HOAN BRIDGE IN-DEPTH INSPECTION PRELIMINARY SUMMARY REPORT

Structure B-40-400
Units 1A/1B through S24

Wisconsin DOT Project ID 1300-00-05

Prepared for

WISCONSIN DEPARTMENT OF TRANSPORTATION
DIVISION OF TRANSPORTATION SYSTEMS DEVELOPMENT
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Hoan Bridge In-Depth Inspection
Preliminary Summary Report

Wisconsin DOT Project ID 1300-00-05
GRAEF Project No. 2010-0333.02

Prepared for: Wisconsin Department of Transportation
Division of Transportation Systems Development
Southeast Region

Site Address: Hoan Bridge
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1. INTRODUCTION AND BACKGROUND

In accordance with the Federal Highway Administration (FHWA) mandate, the 72-month cycle In-Depth Inspection of the Hoan Bridge was performed between July 26 and November 4, 2010, with additional site visits made to inspect the deck top side in December, 2010. Also per FHWA mandate, a 24-month Fracture Critical Inspection was performed on the main span tie girders. This Preliminary Summary Report outlines the main findings and structural concerns of this 52 unit bridge. Language in this report is general in nature, as a detailed In-Depth Inspection report for each unit will be submitted to the WisDOT in the spring of 2011.

It is the State of Wisconsin Department of Transportation's recommendation that a Fracture Critical Inspection with nondestructive testing (NDT) be performed every 72-months on the main span tie girders. WisDOT has been performing the NDT on the Hoan Bridge's critical fracture critical details every two years. Though planned, the NDT portion of this inspection has not yet been performed at the tie girder/arch intersections. The NDT will be conducted in the winter and early spring of 2011.

The approximately 10, 200-ft (1.9-miles) long Hoan Bridge was opened to traffic in 1978. The main span of this structure is a continuous tied arch having a clear span of 600-ft and providing a vertical clearance of 120-ft over the Milwaukee River.

Several series of repair efforts were performed since the bridge's opening to maintain the structure.

- 1978 – eight bolted splices were installed over electroslag butt welds in the tie girder flanges found to contain weld flaws. Tie girder lift lugs were removed or their welds peened, and access doors were installed at the tie girder ends.
- 1986 - embedded downspouts within the piers were plugged and a surface mounted drainage system was installed.
- 1987 to 1989 - an overcoating of blue-color, barium metaborate paint was applied to the steel superstructure members.
- 1992 - the concrete deck was patched and a 2-inch thick asphalt overlay with waterproof membrane was placed on the concrete surface.
- 1994 – plastic netting was installed on the deck underside of the bridge units over the Maier Festival grounds
- 1996 - steel repairs were made to address issues discovered during the 1992/1993 In-Depth Inspection. Repairs include removing tie girder intersecting welds, drilling mouseholes at the arch/tie girder intersect points, grinding butt welds flush, adding bracing at transfer girders, removing backer

bar butt welds, and recaulking bolted field splices. The interior of the tie girders were painted white, and concrete surfaces repairs were made to several piers.

- 2000/2001 – the north span of unit S2A failed due to "brittle fracture" of 2 of the 3 girders. Subsequent repairs included replacement of the failed span, and lower lateral bracing system removal on all of the girder/floor beam/stringer approach units.
- 2001 or 2002 – hanger cables were cleaned and coated with the proprietary product Metalcoat
- 2010 – deck repairs were made at several units south of the Milwaukee Metropolitan Sewerage District
- 2010 - plastic netting was installed on the deck underside of the bridge units over the.Milwaukee Metropolitan Sewerage District

2. FINDINGS

The following sections outline general conditions of the bridge's steel framing systems, reinforced concrete deck, and reinforced concrete substructure units (piers and abutments). Standard terms used to describe steel corrosion are, in order of increasing severity: blush rust, freckle rust, surface rust, scaling rust, laminate rust, rust pits, and through thickness section loss. Photos referenced throughout the report are located in the Appendix.

This report is based on the conditions of the structure that were readily observable at the time of assessment. The nondestructive testing was only performed at specific areas of concern. Our assessment was intended to be an assessment of the visible elements of the structure from areas accessible as described throughout the report.

This report is intended to inventory existing conditions of the observed areas and to provide general recommendations for repair. Any recommendations provided in this report are conceptual in nature and are not intended for construction. Conditions observed on the date of assessment may change if noted deficiencies are not corrected.

2.1. Main Span Steel Framing

The main span of the Hoan Bridge crosses the Milwaukee River, and is comprised of Units 1A (northbound) and 1B (southbound). The main span steel framing consists of the floor system, the east and west tie girders, the east and west arches, and hanger cables.

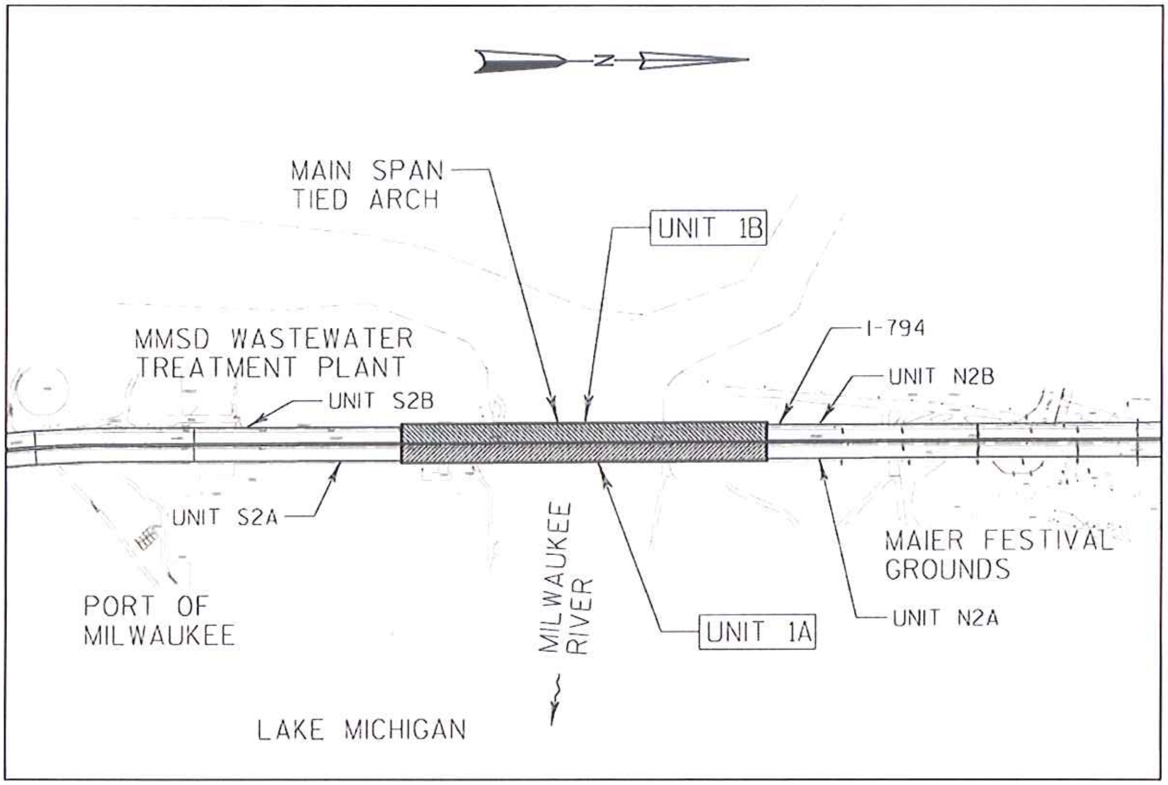

Figure 1: Main span location plan

2.1.1. Floor System

The main span floor system consists of 16 lines of continuous hot rolled noncomposite stringers bearing on floor beams, 26 plate girder floor beams spanning between the tie girders, and lateral bracing trusses which stabilize the floor beams. Peeling of the top paint coat is common on the steel members, however, the middle and base coats are typically intact and are protecting the base metal. Exceptions occur where the paint system has locally failed due to water infiltration at deck cracks or expansion joints.

Stringers - The stringers are generally in good condition. Random blush rust is common on the bottom flanges. Local areas of surface or scaling rust exist where the paint system has failed, resulting in less than 5% section loss. These areas include the fascia stringers adjacent to the tie girders, and top flanges under leaking deck cracks (Photo 1). Scaling and laminate rust is also present at the shiplaps located under the deck compression joints. The most severe shiplap corrosion occurs at the fascia and median stringers where approximately 10% section loss exists (Photo 2). No problems were noted at the stringer diaphragms except for the scaling rust and laminate rust present at the shiplap diaphragms. Approximately 10% section loss exists at these elements.

Floor Beams - The floor beams are generally in good to fair condition. Local blush rust is common on the top and bottom flanges. Paint system failure and flange scaling rust is present in random places with section loss estimates being less than 5% (Photo 3). Localized laminate rust has formed primarily on the floor beam webs and bottom flanges near the tie girder connections. These typically are found at the bottom of web stiffeners and stringer bearing stiffeners and have caused less than 5% section loss (Photo 4). Bottom flange rust pits, caused by ponded water trapped by the bottom flange/stiffener/web intersection, was observed at some of these locations. Maximum rust pit depths were estimated to be ¼-inch. Surface rust is common on the floor beam web's transverse and bearing stiffeners adjacent to the tie girders, and have produced less than 5% section loss on these elements. Some of the floor beam to tie girder connection bolts have surface rust, primarily affecting bolts attaching the floor beam bottom flange to the gusset plate.

Panel points 8 and 8' are locations where the floor beam bottom flange is a steel box element that also serves as a tie between the arches. Access to the interior of these box bottom flanges is provided by midspan hatches. The bottom flange's exterior condition is generally good with blush rust occurring at localized areas on the top surface (Photo 5). This blush rust is likely caused by ponded water trapped by the floor beam bottom flange/stiffener/web intersection. Surface rust, which has caused less than 5% section loss, is also present on the top and side surfaces at the tie girder connection points (Photo 6).

Both bottom flange boxes were entered to observe their interior conditions. Light rust stains were observed at the field splices of the north box along panel point 8'. The interior of this box was dry at the time of inspection. The south box along panel point 8 was wet inside due to condensation. This condensation is causing surface rust to form at the field splices and along the bottom plate (Photo 7).Section loss of the box element due to the surface rust is less than 5%. Rust stains have also formed from behind the splice plate bolt heads. Pigeon waste is present on the bottom plate due to the access hatch being left open at some time in the past. Closing the hatch from the catwalk at panel point 8 was difficult and posed safety concerns due to the poor accessibility in reaching the latch combined with its weight.

Lateral Bracing Trusses – The lateral bracing trusses are of welded construction using channel shape chords and WT webs. The top and bottom chords of most braces are covered with blush or surface rust and have produced less than 5% section loss to these members (Photo 8). Local paint system failures are also common. These paint failures result in scaling rust or laminate rust and are primarily located at the top and bottom chord gusset plate connections resulting in approximately 10% section loss. Approximately half of these connections, located at the tie girders and floor beam flanges, are affected by this heavy corrosion. At many connections, pack rust has formed between the gusset plate and chords, bending the chord channel flanges (Photo 9). It is estimated that connections having pack rust have caused about 15% to 20% section loss of the chord cross section area.

The diagonal web members are typically in good condition, with only a few exhibiting surface or blush rust. One member was found to be significantly bent (Photo 10). It is likely that this condition has existed since the bridge was first erected.

2.1.2. Tie Girders

The continuous tie girders are welded box sections 4-feet wide by 14-feet deep. Main span floor beams frame into the side of each tie girder at approximately 44-feet centers. At each floor beam panel point, cable hangers support the tie girder from the upper arch where the arch projects above the roadway. Columns support the tie girder from below to the lower arch where arch segments are located below the roadway.

Exterior - On the tie girder's top flange exterior surfaces, freckle rust is prevalent. Local paint failure has lead to corrosion of the top flange field splice plates, bolt heads, and cable covers (Photo 11). Paint had been removed at the top flange electroslag welds for nondestructive testing during previous In-Depth Inspections. Though these were repainted at that time, surface rust has formed causing less than 5% section loss. Conditions on the bottom flange are good with only random areas exhibiting spot rust.

On the tie girder webs, laminate rust and rust pitting exist at about half of the floor beam/lateral brace gusset plates. Specifically, the corrosion occurs in the gap between the tie girder web and end of the floor beam bottom flange has produced less than 5% section loss of the web (Photo 12). The gap is only about an inch wide, and tends to slow the evaporation rate of moisture on these surfaces. The remaining web exterior surfaces exhibit surface rust primarily at the field splices. Freckle rust appears at random locations only. A dent on the east tie girder exists between panel points 11 and 12 (Photo 13). Original paint suggests that this dent has likely existed since the bridge was first erected.

The north bearing for the east tie girder is laterally misaligned by about ¼". This condition allows the roller gear stud bolts to rub against the lower pin plate, creating 1/16" wear grooves in the pin plate and a loud squeaking noise under live loads.

Interior - At the tie girder interiors, approximately 50% of the bolted field splices have gaps allowing water to leak through and causing rust stains to occur inside of the box sections (Photo 14). Water enters through the top corners, runs down the web splice plate, and then ponds on the bottom flange or flows down to the bottom flange drain holes. At a few field splice locations, corrosion has advanced such that the paint system has failed, resulting in scaling and laminate rust at the tie girder's bottom corners (Photo 15). This corrosion has caused less than 5% section loss to the tie girder. Fabrication or construction related irregularities noted in previous In-Depth Inspection reports, such as bent longitudinal stiffeners and weld flaws, still exist. No new problems were noted at these bent elements.

In the east tie girder, the lighting was not functioning in several areas. It appeared that a circuit breaker had been tripped since lighting in about every other bay was functioning. The lighting was operational in every bay of the west tie girder.

2.1.3. Arches

The continuous arches are welded box sections 4-feet wide by 5-feet deep. The upper arch located above the roadway supports the tie girder using cable hangers, whereas the lower arch below the roadway receives tie girder loads from above by way of columns. Eighteen transverse box shaped vierendeel struts tie the east and west arches together. Six struts span between the upper arches, and twelve struts (six at the north end and six at the south end) span between the lower arches.

Exterior – Corrosion has formed at localized areas of the arches. Paint system failure and surface rust occur at the base of about half the columns above the lower arches, predominantly on the east arch (Photo 16). This same condition exists at many of the lower arch field splices and access hatches (Photo 17). Less than 5% section loss has occurred at these columns.

On the upper arch, corrosion is less severe. Surface rust is present on the field splice plates and bolts located just above the tie girders, causing less than 5%

section loss to the arch (Photo 18). Small areas of freckle rust exist on the cable covers

Interior – Bolted field splices along the arch and at the arch strut ends allow water to enter the arch interiors. This problem is likely due to failed caulk at the gap between the main members of the field splice. Corrosion has formed where water runs along the arch's lower corners, and at transverse stiffener plates where the water ponds. Severity of the corrosion is generally limited to surface rust and scaling rust, though laminate rust has formed in some locations (Photo 19). Total section loss to the arch is less than 5%. Water infiltration at the field splices is an ongoing problem, as it was observed that these regions were cleaned and spot painted in the past.

Existing mouseholes had been drilled for previous In-Depth Inspections to test for and monitor cracks at the arch/tie girder intersections. Cracks had been discovered at many of these areas during the earlier inspections, and markings written on the members indicate the crack limits when the marks were made. Nondestructive testing techniques (magnetic particle testing and ultrasonic testing) had been used to establish these limits. Visually identifying the cracks around the mouseholes was difficult if not impossible, though visual confirmation was clearly possible at a few (Photo 20). An independent consultant will be performing nondestructive testing of all arch/tie girder intersection details in the winter/spring of 2011.

The interiors of only six vierendeel arch struts were accessible for inspection. On the upper arches, these include the struts at panel points 11 and 11'. On the lower arches, accessible struts are located above piers 1N and 1S (two at each pier). These six struts have access holes cut through the interior diaphragm plates, allowing access from one arch to the other.

At the lower arch transverse struts, access hatches are provided midspan through the top plate of the box. Pack rust has formed between the strut top plate and access hatch edge. This pack rust has compromised the hatch seal and allowed water to enter the strut interior. The moisture has caused scaling rust to form on the bottom box plate below the access hatch. It is estimated that less than 5% section loss has occurred to this plate.

2.1.4. <u>Hanger Cables</u>

The steel "bridge strand" style cables are 2-1/4 inches in diameter and suspend the tie girder from the upper arch at 8 locations. There are 4 cables used at each location. In addition to being galvanized, each cable is coated with a proprietary product to provide further protection against the elements.

There are random local areas of surface rust along the length of the cables exposed to the elements (Photo 21). Approximately 5% to 10% of the surface area is rusted, resulting in less than 5% section loss. Inside of the tie girders, rust staining is present on about 20% to 30% of the cable end fittings. On about two cables, one broken wire was found near the lower anchorage due to corrosion. The broken wire accounts for less than 5% section loss of the cable element. Pack rust was found between the shim plates at approximately 10% of the cable end fittings/anchorage castings (Photo 22). Varying degrees of surface rust were noted at most of the cable covers inside of the tie girders (Photo 23).

One cable at the east arch was found to be vibrating excessively relative to others on a day with wind gusts near 25 mph. This was the southeast cable of panel point 12 of the east arch. The excessive vibrations were a sign that the tension in the cable is different than the neighboring three.

2.1.5. Miscellaneous

The main span catwalks deflect an excessive amount laterally when used. This occurs at both the main catwalk located under the roadway median, and at the secondary catwalks leading to the tie girder access hatches. This is a potential safety issue such that a worker may be thrown off balance should the catwalks resonate. Laminate rust exists near the end anchorage points of the catwalk cable guardrails (Photo 24).

2.2. Girder/Floor Beam/Stringer Approach Span Steel Framing

Units S2A, S3A, S4A, N2A, N3A and N4A comprise the northbound units of the approach spans, and units S2B, S3B, S4B, N2B, N3B and N4B comprise the southbound units of the approach spans. The approach units begin at the north and south ends of the main span units 1A and 1B.

The approach unit steel framing system consists of plate girders, floor beams and stringers. Peeling of the top paint coat is common on the steel members, however, the middle and base coats are typically intact and are protecting the base metal. Exceptions occur where the paint system has locally failed due to water infiltration at deck cracks or expansion joints.

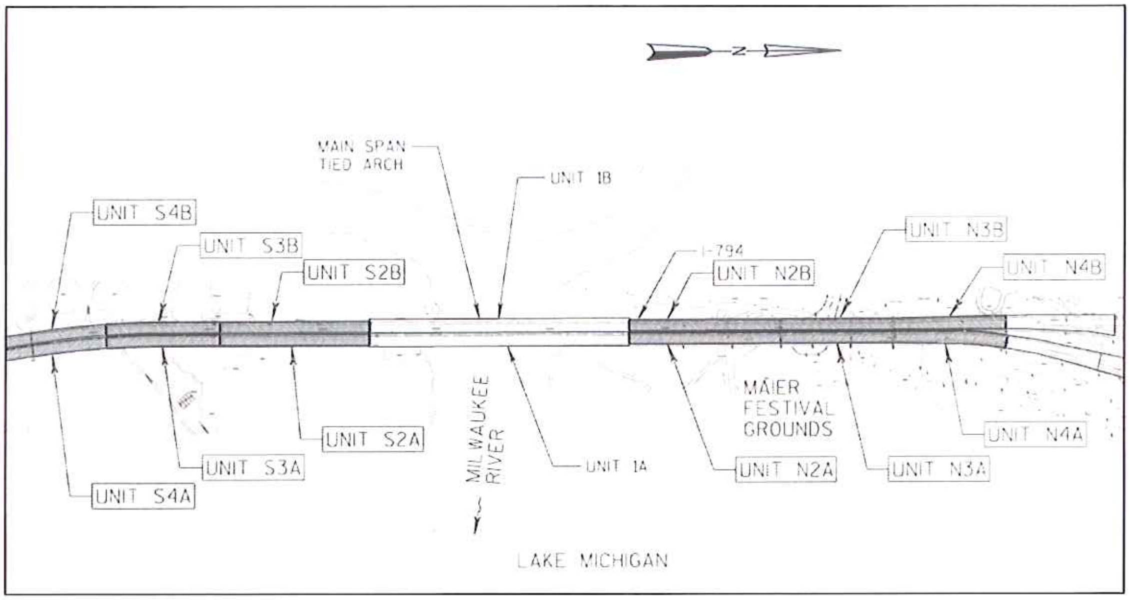

Figure 2: Plate girder/floor beam/stringer approach span units location plan

2.2.1. Girders

The approach span girders are welded steel plate girders, and are typically in good to fair condition. Lower lateral bracing shelf plates were originally welded to the girder webs at the floor beam connection plates, but were removed in 2001 due to associated performance problems. During the 2001 retrofit work, several crack arresting mouseholes were drilled through the girder webs adjacent to most of the tension region lower lateral shelf plate details. The lower lateral shelf plates have all since been removed.

In general the top coat of paint is peeling, and blush rust is common in random areas (Photo 25). The prime and middle coats of the paint system are typically intact, although local failures and scaling rust are present in random locations. This scaling rust has resulted in less than 5% section loss, and is normally located below or adjacent to leaking cracks, spalls or delaminations in the deck. Local paint failure with surface or scaling rust was also observed at locations of previous steel retrofits (mousehole drilling, grinding, magnetic particle testing, etc). These locations were field painted at the time the repairs were made, but the paint in these locations is generally in poor condition (Photo 26). Section loss is less than 5% of the element area.

Scaling and laminate rust is typical at the girder shiplaps below the finger plate joints (Photo 27). Section loss was measureable, approaching 50% in localized areas of some stiffeners. In general, it is estimated that 1/16" of section loss has occurred on all surfaces of the shiplaps. A structural analysis of the ship laps, accounting for the section loss, is warranted to determine the remaining load capacity. Other elements near the shiplap and finger plate joints have scaling rust and laminate rust as well. This corrosion is present on the transfer beams, end diaphragms, bearings and cross frames. Section loss to individual elements of the members is estimated to be 15% to 20%. The presence of scaling rust and laminate rust made visual crack detection very difficult in these areas.

At the shiplap joints the curved surfaces of many of the rocker bearings had rust pits, resulting in higher than designed bearing stresses and pockets for moisture and debris to accumulate (Photo 28). Maximum rust pit depths is estimated to be 1/8". A few rockers appeared excessively tilted for the measured temperature. In general, however, most of the rockers were observed to be moving as designed and were tilted within an acceptable range.

About 50% of the fascia girder bolted field splices are experiencing scaling rust and laminate rust on the bottom flange, on the bottom flange inside splice plate, and at the base of the web (Photo 29). Local rust pitting up to ¼" deep was observed in some locations. Section loss to the bottom flange is less than 5%. On some of the corroded field splices, pack rust is bending the bottom flange splice plates, causing unintended prying forces on the splice bolts (Photo 30). Missing splice bolts were noted in a few locations, but slip between the plates was not observed.

Some fatigue cracks were observed on the girders, and confirmed with NDT (grinding and magnetic particle, Photo 31). The cracks typically appear to be the result of distortion induced fatigue. Girder web out-of-plane bending is occurring at floor beam connection plates as the floor beam ends rotate under live loads. Some of these cracks were ground out and removed during the NDT, but others were too deep for complete removal. All confirmed steel cracks on the bridge were located within the plate girder/floor beam/stringer system approach spans (Units N2-N4 and S2-S4). A separate, detailed NDT report describing the findings is being prepared by a sub-consultant.

Most of the plate girder web cracks have formed at the first or second floor beam connection from the pier, which corresponds to the location of the flange shop splice. Reduced lateral stiffness of the girder in this area may contribute to the problem. Numerous cracks in the paint were observed in similar locations, but nondestructive examination did not reveal any cracks in the steel. This suggests that out-of-plane forces are present and have cracked the paint, but steel stresses have been low enough so as not to have produced fatigue cracks. The potential for fatigue cracks is present, and these areas should be closely monitored in the future.

During earlier retrofit work in 2001, mouseholes were drilled through steel elements to arrest cracks and intersecting welds were ground out in regions of tensile stress. In other areas, bolted splice plates were installed to replace steel area lost due to the formation of large cracks (Photo 32). The strengthened areas were inspected to confirm they are performing as intended. No problems were observed at the previously repaired locations other than paint failure and minor corrosion.

Bent or dented flanges, webs and stiffeners were observed in many locations. These appear to be the result of the original steel erection and have a low probability of developing into significant problems. Wavy bottom flanges were observed near the piers (Photo 33), but symmetry of the flanges in adjacent girders indicates this was likely a fabrication issue.

2.2.2. Floor Beams

The floor beams are hot-rolled steel W sections and are typically in good to fair condition. Blush rust is common on the bottom flanges (Photo 34). The paint system is generally intact, although local areas of failure and scaling rust are present in random locations. These areas are typically located below or adjacent to leaking cracks, spalls and delaminations in the deck. The resulting section loss is estimated to be less than 5% of the element area.

The floor beams are bolted to the girder connection plates. In some locations a small gap between the floor beam web and the connection plate was observed, suggesting that the bolts were not tightened properly during construction or fabrication tolerances were not maintained (Photo 35). In all cases, however, the connection appeared to be performing adequately. Rust caused by the fretting of the metal surfaces was also observed at random locations between the connection plate and floor beam web, indicating relative plate movement (Photo 36).

In some locations the floor beam top flange was coped at the connection plate to allow for easier steel erection. Prior to the repairs in this area, cracks were observed at some of these copes, and the cracks were subsequently ground out (Photo 37). No issues were noted at any of the copes, including those which have been previously repaired.

2.2.3. Stringers

The stringers are generally in good condition. Random blush rust is common on the bottom flanges (Photo 38). Local areas of scaling rust or laminate rust exist where the paint system has failed (Photo 39). These areas are typically located on the top flange, below or adjacent to leaking cracks, spalls and delaminations in the deck. The corrosion is generally limited to lengths of one to two feet, and accounts for less than 5% section loss of the flange element. No problems were noted at the bolted field splices.

Scaling and laminate rust are present at the twelve stringer shiplap ends and bearings below the deck's finger plate joints at units N2A, N2B, S2A, and S2B (Photo 40). Measurable element section loss exists in localized areas with loss estimates being in the range of 15% to 20%. The finger plate joints allow water and deicing chemicals to spill onto the stringers, accelerating deterioration of the paint system and the base metal. Elements affected by this section loss should be analyzed to determine their remaining load carrying capacity.

2.3. Multigirder Approach Span Steel Framing

Twenty-six units comprise the multi-girder approach spans carrying the northbound and southbound mainline and the south end ramps. Units S5A to S7A, S5B to S7B, and S8 through S24 begin at the north end of the Port of Milwaukee property and end at the south end abutments. Units 5A, 6A, and 6B are located at the far north end of the bridge adjacent to the Lake Interchange. Construction of these units, which is similar to that found on typical steel highway overpasses, consists of welded steel plate girders having bolted field splices, diaphragms, and lateral bracing. The mainline units consist of seven lines of girders northbound and seven southbound, with added girders where the structure widens out for the ramps. The ramp units have three lines of girders. Girders are continuous over three piers (four piers support a single unit in one location) with shiplap expansion joints between adjacent units. At the ramp widenings, transfer girders (oriented at right angles to the main girders) support the additional lines of longitudinal girders at the shiplaps.

Beyond areas of rust, the top coat of the paint system is peeling. In general, however, the prime and middle coats are protecting the base metal.

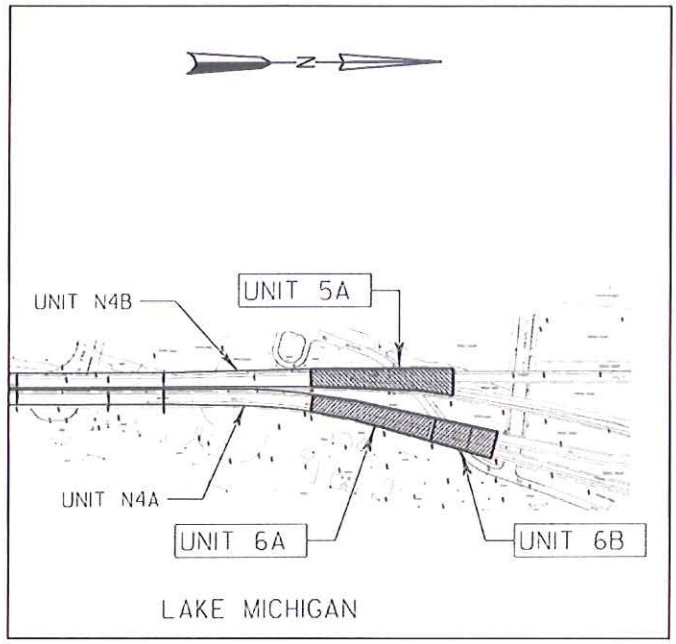

Figure 3: North end multigirder units location plan

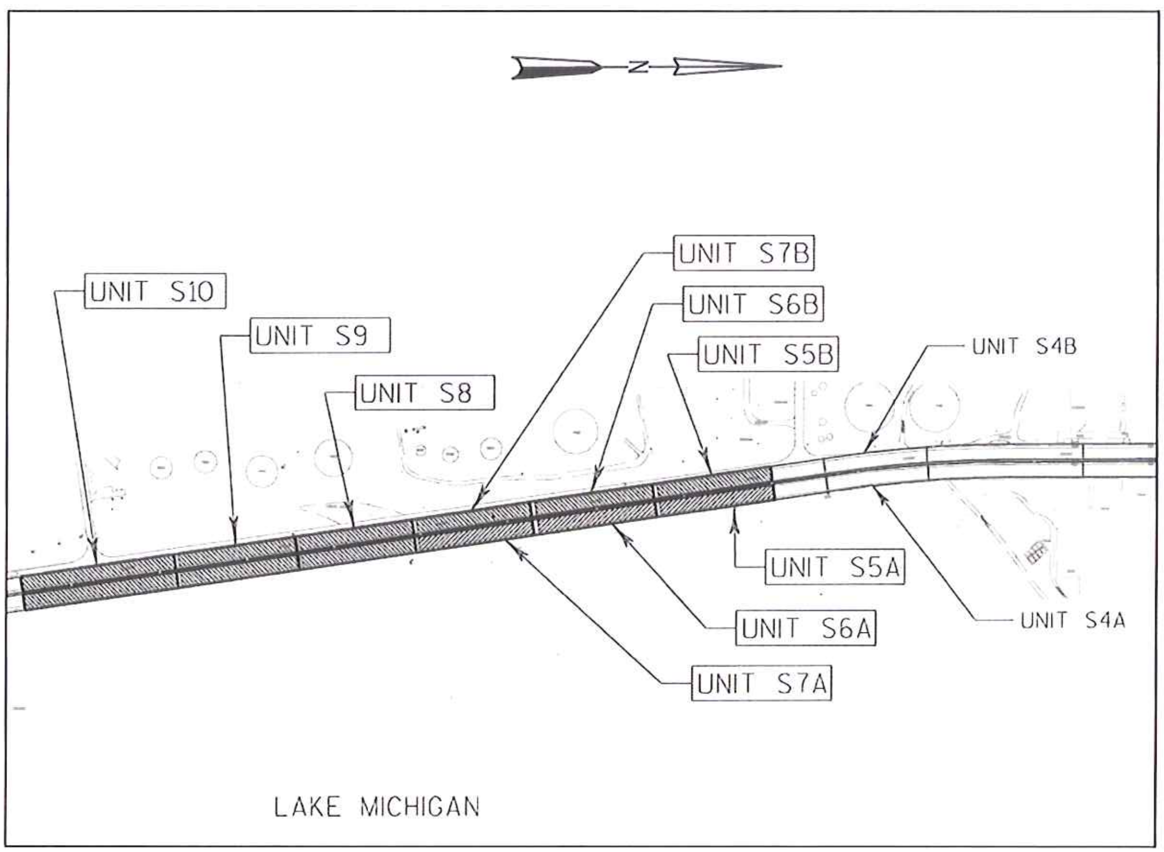

Figure 4: Near south end multigirder units location plan

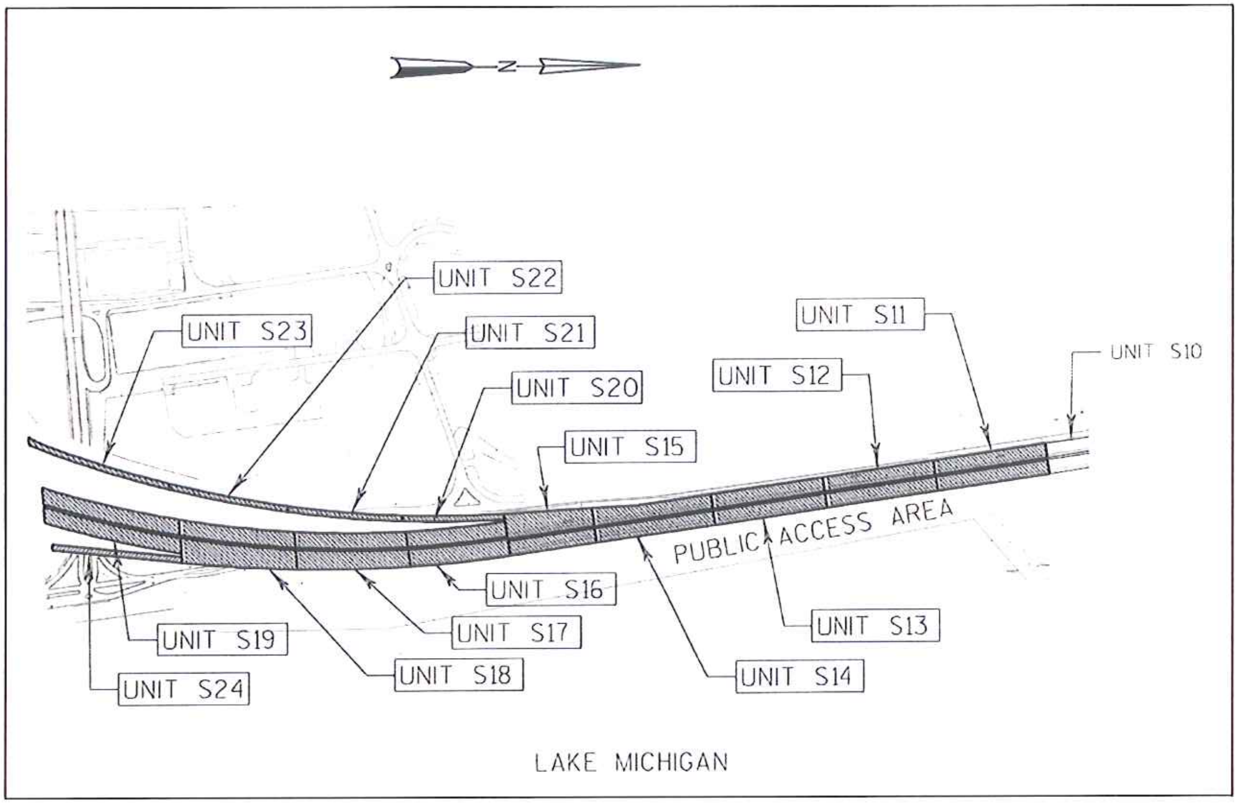

Figure 5: Far south end multigirder units location plan

2.3.1. Girders

The plate girders are the main structural members and are constructed from welded steel plate elements. Generally the girders are in fair to poor condition with deficiencies that include local areas of laminate rust in regions of the top flange, primarily where deck cracks, delaminations, or spalls exist (Photo 41). The laminate rust has produced less than 5% section loss in these regions. The top coat of paint is peeling in most areas of the structure, and blush rust exists on many of the girder bottom flanges. Loose bolts on the bottom flange field splices and lateral brace gusset plates occur at less than 1% of the total locations (Photos 42 and 43). No slip was detected where the loose bolts occur.

On the fascia girders, about 50% of the field splices have localized laminate rust and rust pits (Photo 44). The corrosion is generally limited to the bottom flange inside splice plate, bottom flange, and web bottom. Measurable corrosion pit depths range from 1/32" to 5/16" (Photo 45). Pack rust is bending some of the bottom flange splice plates, creating unintended prying forces on the splice bolts.

The NDT team ground out one crack discovered at a stiffener on unit 6A. All other suspected cracks tested negative.

2.3.2. Shiplaps

The shiplaps, which are located directly below the deck expansion joints, are in poor condition. Laminate rust and corrosion product covers most of the shiplaps (Photo 46). This corrosion made visual crack detection difficult or impossible. The most significant corrosion occurs on the lower supporting lap, both on the main girder and on the diaphragms directly supporting the deck. Knife edges or through thickness section loss was observed at several "end plates" located at the lap ends. Transfer girders, oriented at right angles to the main girders, are used at shiplaps where the number of girders on either side of the joint varies. The transfer girders are also experiencing scaling rust or laminate rust due to leaking expansion joints above (Photo 47). In general, it is estimated that 1/16" of section loss has occurred on all surfaces of the shiplaps. A structural analysis of the ship laps, accounting for the section loss, is warranted to determine the remaining load capacity.

Scaling rust or laminate rust has built up on the sole plate of most lower laps. The corrosion product traps moisture which promotes further rusting, and it may restrict free movement of the rocker bearing. The curved bearing surfaces of some rockers were found to have rust pits, resulting in higher bearing or contact stresses and pockets for moisture and debris to accumulate (Photo 48). Rust pit depths are estimated to be 1/8" Some rockers seemed to be excessively tipped for the given ambient temperature (Photo 49).

2.3.3. Cross-frames

The cross-frames (diaphragms), consisting of two diagonal single angles and a bottom chord single angle, are in good condition. Deficiencies include surface or scaling rust where a deck crack located directly above allows top side water to pass through onto the angles. This condition is the exception rather than the norm, with the corrosion accounting for less than 5% section loss to the members.

Weld cracks or broken welds were found at a few of the cross-frames where the diagonal intersection spacer plate is welded to the diagonals (Photo 50). Incomplete or missing welds at the cross-frame member to girder connection plates were also noted (Photo 51). At one location, all welds were missing and only the erection bolt is supporting the cross-frame member (Photo 52).

2.3.4. Lower Lateral Bracing

In the exterior bay horizontal planes between the cross-frames, WT section lower lateral bracing is laid out in an X-type configuration near the girder bottom flanges. The lower lateral bracing is in fair to poor condition. Deficiencies include localized failure of the paint system, and scaling rust or laminate rust near some of the shiplap expansion joints. Rust near the ship laps have caused less than 5% section loss.

2.4. Deck

The reinforced concrete deck is part of the original mid-1970's construction. In 1992, deck repairs were performed and a 2-inch thick bituminous overlay with waterproof membrane was placed. Since that time, overlay repairs have been performed, with the most recent work starting in the fall of 2010.

2.4.1. Underside

The deck underside is overall in fair to poor condition. Delaminations, spalls, and cracks with efflorescence are common and have caused local steel corrosion of the girders and stringers (Photos 53 and 54). Most spalls reveal exposed reinforcing steel, with some exposing both layers of the bottom bar mat (Photo 55). Cracks with leaching, delaminations and spalls occur in approximately 30% to 40% of all bays on this bridge, with a bay consisting of the area bounded by adjacent girders/stringers/floor beams/cross frames (Photo 56).

At most locations of the plate girder/floor beam/stringer approach spans, a "deck gap" exists between the concrete deck bottom and the top surface of the girder's top flange (Photo 57). Gaps typically measured 1/8" to ¼", and extend across the entire top flange width. These girders are non-composite and no shear studs tie the bridge deck to the steel floor system.

In the past, spalled pieces of concrete have fallen onto areas of the Maier Festival grounds where the general public or workers have access. To protect people from falling debris, plastic netting was installed on the deck underside of all units north of 1A and 1B. This work was performed circa 1996. In 2010, the same type of plastic netting was installed on units S2A to S3A, S2B to S3B, and on portions of unit S4B to protect Milwaukee Metropolitan Sewerage District (MMSD) personnel.

Netting over the Maier Festival grounds appears to be holding up well, except for areas exposed to direct sunlight. These areas are along the deck overhang edges. Since these areas are subjected to direct sunlight, ultraviolet radiation has deteriorated the plastic (Photo 58). A few loose deck fasteners exist (less than 1% of the total) and are ineffective in securing the netting to the deck. It was noted that several pieces of concrete have been caught by the netting.

There are localized areas of the deck underside that are in good condition. These good deck areas are generally limited to one span of a unit or a few bays within a span. Minimal (if any) delaminations or spalls characterize these regions, though cracks are still present.

2.4.2. Top Side

Overlay repairs are prevalent on the asphalt overlay. On any given unit, the quantity of asphalt repairs ranges from about 1% up to 50% of the deck area. The higher percentage occurs generally in the northbound lanes at the south end of the bridge from units S17 to S13 (Photo 59). Repaired areas totaling 5% to 10% of the deck surface area is about average for the entire bridge.

In addition to overlay repairs, unfilled cracks and cracks that have been filled with tar exist in the asphalt overlay. The cracks are primarily concentrated in the right hand lane of the southbound units, and in the middle and right hand lanes of the northbound units. This seems to correspond to the lanes used most often by the salt trucks entering and leaving the Port of Milwaukee, where much of the region's road salt is stored.

There is moderate to advanced deterioration of the reinforced concrete parapets. This deterioration affects most units, with severity ranging from continuous horizontal cracks along the top third of the parapets to widespread delaminations and spalls with exposed/corroded reinforcing steel (Photos 60 to 62). Deterioration occurs on both the median and outside parapets. Approximately 80%-90% of the total parapet length contains horizontal cracks, whereas approximately 30% to 40% of the total length is delaminated or spalled. Localized areas of past parapet repairs exist, generally on the units towards the bridge's north end.

In some locations the outside face of the parapets is spalled or heavily delaminated around the Maier Festival grounds. The potential for future falling concrete from the parapets presents a safety concern to workers and festival goers.

2.4.3. Expansion Joints

The expansion joint types for most of the bridge units are sliding plates. Finger plate joints are used at the ends of units N2A, N2B, S2A, and S2B. On units 1A and 1B, the deck is subdivided longitudinally into seven segments separated by compression seal joints.

Failed joint seals are causing advanced corrosion and section loss at most girder shiplaps. At the sliding plate joint assemblies, a black sealant was poured into the opening above the lower fixed plate. This sealant has debonded from the joint assembly, allowing water to flow freely through the joint and onto the structural steel below (Photo 63).

Original design drawings indicate that steel troughs used to catch water passing through the finger plate joints. These troughs no longer exist. The lack of a drainage system allows water and deicing chemicals to spill through the existing fingerplate joints directly onto the stringers and girders below. Heavy corrosion and laminate rusting of these members has resulted.

Local laminate rust on the stringer shiplaps of units 1A and 1B suggest that the compression seal joints above are failing. The heaviest corrosion is generally limited to the fascia stringers at the ends of the joints. Locally corroded shiplap areas within the joint length also exist, suggesting intermediate areas of compression seal debonding.

2.5. Substructure Units

2.5.1. Piers, Downspouts, and Fixed Bearings

Piers - Flexural and shear cracks are common at delta piers 2S to 14S and 2N to 9N (Photo 64). Some of the delta piers located near the main span contain numerous delaminations, particularly piers 2N and 2S which are situated directly below finger plate expansion joints (Photo 65).

Solid piers 10N to 4-5A, 2-6B, and S15 to 56S/68S/71S, are generally in good condition, and contain only nominal cracks (Photo 66). Approximately 10% of these solid piers were found to have local delaminations or surface spalls with exposed rebar. Pier 4 of Unit 5A and Pier 3 of Unit 6B are in considerably worse condition than the other solid piers. These piers are directly below the expansion joint at the beginning of the concrete box girder units at the north end of the bridge, and contain numerous concrete delaminations.

Downspouts - Most downspouts are clogged and inoperable (Photo 67). The rubber expansion boots on some downspouts are crushed, restricting the flow of water and trapping debris. A missing expansion boot and broken/disconnected downspouts were noted at a few piers. Many of the downspouts are disconnected at the joints, or have failed hanger connections.

Fixed Bearings - Laminate rust exists on approximately 20% of the fascia girder fixed bearing pins and masonry plates (Photo 68). Paint problems on all of the bearing shoes and sole plates are limited to surface rust. About 10% to 20% of the anchor bolts securing the masonry plates to the pier pedestals are either tilted or raised such that the bolt nut is not in contact with the plate.

2.5.2. Abutments

The abutments are in fair condition. Moderate density medium to wide cracks with rust stains and efflorescence exist in both the backwalls and breast walls.

3. CONCLUSIONS

Steel corrosion and section losses occur on this bridge primarily at the approach span shiplaps. **Laminate rust and measurable section loss at the shiplaps indicate a loss of structural capacity and that member strengthening will likely be needed as part of future rehabilitation work.**

Though laminate rust was noted locally on the top flanges of girders and stringers located under leaking cracks in the deck, the amount of section loss does not warrant a structural evaluation at these locations. Laminate rust and pits are also present at several fascia girder bottom flange field splices. Since the field splices are located near inflection points and the stresses within the bottom flanges are low, a structural analysis is not required.

The paint system, which is part of the original 1970's era construction and approximately 35 years old, is showing various stages of deterioration. Many areas exhibit peeling of the top coat of paint and/or blush rust. Complete paint system failure occurs at most shiplap joints and locally at the girder/stringer top flanges where deck cracks allow water to infiltrate. **To protect the steel superstructure from continued deterioration, the entire bridge should be blast cleaned and repainted.** It was noted during the inspection that the prime coat of paint has a bright orange color. This strongly suggests that the prime coat is the "red lead" commonly used on bridges constructed in this period.

Similar to the paint system, the bridge deck is approximately 35 years old. Advanced deterioration was seen during the inspection, confirming that it is near the end of its useful life. The deterioration includes deck and parapet delaminations, spalls, cracks with leaching water, plus deck overlay cracks and repairs. The most serious deterioration occurs at the expansion joints. Almost all of them are leaking, and the leaks have allowed advanced corrosion to take place at the girder shiplaps. **The extent and progression of the deck deterioration is such that local or piece meal repairs of the concrete deck and parapets are not economically warranted.**

4. RECOMMENDATIONS

Rehabilitation recommendations are divided into three categories: Emergency Items, Critical Items and Non-Critical Items. Emergency Items designate repairs that require immediate attention because the existing condition presents an immediate safety risk for the public. Critical Items designate repairs that will require attention soon before further deterioration creates a future safety risk. The Critical Items are recommended for repair within the next 5 years. Non-Critical Items designate repairs that will improve the longevity and serviceability of the bridge, but do not present any existing or near future safety risk at this time. It is recommended that Non-Critical Items be addressed within the next 10 years.

No Emergency Items are recommended for repair at this time. Critical and Non-Critical Item repairs are outlined in the following sections.

4.1. Main Span, Tie Girder, and Arch Steel

Critical Items:

1. Sandblast, clean, and repaint all main span steel members (stringers, floor beams, and lateral bracing).
2. Sandblast, clean, and repaint all exterior exposed surfaces of the tie girders and arches.
3. Clean and spot paint the interiors of the tie girders, arches, and floor beam box bottom flanges.
4. Recaulk/fill gaps at the field splices of the tie girders, arches, floor beam box bottom flanges, and tie girder cable covers.
5. Remove the pack rust at the arch access hatches and replace the hatch gaskets.

Non-Critical Items:

6. Attach ropes inside the lower arch members at these points where no steel hand rail currently exists. Inspection was difficult inside the steepest part of the lower arches above piers 1N and 1S due to a lack of hand rails.
7. Lubricate the north bearing of the east tie girder where the roller gear stud bolts rub against the lower pin plate.
8. All catwalks should be laterally braced to eliminate the lateral deflection serviceability issue.
9. Bring catwalks up to current safety codes by installing toe kick plates and overhead tie-off cables.
10. Install a platform to provide a safe access from the cawalk to the floor beam box bottom flanges located at panel points 6 and 6'.
11. Repair or replace the corroded catwalk cable handrail terminations.
12. Fix the lighting inside of the east tie girder.
13. Clean and spot coat the hanger cables where surface rust exists.

4.2. Girder/Floor Beam/Stringer Approach Span Steel

Critical Items:

1. Sandblast, clean, and repaint all steel members (stringers, floor beams, and girders).
2. Rehabilitate fascia girder field splices at the bottom flange and base of web. Sandblast, remove pack rust between bottom splice plates and repaint.
3. Remove and replace the existing heavily corroded shiplap rocker bearings, sole plates, and attachment bolts.
4. Perform a structural analysis to check the remaining structural capacity of the shiplaps, assuming 1/16" of section loss on all steel surfaces. If required, strengthen the shiplaps using hot rolled angles bolted to the existing shiplap components.
5. Perform an Interim Inspection of all shiplap joints after cleaning and repainting to check for cracks hidden by the laminate rust.
6. Drill or grind out the fatigue cracks that were observed during this inspection but were unable to be ground out completely during the NDT.
7. Remove existing loose connection bolts found at field splices or cross-frame connections. Replace with the same sized slip critical A-325 bolts.

Non-Critical Items:

None

4.3. Multigirder Approach Span Steel

Critical Items:

1. Sandblast, clean, and repaint all multigirder approach span steel members.
2. Rehabilitate fascia girder field splices at the bottom flange and base of web. Sandblast, remove pack rust between bottom splice plates and repaint.
3. Remove and replace the existing heavily corroded shiplap bearings, sole plates, and attachment bolts.
4. Perform a structural analysis to check the remaining structural capacity of the shiplaps, assuming 1/16" of section loss on all steel surfaces. If required, strengthen the shiplaps using hot rolled angles bolted to the existing shiplap components.
5. Perform an Interim Inspection of all shiplap joints after cleaning and repainting to check for cracks hidden by the laminate rust.
6. Prior to new deck placement, weld cross frame members onto the connection plates where no weld currently exists, or where lack of fusion exists.

7. Remove existing loose connection bolts revealed at field splices, cross-frame connections, or lower lateral bracing connections. Replace with the same sized slip critical A-325 bolts.

Non-Critical Items:

None

4.4. Deck

Critical Items:

1. Prior to new deck placement, remove concrete delaminations from the deck underside and outside faces of the parapets over the Port of Milwaukee Property and public area to the south. Remove concrete delaminations from the outside faces of the parapets over the Maier Festival grounds. Delaminations are numerous and present a safety concern for Port workers and the general public.
2. Redeck the entire bridge using a standard concrete mix.
3. Replace the existing finger plate joints with modular joints at both ends of units N2A, N2B, S2A, and S2B.
4. Replace the existing sliding plate joints of the approach span units with strip seal expansion joints.
5. Replace the existing compression seal joints on the main span with strip seal expansion joints (the current Wisconsin Bridge Manual states that compression seals are only to be used with fixed joints).

Non-Critical Items:

None

4.5. Substructures

Critical Items:

1. Barricade areas around heavily delaminated pier 2S to prevent MMSD workers from entering or storing equipment inside the zone where concrete pieces may fall. Barricade a similar area around piers 2N, 4-5A and 2-6B to prevent Maier Festival grounds workers from entering a similar zone.
2. Remove delaminations from piers located in public access areas, Maier Festival grounds, MMSD property, and Port of Milwaukee property.

Non-Critical Items:

1. Clean out the downspouts, and replace the flexible rubber boots that are crushed, loose, or missing.

4.6. Possible Ramifications of Delayed Maintenance

Any delay in addressing the existing bridge deterioration issues can lead to significant safety concerns and cost increases for future rehabilitation. Addressing these issues in a timely fashion will minimize the concerns. Possible consequences of not taking action on the Critical Items include the following:

1. **Inaction to blast and repaint the bridge steel -** Continued paint system failure and unheeded corrosion will lead to local structural failures due to reduced steel areas/structural strength.
2. **Inaction to rehabilitate the ship lap joints –** Section loss and reduced structural strength is present on most ship laps. Continued corrosion will reduce the structural strength further, possibly leading to a local failure in one ship lap and overloading the adjacent ship laps. Potential cracks that are hidden by the existing heavy or laminate rust will not be found and arrested. The cracks may grow to a dangerous length that compromises the member's strength.
3. **Inaction to rehabilitate the ship lap rocker bearings –** Corrosion pitting on the curved surfaces of the bearings has already caused overstresses on the rockers and sole plates. Further pitting will increase the stresses, damaging the rocker or sole plate. This could inhibit the proper expansion/contraction of the joint, thereby introducing unintended stresses into the girders.
4. **Inaction to remove delaminations from the existing deck underside and parapets –** Delaminated concrete on the deck underside may spall off, fall, presenting danger to the general public.
5. **Inaction to replace the parapets -** Safety to the travelling public is compromised due to weakened parapets and an ongoing process of deterioration. Concrete spalls create a rough parapet surface and create snag points for an errant vehicle.
6. **Inaction to replace the concrete deck -** Delaminations will continue to form on the deck underside, leading to concrete spalling and the danger these falling pieces of concrete present to the general public. Deck cracks will continue to allow water and deicing chemicals to pass through, corroding the deck reinforcing steel and reducing the deck's structural strength. The presence of the deck cracks also increases the rate of paint failure and corrosion on the steel superstructure. Local punching shear

type deck failures from vehicle wheel loads will be possible. Expansion joints will continue to leak and corrode the steel superstructure.

7. **Inaction to remove pack rust from field splice plates –** Pack rust will continue to develop and add more unintended prying forces to the bottom flange splice plate bolts. Eventually the bolts will begin to fail, reducing the splice strength and possibly leading to a local girder failure.
8. **Inaction to replace loose or missing bolts –** Connections with loose or missing bolts do not have the strength as intended in the original design. A future structural overload may cause damage or a failure to the connection.
9. **Inaction to grind out existing cracks –** There are some cracks identified during this inspection. Though grinding was performed to remove these defects, the entire crack could not be ground out for a few of the tested areas. These remaining cracks have the potential to grow, potentially causing a local member failure.
10. **Inaction to provide weld at cross-frame members –** There are some cross frame members that have undersized, cracked, or missing welds. The cross frames are required to keep the girders in-line and to provide proper load distribution during the proposed concrete deck pours. A broken, undersize, or missing weld may allow for a cross frame member failure during a concrete deck pour.
11. **Inaction to remove delaminations from the piers accessible to the public –** Delaminated concrete on the piers may spall off. The falling concrete pieces will present a danger to the general public.
12. **Inaction to barricade delaminated piers not accessible to the public –** Delaminated concrete on the piers may spall off. The falling concrete pieces will present a danger to Maier Festival, MMSD, or Port of Milwaukee employees.

4.7. On-going Maintenance Program

After major rehabilitation work is completed, an on-going maintenance program could be developed to extend the life of the bridge, improve serviceability, and potentially reduce the overall future maintenance costs. Timely rehabilitation of certain maintenance items can reduce the quantity and severity of future deterioration. A regularly scheduled on-going maintenance program is recommended to include:

1. Spot cleaning and painting of the steel framing system, with a primary focus on the ship lap joints.
2. Cleaning out and repairing of deck drains and downspout system to ensure proper drainage.

3. Removing delaminated concrete on the piers, deck, and parapets in areas accessible to the public.
4. Cleaning and repairing expansion joints to minimize water leakage.
5. Other work as determined during regular inspections.

5. PRELIMINARY CONSTRUCTION COST ESTIMATES

For our estimates, we have assumed that the primary bridge rehabilitation tasks will include:

- Complete bridge redeck with an 8-1/2 inch thick concrete deck.
- Complete blasting of the existing paint system and repainting of the structural steel elements. The interiors of the tie girders and arches are not included.
- Strengthening of all shiplap joints.
- Expansion bearing replacement.
- Concrete surface repair of the piers.
- Removal and replacement of four sign structures.

Costs provided within this report should be considered engineer's opinion of probable construction costs and should be used for planning purposes only. The actual construction costs can vary dramatically depending on the scope of work to be performed, varying costs of materials and labor, access and other site restrictions and the schedule of work.

APPENDIX

(Photos)

Photo 1: Main span stringer - local heavy rust at the top flange and typical bottom flange blush rust

Photo 2: Main span stringer shiplaps - typical condition

Photo 3: Main span floor beam - local paint failure/surface rust at the bottom flange

Photo 4: Main span floor beam - laminate rust near tie girder connection at stiffeners

Photo 5: Main span - floor beam box bottom flange local rust

Photo 6: Main span - floor beam box bottom flange rust at the tie girder connection

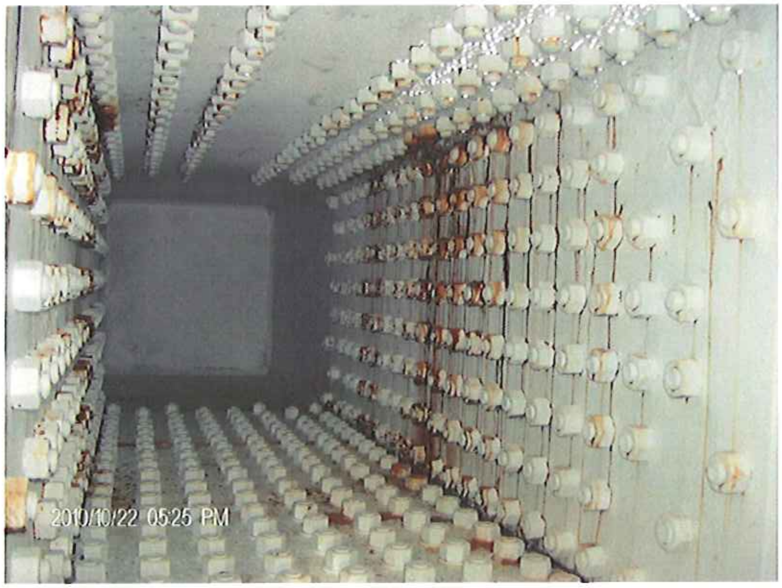

Photo 7: Main span - floor beam box bottom flange field splice interior w/ condensation and rust staining

Photo 8: Main span - typical lateral brace condition with bottom chord surface rust

Photo 9: Main span - pack rust at lateral brace bottom chord connection to gusset

Photo 10: Main span - bent lateral brace web diagonal

Photo 11: Tie girder - typical top flange corrosion at splice and cable covers

Photo 12: Tie girder web - typical laminate rust at floor beam bottom flange gusset

Photo 13: East tie girder - dent near bottom flange

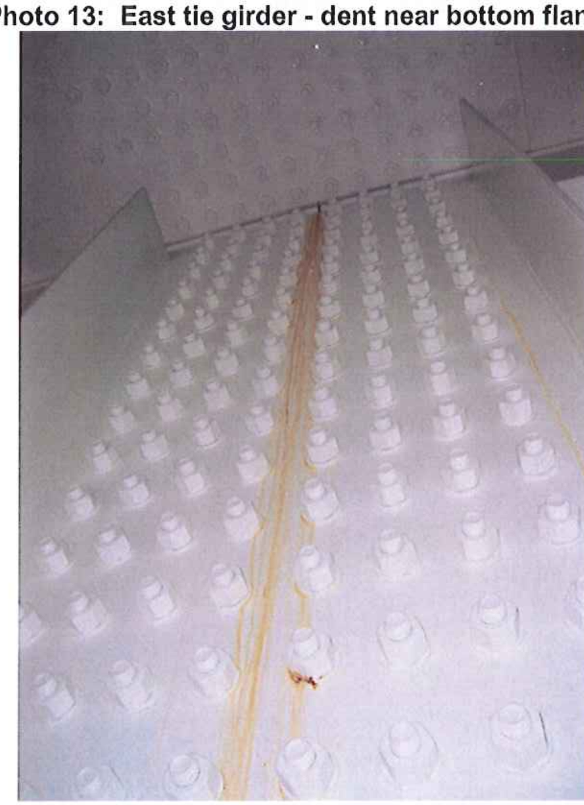

Photo 14: Tie girder interior- typical rust stains at field splice the

Photo 15: Tie girder splice - heavy corrosion at bottom flange corner

Photo 16: Lower arch column base - surface rust (note pack rust at access hatch in background)

Photo 17: Lower arch - typical field splice paint failure and surface rust

Photo 18: Upper arch - typical surface rust at field splice above tie girder

Photo 19: Lower arch interior - local laminate rust at the arch/strut fillet due to leaking hatch above

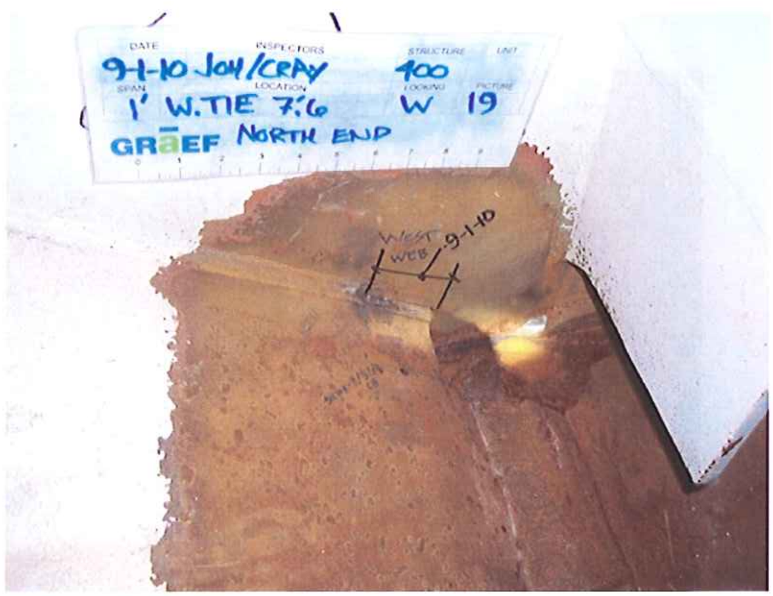

Photo 20: Arch/tie girder intersection - visually evident longitudinal crack along arch to tie girder web weld

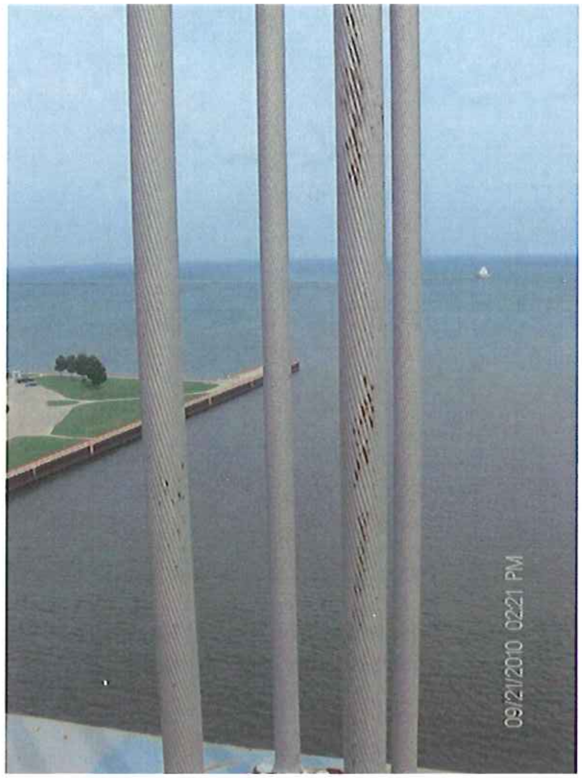

Photo 21: Hanger cables - spot rust and typical condition

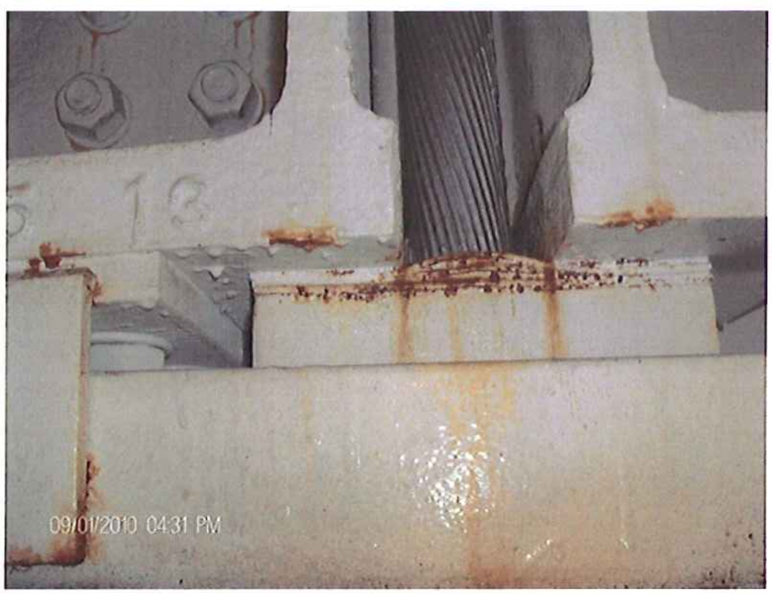

Photo 22: Hanger cable anchorage - pack rust between shim plates

Photo 23: Leaking tie girder cable cover - corrosion causing rust stains on hanger cable (typical condition)

Photo 24: Catwalk guardrail - laminate rust at anchorages

Photo 25: Peeling Paint on girder webs and flanges.

Photo 26: Failed paint at previous girder repair (mouseholes at top of web)

Photo 27: Laminate rust (typical) at girder in shiplap.

Photo 28: Corrosion and pitting on rocker bearing in shiplap.

Photo 29: Corrosion at fascia girder field splice at bottom flange and base of web.

Photo 30: Pack rust at fascia girder bottom flange field splice

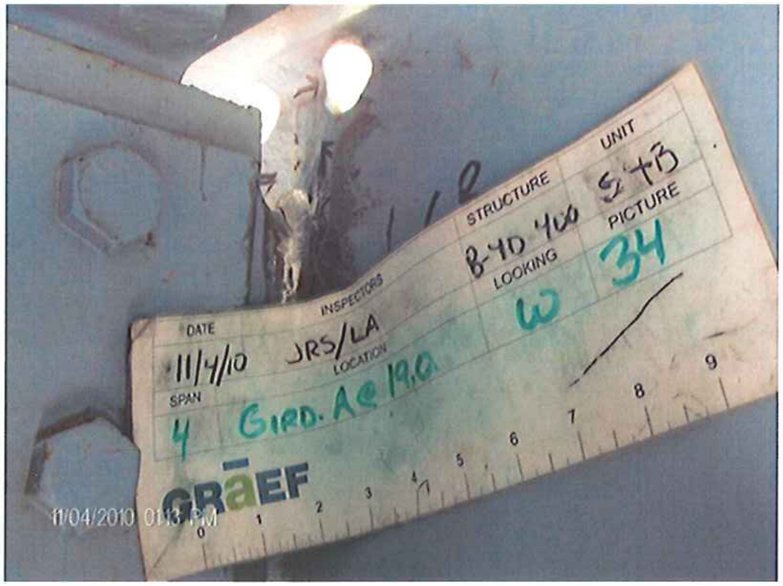

Photo 31: Fatigue crack at top of girder connection plate after completed NDT.

Photo 32: Large existing crack through web of girder arrested by mouseholes and bottom flange splice plates.

Photo 33: Wavy bottom flange of girder near pier.

Photo 34: Blush rust on underside of floor beam bottom flange (typical).

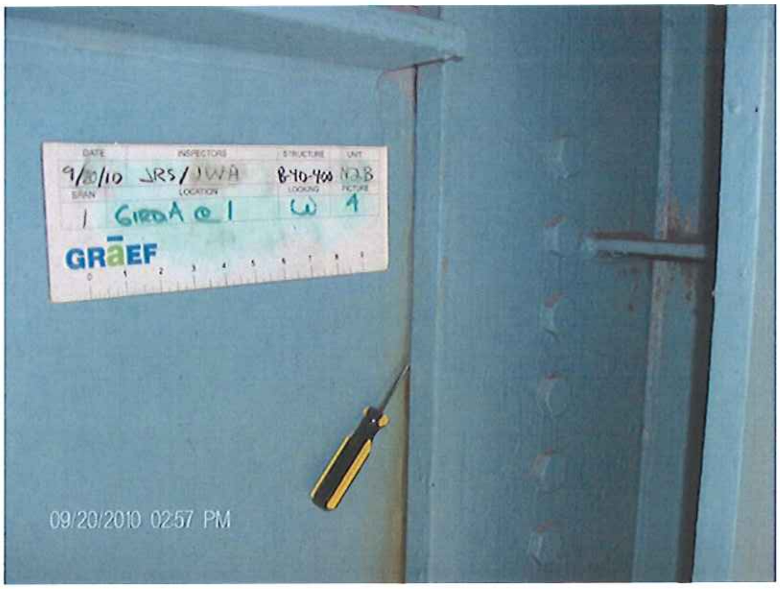

Photo 35: Gap between floor beam and girder connection plate.

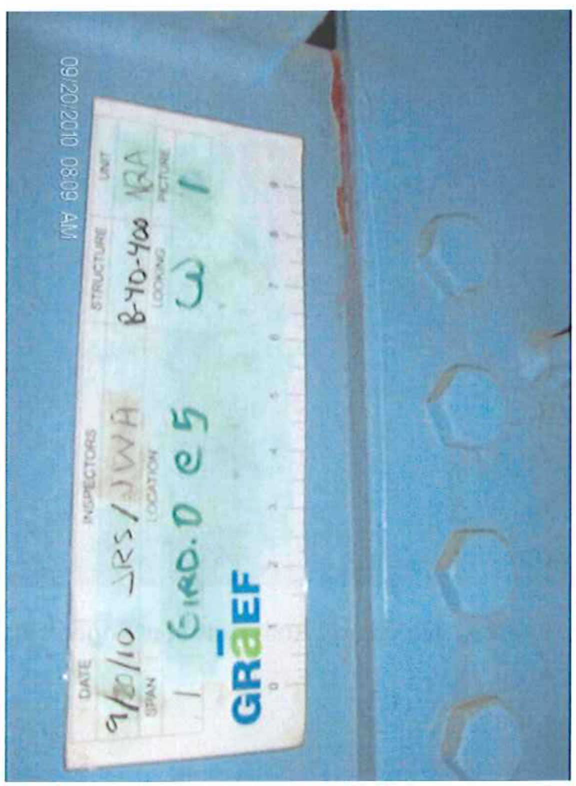

Photo 36: Fretting rust between floor beam and girder connection plate.

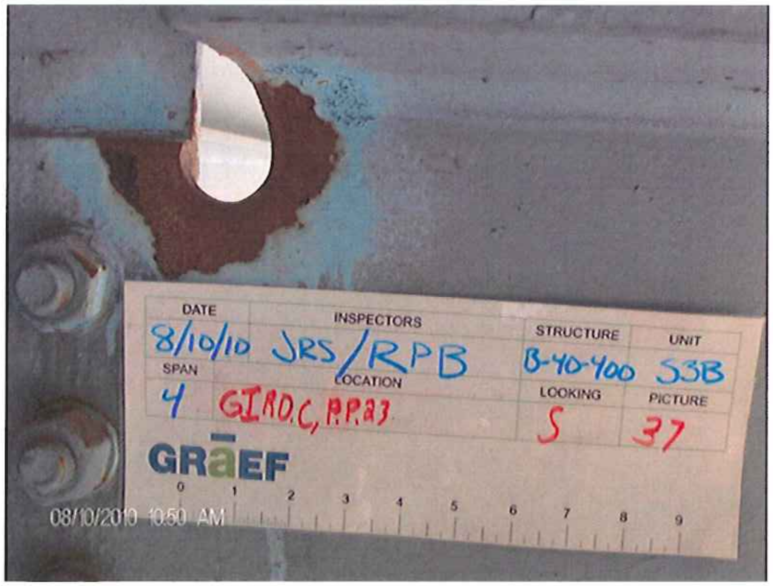

Photo 37: Ground out area at floor beam top flange cope.

Photo 38: Peeling paint and blush rust at stringer bottom flange (typical).

Photo 39: Local laminate rust on stringer top flange at deck spall.

Photo 40: Laminate rust at end of stringer and end diaphragm in shiplap below finger plate joint.

Photo 41: Multigirder top flange corrosion typical at deck delaminations

Photo 42: Three loose bolts on bottom flange splice plate

Photo 43: Loose bolt on lower lateral brace gusset plate

Photo 44: Fascia girder in multigirder approach spans – bottom flange pit due to corrosion

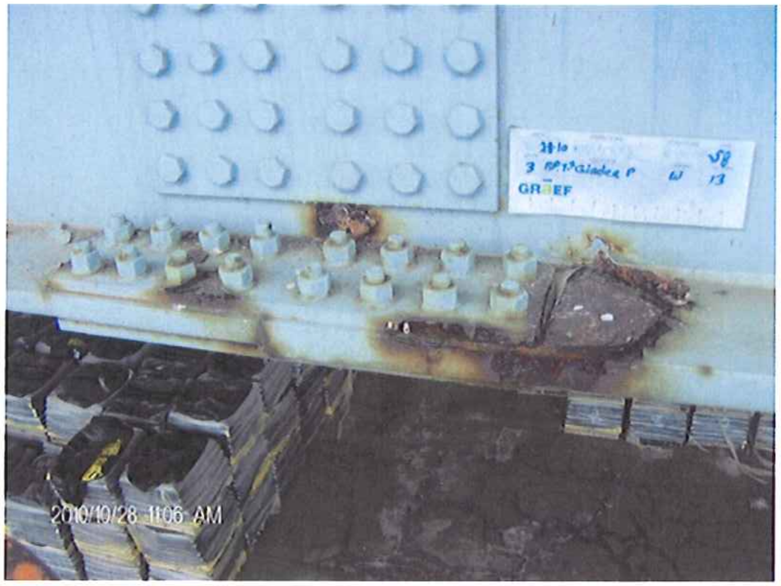

Photo 45: Multigirder fascia - laminate rust at bottom flange splice plates

Photo 46: Typical rust at multigirder unit shiplap

Photo 47: Heavy surface rust at shiplap transfer girder top flange

Photo 48: Multigirder unit rocker bearing with pitting on curved bearing surface

Photo 49: Excessively tipped multigirder rocker bearing

Photo 50: Cross-frame broken weld at diagonal spacer plate to angle

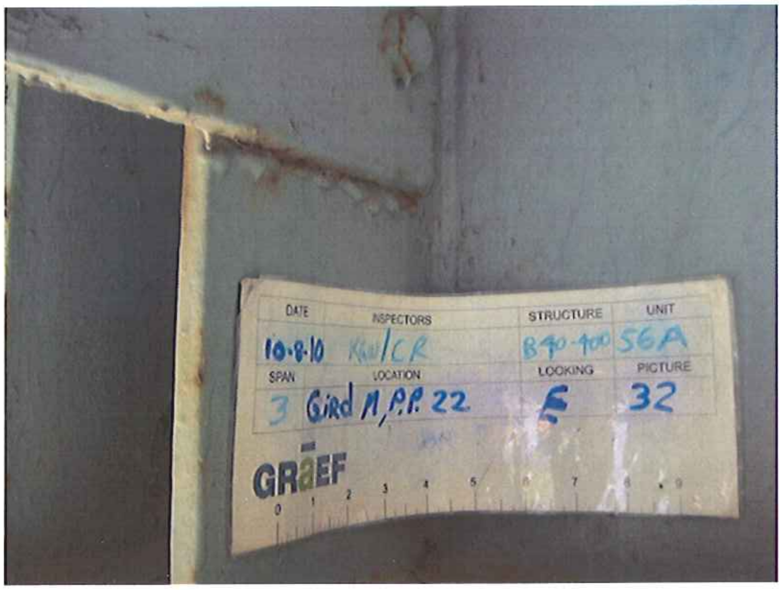

Photo 51: Incomplete fusion at cross-frame bottom chord to connection plate

Photo 52: Missing weld at cross-frame bottom chord to connection plate

Photo 53: Deck underside - typical cracks with leaching and stringer top flange corrosion

Photo 54: Deck underside – typical overhang delaminations with corrosion at girder top flange

Photo 55: Deck underside - spall with 2 layers of bottom mat reinforcing steel exposed

Photo 56: Deck underside - typical condition with delaminations and spalls

Photo 57: Deck underside - typical deck gap at girder

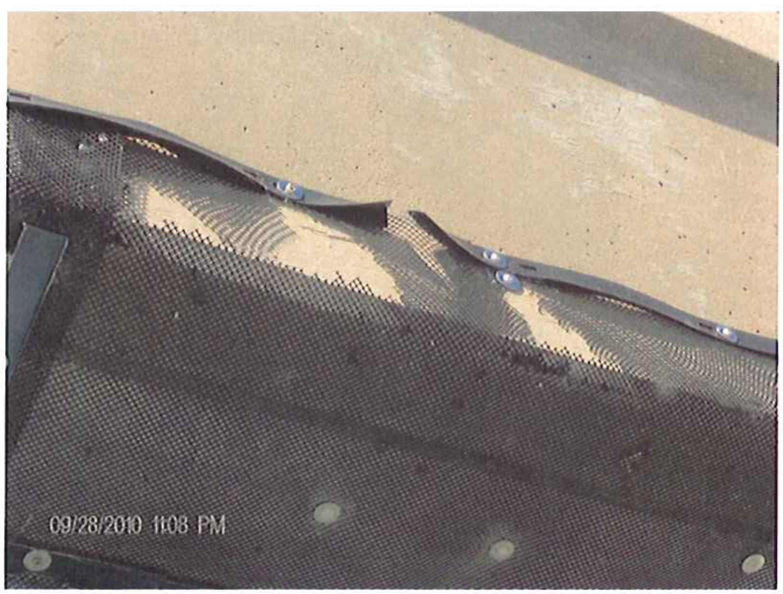

Photo 58: Deck underside - deteriorated netting over Maier Festival grounds

Photo 59: Deck top side - heavy asphalt overlay repair on south end, northbound units

Photo 60: Parapet – typical spall with exposed reinforcing steel on the front face

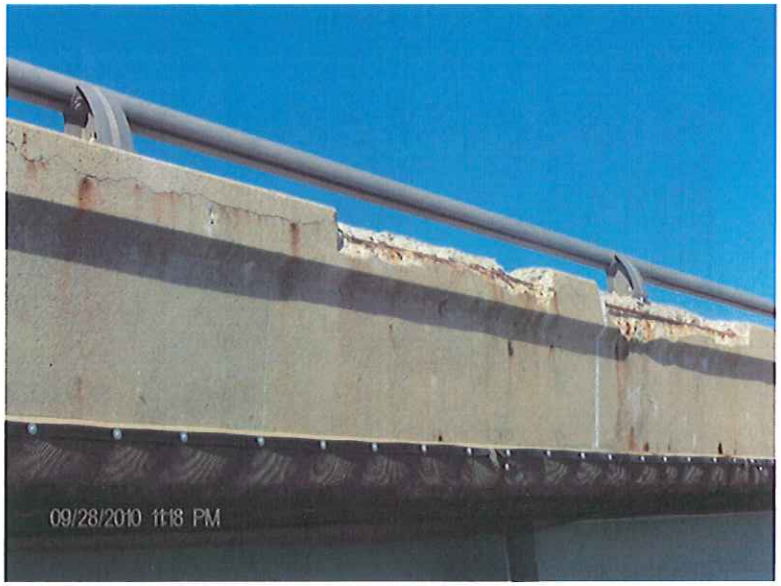

Photo 61: Parapet - spalls with exposed reinforcing steel on the exterior face

Photo 62: Parapet - typical horizontal cracks and delaminations on the median parapet

Photo 63: Sliding plate joint - pourned sealant failure (typical)

Photo 64: Typical delta pier

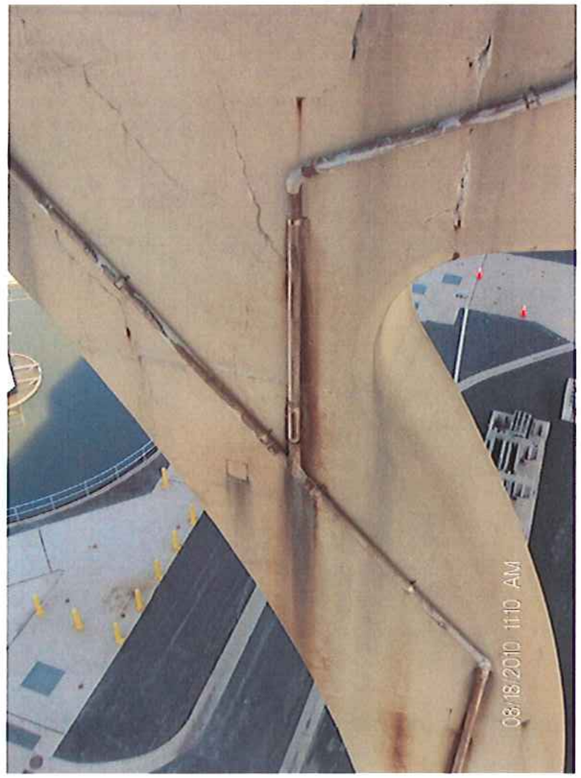

Photo 65: Pier 2S - cracks and leaking downspouts

Photo 66: Typical solid pier

Photo 67: Misaligned downspout and crushed expansion boot

Photo 68: Laminate rust on fixed bearing pin