BRIDGE INSPECTION REPORT

Sylvan Slough Bridge Crossing - Moline, Illinois

for

CITY OF MOLINE – MOLINE, IL

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SYLVAN SLOUGH BRIDGE CROSSING

LOCATED BETWEEN SYLVAN ISLAND AND MOLINE, ILLINOIS AT 2ND STREET

INSPECTION FOR:

CITY OF MOLINE, ILLINOIS

INSPECTION TYPE: GENERAL (FULL)

INSPECTION DATE: JUNE 3RD – 7TH, 2013

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MISSMAN PROJECT NO.: A13Y010

BRIDGE INSPECTION REPORT

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IN MAY 2013, MISSMAN, INC. WAS ENGAGED BY THE CITY OF MOLINE TO INVESTIGATE THE CAPACITY OF THE EXISTING BRIDGE AT THE ENTRANCE TO SYLVAN ISLAND PARK. THIS REPORT CONTAINS THE PERTINENT INFORMATION OBTAINED FROM THE FIELD INSPECTION, A DESCRIPTION OF THE PHYSICAL PROPERTIES OF THE STEEL MEMBERS, A LOAD RATING SUMMARY, AND RECOMMENDATIONS FOR REPAIRS/REPLACEMENT FOR FUTURE USE.

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I. GEOGRAPHICAL & ADMINISTRATIVE DATA

Structure Number: N/A
County: Rock Island
Route Carried: Pedestrian Access to Sylvan Island
Feature Crossed: Sylvan Slough (Mississippi River Canal)

Weather: 65-75°F

Inspection Method: Visual. Flooded river conditions at the time of inspection allowed for boat access and hands-on inspection of the underside floor system and all lower truss members. Upper truss members were hands-on and visually inspected from a scaffold. A chipping hammer and wire brush were used to remove surface rust and other detritus prior to determining the remaining section of all members inspected. Bearings and abutment caps were cleaned. No diving for substructure inspection or NDE for superstructure was conducted.

The substructure was additionally inspected at low-flow conditions after flood waters had receded.

II. STRUCTURE DATA

ADT (current): Pedestrian & Bicycle Access
ADTT (current): - - - -
Load Rating (H-15): H-4.87 (110' Truss-Vertical Connection, 1969)
Pedestrian Rating: 53 psf

STRUCTURE DESCRIPTION

Type: Pratt-style, Thru-Truss
Span Arrangement: 2-span, simply supported
Length and Width: 89'-0" and 110'-10" | 12'-10" c-c truss, 12'-0" clear
Skew: 11°20'
Abutments: Concrete, bearing on shale bedrock.
Piers: Stone masonry, bearing on shale bedrock.

CONSTRUCTION / RECONSTRUCTION / REPAIR HISTORY

Year Constructed: **1901** (Drawing No.: RIA S162-1) – Two pin-connected, Pratt-style, thru-trusses of 89'-0" and 110'-10" spans respectively. The trusses, spaced at 12'-10" are on an 11°20' skew, and are supported by two concrete abutments and one masonry pier. Originally, decking was 2x10 wood planking on steel stringers.

Year/s Reconstructed: **1945** (Drawing No.: 162-100) – Bearing plate repairs. According to repair plans the original riveted sole plate, rollers, and bed plate were replaced. Abutment stems were widened, new welded sole plates, s/s rollers, and bed plates were installed.

1953 (Drawing No.: RIA S162-9) – Original timber planking of bridge floor were replaced with corrugated metal decking surfaced with asphaltic concrete. According to repair plans, additional steel stringers were proposed to be installed at this time. However, a combination of steel and timber stringers was selectively installed to strengthen the floor system rather than all new steel. Turnbuckles were also added to some of the diagonal tension members at this time.

1964 (Drawing No.: RIA S162-12) – Floor system and floorbeam support repairs. Neoprene bearing pads were added to stringer bearings at abutments. Exterior stringer bearings were reinforced with welded angle attachment at floorbeams. 5" channel at supporting ends of floorbeams was replaced with 5" x 1" plates, included replacement of U-bolts and painting of assembly.

III. FIELD INSPECTION & PHYSICAL EVALUATION

SUPERSTRUCTURE

Floor System: Corrugated 12-ga. metal decking surfaced with asphaltic concrete on stringers, originally installed in 1953. Structural rating: "fair-to-poor". Overall loss of bituminous paint coating; moderately severe surface corrosion and localized section loss. Several full-depth holes observed. Cracking in asphaltic concrete observed at floor beams is therefore allowing water to penetrate structural plating joints, leading to primary cause for deterioration of stringers and floorbeams (Figure 25-Figure 26).

Stringers: Steel and timber: varying member sizes S6 thru S8 beams with timbers (interior); C7 channels with timber and S6 beams (exterior). In general, steel stringers show moderate-to-severe corrosion of top flange with light-to-moderate corrosion of remaining member (Figure 21). Outside original channel stringers have advanced section loss and bearing. Some original stringers were replaced or "doubled-up" with S6x12.5 members in 1953 rehab work. These replacement S6x12.5 members are generally in good condition with minor surface corrosion. The timber stringers (2x7's w/ 4-bolted connection) added in 1953 to reinforce original stringers are generally in good condition and structurally sound. A significant number of the interior stringers are not readily accessible to close inspection because they are tightly sandwiched between timber stringers. Visible bottom flanges typically show moderate section loss with remaining member assumed in similar condition. Loss of stringer bearing capacity at floorbeams was seen at majority of locations due to section loss of floorbeam top flange area. Some stringers had no bearing contact with floorbeams. Several stringers were rotated out of plane due to severe pack rust at the bearing locations. (Figure 22).

Floorbeams: Steel I-beams, 12”x31.5: Generally seen to have moderate-to-advanced section loss of top flange at stringer locations (Figure 20-Figure 23). Light-to-moderate surface corrosion of webs and bottom flange; localized section loss on several members. Out-of-plane bending and global rotation evident on several members with measurements found to be 1-inch out of plumb. Field measurements indicated up to 50% loss of top flange section at multiple locations, typically at stringer bearings.

The bottom lateral bracing members were observed to be largely intact, although several of them have been bent. Some members are loose and cannot be relied upon for any structural engagement.

Trusses: In general, all main truss members were found to be in satisfactory condition with minor surface corrosion despite the total loss of asphaltic paint coating. However, severe pack rust and section loss was locally observed at majority of connection locations. Built-up top chord members have complete section loss of top plate at all strut connections (Figure 36-Figure 40). Vertical-to-Top chord connection plates have instances of severe corrosion and even reduced connection capacity above fifty percent (Figure 39). Pin connections and plating typically found to have moderate surface corrosion and pitting with some section loss.

Bottom chord members were found to be in critical condition at the abutment bearings due to advanced corrosion, pitting, and section loss exceeding 50% of the thickness at the eyebar pin connection. In a typical bridge inspection, the deterioration noted at these locations would have warranted a “critical finding” and immediate closure of the bridge. These members could easily be overstressed. Furthermore, being fracture critical members means that the bridge would collapse upon their failure. Other bottom chord members were observed to be in fair condition with surface corrosion and some section loss noted at the pin connections.

Laced channel vertical members have experienced severe deterioration at the lower ends near lower pin connections (Figure 31-Figure 32). This deterioration and pack rust was also observed on the bottom chord eyebar ends. Generally, all lower pin connections have severe corrosion and pack rust that encompasses the ends of all connecting members.

Truss diagonal members were found to generally be in good condition. However, several of the members, all of which should be in tension, were found to have no engagement and were “loose”. This was especially true at 2-bar members with only one bar having a turnbuckle installed. Presently, there is a turnbuckle in one bar of every double bar diagonal and in each of the single bar counters.

Top chord bracing struts and laterals were observed to be severely deteriorated. The struts were seen to have near complete section and connection capacity loss at the top chord members in addition to severe pack rust between double angle and knee bracing interfaces. Fractures caused by pack rust expansion were also observed. Lateral bracing members were generally not engaged and had extremely limited to zero connection capacity to the top chord members.

Portal frames exemplified similar deterioration as seen on top strut members, although gusset plate connections to top chord members were in good shape. Severe pack rust leading to section loss and fractures were observed on double angle segments (Figure 45-Figure 46). Several top and bottom portal segments previously replaced original double angles with "T" bars are in good condition.

Railings: Railing consists of 3'-0" tall section made up of steel angles and welded lacing members; top of the rail at 3'-3" from wearing surface. Overall, railings are in good condition; including connection to truss members.

Pins: Pins were found to have moderate surface corrosion and pitting with negligible section loss assumed. However, severe pack rust between pin plates and connecting members is significantly affecting the connection capacity.

Bearings: Abutment bearings are in a severely deteriorated condition. Severe pack rust observed between adjacent members at pin connections; consequentially resulting in significant section loss and reduced capacity of bottom chord members (Figure 27-Figure 30). Roller bearings assumed to be in a frozen condition. A tree was even found to be growing in/around the northeast bearing assembly (Figure 29). Bearing pins have severe surface corrosion and pitting with some section loss. Structural capacity of bearing is there for reduced.

Pier bearings found to be in good condition, with limited surface corrosion that has occurred. It appears the pier bearings have been repaired previously due to welded construction, whereas, the original bridge was constructed with rivets. Bearings are fixed and have maintained a relatively clean and dry condition.

Joints: Prior to the cleaning processes that occurred during the inspection, the expansion joints and abutment caps were inundated with debris, dirt, and vegetation. Wet conditions were undoubtedly existent for extended periods which adversely led to some of the deterioration observed at the expansion bearings and adjacent stringer ends.

General: The entire superstructure vibrates under the loading of two men walking. A single man jumping up and down on the deck creates noticeable vibrations throughout the truss.

SUBSTRUCTURE[2]

Abutments: Both abutments were observed to be in a structurally sound condition. Minor cracks are located at the interface between the wingwall and the front face. The concrete itself exemplifies only typical surface deterioration. (Figure 7-Figure 8 and Figure 13-Figure 14)

Pier: The single pier was observed to be in a similar state as noted by 1969 Teng report. Square stone masonry, observed to be vertical and sound; sitting atop a concrete encased footing that is founded on bedrock. Minor deterioration of the masonry joints was seen; no evidence of scouring or undercutting of the foundation was found.

Slope Protection: Combinations of riprap and shale embankments; no erosion, undercutting, or scour was observed. Embankments are further protected with gabion walls that extend from the wingwall ends; all segments found to be in good condition. (Figure 11-Figure 14)

INSPECTION HISTORY

Prior to 1960: No records are currently known, it is assumed that the Rock Island Arsenal (RIA) regularly inspected the structure given that they approved repair plans in 1945, 1953, and 1964. Inspection reports also show their apparent ownership thru June of 1972, after which, ownership transferred to the City of Moline.

1960: The City of Moline hired Robert Hofmann and Associates to perform an inspection and analysis of the structure. According to the summary letter the following load ratings were determined:

Floor System: H-7 (inventory); H-12 (operating)
Trusses: H-10 (inventory); H-20 (operating)

1964-1969: Teng & Associates, Inc.: Hired by RIA. (inventory ratings)

Floor System:	H-10.5	structural floor plate decking
	H-1.45	exterior 7” channel stringers
	H-7.3	interior stringers
	H-7.0	floorbeam (bending)
Trusses:	H-12.3	89’ Truss
	H-4.87	110’ Truss

[2] Further inspections were conducted following the conclusion of receded floodwaters.

Teng references a 1964 test report conducted by the Rock Island Arsenal for the mechanical and chemical composition of the steel. For the test, samples of one lacing bar and one tie plate were evaluated. From the report, a copy of which was included in Teng's report, it was apparent that the steel used in construction of the bridge is very close to ASTM A-7 structural carbon steel.

1971: D.L. Logsdon – Rock Island Arsenal, Structural Section: Report summarized in terms of repairs required to return structure to certain H-loadings. Ratings referenced from 1969 Teng & Associates report. The bridge was reported to have been closed to all vehicular traffic following the completion of the assessment report from Teng & Associates, in 1969.

1972: Report from Division Engineer – Rock Island Arsenal: Letter discusses 1971 RIA reports and estimated costs for repairs, removal and replacement of existing structure. Ownership appears to have been transferred to the City of Moline at this time.

1993: Missman, Stanley & Associates, Prof. Corp.: Inspection and evaluation of existing conditions relative to past reported inspections and studies. Ratings remained the same as noted in 1969 Teng report with additional note to pedestrian loading. 1969 Teng report noted governing truss vertical members at H-4.87, correlating to a pedestrian live load reduction of 85 PSF down to 53 PSF. Minor repair costs estimated and regular inspection recommended.

April 2013: Bridge was closed to all access by the City of Moline due to safety concerns.

IV. ENGINEERING ANALYSIS

Evaluation for the current live load carrying capacity of the structure was carried out with an Allowable Stress Rating approach in accordance with *The Manual for Bridge Evaluation (MBE), Second Edition (2010)*, including 2013 Interim Revisions. Items not explicitly covered in the MBE were in accordance with the 2002 AASHTO Standard Specifications for Highway Bridges. Additional engineering judgment was used in translating field observations to an analytical application for a reasonable load rating for the structure and its individual elements. Member capacities, dead load and live load stresses and resulting load ratings were determined using hand calculations and engineering software. Live load analysis was conducted on the basis of an H-15 truck or an 85 psf pedestrian loading.

We established inventory ratings using the H-15 truck (15 ton vehicle) to maintain consistency with past inspections and load ratings. Inventory ratings is the classification of the bridge in terms of the standard "H" loadings for bridges, and represents the safe load limit of the bridge for continued use. If an operating rating is mentioned in past reports, this represents a safe load carrying capacity for the maximum load that could pass over the bridge on an infrequent basis.

For pedestrian bridges, the standard design load has been 85 psf, along with a minimum of an H-5 Truck (5 ton vehicle) on bridges 6' to 10' wide, and H-10 truck (10 ton vehicle) on bridges wider than 10 feet.

Similar to Teng, and in agreement with the laboratory tests references, the allowable steel stresses were taken to be the same as those allowed for A-7 structural carbon steel (Fy = 33,000 psi). The allowable stresses in the timber stringers were taken as 1,200 psi.

SUPERSTRUCTURE

Floor System: No load rating was conducted for the current condition of the structural deck plate. Due to its deteriorated condition with local failures; and that any proposed repairs would be limited to complete replacement.

Stringers: The interior stringers are rated at H-6.36 using steel section reductions (50% top and bottom flanges) and full timber stringer capacity. If timber stringers are only considered, the rating drops to H-4.68. This rating accommodates a 152 psf deck loading. Additionally, the AASHTO deflection requirements of 1/800 of the span that the Teng report commented on are even further reduced with current estimates at 1/159; comparable to an H-1.25 rating.

The remaining original exterior 7" channel stringers are considered to have no capacity and rely entirely on the timber stringers which are rated at H-1.37. This is based on the AASHTO requirement that wheel loads are two feet from the edge of the roadway. If the assumption is made that the rating vehicle's wheel came to 1'-0" from the roadway edge, then the rating would drop to H-0.31. The replacement 6" I-beam stringers installed in 1953 have a rating of H-15.64 with no section reductions and H-3.37 when live load is assumed at one foot from edge. Based on the controlling timber stringers, a comparable pedestrian rating of only 21 psf can be used.

Floorbeams: The floorbeam rating is controlled by the 110' span and the reduced section observed at mid-span of the individual beam. Section loss (reduced area = 7.33 in^2) has reduced the rating to H-3.05 and a comparable 34 psf pedestrian loading. Due to the location of section loss being at mid-span, the full moment can be assumed in the rating computation. If the deterioration were to have been found elsewhere along the member (as was also the case) the moment used in the rating would be proportionally reduced. It should also be noted that several other floorbeams were found to have similar deteriorations. Original floorbeam design with no reductions, would amount to a total pedestrian rating of 115 psf.

Trusses: The overall load rating of the trusses resulted in a nominal capacity of H-3.5 and H-2.1, respectively for the 89' truss and 110' truss. Both ratings were controlled by the bottom chord members and the severe deterioration seen at the abutment bearing eyebar connections (B1-members, near 50% reduction see Figure 27 thru Figure 30).

Deterioration of other bottom chord eyebar connections were observed the most critical being on the B2 members of the 110' span with a corresponding rating of H-7.9.

Truss end-bearing capacity was an additional concern at the abutment bearings. Previously repaired in 1953, pin-plates that are welded to the channel webs are the sole connection to the bearing pins. Reductions for deterioration of the connection allow for an H-13.4 and H-10.4 rating of the 89' truss and 110' truss, respectively.

Another controlling rating was found on the V2 vertical member to the lower pin connection. Severe deterioration of the section and significant pack rust led to a reduced capacity that yielded an H-7.4 rating. The corrosion of this connection was found to be greater than that assumed in the Teng report, however, the truss analysis found differing stress levels in the member itself that approached 50%. Therefore, rather than the H-4.87 rating that Teng calculated even with a reduced member capacity, led to an increase in the load rating due to the decreased applied stress.

These results do not take into account several indeterminate factors that undoubtedly affect the load rating. The first of which is the flexibility and "looseness" of the structure. This is the result of a number of items associated with deteriorated connections, missing or not engaged lateral members, and elongation of steel over time. The most appropriate way to quantitate this is with an increased impact factor on the live load stress. This will further reduce the ratings. For example, the H-2.1 rating found on the 110' truss bottom chord member using the standard AASHTO 30% impact. This would further be reduced to H-1.4 with an impact of 100%. Teng also used this comparison in their report to better demonstrate the vibration susceptibility.

An additional allowance to possibly further reduced load ratings is the main diagonals that consist of two separate steel bars; only one of which is fitted with a turnbuckle. In every instance where this occurs, the bar not fitted with a turnbuckle is loose and not structurally engaged with the truss. The two bars can only participate equally if both members are engaged. Additional turnbuckles should have been installed. This fact essentially eliminates the second bar in the capacity of the overall member. If this assumption was made, a consequential load rating as low as H-1.4 could be found.

General: Specific wind and vibration analysis was not completed as well as specific deflection calculations that could affect the resultant load ratings. Teng noted certain members could become overstressed due to dead and wind loading, however, this was not found to be a critical concern.

SUBSTRUCTURE

No plans or specifications were available for the substructure elements. In lieu of a complete quantitative analysis for the substructure, a determination was made on the basis of engineering judgment. The absence of any visual structural deficiencies allows the conclusion to be made that the substructure is in acceptable condition to carry the superstructure.

SUMMARY OF STRUCTURE RATINGS

The overall controlling ratings are as follows:

EXTERIOR STRINGERS	7" Channel (timber ONLY)	6" S-stringer (steel replacement)
Pedestrian = 21 psf	*H-1.37*	*H-15.64*
INTERIOR STRINGERS	7" S-stringer + timber	7" S-stringer ONLY
Pedestrian = 152 psf	*H-6.36*	*H-4.68*
FLOORBEAMS	12" S-beam	
Pedestrian = 34 psf	*H-3.05*	
TRUSS	89'-0" Truss	110'-10" Truss
Bottom chord members	*H-3.5 | 23 psf*	*H-2.1 | 13 psf*

V. POTENTIAL SCOPE OF WORK DETERMINATION

In its current state of deterioration, the bridge is not safe for pedestrian or bicycle use. Local failures of the floor system are possible as is a complete structure collapse due to section loss observed at critical members. There are two avenues the bridge owner may take to return serviceable access to Sylvan Island; repairs or complete replacement. See Attachment C for a summarized cost estimate of both options.

REHABILITATION AND REPAIR OF EXISTING STRUCTURE

There are significant repairs that are proposed in order to return this bridge to a level of service acceptable to standard safety measures. A number of repairs noted by Teng in 1969 are still applicable today, however, significant deterioration over the past 44 years has added to the list. To start with, the bottom chord members at the abutment truss bearings that were found in critical condition need to be replaced. This would include the replacement of the bearing pins; rehab work to the bearing assembly itself; and any repairs/strengthening to the end post pin-plates. Following that, all the lower pin connections (verticals-to-floorbeams) should be replaced. At a minimum, the laced vertical member connections are severely corroded and existing pack rust poses a threat of pushing members off of the pins. The corrosion at this location also requires the repair and strengthening of the bottom portion of the laced verticals.

Complete floor system replacement including the steel decking with asphalt wearing surface and all steel stringers. Deficiencies of the deck that allowed for notable issues on the floorbeams back in 1969 have only worsened over time. Water leaking through the cracks in the decking has led to section loss of the top flange on all floorbeam members. This section loss now requires all floorbeams to have their top flanges reinforced. We assumed a similar deck system of pans and asphalt, or concrete, would be

installed, however, the City could consider lightweight grating or timber. The grating and timber would reduce dead loads and increase load ratings for certain items. Of the three in terms of upfront cost, grating is the most expensive option, followed by pans & concrete, then timber.

All truss diagonals are also recommended to be replaced due to unequal stressing. After such a long period of only one of the two bars in a two-bar configuration being stressed it is not recommended to simply install new turnbuckles. All single-bar diagonal members should be replaced as well and include a turnbuckle for proper tensioning. Portal frames on the upper portion of the truss ends along with all intermediate strut members along the length of the truss need to be replaced. Severe deterioration, pack rust accumulation, and fractured members were a constant observation. Additionally, all upper strut connections to the top chord members are essentially gone. Pack rust and section loss has not only affected the strut members, but has also caused local section loss to the top chord cover plate. Similar deterioration was found at the knee brace member connections. Both of these deficiencies require the top chord cover plate to be locally replaced. Repair efforts may even require select top pin replacements depending on the amount of corrosion found at the vertical-to-top chord gusset plates at the time of construction. Segments of the portal frames themselves previously replaced are in good condition, however, partial replacement would likely be more costly than a full portal frame replacement. Lateral cross-bracing members should be re-tensioned or replaced appropriately.

A complete cleaning and painting of the structure should also be considered, and has been included in the budget estimate. There are advantages and disadvantages to this process that will later be discussed.

The estimated construction cost of a rehabilitating the structure is $1,772,000.00. A detailed cost analysis for the option is included within this report, see Attachment C.

COMPLETE STRUCTURE REPLACEMENT

A complete replacement of the existing structure with a pre-fabricated, two-span truss is proposed. No matter the type and size of the replacement structure, significant work is entailed with removing the existing truss. Due to the current deteriorated condition, labor intensive efforts are also required to safely remove the bridge and maintain environmental standards. Removal is not as simple as blowing the bridge up and picking up the remains from the channel. Hand removal of the floor system is even required because the bridge cannot structurally handle equipment loads. When removing the truss, the preferred option would be to pick each of the truss spans and place on either a barge or move to the shoreline prior to disassembly. Each option requires the construction of a causeway across the channel width for access and equipment accommodation.

Once the existing structure is removed, several modifications and possible repairs to the existing substructure would be made to accommodate the new superstructure. The new pre-fabricated truss would then be swung into place from the causeway; assuming the proposed pre-fabricated truss is selected as the replacement structure. A new deck surface would then be installed; approach reconstruction completed; and park bike path and landscaping restored. It is assumed for estimating purposes, that the replacement structure would be a two-span truss with a 12’ clear width.

A consideration to keep in mind is that replacement costs are apt to be elevated due to basic logistic issues at the site. These issues would include: the channel depth to bedrock creating shoring concerns; the proximity of MidAmerican property and the power lines on three sides of the site; construction access through the city-owned park up to the bridge with temporary bike path relocation; and finally, shoreline modifications to gain access for causeway construction and use. There is also the possibility for environmental mitigation; if for example, a mussel bed is determined to be in proximity to the site requiring relocation and monitoring efforts.

The estimated construction cost of a complete replacement structure is $1,183,400.00. A detailed cost analysis for the option is included within this report, see Attachment C.

VI. DISCUSSION OF RECOMMENDED OPTIONS

REHABILITATION & REPAIR

This option is acceptable, but not the preferred option due to high cost versus the remaining life expectancy of the 112 year old structure. Even with the extensive repairs and rehabilitation recommended in this report to bring the bridge back into service, we would anticipate the City would have long term maintenance and repair obligations for the remaining life of the bridge.

We have identified critical areas that need to be repaired or replaced, which was determined from a visual hands on inspection. It is our experience on older structures, that once construction starts and members are cleaned, connections taken apart, etc., the scope of work will creep, and in some cases can add significant cost to the project.

Painting of this structure would not normally be recommended due to high cost versus extended life benefits. In essence, painting will not cost effectively add service life to the bridge. That being said, several areas on the structure do need to be blast cleaned in order to determine additional deterioration and possible additional repairs, which adds to the concern mentioned above about scope creep. With partial areas requiring cleaning, as well as new steel repairs needing corrosion protection, we would recommend the entire structure be cleaned and top coated for consistency.

There may be an element of historical aesthetics that may be beneficial to the City for the end user. This is an old bridge that fits in well with its surroundings, though more expensive to maintain, may be worth the extra cost to the City. That being said, aesthetics and historical significance determinations can be very subjective, thus, beyond the scope of this report and any recommendations we may make.

COMPLETE REPLACEMENT

This is the preferred option. Due to the existing substructures being sound, they would only require minor modifications and repairs to accommodate a new superstructure. By going with a new superstructure, the anticipated life span would approach 75 years with reduced maintenance costs, especially, in the first 25 years. The new bridge would also be more durable, designed to today's standards and load requirements, thus giving a peace of mind to the City that would be difficult to obtain with rehabilitating a 112 year bridge. This option is also the most economical option as compared to rehabilitation & repair.

VII. CONCLUSIONS AND RECOMMENDATIONS

CONCLUSION

After inspecting this bridge and noting the extensive deterioration that has occurred, Missman concurs that closing this bridge to all access was an appropriate decision made by the City in April of 2013. Several members on both spans show significant corrosion, which has reduced the capacity of the bridge to safely carry pedestrian and bicycle traffic. One specific area of serious concern to note, are the bottom chord members near the abutments. These tie bars have over 50% section loss, and are a main load carrying members of the trusses. These are primary tension members, and are considered fracture critical. By definition, if any one of these members were to fail, there would be a catastrophic failure to the structure.

Per Teng & Associates report in 1969, they had serious concerns about the safety of the bridge, which led to the structure being closed to all vehicle traffic. A direct quote from the report: "*The fact that no catastrophic failure has occurred so far must be attributed to the load distributing effect of the corrugated steel deck and most likely also the absence of heavy eccentric loading. These conditions, however, cannot be relied upon indefinitely*." It has been 44 years since this report was completed, and with little to no maintenance completed in that time, Missman would add that same quote in this report. Teng still had observed paint on the structure in 1969, but in 44 years, that paint system is gone and significant surface and pack rust has occurred. The surface rust on the main load carrying members of the trusses are not as much of a concern, as the severe pack rust that has developed in the connections. Pack rust can hide serious section loss and cracks. Some of these connections where this has occurred, specifically, at the bottom chord/floor beam locations, are of a critical nature where a failure could lead to a minimum of a partial deck failure, or worse, a complete span failure. Pack rust can also exert prying/wedging forces in directions the connection was not designed for, thus causing another concern to the actual capacity of the connection. To note, Missman made no attempt to clean the connections, and made assumptions to the amount of section loss in our ratings and that no cracks were present.

Another area of concern is the amount of flexibility the two spans exhibit. As noted in 1969, a single man jumping up and down on the bridge deck can cause the bridge to vibrate/shake. Missman inspectors also checked this, and determined it to be true. It is our understanding that there are annual bicycle and running events held around the Sylvan Island recreation area. Groups of runners can cause unusual vibrations in a structure, and that is a concern on these old shaky spans.

If the extensive list of proposed repairs is made to the structure it can be conservatively assumed that the bridge will return to a rating condition as originally constructed in 1901. Stringers would be assumed at H-8.9 (212 psf) and H-11.3 (170 psf), respectively, for the interior and exterior members. Floorbeams would conservatively be rated at H-10.3 (115 psf); assuming no additional capacity from proposed re-plating during rehabilitation. The controlling rating would be governed by the truss and the bottom chord members at mid-span with ratings of H-13.0 (83 psf) and H-11.7 (70 psf), for the 89' and 110' truss spans, respectively. These results are acceptable, with the exception of the controlling truss rating which is below the pedestrian loading requirement of 85 psf set by AASHTO.

RECOMMENDATIONS

Because of the bottom chord critical finding, along with several other issues, Missman recommends that this bridge remain closed. If the City decides to save the structure, it should not be reopened until an extensive rehabilitation plan is developed and constructed.

Due to economics and long term durability, we would encourage the City of Moline to consider the complete replacement option. We have assumed pre-engineered trusses for this report and our cost estimates, but there are other types available that may offer different aesthetics or recreational uses.

In closing, we recommend that the existing structure be removed in its entirety, and replaced with a new superstructure sitting on the existing substructures.

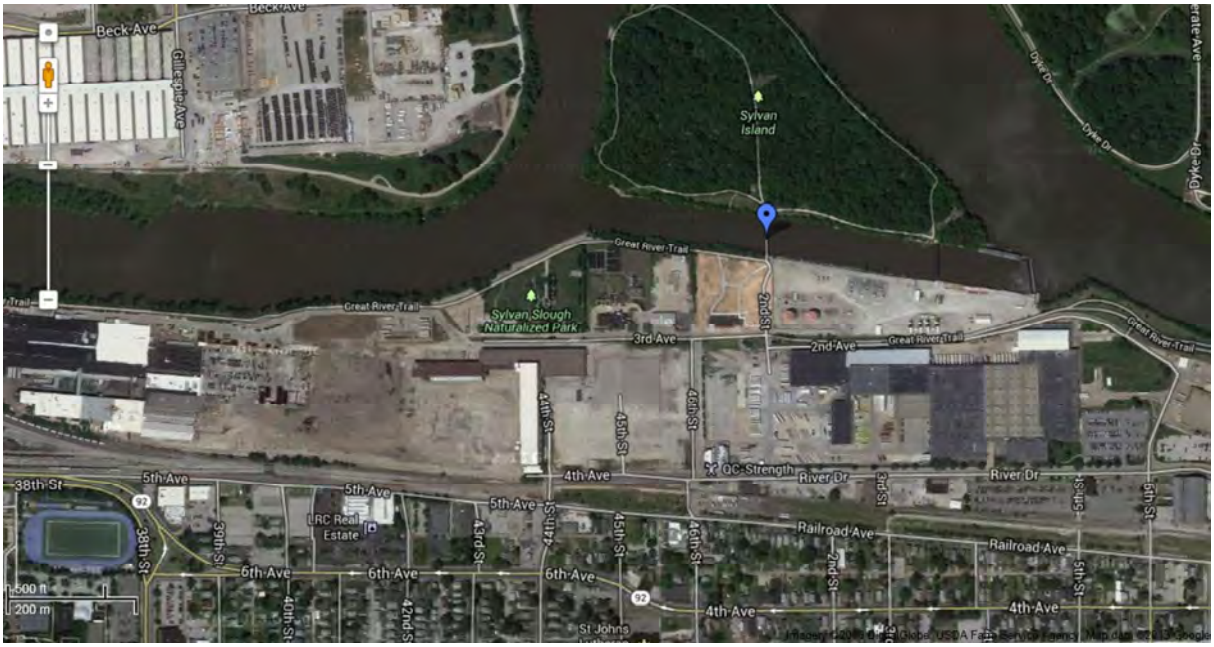
ATTACHMENTS

- **Attachment A: Location Map**
- **Attachment B: Structural Assessment Summary**
- **Attachment C: Cost Estimates**
- **Attachment D: Structure Photos**
- **Attachment E: General Plans & Member Labeling**

ATTACHMENT A: LOCATION MAP

(CITY, COUNTY, STATE): Moline, Rock Island County, Illinois

Descriptive Location: Sylvan Slough Crossing to Sylvan Island. Near Rock Island-Moline City Limits



(maps.google.com)

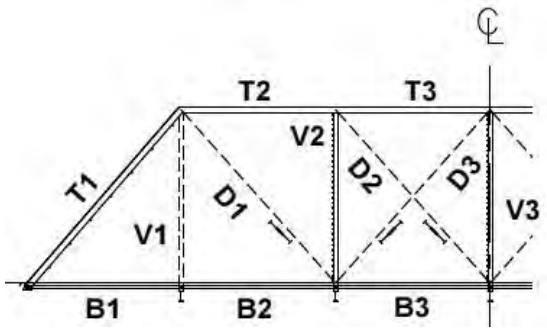
Prepared by:

Missman, Inc.
Professional Engineers & Land Surveyors

ATTACHMENT B: STRUCTURAL ASSESSMENT SUMMARY

89'-0" Truss

CURRENT CONDITION (w/ Reductions)							
					H-15 Live Load		
Member	Total Reduced Capacity (kip)	Governing Condition	DL Stress (kip)	LL Capacity (kip)	LL (H-15) Stress (kip)[1]	Inventory Rating Factor	Inventory Rating (H-15)
T1	-52.65	*BEARING ON PIN*	-21.5	-31.2	-34.8	0.89	H-13.4
T2	-67.28	" "	-22.5	-44.8	-30.9	1.45	H-21.7
T3	-57.41	*SPLICE PLATE*	-25.0	-32.4	-33.8	0.96	H-14.4
B1	18.59	*NET AREA*	14.0	4.6	19.5	***0.24***	H-3.5
B2	31.10	" "	14.0	17.1	19.5	0.88	H-13.2
B3	49.50	" "	22.9	26.6	30.7	0.87	H-13.0
V1	27.56	" "	6.5	21.1	19.0	1.11	H-16.6
V2	-11.29	*CONNECTION*	-3.0	-8.3	-7.5	1.10	H-16.5
V3	-11.29	" "	-0.7	-10.6	-1.6	6.79	H-101.8
D1	35.44	*NET AREA*	13.0	22.4	23.7	0.95	H-14.2
D2	20.25	" "	3.8	16.5	10.3	1.60	H-24.0
D3	7.95	" "	-0.5	8.5	4.4	1.91	H-28.7

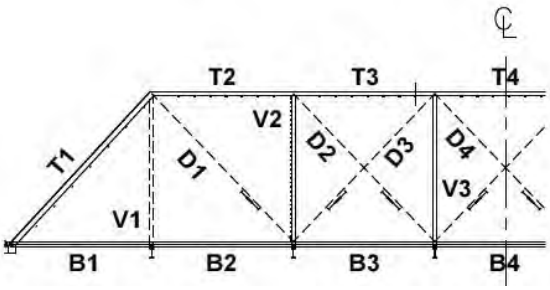


ORIGINALLY CONSTRUCTED CONDITION				
Member	Total Capacity (kip)	LL Capacity (kip)	Inventory Rating Factor	Inventory Rating (H-15)
T1	-68.44	-46.9	1.35	H-20.2
T2	-67.28	-44.8	1.45	H-21.7
T3	-57.41	-32.4	0.96	H-14.4
B1	35.13	21.1	1.08	H-16.3
B2	31.10	17.1	0.88	H-13.2
B3	49.50	26.6	***0.87***	H-13.0
V1	27.56	21.1	1.11	H-16.6
V2	-25.65	-22.7	3.00	H-45.1
V3	-25.65	-25.0	15.99	H-239.9
D1	35.44	22.4	0.95	H-14.2
D2	20.25	16.5	1.60	H-24.0
D3	7.95	8.5	1.91	H-28.7

ATTACHMENT B: STRUCTURAL ASSESSMENT SUMMARY

110'-10" Truss

CURRENT CONDITION (w/ Reductions)							
					H-15 Live Load		
Member	Total Reduced Capacity (kip)	Governing Condition	DL Stress (kip)	LL Capacity (kip)	LL (H-15) Stress (kip)[1]	Inventory Rating Factor	Inventory Rating (H-15)
T1	-58.50	*PIN PLATE CONN.*	-29.6	-28.9	-41.9	0.69	H-10.4
T2	-76.49	" "	-33.6	-42.9	-40.6	1.06	H-15.9
T3	-114.82	*L/r*	-39.5	-75.3	-46.4	1.62	H-24.3
T4	-114.80	" "	-40.9	-73.9	-48.5	1.52	H-22.9
B1	23.62	*NET AREA*	20.1	3.5	24.7	***0.14***	H-2.1
B2	33.12	" "	20.1	13.0	24.7	0.53	H-7.9
B3	67.50	" "	35.0	32.5	41.7	0.78	H-11.7
B4	81.00	" "	39.7	41.3	46.9	0.88	H-13.2
V1	27.56	" "	7.2	20.4	19.0	1.07	H-16.1
V2	-11.29	*CONNECTION*	-6.0	-5.3	-10.8	0.49	H-7.4
V3	-11.29	" "	0.2	-11.5	-3.0	3.84	H-57.6
D1	45.00	*NET AREA*	19.7	25.3	29.8	0.85	H-12.7
D2	27.56	" "	8.8	18.8	14.7	1.28	H-19.2
D3	10.12	" "	1.0	9.1	3.1	2.92	H-43.8
D4	18.00	" "	0.7	17.3	7.2	2.42	H-36.3



ORIGINALLY CONSTRUCTED CONDITION				
Member	Total Capacity (kip)	LL Capacity (kip)	Inventory Rating Factor	Inventory Rating (H-15)
T1	-85.5	-55.9	1.34	H-20.0
T2	-80.5	-46.9	1.16	H-17.3
T3	-114.8	-75.3	1.62	H-24.3
T4	-114.8	-73.9	1.52	H-22.9
B1	40.5	20.4	0.83	H-12.4
B2	40.5	20.4	0.83	H-12.4
B3	67.5	32.5	***0.78***	H-11.7
B4	81.0	41.3	0.88	H-13.2
V1	27.6	20.4	1.07	H-16.1
V2	-25.7	-19.7	1.82	H-27.3
V3	-25.7	-25.9	8.65	H-129.7
D1	45.0	25.3	0.85	H-12.7
D2	27.6	18.8	1.28	H-19.2
D3	10.1	9.1	2.92	H-43.8
D4	18.0	17.3	2.42	H-36.3

ATTACHMENT B: STRUCTURAL ASSESSMENT SUMMARY

Associated equivalent pedestrian loading

89 ft Truss

CURRENT CONDITION (w/ Reductions)								ORIGINALLY CONSTRUCTED			
					Pedestrian Live Load						
Member	Total Reduced Capacity (kip)	Governing Condition	DL Stress (kip)	LL Capacity (kip)	LL (Ped.) Stress (kip)^2	Inventory Rating Factor	Inventory Rating (psf)	Total Capacity (kip)	LL Capacity (kip)	Inventory Rating Factor	Inventory Rating (psf)
T1	-60.11	*BEARING ON PIN*	-21.5	-38.6	-26.2	1.47	125	-68.44	-46.9	1.79	152
T2	-67.28	*" "*	-22.5	-44.8	-26.8	1.67	142	-67.28	-44.8	1.67	142
T3	-57.41	*SPLICE PLATE*	-25.0	-32.4	-29.7	1.09	93	-57.41	-32.4	1.09	93
B1	18.59	*NET AREA*	14.0	4.6	17.2	***0.27***	23	35.13	21.1	1.23	104
B2	31.10	*" "*	14.0	17.1	17.2	0.99	85	31.10	17.1	0.99	85
B3	49.50	*" "*	22.9	26.6	27.4	0.97	83	49.50	26.6	***0.97***	83
V1	27.56	*" "*	6.5	21.1	8.6	2.45	208	27.56	21.1	2.45	208
V2	-11.29	*CONNECTION*	-3.0	-8.3	-3.2	2.59	220	-25.65	-22.7	7.08	602
V3	-11.29	*" "*	-0.7	-10.6	1.2	8.83	750	-25.65	-25.0	20.79	1,767
D1	35.44	*NET AREA*	13.0	22.4	14.7	1.53	130	35.44	22.4	1.53	130
D2	20.25	*" "*	3.8	16.5	4.3	3.83	325	20.25	16.5	3.83	325
D3	7.95	*" "*	-0.5	8.5	-0.7	12.07	1,026	7.95	8.5	12.07	1,026

110 ft Truss

CURRENT CONDITION (w/ Reductions)								ORIGINALLY CONSTRUCTED			
					Pedestrian Live Load						
Member	Total Reduced Capacity (kip)	Governing Condition	DL Stress (kip)	LL Capacity (kip)	LL (Ped.) Stress (kip)^2	Inventory Rating Factor	Inventory Rating (psf)	Total Capacity (kip)	LL Capacity (kip)	Inventory Rating Factor	Inventory Rating (psf)
T1	-36.58	*PIN PLATE CONN.*	-29.6	-7.0	-34.3	0.20	17	-85.5	-55.9	1.63	139
T2	-76.49	*" "*	-33.6	-42.9	-38.0	1.13	96	-80.5	-46.9	1.23	105
T3	-114.82	*L/r*	-39.5	-75.3	-44.8	1.68	143	-114.8	-75.3	1.68	143
T4	-114.80	*" "*	-40.9	-73.9	-46.3	1.60	136	-114.8	-73.9	1.60	136
B1	23.62	*NET AREA*	20.1	3.5	23.3	***0.15***	13	40.5	20.4	0.88	74
B2	33.12	*" "*	20.1	13.0	23.3	0.56	47	40.5	20.4	0.88	74
B3	67.50	*" "*	35.0	32.5	39.2	0.83	70	67.5	32.5	***0.83***	70
B4	81.00	*" "*	39.7	41.3	45.1	0.92	78	81.0	41.3	0.92	78
V1	27.56	*" "*	7.2	20.4	9.2	2.21	188	27.6	20.4	2.21	188
V2	-11.29	*CONNECTION*	-6.0	-5.3	-7.0	0.76	64	-25.7	-19.7	2.81	239
V3	-11.29	*" "*	0.2	-11.5	0.4	28.73	2,442	-25.7	-25.9	64.63	5,493
D1	45.00	*NET AREA*	19.7	25.3	21.6	1.17	100	45.0	25.3	1.17	100
D2	27.56	*" "*	8.8	18.8	9.6	1.95	166	27.5	18.7	1.95	166
D3	10.12	*" "*	1.0	9.1	-1.3	7.02	596	10.1	9.1	7.02	596
D4	18.00	*" "*	0.7	17.3	-0.8	21.63	1,838	18.0	17.3	21.63	1,838

ATTACHMENT C: COST ESTIMATES

Sylvan Slough Truss Bridge Repairs

Budgetary Bridge Repair Estimate

The existing 1901 truss bridge would have repairs made to return bridge servicibility back to an acceptable standard.

ITEM	Quantity	Unit	Unit Price	Item Total
1. Replace all Lower Pin Connections.....	30.0	EA	$ 4,500.00	$ 135,000.00
Includes repairs to lower truss verticals Pins, U-bars, etc.				
2. Rehabilitate Abutment Bearings.....	4.0	EA	$ 3,200.00	$ 12,800.00
3. Replace select Bottom Chord Members.....	4.0	EA	$ 17,800.00	$ 71,200.00
4. Existing Deck & Stringer Removal.....	1.0	LS	$ 62,000.00	$ 62,000.00
5. Floorbeam Strengthening.....	11.0	EA	$ 4,000.00	$ 44,000.00
6. Replace all Truss Diagonal Members.....	1.0	LS	$ 130,000.00	$ 130,000.00
7. Replace Truss Portal Frames & Upper Struts.....	1.0	LS	$ 86,000.00	$ 86,000.00
Includes Repairs to Top Chord Cover Plates				
8. New Concrete Deck & Stringers.....	2,400.0	SF	$ 75.00	$ 180,000.00
9. Minor Substructure Repairs.....	1.0	LS	$ 20,000.00	$ 20,000.00
10. Mobilization.....	1.0	LS	$ 165,000.00	$ 165,000.00
11. Complete Bridge Cleaning & Painting.....	1.0	LS	$ 275,000.00	$ 275,000.00
			Construction Subtotal =	**$ 1,181,000.00**
			Contigency (20%)	$ 236,000.00
			CONSTRUCTION TOTAL =	**$ 1,417,000.00**
			Engineering Services (15%)	$ 213,000.00
			Construction Observation (10%)	$ 142,000.00
			ESTIMATED PROJECT COST =	**$ 1,772,000.00**

Notes:

Prepared by:

Missman, inc.
Professional Engineers & Land Surveyors

ATTACHMENT C: COST ESTIMATES

Sylvan Slough Truss Bridge Replacement

Budgetary Bridge Replacement Estimate

The existing 1901 truss bridge would be removed, existing substructure rehabilitated, and a new superstructure constructed. The new structure is anticipated at either a 10' or 12' width and designed for pedestrian loading.

ITEM	Quantity	Unit	Unit Price	10' Wide Item Total	12' Wide Item Total
REMOVAL OF EXISTING STRUCTURE					
1. Removal of Existing Structure	1.0	L.S.	$ 200,000.00	$ 200,000.00	$ 200,000.00
CONSTRUCTION ACCESS					
2. Clearing & Grubbing (Stream bank)	1.0	L.S.	$ 5,000.00	$ 5,000.00	$ 5,000.00
3. Construction Access (causeway)	1.0	L.S.	$ 120,000.00	$ 120,000.00	$ 120,000.00
4. Bike Path Re-route	1.0	L.S.	$ 5,000.00	$ 5,000.00	$ 5,000.00
PERMITTING					
5. 404 Permit Administration	1.0	L.S.	$ 5,000.00	$ 5,000.00	$ 5,000.00
6. *Mussel Survey	1.0	L.S.	$ -	$ -	$ -
REPLACEMENT STRUCTURE					
7. Substructure Rehabilitation	1.0	L.S.	$ 20,000.00	$ 20,000.00	$ 20,000.00
8. Approach Reconstruction	1.0	L.S.	$ 5,000.00	$ 5,000.00	$ 5,000.00
10' WIDE BRIDGE					
9.A Pre-Engineered Truss (10' wide)	1,990.0	S.F.	$ 175.00	$ 348,250.00	$ -
12' WIDE BRIDGE					
9.B Pre-Engineered Truss (12' wide)	2,388.0	S.F.	$ 175.00	$ -	$ 417,900.00
SURFACE RESTORATION					
10. Surface & Stream Bank Restoration	1.0	L.S.	$ 25,000.00	$ 25,000.00	$ 25,000.00
11. Bike Path Reconstruction	1.0	L.S.	$ 13,500.00	$ 13,500.00	$ 13,500.00
MISCELLANEOUS					
12. **Utility Mitigation	1.0	L.S.	$ 5,000.00	$ 5,000.00	$ 5,000.00
13. ***Mobilization *0.0%*	1.0	L.S.	$ -	$ -	$ -
			Construction Subtotal =	**$ 751,750.00**	**$ 821,400.00**
			Contigency (20%)	$ 150,000.00	$ 164,000.00
			CONSTRUCTION TOTAL =	**$ 901,750.00**	**$ 985,400.00**
			Engineering Services (10%)	$ 90,000.00	$ 99,000.00
			Construction Observation (10%)	$ 90,000.00	$ 99,000.00
			ESTIMATED PROJECT COST =	**$ 1,081,750.00**	**$ 1,183,400.00**

Notes:

* Does not include mitigation, relocation, and monitoring efforts if a mussel bed is actually found.

** No communication with utility company, this may be more involved depending on easements.

*** Mobilization is included elsewhere within the pay items.

Prepared by:

Missman, inc.
Professional Engineers & Land Surveyors

ATTACHMENT D: STRUCTURE PHOTOS

Comments:
Bridge was closed to all access in April 2013. Fenced at both ends.

Figure 1: Bridge/Park Entrance from Moline (IMG_0626)

Comments:
Mississippi River at Flood Stage presently
(15.00 ft. @ Rock Island)

Figure 2: West Elevation View (RT-Moline; LT-Sylvan Island) (IMG_0328)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

Figure 3: West Elevation, 110' Truss (IMG_0326)

Comments:

Figure 4: West Elevation, 89' Truss (IMG_0327)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:
MidAmerican owned property to the left.

Figure 5: From bridge, looking upstream (IMG_0187)

Comments:
Moline – Left
Sylvan Island - Right

Figure 6: From bridge, looking downstream (IMG_0188)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

Figure 7: Sylvan Island Abutment (IMG_0231)

Comments:

Figure 8: Moline Abutment (IMG_0324)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:
Squared masonry stone, found to be vertical and sound. Joints appeared sound with very few surface cracks.

Figure 9: North Pier Elevation (IMG_0224)

Comments:

Figure 10: South Pier Elevation (IMG_0325)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

Figure 11: East Elevation & Sylvan Island Shoreline (IMG_0709)

Comments:

Figure 12: West Elevation & Moline Shoreline (IMG_0711)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

Figure 13: South Abutment & SE Wingwall Elevation (IMG_0707)

Comments:

Figure 14: North Abutment & NW Wingwall Elevation (IMG_0713)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

Figure 15: West Elevation Truss-Pier (IMG_0331)

Comments:

Figure 16: Typical Deck-Stringer-Floorbeam Configuration (IMG_0149)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

Figure 17: Floor System – Span 1-2 (IMG_0087)

Comments:

Figure 18: Stringer C1-2 @ FB-2 (IMG_0100)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

Figure 19: Stringer G2-3 (IMG_0118)

Comments:

Figure 20: Stringer A2-3 @ FB-2 (IMG_0105)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:
Stringer B3-4, Span 3-4
7" I (S7x15.3)
Flange Thickness = 0.392
FIELD: t_{tf} = 15% loss (0.33")
t_{bf} = 0.36"

Figure 21: Stringer B3-4 (IMG_0142)

Comments:
Stringer F6-7, Span 6-7
7" I (S7x15.3)

- Top flange is completely corroded away.

Stringer F7-8, Span 7-8
7" I (S7x15.3)

- Entire stringer is rotated out of plumb due to severe pack rust at bearing. Stringer in general has 50% total section loss due to severe corrosion.

Figure 22: Stringer F6-7 at FB-7 (IMG_0209)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:
Floorbeam top flange section loss.

Figure 23: Floorbeam FB-5 East Top Flange at Stringer (IMG_0186)

Comments:
Severely deteriorated stringer members.

Figure 24: Stringer F10-11 at mid-span (IMG_0277)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:
No stringer bearing and holes in decking material.

NOTE THE DAYLIGHT

Figure 25: Stringer B13-14 at FB-13 (IMG_0307)

Comments:
Holes in decking material.

NOTE THE DAYLIGHT

Figure 26: Stringer A13-14 at South Abut. (IMG_0306)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

CRITICAL FINDING
Bottom chord section loss

Figure 27: NW Truss Brg. at North Abut. (IMG_0232)

Comments:

CRITICAL FINDING
Bottom chord section loss

Figure 28: NW Truss Brg. Detail at North Abut. (IMG_0233)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

CRITICAL FINDING
Bottom chord section loss

Tree growing out of bearing assembly

Figure 29: Stringer G1-2 at North Abut. (IMG_0241)

Comments:

CRITICAL FINDING
Bottom chord section loss

Figure 30: East Truss Brg. Detail at South Abut. (IMG_0316)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

Pack rust affecting capacity of connection

Figure 31: West pin connection at FB-6 (IMG_0198)

Comments:

Severe pack rust affecting member and connection capacity.

Figure 32: East pin connection at FB-10 (IMG_0272)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

Pack rust

Figure 33: East pin connection at FB-11 (IMG_0281)

Comments:

Pack rust

Figure 34: West pin connection at FB-11 (IMG_0284)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

Figure 35: North bridge span top structure framing looking south (IMG_0548)

Comments:
Typical pack rust and deterioration of top chord cover plate at strut connections.

Figure 36: East pin connection at V-2 South span (IMG_0370)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:
Typical section loss of strut ends at bearing/connection

Figure 37: East pint connection at V-3 South span (IMG_0386)

Comments:
Pack rust and top chord cover plate failure at vertical connection.

Note the loose bolts at gusset plate connection to vertical member. Severe corrosion on interior face of connection.

Figure 38: East pin connection at V-3 looking west South span (IMG_0389)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:
See Figure 38.

Figure 39: East pin connection at V-3 looking west South span (IMG_0392)

Comments:

Figure 40: West pin connection at V-4 looking west South span (IMG_0413)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:
89' Truss, northeast upper pin connection at end post

Figure 41: East pin connection at U-5 (IMG_0422)

Comments:
110' Truss, southeast upper pin connection at end post

Figure 42: East pin connection at U-6 (IMG_0442)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

Figure 43: East pin connection at U-8 (IMG_0478)

Comments:

Figure 44: East pin connection at U-9 (IMG_0498)

ATTACHMENT D: STRUCTURE PHOTOS

Comments:

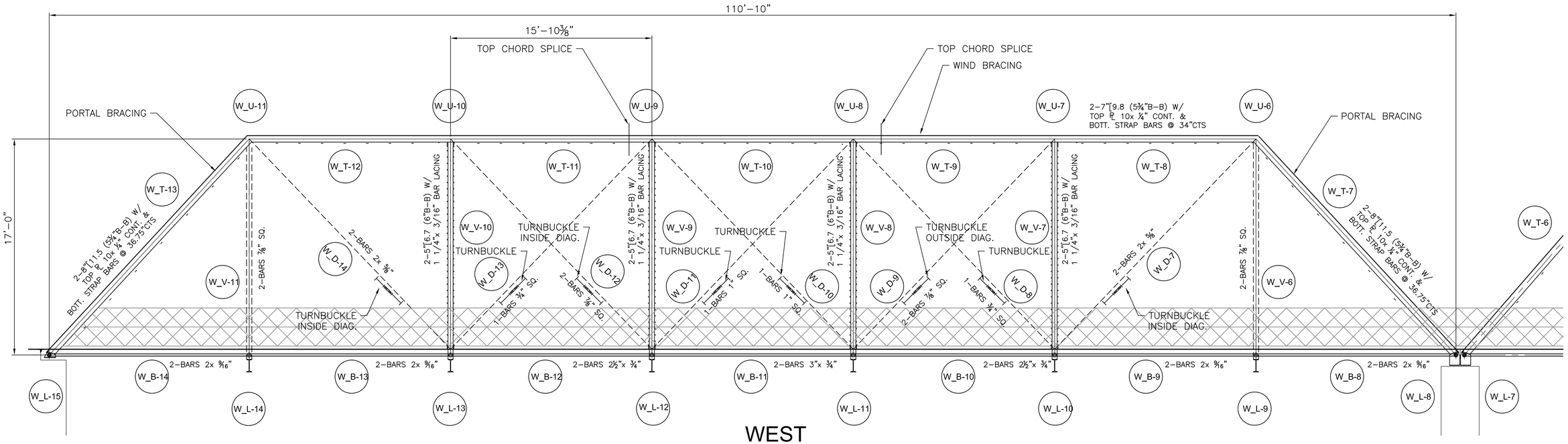
Pack rust expansion observed ¼" to 1½"

Figure 45: Portal 3 angle connection to diag. strut (IMG_0419)

Comments:

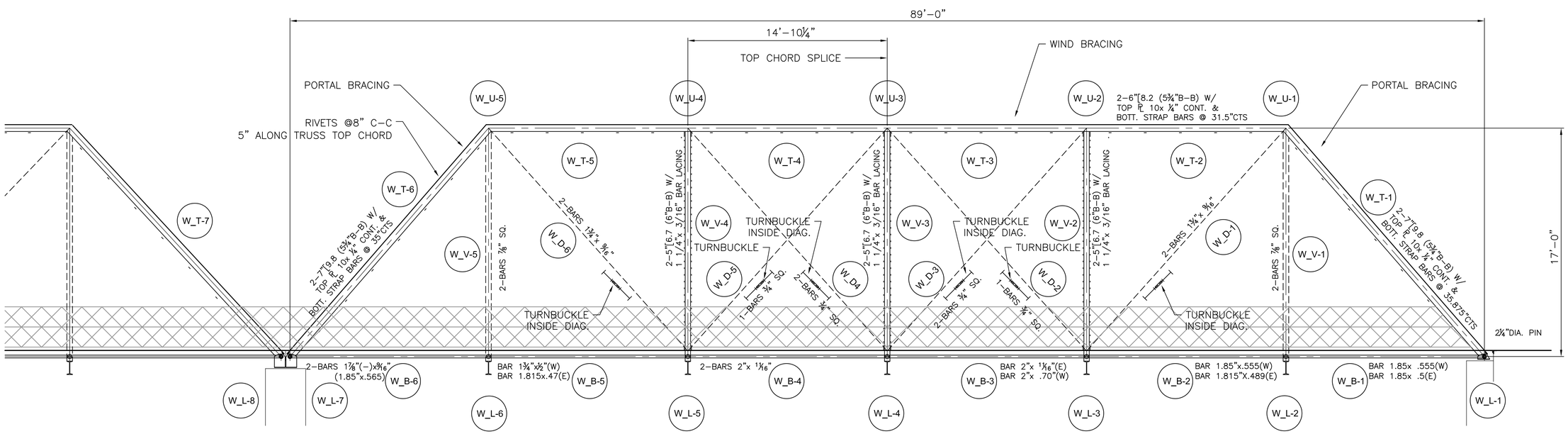
Fractured double angle segment of portal frame at end post connection

Figure 46: Portal 1 diag. strut connection to West T-13 (IMG_0542)



WEST

WEST ELEVATION - NORTH TRUSS



WEST

WEST ELEVATION - SOUTH TRUSS

ATTACHMENT E

IL DESIGN FIRM NO. 184-000843

FILE NAME =	USER NAME =	DESIGNED —	REVISED —
		DRAWN —	REVISED —
	PLOT SCALE =	CHECKED —	REVISED —
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F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
		CONTRACT NO.		

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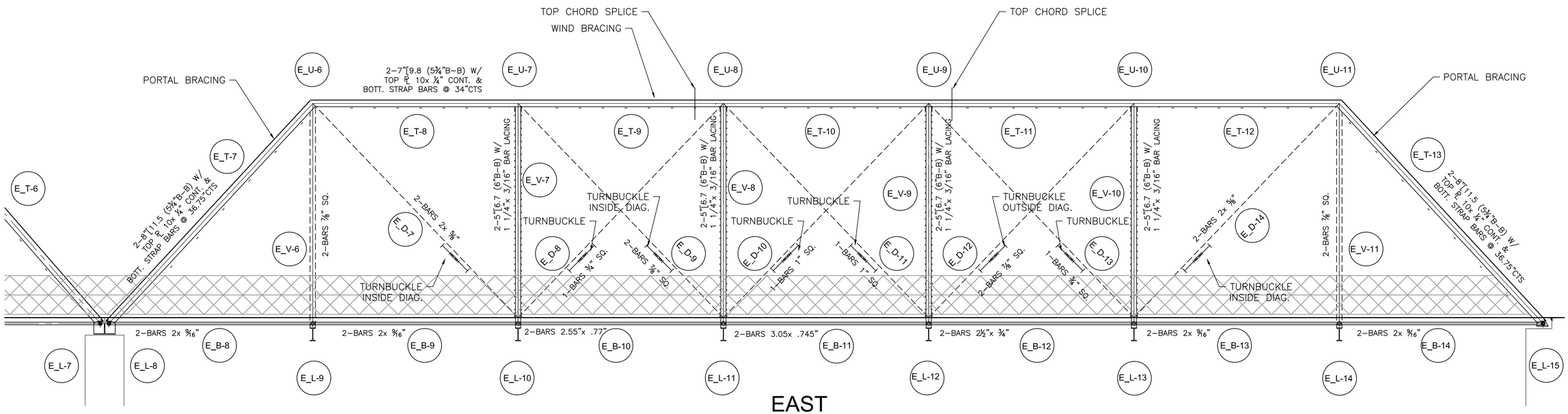
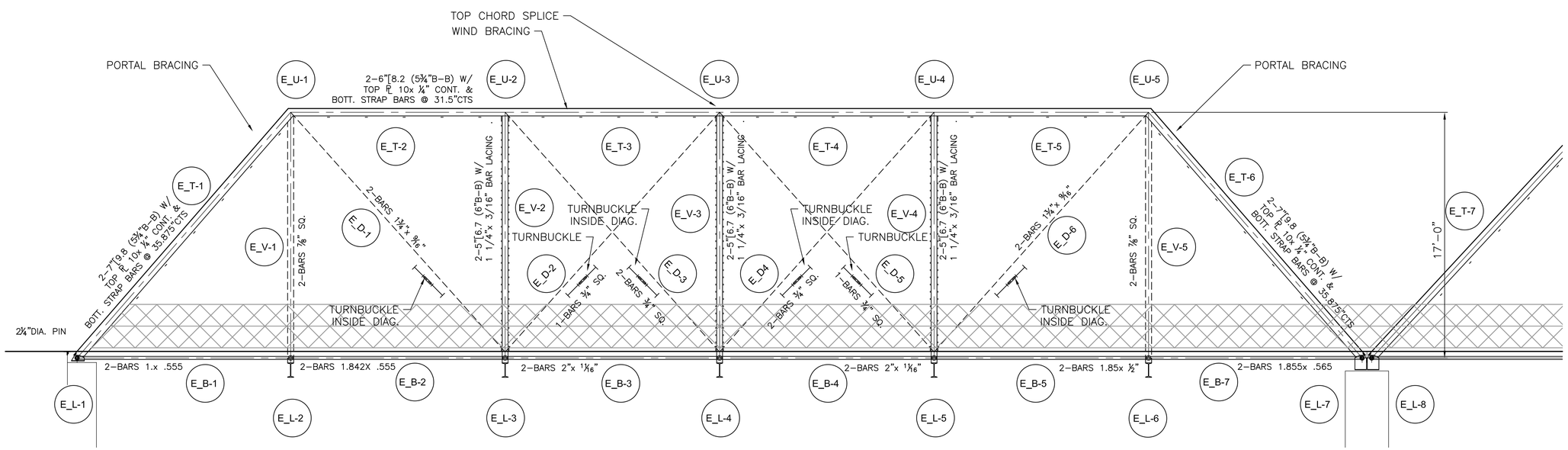


EAST

EAST ELEVATION - NORTH TRUSS



EAST

EAST ELEVATION - SOUTH TRUSS

ATTACHMENT E

IL DESIGN FIRM NO. 184-000843

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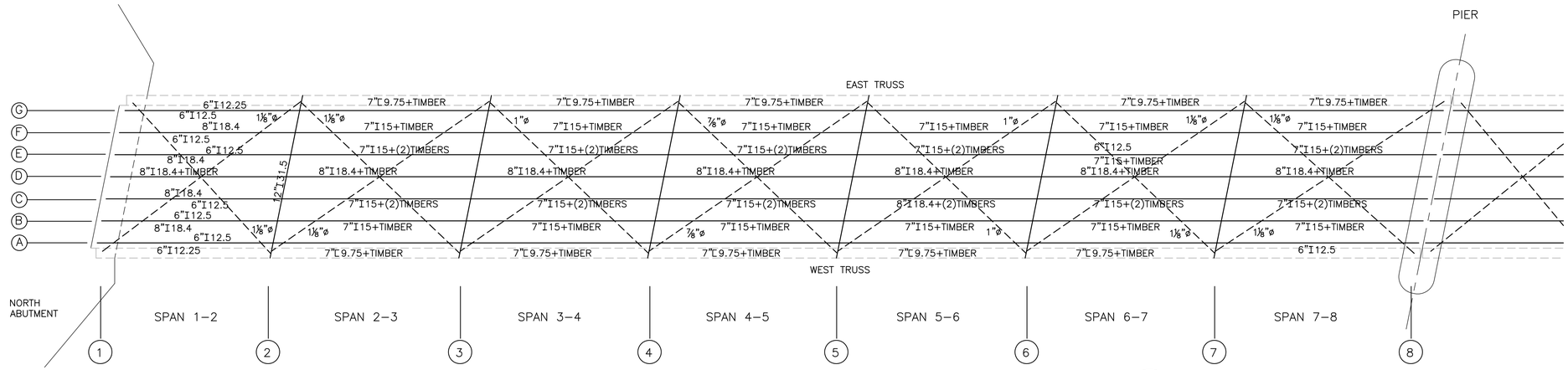
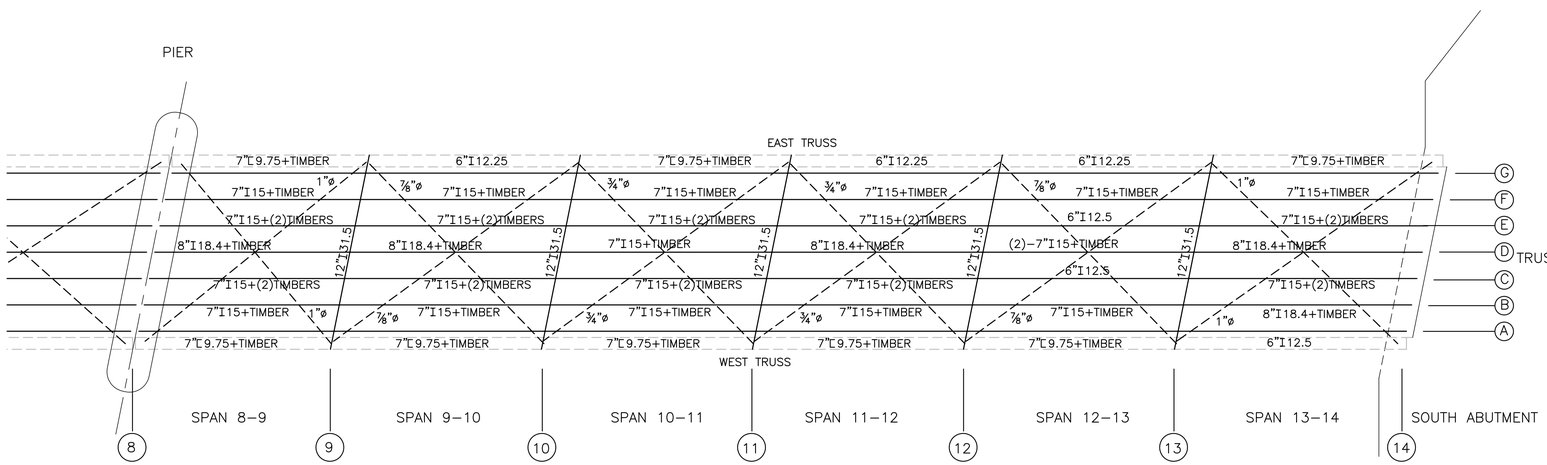
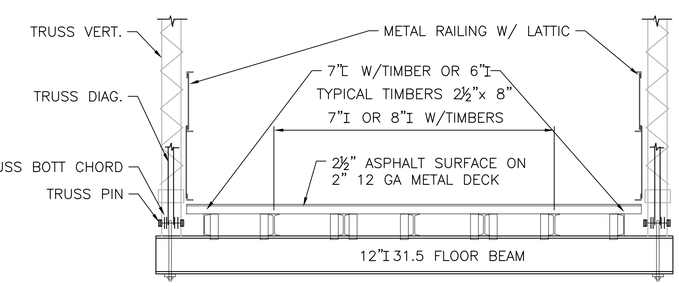


FLOOR BEAM FRAMING PLAN - NORTH 110' TRUSS SPAN



FLOOR BEAM FRAMING PLAN - SOUTH 89' TRUSS SPAN



TYPICAL SECTION BRIDGE

ATTACHMENT E

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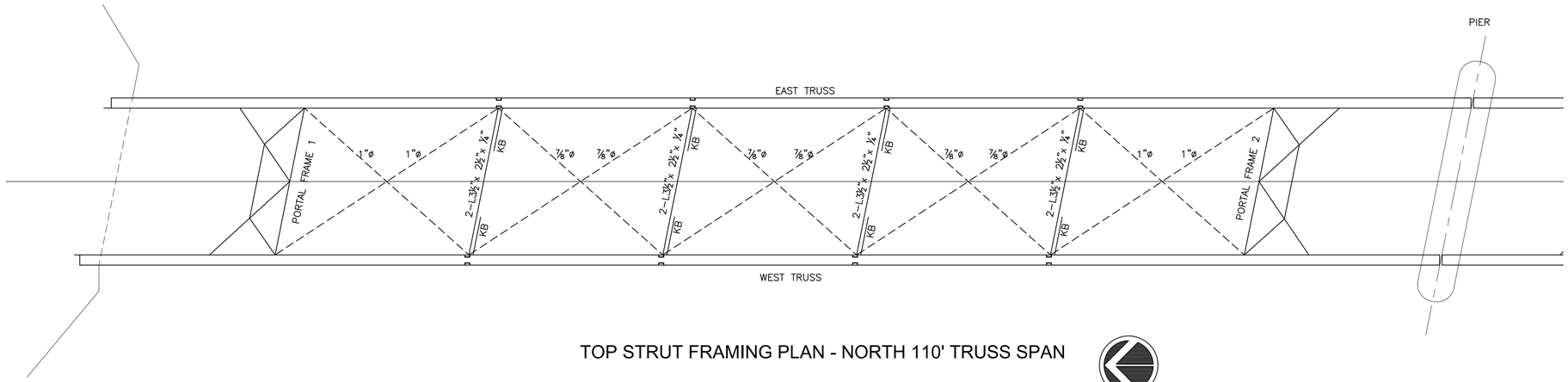
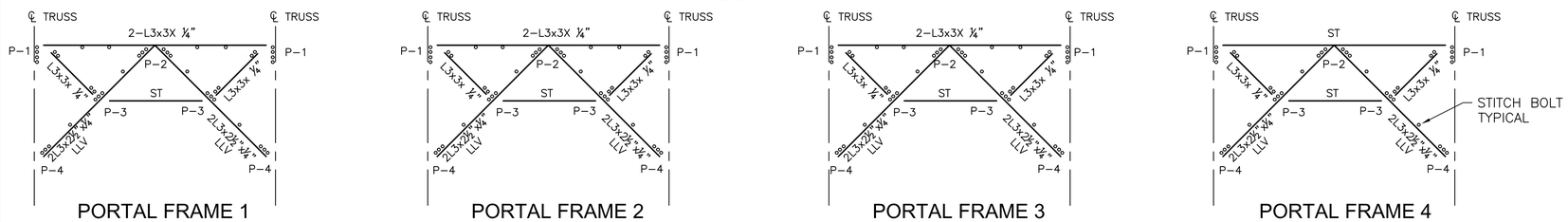
	PLOT DATE =	DATE —	REVISED —	Rock Island, IL (309) 788-7644 · Bettendorf, IA (563) 344-0260 · www.missman.com · Rockford, IL (815) 965-6400 · Sycamore, IL (815) 895-3825	SHEET NO. OF SHEETS	FED. ROAD DIST. NO. _	ILLINOIS	FED. AID PROJECT		

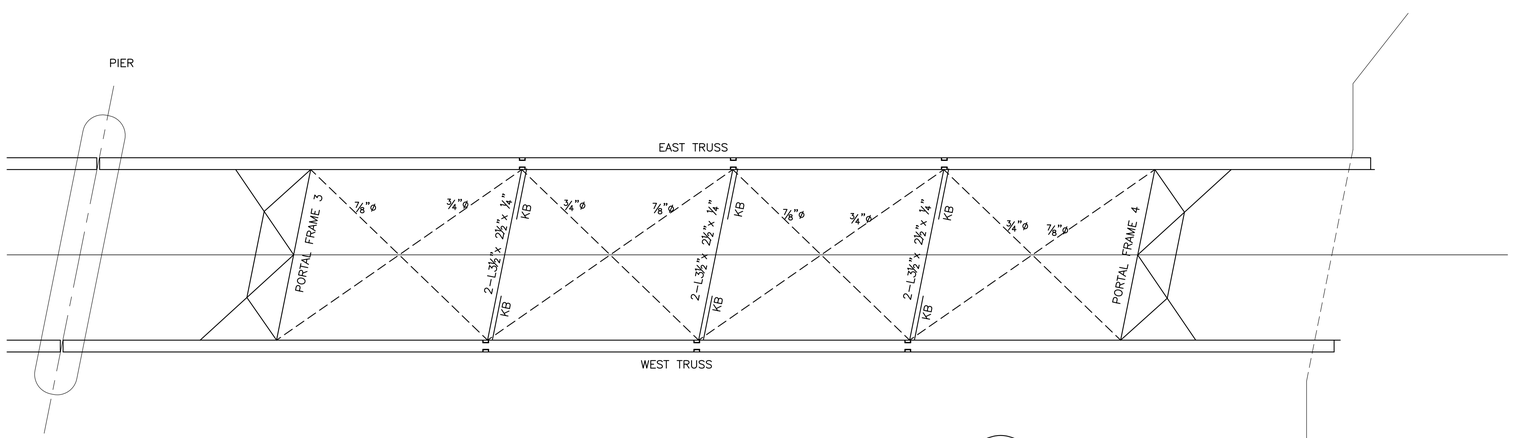


TOP STRUT FRAMING PLAN - NORTH 110' TRUSS SPAN

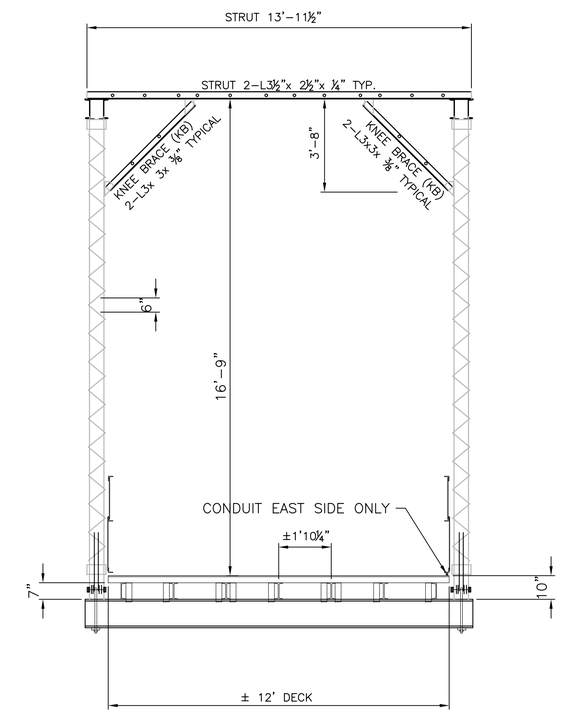
ST = STL TEE FLG 2.75x.31, WEB 2"x.658
P1 = PLATE 6"x ¼"x 12"
P2 = PLATE 7½"x ¼"x 18"
P3 = PLATE 8"(TAPER TO 6")x ¼"x 16"
P4 = PLATE 7¾"x ¼"x 13"



PORTAL FRAME 1 PORTAL FRAME 2 PORTAL FRAME 3 PORTAL FRAME 4



TOP STRUT FRAMING PLAN - SOUTH 89' TRUSS SPAN



TYPICAL SECTION

ATTACHMENT E

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