any comments about the abutment ends were made based on viewing from the pier. Our efforts were concentrated on the first three interior box beams for Z03 and the first four box beams for Z01 and Z02. The difference in our concentration was due to the different designs, i.e., how the parapet wall forces are carried. For Z03, the fascia beam carried the entire weight of the earth fill and parapet wall. The second interior beam resisted the earth pressure torsional forces through a turnbuckle assembly that was attached to the parapet wall and the second interior beam, refer to Figure 1. For Z01 and Z02, the parapet wall forces, superimposed dead loads and torsional loads, were distributed through the first four beams that were transversely post-tensioned together at four locations along the span length, refer to Figure 2.

Through our inspections, we discovered that all three structures contained beams with numerous diagonal cracking at the beam ends. The amount and size of the cracks appeared to be less on Z03 when compared with the other two structures, refer to Figures 3 through 14. Also, the interior beams of Z01 and Z03 contained many beams with diagonal shear cracks. In one location on Z03, a bottom flange crack measured wider than 0.060 inches. The inspection on top of the structure revealed sprinkler systems, numerous trees, playground equipment, sidewalks, roadway (Z03 only), and earth fill greater than 4 feet. Only 4 feet of earth fill was detailed in the original contract plans. During our inspection of Z01 and Z02, we extracted part of the wet earth fill and brought it back to the laboratory for analysis. From these samples we learned that the clayey sand fill had a natural moisture content of 17.6 and 22.6 percent, for Z01 and Z02 respectively. Through a visual inspection performed by Dan Troia (MDOT Soils Engineer), it was suggested that the unit weight of the soil was closer to 140 pcf rather than the 120 pcf stated in the design notes. Refer to Appendix A for the detailed inspection reports.

During our inspection, we also discovered that most of the weep holes in the inspected beams still had the plastic cap in tact. With concern of water being retained in the beam cavity, we attempted to remove the caps. As we removed the plastic caps, water drained from a couple of the beams. Since most of the cracks on the fascia beams were outlined with leachate and/or water at the time of our inspection, it was not surprising to discover that water was entering the beam cavity.

As a result of finding the plastic caps still in tact and moisture collecting in a couple of the inspected beams, the Auburn Hills Special Crew, MDOT, was asked to probe the weep holes in every beam on all three structures. We were concerned with moisture collecting in the interior beams and substantially increasing the applied load. During this operation, the Auburn Hills Special Crew unplugged numerous weep holes that had the plastic cap still in tact but never found standing water. However, some of the beams did not have weep holes as specified in the plans. For these beams, we did not attempt to create a weep hole due to concern with damaging the prestressing strands. Therefore, even though standing water was not found in the beams that had the weep holes unplugged, we can now confidently state that if and when moisture collects in the interior of these beams it will have a path to drain. However, for the beams without weep holes, there is still exists a concern with moisture collecting.

File Review

After our detailed inspections, an extensive file review was performed. Construction history, design data, and the material documentation files were reviewed. The twenty-six boxes of construction history for job numbers 21960, 21961, and 21858 were requested for Z01, Z02 and Z03 of 63102, respectively. From these files, we were able to determine that problems with the plaza structures appeared either during construction or soon there after. Problems with the fascia beams on Z03 initiated with the placement of the 10-foot tall concrete barrier wall. As the west parapet wall was poured, the west fascia beams cracked. Although this was noticed, the placement of the concrete continued. Due to this problem with Z03, the design of Z01 and Z02 was modified in an attempt to prevent the cracks from occurring. The first four box beams were transversely post-tensioned at four locations along the span, unlike Z03 that had two tie rods placed transversely along the span. However, modifying the design did not stop the formation of cracks. Soon after the placement of the parapet wall, cracks formed in both of the east fascia beams of Z01. With concern about crack growth, the Structural Research Unit was asked to instrument and monitor the cracks for any changes. The cracks in Z01 fascia beams were monitored from July 1989 through March 1991 during which time it was found that the cracks were stable and epoxy injection of the cracks was recommended. Since the design modification for Z01 failed to produce a crack free fascia beam, further modifications for Z02 were designated. Light weight fill was to be used around the parapet wall to reduce the dead load. Through our field inspection, we found this not to be the case. The fill material found on the Z02 structure was the same as the other two structures. Therefore, as with Z01, soon after constructing the parapet wall, cracks developed on the Z02 fascia beams. These cracks were not monitored.

The eleven folders of design files revealed that all three structures were designed using MDOT's Standard Bridge Design Computer Program (MDOT Bridge Program). The applied dead loads were 4 feet of 120 pcf of earth fill, 1 foot of 40 pcf of snow, and a future wearing surface at 25 psf. The dead load created by the 10-feet high concrete parapet wall was not included in the computer design calculations. Instead, a hand calculation was performed to verify that the beam had sufficient flexural strength to carry the parapet wall. From this it was found that the beam was sufficient in flexure. However, the effects of shear were never considered. Also, nowhere in the design files were sidewalks, trees, playground equipment, nor water from the sprinkler systems noted. Additionally, there was no mention nor accounting for the combined shear and torsional loads produced by the concrete parapet wall. As a result, this left some questions that could only be answered through in-depth calculations. One item that we found in the design files was a partial set of shop drawings for Z03, which indicated welded wire fabric was substituted for the reinforcing bars shown on the contract plans. Through all of our searching, this was the only set of shop drawings for any of the three structures that we were able to locate in the department's files.

Since we were unable to locate all of the shop drawings, we searched the Construction and Technology's microfilm files for material information that might be of use. We were unable to find any information about Z03 of 63102 in the files. In an attempt to locate the microfilm for job number 21858 (Z03), we searched microfilm roll numbers 1893 through 1960 and were unable to locate any records corresponding to Z03. The project was finalized in 1990, so the records should have been microfilmed in 1990 or 1991. The files covering this time period are in rolls 1909 to 1925. Although we were unable to find files on Z03, we were able to locate the material documentation files for Z01 and Z02. As a result, we were able to modify the yield strength of the stirrups and verify the prestressed concrete strength for Z01. During our search for the stirrup strength of Z02, we discovered that welded wire fabric (WWF) was used for the construction of the prestressed box beams. This differed from the *As Constructed Plans*. Personal communication with Mr. Bob Livesay (MDOT shop inspector at the time) confirmed that WWF was used. As with Z01, we were able to verify the prestressed concrete strength for Z02.

Due to our unsuccessful search in finding the shop drawings in MDOT's record center, we contacted Mr. Jim Clapper. Mr. Clapper worked for Superior Products Company, who made the beams, at the time of the fabrication. Mr. Clapper stated that last he knew the files for Z02 and Z03 were in the basement of Cole Pipe. Cole Pipe bought Superior Products Company some time after the completion of the plaza structures. Therefore, in June 1998, Mr. Randy Anteau at Cole Pipe was contacted for possible use of their structure files. Mr. Anteau indicated that the old files that were stored in the basement were destroyed in May 1998. On a positive note, on October 20, 1998, Mr. Paul Malloure of C.A. Hull Co., Inc. sent us a copy of the shop drawings for Z01 of 63102, which indicated 9/16-inch diameter prestressing strands were substituted for the ½-inch diameter strands shown on the contract plans.

Calculations

After the inspection and file review were performed, the next step was to analyze the structures and determine if they were structurally adequate to carry the existing loads. For the analysis, we referenced both American Association of State Highway and Transportation Officials LRFD Bridge Design Specifications (AASHTO LRFD) 1998 and Arthur H. Nilson, *Design of Prestressed Concrete*, 1978 (Nilson). AASHTO LRFD was used as the primary code for analysis with Nilson used as a check. For both analyzes, factored loads were used. For the Nilson analysis the loads were factored according to AASHTO Standard Specifications for Highway Bridges (AASHTO Standard) 1996 and compared with the ultimate strength using capacity reduction factors from AASHTO Standard and/or American Concrete Institute Building Code Requirements for Structural Concrete 318-95 (ACI). Our analysis differed from the original design methodology because during the original design of the plaza structures in the 1980's, neither AASHTO Standard nor ACI addressed design of prestressed concrete beams for combined shear and torsion.

For our analysis of the fascia beams we used 5 feet of earth fill at 140 pcf, 1 foot of snow at 40 pcf, 10-foot tall concrete parapet wall, and the self weight of the box beam(s). A further refinement of our analysis included a temperature loading to account for all supports being fixed against thermal movement. The earth fill of 5 feet at 140 pcf was used to match the current field conditions. This depth differs from the maximum depths as shown on the plans. The contract plan sheets show that each structure has a different design earth fill depth. The design loading for Z01 was the greater of either 3 feet 6-inches of earth fill or HS20 plus 2 feet 3 inches of earth fill. For Z02 the design loading was the greater of either 3 feet 9 inches of earth fill or HS20 plus 2 feet 9 inches of earth fill and the design loading for Z03 was the greater of either 4 feet 0 inches of earth fill or HS20 loading plus 2 feet 9 inches of earth fill. The unit weight of the designed earth fill was 120 pcf. For our analysis, we focused on the earth fill option and not the live loads because after inspecting the structures we found only one structure, Z03, that allows only cars. Truck traffic is restricted. Therefore, we concentrated our analysis on the average fill depth of 5 feet as determined though our inspection. Reviewing the MDOT Bridge Program input verification sheets for the prestressed box beams, we found that 1 foot of snow load at 40 pcf and a future wearing surface at 25 psf were also accounted for in the applied design loads. The shear load produced by the 10-foot tall concrete parapet wall was not completely accounted for in the design of the fascia beams on all three structures.

Wind loading on the exposed face of the parapet wall (about 5 feet) had been omitted in the design and analysis. Even though wind loading is included in the AASHTO load cases, we omitted its affect due to the proximity of pine trees that line the parapet walls. When the wind loads are applied to the exterior face of the parapet wall, the torsional loading produced by the dead load weight of the parapet wall is reduced. When the wind blows across the interior portion of the plaza structures, it is diffused by the pine trees, playground equipment, and mounds of earth fill that are present on all three structures. However, if the pine trees are removed along the parapet walls to reduce the total dead load applied to the fascia beams, then the affects of wind loading need to be considered in the analysis.

For the fascia beams analyzed, the factored loads, using 5 feet of earth fill at 140 pcf, 1 foot of snow load at 40 pcf, 10-foot tall concrete parapet wall, and the self weight of the beam(s), are greater than both the shear strength of the concrete and the nominal design strength of the fascia beam(s) when the beams are assumed to carry 100 percent of the superimposed dead load. Since the shear strength of the concrete was exceeded, diagonal shear cracking was expected on all of the fascia beams on Z01, Z02, and Z03. Keep in mind that a cracked beam does not mean that it is structurally insufficient. If the steel stirrups and the concrete compression struts can carry the excess load, then the beam is structurally sufficient.

In addition to the nominal design strength of the fascia beam(s) being exceeded, the stirrups were not detailed appropriately for torsion. For each structure, the stirrup lap of the top and bottom bars (or WWF) occurred in the beam webs. A properly detailed stirrup for torsion consists of one bar wrapped around the entire beam and anchored to the longitudinal reinforcing steel with a 135-degree hook. This detail is preferred because when a rectangular beam fails in torsion, the corners of the beam tend to spall off due to the inclined compressive forces. If the stirrups are not properly anchored when the concrete spalls, the reinforcing steel will not provide the required resistance. Fortunately, for the three plaza structures, only about 30 percent of the load on the stirrups is required for torsion in the fascia beams. Therefore, the beams are subjected to a greater shearing force than torsional force. However, if any slip occurs in the stirrups as a result of an improper detail for torsion, this could be detrimental to the shear strength of the beams.

Also, the lap length for the WWF shear reinforcement for Z03 was not properly detailed. At the beam ends the required lap length is 7-7/8 inches compared the detailed lap of only 7 inches. Therefore, the web shear forces may not be properly restrained. This improper lap length could lead to premature slipping of the stirrups.

According to AASHTO LRFD, if WWF, with wires located perpendicular to the axis of the member, is used for transverse reinforcement, the transverse wires must have a minimum elongation of 4 percent measured over a 4-inch gage length including at least one cross wire. Reviewing ASTM A185 and A82, the specifications cited for WWF, we did not find any elongation requirements. Therefore, this criterion, although not required during the original design, was not checked. If the WWF cannot meet the elongation requirements and if the WWF was not stress relieved after fabrication, it may fail before the required strain is obtained.

Throughout the analysis, the tensile requirements of the longitudinal reinforcement were determined. It should be mentioned that the longitudinal reinforcement must be distributed around the perimeter of the closed stirrups for proper placement regarding torsion. For these structures, the majority of the longitudinal resistance is provided by the prestressing strands, which are concentrated in the bottom flange. For Z02 and Z03, which use WWF, the small amount of longitudinal reinforcement does not provide a significant amount of resistance.

Z01 of 63102

The first structure analyzed, Z01, was the only one of the three structures constructed with Grade 40 #4 bar stirrup cages. With the cooperation of C.A. Hull Co., Inc. we obtained a copy of the shop drawings. From these drawings, we determined that the stirrups were similar to that detailed in the contract plans. However, the prestressing strands were increased from a ½-inch diameter as stated in the contract plans to 9/16-inch diameter, along with a corresponding decrease in the number of strands. Through our file review, we found beams designed with both ½-inch and 9/16-inch diameter prestressing strands. For determining the prestressing forces, we used the initial prestress force per shop drawings and total losses determined through the MDOT Bridge Program. Also, by reviewing the mill certifications, we were able to modify the yield strength of the stirrups to 45 ksi using a statistical method based on the final draft of the Canadian Bridge Design Code.

Three different fascia beams (Beams P1-1, P5-24, and P10-24 as designated on the plans), along with ten interior beams, were analyzed to determine if they are adequate to carry the existing loads. These beams represented the maximum span length for the beam depth and prestressing strand combination.

Beam P1-1

The first beam, P1-1, was the west fascia beam of span 1. This 48-inch box beam, had a center of bearing to center of bearing span length of 86'-6inch and a total of thirty-four prestressing strands with twenty-two bonded at the beam end. Since the first four box beams were post-tensioned together, we had to first determine if the four beam combination acted as a unit. This was performed by determining the torsional shear force present between the beams and comparing it with the frictional force created by the transverse post-tensioning. Using the applied loads, it was found that the frictional force was adequate to carry the torsional shear force. Once we found that the beam combination acts as a unit, a model of the beams was generated in a finite element program, GTSTRUDL, and the proportion of the applied loads carried by the fascia beam was determined. From the finite element program, we discovered that the fascia beam carried 23 percent of the earth and snow load located above the box beam combination and 80 percent of the parapet wall. We expected a large percentage of the parapet weight to be carried by the fascia beam because there was not enough area between the parapet wall and fascia beam to evenly distribute the load to the four beam combination.

Once the distribution factors of the applied loads were determined, the next step was to check if the reinforcing steel in the exterior fascia beam web was sufficient to carry the loads. The exterior web of the fascia beam was analyzed because at this location the shear and torsional forces are additive. During our analysis, we checked the beam for combined shear and torsion according to design procedures in the AASHTO LRFD as well as analyzed the beam with the measured crack angles that were obtained though our field inspection. AASHTO LRFD addresses combined shear and torsion section using the modified compression theory, which accounts for the contribution of tensile stresses in the concrete between cracks. Therefore, the crack angle plays an important role in the design and/or analysis. Since the design angle differed from the measured angle, we performed two different analyses. Therefore, using the combined shear and torsional design per AASHTO LRFD, we found that for Grade 45 (modified per the final draft of Canadian Bridge Design Code) stirrups, the box beam combination requires 0.21 in^2 of steel per 7-inch stirrup spacing for the vertical steel per design. Per analysis, the exterior fascia web requires 0.28 in^2 of steel per 7-inch stirrup spacing. Both requirements exceeded the existing stirrup reinforcement steel of 0.20 in^2 of steel per 7-inch stirrup spacing. Since the required reinforcement was greater than the existing reinforcement, we performed further analysis to determine if the 10-foot tall concrete parapet wall would act composite with the four beam combination and take part of the superimposed dead load (earth fill and snow loads). Using the interface shear transfer - shear friction theory from AASHTO LRFD, we found that the parapet wall did act composite with the four beam combination and could take a portion of the superimposed dead load. Our next step was then to determine the amount of superimposed dead load that the wall carried. For this, we proportioned the superimposed dead between the parapet wall and four beam combination using relative stiffness and the effective moment of inertia. After three iterations using the effective moment of inertia, we found that the parapet wall can take about 42 percent of the superimposed dead load. It should be noted that the parapet wall did not have enough flexural capacity to withstand the imposed superimposed dead load but would show signs of distress when overloaded that would alert inspectors to take corrective action. Subtracting the portion of the superimposed dead load carried by the parapet wall and applying the distribution factor of 23 percent obtained from finite element analysis, we found that 0.11 in^2 of steel per 7-inch stirrup spacing was required for the design of the exterior web of the fascia beam web and that 0.22 in^2 of steel per 7-inch stirrup spacing was required per analysis. Although the design requirement was less than the existing 0.20 in^2 of steel per 7-inch stirrup spacing, the required reinforcing steel per analysis using measured crack angles was insufficient.

Using the measured crack angle and the proportioned superimposed dead loads, we found that the longitudinal tensile resistance provided at the critical section d_v and at the bearing was sufficient to overcome the respective longitudinal tensile forces.

As stated, two different methods were performed during the analysis of these beams. For the analysis method according to Nilson, the total load applied was divided evenly between the four box beams; i.e., the parapet wall was distributed evenly to the four beams. From this analysis, we found that prior to proportioning the superimposed dead load between the box beam combination and parapet wall, the exterior web of the fascia beam required 0.27 in^2 of steel per 7-inch stirrup spacing. After proportioning the superimposed dead load, the exterior web of the fascia beam required 0.17 in^2 of steel per 7-inch stirrup spacing.

Using AASHTO LFRD and the measured crack angles, we determined that the maximum earth fill based on the existing longitudinal steel resistance at the bearing (controlling case) along with the existing stirrups. When the entire superimposed dead load was considered, the maximum fill based on the longitudinal steel resistance was 3.5 feet and the maximum fill based on the existing stirrups was 3.7 feet. Proportioning the superimposed dead load between the parapet wall and the box beam combination, the maximum fill based on the longitudinal resistance was 6.2 feet and the maximum fill based on the existing stirrups was 6.6 feet.

A summary of analysis for Beam P1-1 is shown in Figure 15.

Beam P5-24

The second beam analyzed for Z01 of 63102, P5-24, was the east fascia beam of span 1. This 60-inch box beam had a center of bearing to center of bearing span length of 97 feet 9-3/4 inches and a total of thirty-four prestressing strands with twenty-two bonded at the beam end. Using the same methods as beam P1-1, we found that the transverse post-tensioning force provided sufficient frictional force to overcome the torsional shear force presented with the applied loads. Through a finite element analysis of the four beam combination, we found the fascia beam carried 23 percent of the earth and snow loading, and 80 percent of the parapet wall loading.

Using the distribution factors obtained through finite element analysis and AASHTO LRFD combined shear and torsion specifications, we found for design, the exterior fascia beam web required 0.24 in^2 of steel per 9-inch stirrup spacing and for analysis using measured crack angles the exterior fascia beam web required 0.33 in^2 of steel per 9-inch stirrup spacing. Both requirements exceeded the existing stirrup reinforcement steel of 0.20 in^2 of steel per 9-inch stirrup spacing. Since the required reinforcement was greater than the existing reinforcement, we proportioned the superimposed dead loads between the parapet wall and the four beam combination as previously stated. From this, we found that the parapet wall could take about 30 percent of the superimposed dead load. Subtracting this from the total superimposed dead load and applying the distribution factor of 23 percent obtained from finite element analysis, we found that 0.15 in^2 of steel per 9-inch stirrup spacing was required for design and 0.28 in^2 of steel per 9-inch stirrup spacing was required per analysis. Although the design requirement was less than the existing 0.20 in^2 of steel per 9-inch stirrup spacing, the required reinforcement steel per analysis using measured crack angles was insufficient.

As with the previous beam, by using the measured crack angle and the proportioned superimposed dead loads, we found that the longitudinal tensile resistance provided at the critical section d_v was sufficient to overcome the longitudinal tensile forces at this location. However, at the bearing location, the longitudinal tensile resistance was not sufficient to restrain the longitudinal tensile forces. As a result of the insufficient longitudinal tensile resistance, once a shear crack develops, the longitudinal steel could not prevent the crack from opening.

Analyzing the fascia beam according to Nilson and distributing the total applied loads evenly between the four beam combination, we found that the exterior fascia beam web required 0.28 in^2 of steel per 9-inch stirrup spacing. When the superimposed dead load was proportioned between the parapet wall and four beam combination, the exterior fascia beam web required 0.20 in^2 of steel per 9-inch stirrup spacing.

Using AASHTO LFRD and the measured crack angles, we determined that the maximum earth fill based on the existing longitudinal steel resistance at the bearing (controlling case) along with the existing stirrups. When the entire superimposed dead load was considered, the maximum fill based on the longitudinal steel resistance was 1.9 feet and the maximum fill based on the existing stirrups was 2.9 feet. Proportioning the superimposed dead load between the parapet wall and the box beam combination, the maximum fill based on the longitudinal resistance was 2.8 feet and the maximum fill based on the existing stirrups was 4.2 feet.

A summary of analysis for Beam P5-24 is shown in Figure 15.

Beam P10-24

The third and final beam for Z01 of 63102, P10-24, was the east fascia beam of span 2. This 60-inch box beam, had a center of bearing to center of bearing span length of 113'-3 5/8-inch and total of forty-two of prestressing strands with twenty-four bonded at the beam end. Using the same methods as previously discussed, we found that the transverse post-tensioning force provided sufficient frictional force to overcome the torsional shear force presented with the applied loads. Through a finite element analysis of the four beam combination, we found the fascia beam carried 23 percent of the earth and snow loading, and 79 percent of the parapet wall loading.

Using the distribution factors obtained through finite element analysis and AASHTO LRFD combined shear and torsion specifications, we found for design the exterior fascia beam web required 0.28 in^2 of steel per 7-inch stirrup spacing and for analysis of the box beam using measured crack angles the exterior fascia beam web required 0.32 in^2 of steel per 7-inch stirrup spacing. Both requirements exceeded the existing stirrup reinforcement steel of 0.20 in^2 per 7-inch stirrup spacing. Therefore, we proportioned the superimposed dead loads between the parapet wall and the four beam combination. From this, we found that the parapet wall could take about 29 percent of the superimposed dead load. Subtracting this from the superimposed dead load and applying the distribution factor of 23 percent obtained from finite element analysis, we found that 0.19 in^2 of steel per 7-inch stirrup spacing was required for design and 0.27 in^2 of steel per 7-inch stirrup spacing was required per analysis. Although the design requirement was less than the existing 0.20 in^2 of steel per 7-inch stirrup spacing, the required reinforcement steel per analysis using measured crack angles was insufficient.

As with the previous beams, by using the measured crack angle and the proportioned superimposed dead loads, we found that the longitudinal tensile resistance provided at the critical section d_v was sufficient to overcome the longitudinal tensile forces at this location. However, at the bearing location, the longitudinal tensile resistance was not sufficient to restrain the longitudinal tensile forces. As a result of the insufficient longitudinal tensile resistance, once a shear crack develops, the longitudinal steel could not prevent the crack from opening.

Analyzing the fascia beam according to Nilson and distributing the total applied loads evenly between the four beam combination, we found that the exterior fascia beam web required 0.31 in^2 of steel per 7-inch stirrup spacing. When the superimposed dead load was proportioned between the parapet wall and four beam combination, the exterior fascia beam web required 0.22 in^2 of steel per 7-inch stirrup spacing.

Using AASHTO LFRD and the measured crack angles, we determined that the maximum earth fill based on the existing longitudinal steel resistance at the bearing (controlling case) along with the existing stirrups. When the entire superimposed dead load was considered, the maximum fill based on the longitudinal steel resistance was 1.6 feet and the maximum fill based on the existing stirrups was 2.9 feet. Proportioning the superimposed dead load between the parapet wall and the box beam combination, the maximum fill based on the longitudinal resistance was 2.3 feet and the maximum fill based on the existing stirrups was 4.2 feet.

A summary of analysis for Beam P10-24 is shown in Figure 15.

Interior Beams

Ten different interior beam sizes were checked for adequate shear strength. The sizes and spans ranged from a 48-inch box beam with a center of bearing to center of bearing span of 89 feet 6 inches to a 60-inch box beam with a center of bearing to center of bearing span of 112 feet 0 inches. For all ten beams, the required vertical reinforcement, longitudinal tensile resistance at the critical section d_v, and the longitudinal tensile resistance at the bearings were determined. In each case, the required vertical reinforcement was less than the existing vertical reinforcement and the longitudinal tensile resistance provided at the critical section d_v was sufficient to overcome the respective longitudinal tensile forces. However, one out of the ten beams, which represents roughly 7 percent of the total number of interior beams, had inadequate longitudinal tensile resistance at the bearing. For the interior beams, we determined the maximum earth fill based on the existing longitudinal steel resistance at the bearing, which was the controlling case. From this we found that the maximum fill for a 48-inch box beam should be 5.9 feet, for a 54-inch box beam the maximum fill should be 5.3 feet, and for a 60-inch box beam the maximum fill should be 4.7 feet.

Refer to Figure 16 for the analysis results of the ten interior beams.

Z02 of 63102

As with Z01, Z02 also utilizes the transverse post-tensioning of the first four box beams. Therefore, the same theory and methodology for analysis were used for both Z01 and Z02. However, through our file review, we found that the prestressed box beams for Z02 were constructed with Grade 60 WWF in lieu of the Grade 40 #4 bar stirrups as detailed in the contract plans. Since we were unable to determine the location and spacing of the WWF without shop drawings, we could not evaluate the beams as constructed. However, assuming the WWF provided the same resistance as the #4 bar stirrups, the box beams were evaluated per the contract plans.

Beam P14-22

The first beam analyzed, P14-22, was the east fascia of span 2. This 48-inch box beam, had a center of bearing to center of bearing span length of 88 feet 8inches and total of forty-two of prestressing strands with twenty-four bonded at the beam end. Using the same theory as discussed for Z01 of 63102, we found that the transverse post-tensioning force provided sufficient frictional force to overcome the torsional shear force presented with the applied loads. Through a finite element analysis of the four beam combination, we found the fascia beam carried 23 percent of the earth and snow loading, and 80 percent of the parapet wall loading.

Using the distribution factors obtained through finite element analysis and AASHTO LRFD combined shear and torsion specifications, we found that for design the exterior fascia beam web required 0.22 in^2 of steel per 6-inch stirrup spacing and for analysis the exterior fascia beam web required 0.35 in^2 of steel per 6-inch stirrup spacing. Both requirements exceed the existing stirrup reinforcement steel of 0.20 in^2 of steel per 6-inch stirrup spacing. Therefore, we proportioned the superimposed dead loads between the parapet wall and the four beam combination. From this, we found that the parapet wall could take about 42 percent of the superimposed dead load. Subtracting this from the total superimposed dead load and applying the distribution factor of 23 percent obtained from finite element analysis, we found that 0.13 in^2 of steel per 6-inch stirrup spacing was required for design and 0.27 in^2 of steel per 6-inch stirrup spacing was required per analysis. Although the design requirement was less than the existing 0.20 in^2 of steel per 6-inch spacing, the required reinforcement steel per analysis using measured crack angles was insufficient.

As with the previous beams for Z01 of 63102, by using the measured crack angle and the proportioned superimposed dead loads, we found that the longitudinal tensile resistance provided at the critical section d_v and at the bearing was sufficient to overcome the respective longitudinal tensile forces.

Analyzing the fascia beam according to Nilson and distributing the total applied loads evenly between the four beam combination, we found that the exterior fascia beam web required 0.29 in^2 of steel per 6-inch stirrup spacing. When the superimposed dead load was proportioned between the parapet wall and four beam combination, the exterior fascia beam web required 0.19 in^2 of steel per 6-inch stirrup spacing.

Using AASHTO LFRD and the measured crack angles, we determined that the maximum earth fill based on the existing longitudinal steel resistance at the bearing (controlling case) along with the existing stirrups. When the entire superimposed dead load was considered, the maximum fill based on the longitudinal steel resistance was 2.8 feet and the maximum fill based on the existing stirrups was 2.1 feet. Proportioning the superimposed dead load between the parapet wall and the box beam combination, the maximum fill based on the longitudinal resistance was 5.0 feet and the maximum fill based on the existing stirrups was 3.9 feet.

A summary of analysis for Beam P14-22 is shown in Figure 15.

Beam P8-1

The final beam analyzed for Z02 of 63102, P8-1, was the west fascia of span 2. This 60-inch box beam had a center of bearing to center of bearing span length of 111 feet 9-3/8 inches and a total of fifty-four of prestressing strands with twenty-four bonded at the beam end. Since this beam combination was similar to that of P10-24 of Z01 of 63102, we used a portion of the analysis obtained from P10-24 for P8-1. The only difference between the two beams as detailed in the MDOT plans was that the center of bearing to center of bearing span length for P10-24 was 113 feet 3-5/8 inches and the center of bearing to center of bearing for P8-1 was 111 feet 9-3/8 inches. Therefore, the shear and torsional forces will be slightly less for Z02, however, the minor difference will not significantly affect the outcome. The previous results for P10-24 of Z01 could not be used to directly compare P8-1 of Z02 because the prestressing strands used for Z01 were 9/16-inch diameter and for Z02 they were ½-inch diameter. However, by using P10-24 of Z01 of 63102, we found that the transverse post-tensioning force provided sufficient frictional force to overcome the torsional shear force presented with the applied loads and through a finite element analysis of the four beam combination, we found the fascia beam carried 23 percent of the earth and snow loading and 79 percent of the parapet wall loading.

Using the distribution factors obtained through finite element analysis and AASHTO LRFD combined shear and torsion specifications, we found that for design the exterior fascia beam web required 0.38 in^2 of steel per 6-inch stirrup spacing and for analysis the exterior fascia web required 0.34 in^2 of steel per 6-inch stirrup spacing. Both requirements exceed the existing stirrup reinforcement steel of 0.20 in^2 of steel per 6-inch stirrup spacing. Therefore, we proportioned the superimposed dead loads between the parapet wall and the four beam combination. From this, we found that the parapet wall could take about 29 percent of the superimposed dead load. Subtracting this from the total superimposed dead load and applying the distribution factor of 23 percent obtained from finite element analysis, we found that 0.25 in^2 of steel per 6-inch stirrup spacing was required for design and 0.29 in^2 of steel per 6-inch stirrup spacing was required per analysis using measured crack angles. Therefore, if the WWF substituted for the #4 bar stirrups provided an equivalent resistance per 6-inch, then the required vertical reinforcing steel exceeded the existing reinforcement.

By using the measured crack angle and the proportioned superimposed dead loads, we found that the longitudinal tensile resistance provided at the critical section d_v was sufficient to overcome the longitudinal tensile forces at this location. However, at the bearing location, the longitudinal tensile resistance was not sufficient to restrain the longitudinal tensile forces. As a result of the insufficient longitudinal tensile resistance, once a shear crack develops, the longitudinal steel could not prevent the crack from opening.

Analyzing the fascia beam according to Nilson and distributing the total applied loads evenly between the four beam combination, we found that the exterior fascia beam web required 0.34 in^2 of steel per 6-inch stirrup spacing. When the superimposed dead load was proportioned between the parapet wall and four beam combination, the exterior fascia beam web required 0.26 in^2 of steel per 6-inch stirrup spacing.

Using AASHTO LFRD and the measured crack angles, we determined that the maximum earth fill based on the existing longitudinal steel resistance at the bearing (controlling case) along with the existing stirrups. When the entire superimposed dead load was considered, the maximum fill based on the longitudinal steel resistance was 1.1 feet and the maximum fill based on the existing stirrups was 2.3 feet. Proportioning the superimposed dead load between the parapet wall and the box beam combination, the maximum fill based on the longitudinal resistance was 1.7 feet and the maximum fill based on the existing stirrups was 3.1 feet.

A summary of analysis for Beam P8-1 is shown in Figure 15.

Interior Beams

Twelve different interior beam sizes were checked for adequate shear strength. The sizes and spans ranged from a 48-inch box beam with a center of bearing to center of bearing span of 89 feet 6 inches to a 60-inch box beam with a center of bearing to center of bearing span of 111 feet 0 inches. For all twelve beams the required vertical reinforcement, longitudinal tensile resistance at the critical section d_v, and the longitudinal tensile resistance at the bearings were determined. In each case, the required vertical reinforcement was less than the existing vertical reinforcement and the longitudinal tensile force at the critical section d_v was less than the longitudinal tensile resistance at d_v. However, eight of the twelve beams, which represents about 42 percent of the total interior beams, had insufficient longitudinal tensile resistance at the bearing. The lack of longitudinal tensile resistance may explain why we saw large crack widths on the interior beams.

For the interior beams, we determined the maximum earth fill based on the existing longitudinal steel resistance at the bearing, which was the controlling case. From this we found that the maximum fill for a 48-inch box beam should be 6.3 feet, for a 54-inch box beam the maximum fill should be 4.5 feet, and for a 60-inch box beam the maximum fill should be 3.5 feet.

Refer to Figure 17 for the analysis results of the twelve interior beams.

Z03 of 63102

The final set of calculations was performed on Z03 of 63102. This was the first of the three plaza structures to be designed and constructed. For this structure only, the fascia beam was not post-tensioned to the interior beams, therefore the 48-inch box beam carried the entire parapet wall and earth fill load placed upon it. However, the torsional loading produced by the horizontal earth pressure was resisted though a turnbuckle assembly attached to the parapet wall and the second interior beam.

From the shop drawings, we discovered that the beams were constructed with Grade 60 WWF in lieu of the Grade 40 #4 bars stirrups stated in the contract plans. The WWF was to meet ASTM A185 according to the shop drawings. Comparing the size, grade and spacing differences, we determined that the WWF was an equivalent alternative to the #4 bar stirrups. Reviewing the lap length requirements per AASHTO LRFD, we discovered that the WWF at the beam end has an inadequate lap length for shear. According to AASHTO LRFD, the required lap length is 7-7/8 inches, however, the detailed lap length was only 7 inches. Therefore, the web shear forces may not be properly restrained. This improper lap length could lead to premature slipping of the stirrups.

For Z03 of 63102, only 48-inch box beams with a total of forty prestressing strands, twenty-four bonded at the end, were used for the fascia beams. Therefore, only one beam was analyzed using the maximum center of bearing to center of bearing span length of 87 feet 4-5/8 inches. Although the MDOT plan sheets specify a 28-day compressive strength (f'c) of 6,000 psi for the prestressed concrete, we used a compressive strength of 5,000 psi as stated on the shop drawings. The MDOT plans do not specify a release strength unlike the shop drawings, which states a release strength of 3,500 psi for the prestressed beams. Comparing this release strength to that of the 1984 Standard Specifications for Construction, we found that it was below the required release strength of 4,000 psi in the standard specifications.

Using the combined shear and torsion specifications according to AASHTO LRFD, we found that the required amount for design of vertical reinforcing steel in the exterior web of the fascia beam was 0.29 in^2 of steel per 12-inch stirrup spacing. An equivalent 12-inch stirrup spacing for the WWF was used for simplicity. The required amount of reinforcing steel in the exterior fascia web from analysis using measured crack angles was 0.26 in^2 of steel per 12-inch stirrup spacing. The existing vertical reinforcement steel was 0.24 in^2 of steel per 12-inch stirrup spacing. Since the required reinforcement was greater than the existing reinforcement, we proportioned the superimposed dead loads between the parapet wall and the fascia beam. From this, we found that the parapet wall could take about 86 percent of the superimposed dead load. Subtracting this from the total superimposed dead load, we found that 0.15 in^2 of steel per 12-inch stirrup spacing was required for design and 0.16 in^2 of steel per 12-inch stirrup spacing was required per analysis. Both required values were less than the existing 0.24 in^2 of steel per 12-inch stirrup spacing. Therefore, by proportioning the superimposed dead load between the parapet wall and the fascia beam, there was sufficient vertical reinforcement.

Using the measured crack angle and the proportioned superimposed dead loads, we found that the longitudinal tensile resistance provided at the critical section d_v and at the bearing was sufficient to overcome the respective longitudinal tensile forces.

Analyzing the fascia beam according to Nilson, we found that the exterior fascia beam web required 0.49 in^2 of steel per 12-inch stirrup spacing. When the superimposed dead load was proportioned between the parapet wall and fascia beam, the exterior fascia beam web required 0.27 in^2 of steel per 12-inch stirrup spacing.

Using AASHTO LFRD and the measured crack angles, we determined that the maximum earth fill based on the existing longitudinal steel resistance at the bearing (controlling case) along with the existing stirrups. When the entire superimposed dead load was considered, the maximum fill based on the longitudinal steel resistance was 1.9 feet and the maximum fill based on the existing stirrups was 6.1 feet. Proportioning the superimposed dead load between the parapet wall and the fascia beam, the maximum fill based on the longitudinal resistance was 15.3 feet and the maximum fill based on the existing stirrups was 44.9 feet. In theory, if 14 percent of the superimposed dead load was carried by the box beam, the longitudinal force at the bearing produced by 15.3 feet of fill could be restrained by the existing steel. Likewise, the existing stirrups in the beam at section d_v could withstand 14 percent of the loading produced by 44.9 feet of fill. However, the parapet wall would not have been able to carry these large amounts of fill and would have failed in both flexure and shear long before reaching these depths.

A summary of analysis for Z03 of 63102 is shown in Figure 15.

Interior Beams

Reviewing the shear strength of the interior beams revealed the required vertical reinforcement, 0.28 in^2 of steel per 12-inch stirrup spacing, was less than the existing 0.48 in^2 per 12-inch stirrup spacing. It was also found that the longitudinal tensile resistance provided at the critical section d_v and at the bearing was sufficient to overcome the respective longitudinal tensile forces.

For the interior beams, we determined the maximum earth fill based on the existing longitudinal steel resistance at the bearing, which was the controlling case. From this we found that the maximum fill for the 48-inch box beam should be 5.2 feet.

Refer to Figure 18 for a summary of the results.

At the end of Phase I, the following list of possible solutions to increase the strength of the beams to address the unaccounted dead loads (i.e., extra earth fill, playground equipment, sidewalks, trees, and sprinkler systems) and steel reinforcement details was generated.

1. Replace affected beams.
2. Remove part of the earth load.
3. Remove the earth load and replace with a light weight fill.
4. Remove trees.
5. Eliminate or reduce the sprinkler system.
6. Reduce the height (mass) of the parapet wall.
7. Re-grade the earth fill so that it water drains away from the structure easier.
8. Cast a concrete deck composite with the beams.
9. Place carbon fiber reinforced polymer (CFRP) sheets on exterior fascia beam webs and bottom flanges of all beams where the maximum earth fill is exceeded.
10. Inject cracks with epoxy.

11. Wrap the beam end with a metal sleeve that is epoxy injected for bond.
12. Vertically post-tension between the beams.

From this list, we recommended that the cracks in the fascia beams as well as any crack in the bottom flange of the interior beams with a width greater than 0.010 inches be epoxy injected and place CFRP sheets to all of the exposed fascia beam webs and all bottom flanges of the fascia beams. The CFRP sheets should be attached to the bottom flanges of the first, second, and third interior beams from the east and west fascia for span 1 and 2 on Z01 and Z02 and on the bottom flanges of any interior beam with a crack width (measured on the bottom flange) greater than 0.040 inches. In all cases, the CFRP should extend a distance not less than twice the beam depth from the end to provide the required shear and torsional reinforcement. A typical application of the CFRP sheets is shown in Figure 19. The CFRP sheets would also serve to mitigate the improper detailing of the torsional reinforcement. The outside fascia beam web should be coated with a concrete coating in the area of the CFRP sheets for esthetics. For the interior beams, the existing cracks in the webs should be sealed to prevent moisture from contacting the reinforcing and prestressing steel by injecting an expandable urethane foam between the beam gaps. The expandable urethane foam should extend a distance at least three times the beam depth from the end of the beam. Also, a topographical survey of the earth fill should be performed. If the existing fill depths exceed the maximum fill depth as stated in Figures 16, 17, 18 (Z01, Z02, and Z03 summary of results, respectively), then the excess fill should be removed. If removing the earth fill is not feasible, then attaching CFRP sheets to the bottom flanges for a distance not less than twice the beams depth from the end is recommended. On top of the structures, the sprinkler systems use should be eliminated.

If the pine trees are removed along the parapet walls to reduce the total dead load applied to the fascia beams, then the affects of wind loading need to be considered in the analysis.

Maintenance should continue to inspect these structures on a six month basis until the repair work has been performed.

Two cost estimates were prepared from our recommendations, refer to Figure 20. The first was derived from removing the excess earth fill and only placing CFRP sheets on all of the fascia beams as well as on the first, second, and third interior beams for Z01 and Z02. This cost estimate was $1,000,000 for all three bridges. The second estimate was based on earth fill removal not being feasible. For this cost estimate we placed CFRP sheets on the bearing ends of every beam as well as on the fascia beam webs. This cost estimate for all three bridges was $3,100,000.

PHASE II

On January 29, 1999, a meeting with various MDOT personnel was held to discuss our findings of Phase I and the direction for Phase II. It was determined that the next logical step was to determine the maximum fill depths on top of each of the three structures. This required an extensive topographical survey that was broken into the following three parts; profile the top of the earth fill, profile the bottom of box beam elevations, and verify the landscaping details with those outlined in the *As Constructed* landscaping contract plans. The fill depth was then determined by subtracting the top of earth fill profile from the top of beam profile. With the assistance of CAICE, a computer program used for surveying, our Design Division was able to adjust the bottom of beam profile to represent the top of beam profile. With the adjustment made, a delta profile, i.e., fill depth, between the top of earth fill and the top of beams was created. Once the delta profile was created, a topographical map of the fill depth was generated.

This phase also included an extensive inspection of the interior beams. With the assistance an MDOT bridge inspector, the number and maximum crack widths on each of the cracked interior beams were recorded. During this inspection, we noticed that a few of the crack widths recorded while performing our May 1998 inspection had increased. The time frame between the two inspections was nine months.

From the survey and inspection, we did not find any correlation between the areas with excess fill and the cracked beams. Assuming the unit weight of the earth fill equals 140 pcf and the maximum allowable fill was determined based on the allowable longitudinal resistance, refer to Figures 16-18, we were unable to associate the areas with excessive fill depths to those areas with cracked beams. Figures 21-23 display the areas that contain cracked beams and the areas where the fill is exceeded.

To obtain a more complete understanding of the existing soil conditions, the MDOT Geotechnical Unit extracted numerous soil samples from all three structures, refer to Figure 24. From these samples the moisture content, soil classification, and unit weight were obtained. The moisture content of the soil samples taken from the three structures were as follows: Z01 varied from 5.5 percent to 31.9 percent, with the average at 15.4 percent; Z02 varied from 5.4 percent to 31.3 percent, with the average at 15.1 percent; and Z03 varied from 2.7 percent to 27.1 percent, with the average at 13.0 percent. The soil type on all three structures ranged from poorly graded sand with silt to clayey sand. One sample obtained from Z03 contained steel furnace slag. The moist unit weight of the soil ranged from 113.6 pcf to 141.8 pcf on Z01, 101.5 pcf to 139.6 pcf on Z02. The method of soil extraction on Z03 was different than on Z01 and Z02, therefore only one moist unit weight was estimated at 136.4 pcf. The average of all unit weights are less than the 140 pcf used for the calculations. Therefore, we were conservative in or estimates.

From Phase II of this investigation, we found that the current loads applied to the interior beams would not cause the magnitude and number of cracks that were found. Therefore, the cracks must have been initiated as a result of fabrication errors or during construction and grown due to the lack of longitudinal reinforcement in the bottom flange. Through conversations with the construction engineer at the time of construction, there were minimal load restrictions placed on the superstructure during the construction of Z03. However, as the successive bridges were built, the load restrictions increased. This was evidenced by the number of cracked beams found on the three structures. The first structure constructed, Z03, has the widest and largest number of cracked beams and was constructed with minimal load restrictions. The second constructed structure, Z01, had some load restrictions in place and has fewer and tighter cracks than Z03. The final structure constructed, Z02, had the most load restrictions and has no cracks on the interior beams.

Our findings of Phase II, along with eight different repair options, were discussed during a March 18, 1999, meeting with MDOT personnel. A brief description of the eight options are as follows:

1. Remove excess earth fill and place CFRP sheets on the exterior webs and the bottom flanges of the fascia beams($1,035,000)
2. Do not remove the excess earth fill and place CFRP sheets on the bottom flanges of all the beams along with the exterior web of the fascia beams($3,120,000)
3. Remove the excess earth fill and place CFRP sheets on the bottom flanges of all the beams along with the exterior webs of the fascia beams($3,220,000)
4. Remove the excess earth fill and replace the fascia beam area (first four beams)($2,682,000)
5. Do not remove the excess earth fill, replace the fascia beam area, and place CFRP sheets on all remaining beams' bottom flanges($4,760,000)
6. Remove the excess earth fill, replace the fascia beam area, and place CFRP sheets on all remaining beams' bottom flange($4,865,000)
7. Remove and replace the first four beams and parapet wall on all three structures, place CFRP sheets on the bottom flange of all cracked interior beams, epoxy inject all bottom flange cracks, and place urethane foam in the beam gaps of all cracked beams($2,380,000)
8. Remove and replace the first four beams and parapet wall on all three structures as well as 56 interior beams on Z03, place CFRP sheets on the bottom flange of all cracked interior beams, epoxy inject all bottom flange cracks, and place urethane foam in the beam gaps of all cracked beams($6,900,000)

Option number 8 was decided upon for the repair of the plaza structures. This option although the most expensive, $6,900,000, will provide the largest factor of safety and greatly extend the service life of the plaza structures.

During this meeting, it was also decided to increase the frequency of the plaza inspections from the current six month schedule to every three months until the structures were repaired.

As part of the repair contract awarded in January 2000, the contractor was required to salvage and load test four of the replaced prestressed box beams to shear failure. The load test results were used to verify the adequacy of the remaining prestressed box beams. The four box beams set up for testing were: beam number 116 (with the west fascia beam being number 1), span 2, from Z01 of 63102; beam number 4 (with west fascia beam being number 1), span 2, from Z02 of 63102; and beam numbers 153 and 166 (with west fascia beam being number 1), span 1, from Z03 of 63102.

In preparation for the load test, the test beams were cleaned with a pressure washer (if necessary) and white washed using a solution of lime and water. The white wash served to highlight the cracks during the test. Once white washed, a total of nine reflectors were placed along the beam to measure deflections. To measure bearing settlement, one reflector was placed at the end of the beam in the center of the end block and one reflector was placed in the web above each bearing pad. Deflections were measured using reflectors placed in the web near the third points and at the midspan. To monitor for off center loading that would cause torsion in the beam, two reflectors were placed on the top of the beam at midspan over each web. Construction and Technology's Survey Unit used a total station surveying instrument and the mounted reflectors to measure the deflections. With the reflectors in place, a tilt sensor was mounted the end of the beam to measure the beam end rotation. The midspan deflection as well as the beam end rotation were monitored throughout the load test to compare the expected results with the actual results.

The contractor proposed to use concrete blocks salvaged from the plaza's parapet wall for the load test's free weights. This was an acceptable method providing that all concrete blocks were weighed using calibrated scales. Depending on the size of the blocks, the weights ranged from 12,150 lbs. to 19,580 lbs. During the load test, the blocks were placed within a distance not less than 4 feet from the inside edge of the bearing to not greater than 35 feet from the inside edge of the bearing. The limits of the placement ensured a shear failure as opposed to a flexural failure. Starting at the beam support, the load was applied in increments consisting of three concrete blocks with a maximum weight of 51,750 lbs. Once the maximum loading increment was obtained, the beam supported this load for five minutes prior to measuring the deflections and beam end rotations. Refer to Figure 25 to view the load test setup.

On April 13, 2000, beam number 153 from Z03 of 63102 was load tested. This beam represented the remainder of the in-service beams on this structure having minor webs cracks that do not extend into the bottom flange. The ultimate proof load at d_v (the critical shear location) was 342 kips. Comparing this to the largest factored dead load plus factored pedestrian loading at d_v equaling 238 kips according to AASHTO LRFD for the remaining 48-inch box beams on Z03 with minor web cracks demonstrates that they have adequate shear strength. Refer to Figure 26 to view the beam after the load test.

The next beam tested, beam number 166 from Z03 of 63102, was removed from service due to both the size and number of shear cracks. During our detailed inspections, we discovered this beam had nine shear cracks in each web and one of the cracks that extended thru the bottom flange had a width of about 0.17 inches. Upon removal we discovered an obvious cold joint and a layer of Styrofoam between the web and the bottom flange near the beam end, refer to Figure 27. For this test the ultimate proof load at d_v was 200 kips. The actual factored dead load plus factored pedestrian loading at d_v for this beam was 203 kips according to AASHTO LRFD. Therefore, comparing the ultimate load at d_v to that of the actual factored dead load plus factored pedestrian load at d_v, this beam had marginal shear strength and did require replacement. Refer to Figure 28 to view the beam after the load test.

Due to the obvious fabrication errors for beam number 166, another severely cracked beam from Z03 was salvaged and load tested. Beam number 27 (with west fascia beam being number 1) span 2, Z03 of 63102, was selected as the additional beam due to the number of web shear cracks (five in the west web and four in the east web), along with the width of the bottom flange cracks. One of the cracks extending thru the bottom flange had a width in excess of 0.12 inches. No obvious fabrication errors were noticed in this beam. The preparation for beam 27 differed from all other tests in that the beam end was repaired according to the contract plans, i.e., the bottom flange cracks were injected with structural epoxy and CFRP sheets were placed longitudinally along the bottom flange. The CFRP sheet extended three feet past the furthest bottom flange crack from the support. The beam end repair was performed by the same subcontractor that was used for the repair project.

The test results from beam 27 were compared to the actual factored dead load plus factored pedestrian loading at d_v for beam numbers 59 thru 76 (with fascia beam being number 1), span 2, on Z03. Through our inspections we discovered this beam had a similar number of web shear cracks as beams 59 thru 76, however the width of the bottom flange cracks were greater. The largest flange crack width on beams 59 thru 76 was 0.020 inches as compared to 0.12 inches for beam 27. Since the cracks in this group of beams are less severe than those in beam number 27, if beam 27 can demonstrate its ability to carry the actual factored dead load plus pedestrian load at d_v for beam numbers 59 thru 76 of 196 kips according to AASHTO LRFD then the strength of these beams would be adequate. From the load test, we found beam 27 had an ultimate proof load at d_v of 215 kips. Therefore, beams 59 thru 76 have adequate shear strength. Refer to Figure 29 to view the beam after the load test.

On April 19, 2000, beam number 116 from Z01 of 63102 was load tested. This 60-inch beam was salvaged from one of the four post-tensioned beam combinations. There were no interior beam removed on this structure. The ultimate proof load at d_v was 390 kips. Comparing this to the largest factored dead load plus factored pedestrian load at d_v according to AASHTO LRFD for the 60-inch box beams on Z01 of 240 kips, the remaining 60-inch box beams have adequate shear strength. Refer to Figure 30 to view the beam after the load test.

As with Z01, no interior beams removed from Z02. However, during the removal process of the Z02 test beam the top flange was inadvertently saw cut. Therefore, beam number 3, span 2, was salvaged and load tested instead. For this 60-inch box beam, the ultimate proof load at d_v was 328 kips. The largest factored dead load plus factored pedestrian load at d_v for the 60-inch box beams on Z02 is 288 kips according to AASHTO LRFD. Therefore, the remaining 60-inch box beams have adequate shear strength. Refer to Figure 31 to view the beam after the load test.

Once all of the load testing was completed, the tested beams were dissected to determine the physical properties i.e., the concrete strength; steel strength; stirrup spacing, size, and lap length; web thickness; number of prestressing strands; and number of debonded prestressing strands. From this analysis, we discovered that all of the physical properties of the beams exceeded the specified design strengths. We also discovered that the lap length for the WWF on the three Z03 beams, about 11 inches, exceeded the required length of 7-7/8 inches. One point of interest is the various web thicknesses. It appears the Styrofoam used to create the hollow portion in some of the box beam had either shifted during the placement of the concrete or it was never centered from the beginning. Another interesting finding was that for the Z01 beam, beam number 3, the outside columns of three prestressing strands each side are not enclosed within the stirrups as specified in the contract plans. Refer to Figures 32 thru 39 for the details of our findings.

Deflections of the two original salvaged beams scheduled for load testing on Z03 of 63102, beam numbers 153 and 166, were measured during three different loading conditions. The first two conditions reflected the in-service loading (i.e., earth fill present) and unloaded conditions (i.e., earth fill removed). The third loading condition reflected the load test deflection measurements taken at the loading increment prior to failure. Comparing the unloaded condition to the loaded condition for beam number 153, we found that the actual load deflected the beam about 1.34 inches at midspan. This deflection was 23 percent the measured deflection of 5.90 inches taken prior to failure during the load test. It should be mentioned that the in-service deflections were measured under a full length uniform load and the load test deflections were measured with the applied load concentrated at one end. Comparing the unloaded condition to the loaded condition for beam number 166, we found that the actual load deflected the beam about 1.54 inches at midspan. This deflection was 71 percent the measured deflection of 2.17 inches taken prior to failure during the load test. Refer to Figure 40 for a complete listing of the survey results.

SUMMARY

Phase I

During Phase I of this investigation, we performed a detailed review of the plaza structures that cross I-696 in Southfield, Michigan. From this, we discovered the 11 to 14-year old structures were experiencing numerous shear cracks on both the fascia and interior beams. Through our detailed analysis, we expected the cracks to occur during construction since the applied vertical loads were greater than the shear strength of the concrete. From construction records we were able to verify that cracks did occur either during construction or shortly there after. The only superimposed dead loads that were accounted for in the design of all three structures were 4 feet of 120 pcf earth fill, 1 foot of 40 pcf snow, and a future wearing surface at 25 psf. The loads produced by the 10-foot tall concrete parapet wall were not completely considered. Also, the additional earth fill, playground equipment, sidewalks, trees, and water from the sprinkler systems were not accounted for in the design. From this investigation, we believe that water from the sprinkler system has added a significant dead load. At the time of our field inspection, there had not been rain in the area for at least a week. However, the soil had a moisture content around 20 percent. We believe that this moisture was placed into the soil from the sprinkler system. With the high moisture content entrapped in the clayey sand used for the earth fill, the unit weight of soil increases from 120 pcf to about 140 pcf. But the water problem does not stop in the soil. Although all three structures have a complex drainage system between the earth fill and the box beams, the water is still entering the cavity of the beams. This was evidenced by the water and leachate stained cracks, along with the water that flowed out of some of the weep holes that were unplugged. When we take the added weight of the water and add the trees, sidewalks, and playground equipment we have a situation that was not contemplated in the design.

Detailed calculations were performed for all three structures using AASHTO LRFD and Arthur H. Nilson, *Design of Prestressed Concrete*, 1978. AASHTO LFRD was used as the primary code for analysis with Nilson used as a check. The analyses of the three structures were slightly different because Z01 and Z02 both have the first four box beams transversely post-tensioned. The fascia beams for Z03 are connected to the adjacent beam using only tie rods. All analyses were performed using 5 feet of earth fill at 140 pcf, 1 foot of snow at 40 pcf, self weight of the box beam(s), and included a 10-foot tall concrete parapet wall for the fascia beams. A further refinement of our analysis included a temperature loading to account for all supports being fixed against thermal movement. The depth and unit weight of the earth fill was modified from the design depth and weight of 4 feet at 120 pcf as a result of our field inspection.

For all fascia and interior beams analyzed, the shear strength of the concrete was exceeded. As a result, shear cracks are expected. However, a cracked beam does not mean that it is structurally insufficient. If the steel stirrups and the concrete compression struts can carry the excess load, then the beam is structurally sufficient.

During the design of the fascia beams, the load produced by the 10-foot concrete parapet wall was not completely considered. We found that all of the fascia beams analyzed had insufficient vertical reinforcement when the entire applied loads were used in the analysis. When the superimposed dead loads were proportioned between the parapet wall and the concrete beam(s), the fascia beams for Z01 and Z02 still had insufficient vertical reinforcement when analyzed using the measure crack angles obtained from our inspection. Through analyzing selected fascia beams, we found that they all had sufficient longitudinal resistance at the critical section d_v. However, the east fascia beams of span 1 and span 2 on Z01 and the west fascia beams of span 1 and span 2 on Z02 had insufficient longitudinal tensile resistance at the bearing. As a result of insufficient longitudinal tensile resistance, once a shear crack develops, the longitudinal steel cannot prevent the crack from opening. If the crack becomes too wide, a reduction in the shear strength will occur.

In addition to the lack of longitudinal resistance provided by the longitudinal reinforcement, it should be mentioned that for proper placement of longitudinal reinforcement required for torsion it must be distributed around the perimeter of the closed stirrups. For all three structures, the majority of the longitudinal resistance was provided by the prestressing strands, which are concentrated in the bottom flange. For Z02 and Z03, which use WWF, the small amount of longitudinal reinforcement does not provide a significant amount of resistance.

Also, on all three structures the stirrups were not detailed appropriately for torsion. For each structure, the stirrup lap of the top and bottom bars (or WWF) occurred in the beam webs. This is not the appropriate detail for torsion. A properly detailed stirrup consists of one bar wrapped around the entire beam and anchored to the longitudinal reinforcing steel with a 135-degree hook. This detail is preferred because when a rectangular beam fails in torsion, the corners of the beam tend to spall off due to the inclined compressive forces. If the stirrups are not properly anchored when the concrete spalls, the reinforcing steel will not provide required resistance. Fortunately for the three plaza structures, only about 30 percent of the load on the stirrups was required for torsion in the fascia beams. Therefore, the beams are subjected to a greater shearing force than torsional force. However, if any slip occurs in the stirrups as a result of an improper detail for torsion, this could be detrimental to the shear strength of the beams.

Aside from the reinforcing steel improperly detailed for torsion, the lap length for the WWF shear reinforcement in Z03 was not properly detailed. The required lap length is 7-7/8 inches, which is greater than the detailed lap length of 7 inches. To prevent premature failure of the WWF when used for transverse reinforcement, the wires must provide a minimum elongation of 4 percent when measured over a 4-inch gage length and be stress relieved after fabrication according to AASHTO LRFD. Reviewing the specifications cited for WWF, we did not find elongation requirements nor requirements for the WWF to be stress-relieved after fabrication. A lack of shear reinforcement lap length, along with the possibility of premature failure of WWF, may also explain the diagonal shear cracks found on the interior beams of Z03. Since shear failures occurs with little warning, proper steel detailing is required to prevent premature failure.

A total of twenty-three interior beams were analyzed for all three structures. Ten were analyzed for Z01, twelve for Z02, and one for Z03. For each beam, the required vertical reinforcement was less than the existing vertical reinforcement and the longitudinal steel resistance provided at the critical section d_v was greater than the longitudinal tensile force. However, from these analyzed beams we found that 7 percent of the interior beams for Z01 and 42 percent of the interior beams for Z02 did not have sufficient longitudinal resistance at the bearing when analyzed with 5 feet of earth fill. This lack of longitudinal steel resistance may explain why we saw large crack widths on the interior beams.

Phase II

From the extensive survey performed during Phase II of this investigation, we found that the current loads applied to the interior beams would not cause the magnitude and number of cracks that were found. Therefore, the cracks must have been initiated as a result of fabrication errors or during construction and grown due to the lack of longitudinal reinforcement in the bottom flange. Through conversations with the construction engineer at the time of construction, there were minimal load restrictions placed on the superstructure during the construction of Z03. However, as the successive bridges were built, the load restrictions increased. This was evidenced by the number of cracked beams found on the three structures. The first structure constructed, Z03, has the widest and largest number of cracked beams and was constructed with minimal load restrictions. The second constructed structure, Z01, had some load restrictions in place and has fewer and tighter cracks than Z03. The final structure constructed, Z02, had the most load restrictions and has no cracks on the interior beams.

Upon the completion of Phase II, there were a total of eight cost estimates generated. A brief description of the eight options are as follows:

1. Remove excess earth fill and place CFRP sheets on the exterior webs and the bottom flanges of the fascia beams($1,035,000)
2. Do not remove the excess earth fill and place CFRP sheets on the bottom flanges of all the beams along with the exterior web of the fascia beams($3,120,000)

3. Remove the excess earth fill and place CFRP sheets on the bottom flanges of all the beams along with the exterior webs of the fascia beams($3,220,000)
4. Remove the excess earth fill and replace the fascia beam area (first four beams)($2,682,000)
5. Do not remove the excess earth fill, replace the fascia beam area, and place CFRP sheets on all remaining beams' bottom flanges($4,760,000)
6. Remove the excess earth fill, replace the fascia beam area, and place CFRP sheets on all remaining beams' bottom flange($4,865,000)
7. Remove and replace the first four beams and parapet wall on all three structures, place CFRP sheets on the bottom flange of all cracked interior beams, epoxy inject all bottom flange cracks, and place urethane foam in the beam gaps of all cracked beams($2,380,000)
8. Remove and replace the first four beams and parapet wall on all three structures as well as fifty-six interior beams on Z03, place CFRP sheets on the bottom flange of all cracked interior beams, epoxy inject all bottom flange cracks, and place urethane foam in the beam gaps of all cracked beams($6,900,000)

Option number 8 was decided upon for the repair of the plaza structures. This option although the most expensive, $6,900,000, will provide the largest factor of safety and greatly extend the service life of the plaza structures.

As part of the repair contract, the contractor was required to salvage and load test four of the replaced prestressed box beams to shear failure. The load test results were used to verify the adequacy of the remaining prestressed box beams. All of the box beams carried more than the expected load except for beam number 166 (span 1) of Z03 of 63102. The ultimate proof load at d_v for this beam was less than the actual factored dead load plus factored pedestrian loading at d_v according to AASHTO LRFD. This low strength was attributed to the obvious cold joint and a layer of Styrofoam between the web and bottom flange. Due to the obvious fabrication errors for beam 166 from Z03 of 63102 beam 27 from Z03 of 63102 was salvaged and load tested. For beam number 27, the ultimate proof load at d_v was greater than the actual factored dead load plus factored pedestrian loading at d_v. The remainder of the load tests demonstrated that the remaining beams have adequate shear strength to carry the applied loads.

With the completion of the load testing, all five salvaged beams were dissected to determine their physical properties, i.e., concrete compressive strength; steel strength; stirrup spacing, size, and lap length; web thickness; number of prestressing strands; and number of debonded prestressing strands. From this analysis, we discovered that all of the physical properties of the beams exceeded the specified design strengths. We also discovered that the lap length for the WWF on the three Z03 beams, about 11 inches, exceeded the required length of 7-7/8 inches. One point of interest is the various web thicknesses. It appears as thought the Styrofoam used to create the hollow portion in some of the box beam had either shifted during the placement of the concrete or it was never centered from the beginning. Another interesting finding was that for the Z01, beam number 3, beam the outside columns of three prestressing strands each side are not enclosed within the stirrups as specified in the contract plans.

References

1. *American Association of State Highway and Transportation Officials LRFD Bridge Design Specifications*, 2nd ed., Washington, D.C., 1998

2. American Concrete Institute, *ACI Building Code Requirements for Structural Concrete* (ACI 318-95) and Commentary (ACI 318R-95).

3. *American Association of State Highway Transportation Officials, Standard Specifications for Highway Bridges*, 16th ed., Washington, D.C., 1996

4. Nilson, Arthur H., *Design of Prestressed Concrete*, John Wiley & Sons, New York,1978.

5. *Canadian Highway Bridge Design Code*, final draft, 1996.

BIBLIOGRAPHY

Cumming, David A., Catherine E. French, and Carol K. Shield. Shear Capacity of High-Strength Concrete Prestressed Girders, Minnesota: University of Minnesota Department of Civil Engineering, May 1998.

Shear Behavior of Full-Scale Prestressed Concrete Girders, PCI Journal, May-June 1996, p.48 thru 62.

Shahawy, Mohsen A., Batchelor, Barrington de V, Shear Behavior of Full-Scale Prestressed Concrete Girders: Comparison Between AASHTO Specifications and LRFD Code * PCI Journal, May-June 1997, p.72 thru 93.

Clancy, Chad M. Kulicki, John M., Eshenaur, Scott R. Thomas, Andrew L., Review of Shear Behavior of Full-Scale Prestressed Concrete Girders: Comparison Between AASHTO Specifications and LRFD Code: Readers Comment Article, PCI Journal, May-June 1997 pp.72-93.

Vecchio, Frank J., and Collins, Michael P., The Modified Compression - Field Theory for Reinforced Concrete Elements to Subjected to Shear, ACI Journal, Technical Paper Title No. 83-22, ACI Journal, Technical Paper Title No. 83-22, ACI Journal March-April 1986.

Hsu, Thomas T. C., ACI Shear and Torsion Provisions for Prestressed Hollow Girders, ACI Structural Journal Technical Paper, Title No. 94-S72, ACI Structural Journal, November-December 1997, pp. 787-799.

Rahal, Khaldoun N., Collins, Michael P., Analysis of Sections Subjected to Combined Shear and Torsion - A Theoretical Model, ACI Structural Journal, Technical Paper Title NO. 92-S44, ACI Structural Journal, July-August 1995, pp.459 - 469.

Rahal, Khaldoun N., and Collins, Michael P., Effect of Thickness of Concrete Cover on Shear-Torsion Interaction-An Experimental Investigation, ACI Structural Journal, Technical Paper Title No. 92-S32, ACI Structural Journal, May-June 1995, pp. 334-342.

MacGregor, J. G., and Ghoneim, M. G., Design for Torsion, ACI Structural Journal Technical Paper Title No. 92-S20, Code Background Paper: ACI Structural Journal March-April 1995 pp.211-218.

Rahal, Khaldoun, and Collins, Michael P., Analysis of Sections Subjected to Combined Shear and Torsion - A Theoretical Model, ACI Structural Journal, Technical Paper Title No. 92-S44, ACI Structural Journal July - August 1995, pp. 459-469.

Rahal, Khaldoun N., and Collins, Michael P., Effect of Thickness of Concrete Cover on Shear-Torsion Interaction - An Experimental Investigation, ACI Structural Journal, Technical Paper Title No. 92S32, ACI Structural Journal May-June 1995, pp 334-342.

Figures

Earth Fill (exceeding maximum depth)
Removed and CFRP Sheets Placed on Fasica Beam Area Only

CFRP Sheets	$200,000
Epoxy Injection (repairing structural crack)	$280,000
Expandable Urethane Foam	$180,000
Removal of Earth Fill	$ 80,000
Traffic Control (3 bridges x $25,000)	$ 75,000
Subtotal	*$815,000*
Mobilization (estimate 10%)	$ 85,000
Contingencies (estimate 15%)	$135,000
Total Cost	**$1,035,000**

Earth Fill
Not Removed and CFRP Sheets Placed on Every Beam

CFRP Sheets	$1,850,000
Epoxy Injection (repairing structural crack)	$280,000
Expandable Urethane Foam	$180,000
Traffic Control (3 bridges x $50,000)	$150,000
Subtotal	$*2,460,000*
Mobilization (estimate 10%)	$250,000
Contingencies (estimate 15%)	$410,000
Total Cost	**$3,120,000**

Figure 20 - Cost Estimate Based on Recommendations
Includes work for all 3 bridges (Z01, Z02, Z03 of 63102)

Appendix A

Z01 of 63102

Although the inspection revolved around the first four post-tensioned box beams, periodic inspections on the interior beams were performed. The first four box beams were post-tensioned together to carry the weight of the 10-foot tall concrete barrier.

SPAN 1 OVER WESTBOUND I-696

During our June 9, 1998, inspection, we found the following:

West side-

1. The water outlined fascia beam shear cracks ranged in size from 0.003 in. to 0.005 in.
2. There were no cracks present on the bottom flanges of the fascia, first, second, or third interior beams.
3. No flexural cracks were visible on the fascia beam.
4. There were some shear cracks that extended into the parapet wall.
5. Even though the pier end of the beam had more shear cracks than the abutment end, leachate was hanging from the abutment end and not the pier end.
6. One shear crack was present on the inside face of the third interior.
7. At the pier end of the beams, all weep holes were plugged and the third interior beam did not have a weep hole. In order for us to unplug the holes we needed to use a hammer drill. After drilling into the fascia, first and second beams we found that the Styrofoam was dry. We drilled 8-½ inches through the bottom flange of the second interior before we hit the interior of the beam. No attempt was made to drill through the third interior beam. We were concerned with cutting prestressing strands.

East side-

1. Along with the flexure cracks, the fascia beam also had shear cracks that ranged in size from 0.005 in. to 0.010 in. However, the majority of the crack widths were 0.005 in.
2. At the pier end of the fascia beam, some of the shear cracks had propagated into the parapet wall.
3. Water outlined most of the cracks at both the pier and abutment ends of the fascia beam.
4. Comparing the pier end with the abutment end of the fascia beam, the spacing of the shear cracks at the abutment end were spaced closer and appeared wider than at the pier end.
5. Moving on to the interior beams, one crack, about 0.003 in., was found on the bottom flange of the second interior beam.
6. The inside web of the third interior beam had a shear crack that has started to wrap around the beam into the bottom flange.
7. Most of the grout placed between the fascia, first, second, and third interior beams, was in good condition. The exception to this was the grout at the pier end between the fascia and first interior beams, which appeared black with efflorescence stains.
8. All of the weep holes on these four beams were plugged. After opening them, we found that only the interior of the fascia beam was wet. The remaining three beams were dry.

9. From the east fascia, beam numbers 24-27 and 32-37 all had shear cracks roughly 0.003 in. wide.

SPAN 2 OVER EASTBOUND I-696

During our May 19, 1998, inspection, we found the following:

West side-

1. The fascia web had lots of surface hairline alligator cracking and diagonal cracks generally about 0.003 in. in size.
2. The grout between the fascia, first, second, and third interior beams was in good condition.
3. No cracks were present on the inside edge of third interior beam.
4. There were no transverse cracks present on the bottom flanges of the first four beams.
5. There was one longitudinal crack along bottom of fascia beam.
6. The weep holes located near the pier in the fascia and fourth interior beams were pre-drilled with a 3/4 -inch diameter hole. The weep hole in the fascia beam is about 1 ½ inches deep and the hole in the fourth interior beam is about 2-½ inches. Neither of these weep holes penetrate the beam cavity since the bottom flange thickness is 6 inches. Note that no prestressing strands were cut in either case.

East side-

1. The fascia web had leachate extruding from the 0.007 - 0.009 in. diagonal cracks.
2. The grout between fascia and first interior beam was darker than the grout between the adjacent beams.
3. When compared to the west side, the east fascia beams were in worse condition.
4. We found one diagonal crack in the inside web of the third interior.
5. No cracks were found on the bottom flanges of the first four beams.
6. The weep holes at the pier end of the fascia, first and second interior beams, were plugged. The weep hole inserts were installed correctly, but never unplugged. As we unplugged the fascia weep hole, water poured from the beam. The cavities of the first interior along with the third interior were dry. We were unable to unplug the second interior weep hole.

TOP INSPECTION-

The top of the structure was inspected on May 28, 1998. During this inspection, a conversation with the city of Southfield park maintenance revealed that the sprinkler heads located on this structure are frequently broken by vandals. The system runs at night for ½ hour cycles using 3 gallon/minute nozzles. The fourteen heads in the parapet wall areas run Monday, Wednesday, and Friday. I was informed that last year the zone above the east fascia failed to shut off and the system ran all night. The entire wood chip area was flooded. The north manhole drain grate was plugged with wood chips so all of the water stayed above the beams until he unplugged the grate the next morning. At the time of the inspection, three of the eight sprinkler heads that line the east parapet wall were broken by vandals. All three broken heads were in span 2. With the unit weight of water at 62.4 pcf this adds significant weight. In addition to flooding, the water has added significant weight to the soil itself. At the time of our inspection, there had not been rain in the area for at least a week. However, the soil had a moisture content of 22.6 percent. This high moisture content is entrapped in the clayey sand that was used for the earth fill. The saturated soil increases the unit weight of soil from 120 pcf to about 140 pcf.

Part of the top inspection involved determining how much earth fill was present. In order for us to determine the fill, we dug three holes along the west parapet wall (south abutment, pier, and north abutment) and four holes along the east parapet wall (south abutment, south of the pier, north of the pier, and north abutment). From each location the following was revealed.

West Parapet

S. abutment - 3 feet 3 inches of damp sand fill
Pier - about 4 feet of damp sand fill
N. abutment - 4 feet 4 inches dry fill.

East Parapet

S. abutment - 3 feet 3 inches dry sand
S. of the pier- 4 feet 4 inches depth (bottom 12 inches was saturated sand)
N. of the pier- 4 feet 7 inches depth (bottom 12 inches was saturated clay)
N. abutment - 3 feet 3 inches depth (bottom 2 feet 3 inches saturated clay)

Visually, the soil elevation along the east parapet wall appears to be about 1 foot below the park elevation. If this is true, then the run off water would drain to the east parapet wall then off to the abutments. During our inspection we extracted some of the wet earth fill to bring back to the laboratory for analysis. From this sample we learned that the natural moisture, percent by weight, was 17.6 percent. Through a visual inspection performed by Dan Troia, MDOT Soils Engineer, it was suggested that the unit weight of the soil was closer to 140 pcf than the 120 pcf stated in the design notes.

Z02 of 63102

Although the inspection revolved around the first four post-tensioned box beams, periodic inspections on the interior beams were performed. The first four box beams were post-tensioned together to carry the weight of the 10-foot tall concrete barrier.

SPAN 1 OVER WESTBOUND I-696

During our June 9, 1998, inspection, we found the following:

West side-

1. The fascia web cracks were generally 0.009 in. wide.
2. One of the numerous outside fascia beam web cracks was 0.025 in. wide. Most of these cracks were very long and pronounced with water at both the pier and abutment end.
3. A few cracks were also present in the parapet wall.
4. Even though there were no visible flexural cracks, this fascia beam had the most shear cracks when compared with all of the fascia beams inspected.
5. Grout was present between the fascia, first, second, and third interior beams. Most of it was in good condition, however, the grout between the fascia and first interior was cracked at the pier end.

6. Although at least one of the two weep holes per beam end (pier end) were plugged, the interior of the beam appeared dry.
7. There were no cracks present on the inside web of the third interior beam.

East side-

1. The 0.002 in. wide fascia web cracks were water stained but were not wet.
2. However, the cracks at the abutment end of the fascia beam were outlined with water.
3. No cracks were found on the bottom flanges of the fascia, first, second, or third interior beams.
4. Also, no cracks were present on the inside web of the third interior beam.
5. Some of the weep holes in the pier end of the first four beams were plugged, however, the interior of the beams appeared dry.
6. Some flexure cracks were present in the fascia beam. Also some shear cracks were present in the retaining wall.
7. The grout between the fascia, first, second, and third interior beam was in good condition.
8. All four post-tensioning ducts had water staining the grout pockets.
9. The interior beams on the east side were in good condition.

SPAN 2 OVER EASTBOUND I-696

During our May 19, 1998, inspection, we found the following:

West side-

1. The 0.009 - 0.016 in. diagonal fascia web cracks were wet with no leachate present.
2. One of the diagonal cracks on the fascia beam extended about 4 feet in the parapet wall.
3. The fascia beam web was also severely alligator cracked.
4. When compared with the surrounding keyway grout, the grout between the fascia and first interior was darker and had more spalls.
5. The beams on this structure had two weep holes per location instead of the single weep hole as Z01.
6. Probing the pier end weep holes in fascia, first, second, and third interior beams revealed that the insides of the beams were dry. However, due to the water stains surrounding the holes it appears that water does drain through the beams.
7. The inside web of third interior was in good condition (no cracks).
8. There were no diagonal cracks in the bottom flanges.
9. Two longitudinal cracks were present on the bottom flange of the third interior beam.
10. The grout between second and third interior looks like it might fall.

East side-

1. At the time of our inspection, the 0.005 in. diagonal fascia web cracks were wet and without leachate.
2. When compared to the west fascia, the east fascia was in better condition.
3. The keyway grout between the first four beams was in good condition.
4. All weep holes at the pier end of the first four beams were dry with efflorescence on the outside surface.
5. No cracks were found in bottom flanges or on the inside edge of the third interior beam.

TOP INSPECTION-

The top of the structure was inspected on June 9, 1998. From this inspection we discovered that a sprinkler system is present, however, no sprinkler heads are present directly in front of the parapet wall. They are placed inward from the wall and spray toward it. There is also a valley between the wall and the sidewalk. It appears as though water drains through the valley toward the parapet wall then off to the abutments.

We arrived at 9:00 a.m. for our inspection. At this time the valley between the sidewalk and west parapet was not saturated. It was just wet. However, the valley near the east parapet wall had standing water, even though that night there had been no rain. The only source of water was the sprinkler system. Numerous sprinkler heads were found between the sidewalk and the parapet wall.

One interesting point, two of the original twenty trees that lined the west parapet wall were cut down. As for the pine trees that lined the east parapet wall, seven of the original seventeen were cut down and four of those seven were located above the pier.

Part of the top inspection involved determining how much earth fill was present. In order for us to determine the fill, we dug three holes along the west parapet wall (south abutment, pier, and north abutment) and five holes along the west parapet wall (south abutment, pier (three holes), and north abutment). From each location the following was revealed.

West Parapet

S. abutment - 6 feet 0 inches of damp sand fill - the last 3 inches was saturated clay
Pier - 4 feet 0 inches of saturated sand
N. abutment - 3 feet 6 inches of fill - the last 1 feet 10 inches was moist clay

East Parapet

S. abutment - 4 feet 0 inches of sand fill - the last 1 foot was wet sand
Pier - 4 feet 0 inches of sand fill - the last 2-½ feet was wet sand
10 feet from the parapet wall above the pier - at 3 feet 0 inches we encountered unmovable moist clay *
15 feet from the parapet wall above the pier - 3 feet 10 inches of fill - 3 feet 2 inches of saturated clay and the last 8inch was saturated sand *
N. abutment - 3 feet 6 inches of moist clay
*After two hours, no water had leached in the holes.

During our inspection we extracted some of the wet earth fill to bring back to the laboratory for analysis. From this sample we learned that the natural moisture, percent by weight, was 22.6 percent. Through a visual inspection performed by Dan Troia, MDOT Soils Engineer, it was suggested that the unit weight of the soil was closer to 140 pcf than the 120 pcf stated in the design notes.

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Although the inspection revolved around the first three box beams, periodic inspections on the interior beams were performed. The first three beams were closely inspected because the fascia beam carries the entire weight of the parapet wall and the second interior beam resists the torsional forces created by the earth fill against the parapet wall. The torsional forces are carried through a turnbuckle assembly that is attached to the parapet wall and the second interior beam. This was the first of the three plaza structures to be constructed.

SPAN 1 OVER WESTBOUND I-696

During our June 9, 1998, inspection, we found the following:

West Fascia-

1. The dry fascia web cracks ranged in size from 0.003 to 0.005 in.
2. Only one crack was found on the bottom flange of the fascia beam. This crack was roughly 0.005 in. wide.
3. No cracks were found on the inside web of the fascia beam. This beam was in great condition when compared with the other fascia beams. It was noticeably better than the east fascia.
4. No cracks were found on the first, second, and third interior beams.
5. The weep holes on the pier end of the fascia beam were plugged.
6. The weep holes on the pier end of the first, second, and third interior beams were all open. The interior of these beams appeared dry.
7. The abutment end of the fascia beam has about the same amount of cracking as the pier end.
8. Four ducts were present for the longitudinal tie rods.
9. No flexure cracks were present on the fascia beam.
   Beam number 54 from the west fascia had a collection of water on the midspan of the bottom flange. The 27^{th} beam from the west fascia did not have cracks. This beam was inspected because the 27^{th} beam from the east fascia in the eastbound half had very large cracks.

East side-

1. The width of the water outlined fascia web shear cracks were mainly 0.005 to 0.009 in.
2. The fascia beam end near the abutment had more cracks than the pier end.
3. Longitudinally across the fascia beam a crack that appeared to outline the bottom flange was visible.
4. No cracks were found on the inside web nor on the bottom flange of the fascia beam.
5. Shear cracks were found on both webs of the first interior and second interior beams.
6. No flexure cracks were visible on the fascia beam.
7. The fascia beam had fewer cracks than the interior beams on this structure.
8. A few shear cracks were present in the barrier wall.
9. From the interior bottom flanges, stalactites have formed from the leachate.
10. Each beam end has two weep holes.

11. All of the first four beam ends located near the pier were dry, however, two of the plastic drain caps were still in place.
12. The beam spacing between the fascia and first interior was 1-3/4 inches this seemed large compared to the adjacent beam spacing of 7/8-inch to 1 inch.
13. From the east fascia beam numbers 4-11, 13, 15-19, and 32-35 all have cracks through the bottom flange in the location of the weep holes. Beams 32-35 have very large shears cracks about 0.040 in. in width. Each of these beam ends have numerous cracks. The shear cracks in the web are offset. Although these beams are in far worse shape than the fascia beams, there is no evidence of water.

SPAN 2 OVER EASTBOUND I-696

During our May 19, 1998, inspection, we found the following:

West side-

1. The diagonal fascia web cracks were mainly 0.007 in. in width.
2. No cracks were present on inside web of fascia beam.
3. The first interior beam had diagonal cracks along both webs along with two longitudinal cracks on the bottom flange. The flange cracks appeared to have originated from the weep holes.
4. The second interior beam had no cracks on the bottom flange nor on the outside face of the web.
5. The interior portion of the fascia, first and second interior beams, were dry at the weep holes locations.
6. At least twelve interior beams have shear cracks at the beam ends. From the west fascia beams number 6, 8, 9, 11, 15, 18, 19, 20, 24, 25, and 27 have transverse cracks in the bottom flanges. Beam number 27 had a shear crack width larger than 0.060 in.
7. On top of the structure, there is a roadway approximately 160 feet to 195 feet from the west fascia (Church Road). This road looks recently re-paved. It is posted no trucks and is very busy with cars. Also, there is a large parking lot about 230 feet from the west fascia. From the plans, beam number 27 is approximately 115 feet from the west fascia. The measured dimensioning for the roadway may be slightly off because we were unsure of the location of the pier. We guessed at the pier location due to a diagonal crack in the west side walk that we believed was the approximate location of the pier.
8. It appears that elevation of Church Road is lower than that of the fill around the west parapet wall.

East side-

1. The water stained, diagonal fascia web cracks were mainly 0.005 in. wide.
2. There were no cracks on the inside face of the fascia beam.
3. The grout between the fascia beams of span 1 and span 2 was saturated.
4. There were no cracks present on the bottom flanges of the fascia, first and second interior beams.
5. No web cracks were present on the first interior or second interior beams.

TOP INSPECTION-

The top of the structure was inspected on May 28, 1998. As part of the inspection, we wanted to determine the amount of fill present. Therefore, we dug three holes along the west and east parapet wall (south abutment, pier, and north abutment). From each location the following was revealed.

West Parapet

S. abutment -	3 feet 8 inches depth (the top 3 feet was very dry and hard and the bottom 6 inches was wet granular material)
Pier -	3 feet 0 inches to large aggregate - we were unable to remove the aggregate (the fill and the aggregate were dry)
N. abutment -	2 feet 0 inches to large aggregate - we were unable to remove to top of beam (fill and aggregate were very dry)

East Parapet

S. abutment -	5 feet 5 inches depth (about 8 inches of wet open graded material past the geotextile blanket)
Pier -	5 feet 2 inches depth (about 8 inches of wet open graded material past the geotextile blanket)
N. abutment -	4 feet 4 inches depth (bottom of hole was moist).

Note: Holes stopped at geotextile blanket, which is on top of open graded drainage course.

The area around the west parapet wall is lined with pine trees. No trees are present along the east parapet wall. Also a sprinkler system is present, however, it appeared as though there are no sprinkler heads that spray water near the parapet walls.