

R-1412

***INVESTIGATION OF STEEL  
BOX GIRDER PIER CAPS  
ON  
S05 OF 63103***

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16. Abstract The I-696 bridge over I-75 (ID S05 of 63103) in Royal Oak, Michigan carries four lanes of traffic in both the east bound and west bound directions. This is a five span bridge built in 1971 with the center spans being a three span continuous, steel girder bridge. Due to delayed construction of I-696, the bridge deck was not open to through traffic until 1989. The pier caps that support the center span consist of a 90"-deep, steel box girder supported by a concrete column at each end. Bottom flanges of the steel girders penetrate and are continued through the steel box girder pier cap, while the top flange is continued above the box girder pier cap. During a routine fracture critical inspection of the bridge, three cracks were found in the webs of the steel box girder pier caps where the longitudinal girder penetrates box girder pier cap. Analysis of the box girder pier cap penetration indicates a low live load stress range with an even lower fatigue resistance. Toughness values of the box girder pier cap compare favorably to the current requirement of 30 ft-lbs at 40 F (Zone 2, 2" to 4" welded plates) for fracture critical members. Retrofitting the box girder pier cap web penetrations where the three cracks were found by coring 2-2" diameter holes, 2 3/8" on center, in the box girder pier cap web adjacent to each side of the longitudinal girder bottom flange is recommended, along with saw cutting vertically between the holes.			
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**MICHIGAN DEPARTMENT OF TRANSPORTATION  
MDOT**

**Investigation of Steel Box Girder Pier Caps on S05 of 63103**

**Roger D. Till, P.E.**

**Testing and Research Section  
Construction and Technology Division  
Research Project TI-1997  
Research Report R-1412**

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Lansing, June 2002**

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## BACKGROUND

The I-696 bridge over I-75 (ID S05 of 63103) in Royal Oak, Michigan carries four lanes of traffic in both the east bound and west bound directions. This is a five span bridge built in 1971 with the center spans being a three span continuous, steel girder bridge (See Figure 1). Due to delayed construction of I-696, the bridge deck was not open to through traffic until 1989. The pier caps that support the center span consist of a 90"-deep, steel box girder supported by a concrete column at each end (See Figure 2). Bottom flanges of the steel girders penetrate and are continued through the steel box girder pier cap, while the top flange is continued above the box girder pier cap (See Figure 3). Bolted field splices connect the girders to the pier cap directly adjacent to the pier cap (See Figure 4). The pier caps are considered fracture critical members. All steel used for the longitudinal girders and the box girder pier caps was ASTM A588, Grade 50 weathering steel.

The 1990 average daily traffic was about 155,000 (estimated to be 71,400 according to the 1970 plans) and the commercial average daily traffic was about 8,400, total for both directions. The 2000 average daily traffic was about 190,000 and the commercial average daily traffic was about 8,400, total for both directions.

## INSPECTION

On April 2, 2002, during a routine fracture critical inspection of the bridge, a crack was found in the fourth girder from the north where the bottom flange of the girder penetrates the east web of the steel box girder cap at pier 3. The crack is in the face of the bottom fillet weld and is about 16" long starting at the edge of the flange (See Figures 5, 6, and 7).

According to the contract plans and shop drawings, the bottom flange is continuous through the box girder pier cap web and fillet welded to the outside of the box girder web plate only. There was a 2 1/8" wide slot with a 1 1/16" radius at the ends in the pier cap web for the 2" thick flange to pass through. A back up bar was fillet welded to the inside of the web at the ends of the slot and the slot ends were plug welded using the back up bar (See Figure 8). These details were verified in the field during the inspection.

On April 3, 2002, an ultrasonic and dye penetrant inspection was done at the girder penetration where the crack was found. The crack was visually located and an 8" radius area adjacent to the edge of the spliced bottom flange was ground to a smooth finish using a 9"-diameter abrasive wheel. An ultrasonic inspection was done using a 0.75" x 0.75"-2.25 MHz transducer attached to a 70 degree wedge. Scanning was done according to AWS criteria.

Due to the location of the crack within the weld area, the crack along the bottom of the spliced bottom flange was not ultrasonically inspected. The ultrasonic inspection did not show any indication from the crack where it extends beyond the longitudinal girder flange into the pier cap web. A flaw indication was located just at the edge of the spliced bottom flange. The scanning output trace of the flaw indication observed at the splice flange edge was recorded.

After the ultrasonic inspection was completed, the area was reground with extra attention to the plug weld location. The plug weld area was slightly depressed in relation to the box girder pier cap web

and required more grinding than the adjacent web area. During this grinding process, a small part of the crack end was removed. This fact is based upon where the crack tip was observed prior to grinding and the location of the crack tip after grinding. Removing the crack tip would agree with not finding the crack with the ultrasonic inspection. The crack would be shallow at the crack tip and not provide a discernible indication. Dye penetrant was then applied to the crack area in the web (See Figures 9 and 10). The tip of the crack was marked with a "V" using a paint pen.

The opposite end of the spliced flange was visually inspected, then ground and visually inspected again (See Figure 11). No cracks were found. Ultrasonic inspection and dye penetrant inspection were not done on this end of the splice flange. No effort was made to inspect the top of the spliced flange during the April 3, 2002, inspection. There was a large quantity of bird droppings on the top flange, which would need to be removed to properly inspect this location.

Bird droppings were removed from all the longitudinal girder flanges near the box girder pier cap and the inspection was completed on April 20, 2002. Two additional crack locations were found in the steel box girder pier cap where the bottom flange of the longitudinal girder penetrates the web. One crack was in the face/toe of the bottom fillet weld and is about 11" long starting at the edge of the flange (See Figure 12) at the fourth girder from the north and in the west web of the steel box girder cap at pier 2. The other crack is in the plug weld at the edge of the flange (See Figure 13) at the sixth girder from the south and in the east web of the steel box girder cap at pier 3.

## ANALYSIS

Pier 3 was analyzed to determine the dead load and live load stress present in the steel box girder pier cap. The live load (including impact) stress range in the box girder pier cap, parallel to the web, and located at the bottom flange of the longitudinal girder near the center of the pier cap is estimated to be 1.8 ksi based on the application of the HS20-44 fatigue truck in 2, 3, 4 or 6 lanes of traffic. The HS20-44 trucks were placed directly over the box girder pier cap and reduced by the appropriate load intensity factor depending on the number of lanes loaded. An HS20-44 truck was used as recommended by Juntunen (1), which was based on weigh-in-motion data of trucks on Michigan trunkline roadways.

The maximum live load (including impact) tensile stress in the longitudinal girder bottom flange perpendicular to the pier cap web is about 10 percent of the dead load compressive stress. In this case, the bottom flange of the longitudinal girder over the pier does not experience a net tensile stress.

According to Fisher et al (2), the weld detail used for the web penetration would be assigned to fatigue Category E' or less. The AASHTO Bridge Code (3) cites a constant amplitude fatigue limit (CAFL) for Category E' as 1.3 ksi and 2.6 ksi for non-redundant and redundant load path structures, respectively. The AASHTO LRFD Bridge Code (4) cites a Log-N intercept coefficient for Category E' as  $3.9 \times 10^8 (A)$  and a constant amplitude fatigue threshold of 2.6 ksi for both redundant and non-redundant members. Using the fatigue design method cited in AASHTO LRFD Bridge Code (4) and the calculated live load stress range of 1.8 ksi, along with an average daily truck traffic of 4200 in one direction, a fatigue life can be estimated. The result is

$$(\Delta F)_n = (A/N)^{1/6} \quad (1)$$

from which the fatigue life (Y) is estimated as

$$Y = A/[(\Delta F)_n^3 (365)(ADTT_{SL})n] = 3.9 \times 10^8 / [1.8^3 (365)(3360)(1)] \\ = 54.5 \text{ years}$$

where  $(ADTT_{SL})$  = single lane ADTT =  $0.8(ADTT) = 0.8(4200) = 3360$  and  $n$  = number of cycles per truck passage.

A fatigue life of about 55 years is estimated for the web penetration detail. It could be concluded that the web penetration detail in this case might have a fatigue resistance less than Category E' or the live load stress range might be somewhat higher than calculated since the web detail is cracked after 13 years of service.

## DISCUSSION

Toughness for the steel was specified to be greater than 15 ft-lbs at 40 F pursuant to longitudinal Charpy V-notch testing. According to the mill test reports there were three heats used for the flange plates of the box girder pier cap with toughness values of 45, 63, and 77 ft-lbs (each is average of three Charpy V-notch specimens) at 40 F and three heats used for the web plates with toughness values of 44, 68, and 105 ft-lbs (each is average of three Charpy V-notch specimens) at 40 F. These toughness values of the box girder pier cap compare favorably to our current requirement of 30 ft-lbs at 40 F (Zone 2, 2" to 4" welded plates) for fracture critical members.

The cracks likely initiated at the interface of the bottom flange and the plug weld since that was the only location ultrasonic inspection located a flaw indication or initiated in the plug weld itself. Cracking then propagated as a root crack to the face of the  $\frac{3}{8}$ " flange-web fillet weld, which would provide the least resistance, and progressed along the bottom of the flange.

Apparently, the  $\frac{3}{8}$ " fillet weld at the bottom flange-web location served to seal the inside of the box girder. The contract plans did not specify a weld at that location (See Figure 3) and there were no design calculations for the weld size. The shop drawings did include the  $\frac{3}{8}$ " fillet weld at the bottom flange-web location on the outside and the inside was shown to have the web corner clipped as shown in Figure 14.

Drilling holes at the tips of fatigue cracks has been found to be an effective means of arresting fatigue crack growth. Fisher et al (2) found that fatigue cracks at the ends of web attachments could be prevented from reinitiating at the drilled holes when the relationship

$$\Delta K / \sqrt{\rho} < 4\sqrt{\sigma_y} \quad (2)$$

is satisfied, where  $\Delta K$  = stress intensity factor fluctuation,  $\rho$  = hole radius, and  $\sigma_y$  = steel yield strength. The term  $\Delta K$  is estimated as

$$\Delta K = S_r \sqrt{\pi a} \quad (3)$$

where  $S_r$  = live load stress range and  $a$  = half of the maximum distance between drilled hole perimeters after retrofitting.



Retrofitting web penetrations with drilled holes typically consists of two holes closely spaced and a saw cut between the holes, which is placed adjacent to the penetrating flange in a vertical orientation. The holes are drilled at the crack tips. The department has used this retrofit technique on at least six steel box girder pier cap bridges (S02, S03, S10, S11, R02, and R03 of 82123). The detail used on S02 of 82123 shown is in Figure 15. By drilling the two holes and saw cutting between the holes the crack tip is blunted, the residual tensile stress field at the flange tip is removed, and the bending stresses in the steel pier cap are diverted around the web penetration, which all assist in preventing the crack from reinitiating. Although the hole drilling increases the stress concentration level up to threefold, the stress intensity at the crack tip is reduced because the crack tip is eliminated.

For S05 of 63103, 1" or 2" diameter holes can be drilled at the edge of the flange with their centers 2" apart and equation (1) is satisfied.

For 1" diameter holes at 2" on center,

$$\Delta K/\sqrt{\rho} = S_r\sqrt{(\pi a)}/\sqrt{\rho} = 1.8\sqrt{(\pi \times 1.5)}/\sqrt{(0.5)} = 5.5 < 4\sqrt{\sigma_y} = 4\sqrt{50} = 28.3$$

For 2" diameter holes at 2" on center,

$$\Delta K/\sqrt{\rho} = S_r\sqrt{(\pi a)}/\sqrt{\rho} = 1.8\sqrt{(\pi \times 2)}/\sqrt{(1.0)} = 4.5 < 4\sqrt{\sigma_y} = 4\sqrt{50} = 28.3$$

The 1" diameter holes could allow high strength bolts to be inserted and fully tightened, which creates a compressive stress field around the hole, but makes inspection for crack initiation more difficult. The 2" diameter holes would need neoprene plugs inserted to seal the area.

The maximum design tensile stress in the bottom of box girder pier cap after retrofitting with the drilled holes is estimated to be 25.7 ksi, which is below the as-designed allowable stress of 27 ksi.

## RECOMMENDATIONS

Retrofit the box girder pier cap web penetrations where the three cracks were found by coring 2-2" diameter holes, 2 3/8" on center, in the box girder pier cap web adjacent to each side of the longitudinal girder bottom flange. Saw cut vertically between the holes. Place neoprene plugs in the holes to seal. Seal the saw cut and the crack in the bottom flange fillet weld with a silicone sealant from the department's qualified product list. Details for this work are shown in Figure 16. A hole saw needs to be used to install the holes and the cores should be examined to assess the crack. The retrofit should be done within three months and should be inspected for crack reinitiation on a six month interval for one year after the retrofit has been completed. The three year, detailed fracture critical inspection frequency may be resumed if no crack reinitiation is found.

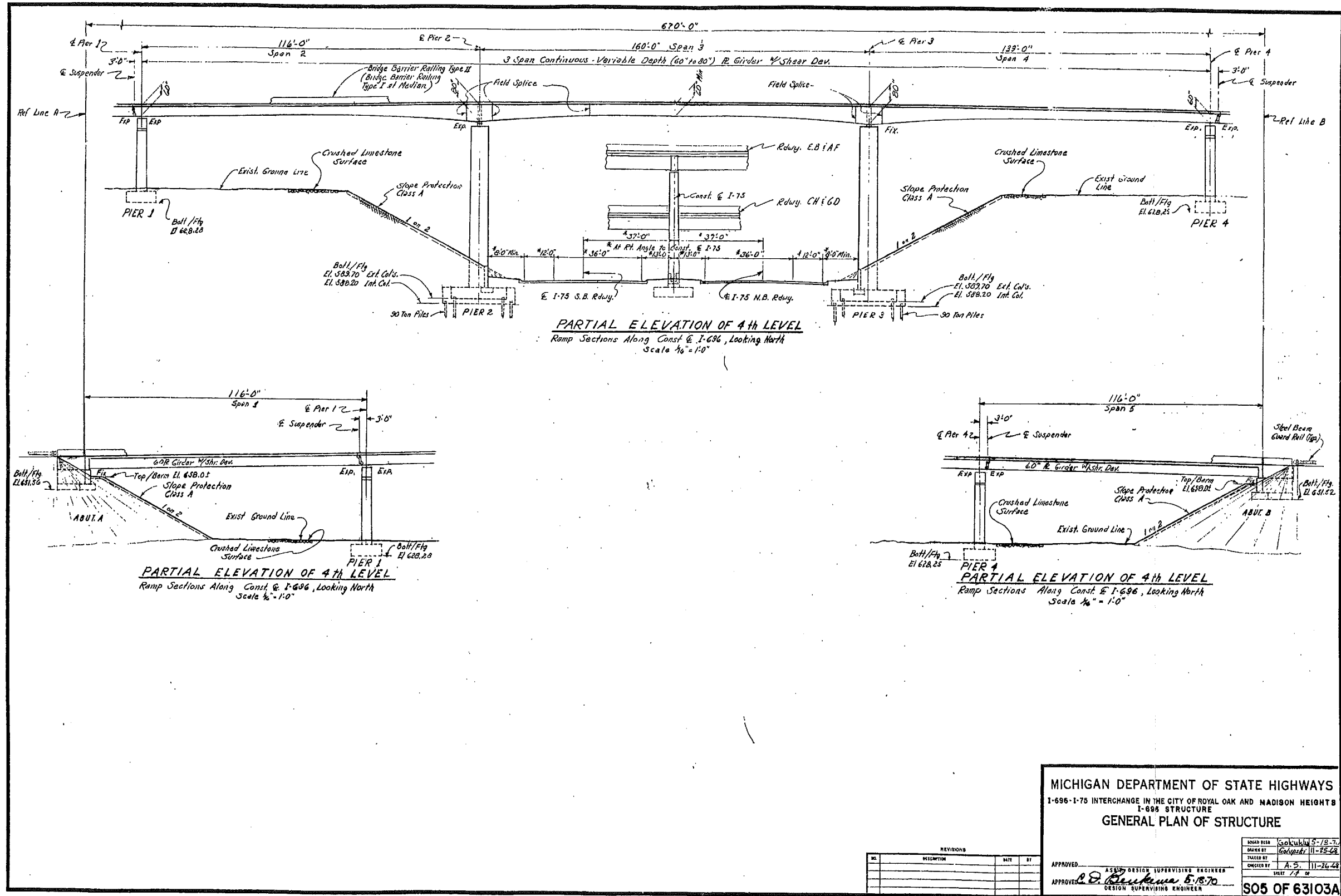
Include retrofitting all the box girder web penetrations according to the detail shown in Figure 16 in the next capital outlay project for this bridge in 2006. This would be consistent with the other steel box girder pier cap bridges the department has retrofitted.

Steel box girder pier caps that have had the web penetrations retrofitted with drilled holes each side of the longitudinal girder bottom flange should be inspected for cracks reinitiating at that location. Six bridges with this retrofit technique applied are S02, S03, S10, S11, R02, and R03 of 82123.

## REFERENCES

1. Juntunen, D., Study of Michigan's Link Plate and Pin Assemblies, Research Report R-1358, Construction and Technology Division, Michigan Department of Transportation, April 1998.
2. Fisher, J. W., et al, Fatigue Behavior of Full-Scale Welded Bridge Attachments, National Cooperative Highway Research Program, NCHRP Report 227, Transportation Research Board, Washington, D.C., 1980.
3. Standard Specifications for Highway Bridges, American Association of State Highway and Transportation Officials, 16<sup>th</sup> Edition, Washington, D.C., 1996.
4. LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, 2<sup>ed</sup> Edition, Washington, D.C., 1998.

## FIGURES



MICHIGAN DEPARTMENT OF STATE HIGHWAYS  
I-696-I-75 INTERCHANGE IN THE CITY OF ROYAL OAK AND MADISON HEIGHTS  
I-696 STRUCTURE  
GENERAL PLAN OF STRUCTURE

REVISIONS			
NO.	DESCRIPTION	DATE	BY

APPROVED: *[Signature]* ASSOCIATION SUPERVISING ENGINEER  
APPROVED: *[Signature]* DESIGN SUPERVISING ENGINEER

ISSUED BY	5-18-70
DESIGNED BY	11-28-68
TRACED BY	A.S. 11-26-68
CHECKED BY	11-26-68
DATE	11-26-68

SHEET 17 OF 18  
S05 OF 63103A

Figure 1 - I-696 bridge over I-75 (ID S05 of 63103) in Royal Oak, Michigan

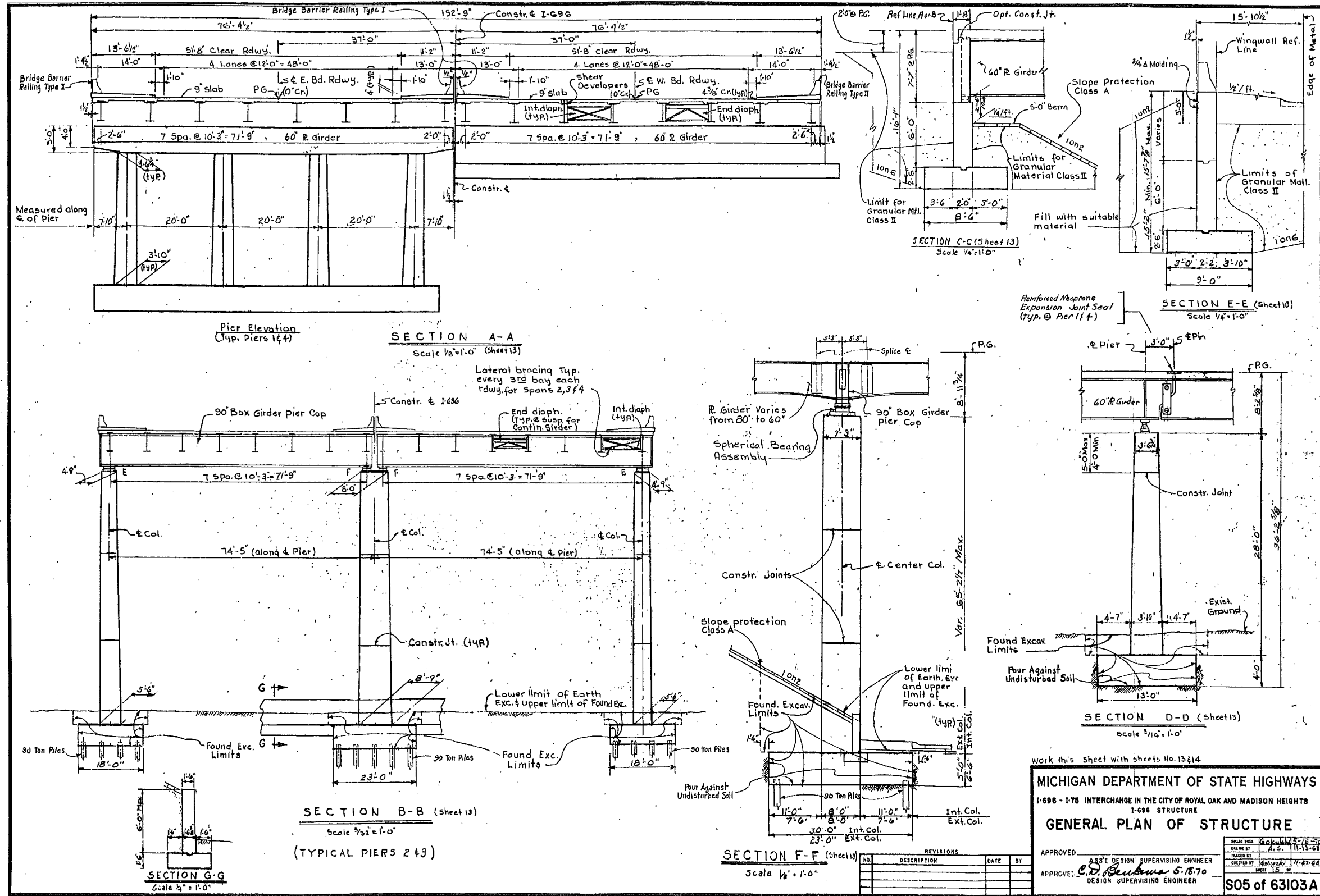


Figure 2 - S05 of 63103 with 90"-deep, steel box girder

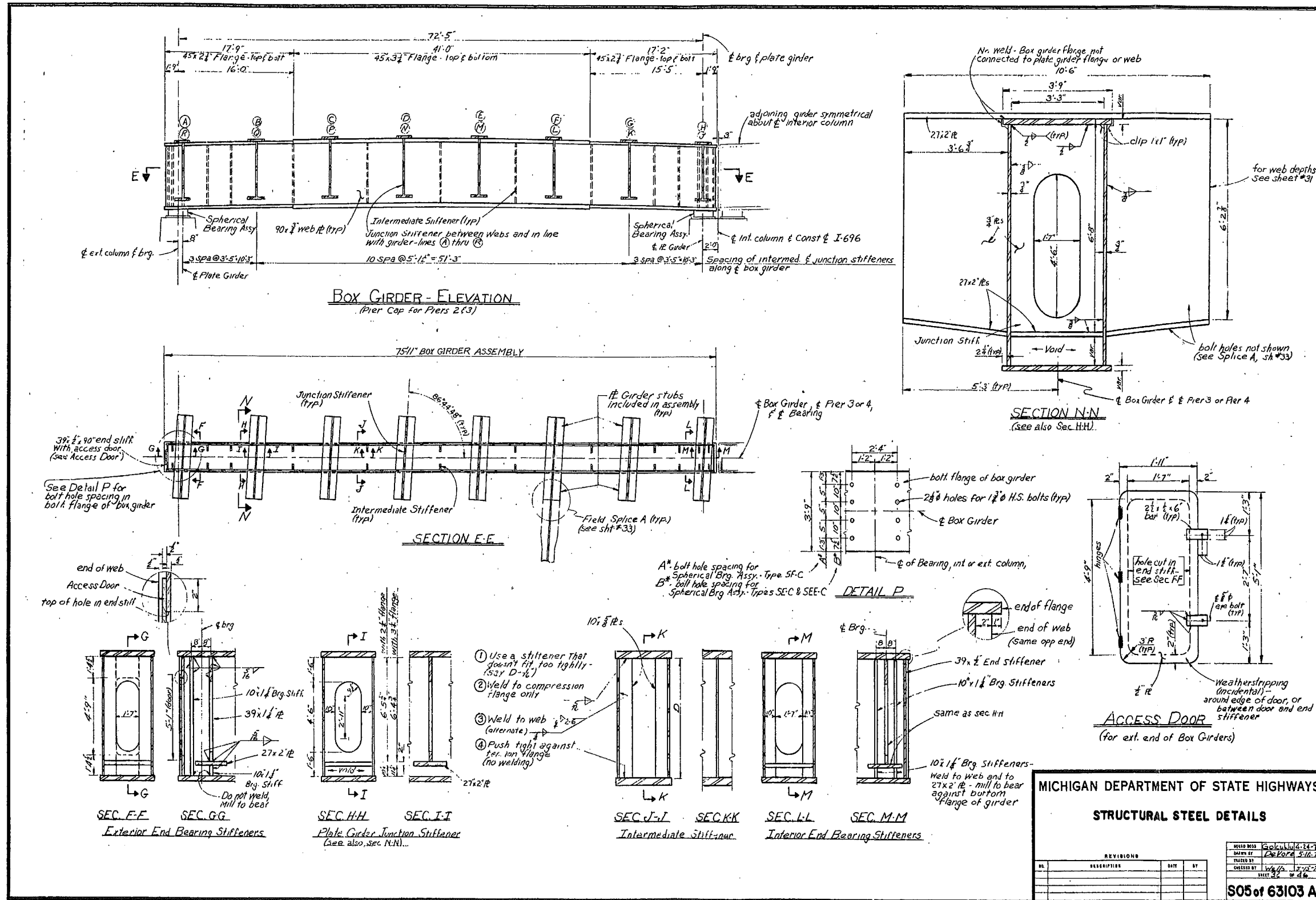


Figure 3 - S05 of 63103 with 90"-deep, steel box girder

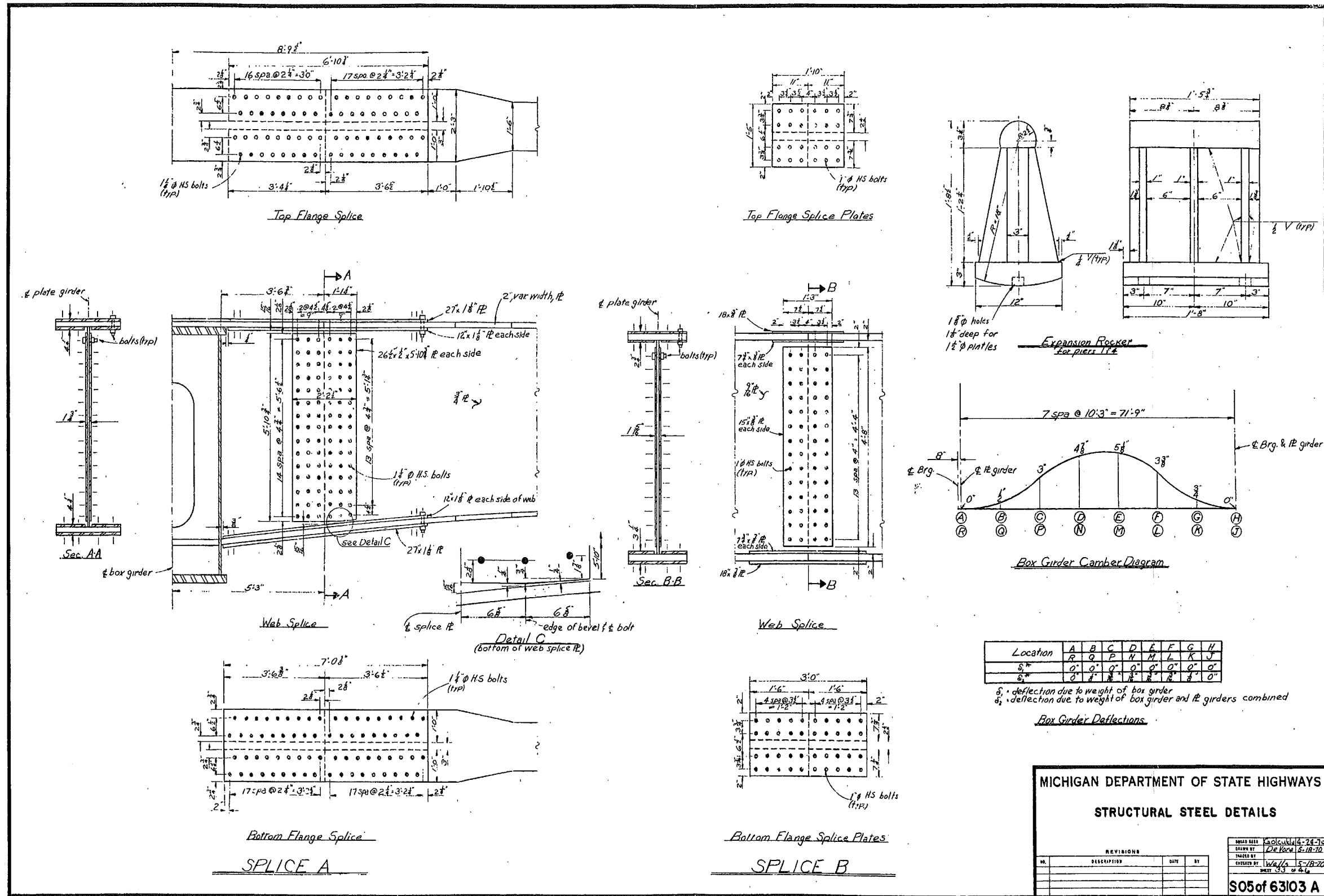
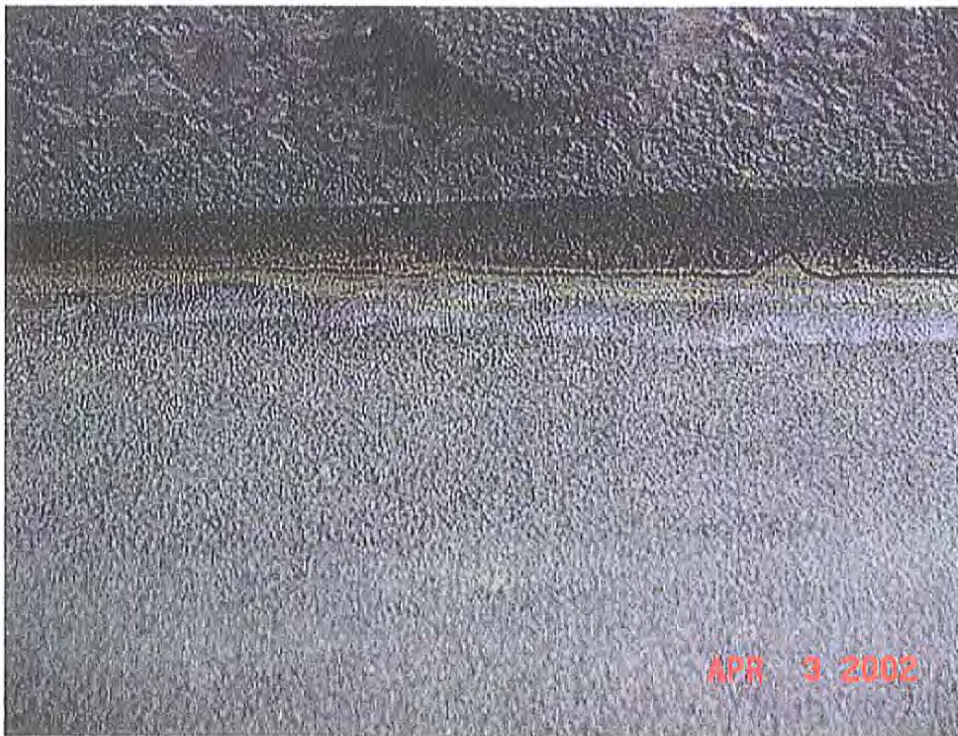


Figure 4 - S05 of 63103 with 90"-deep, steel box girder



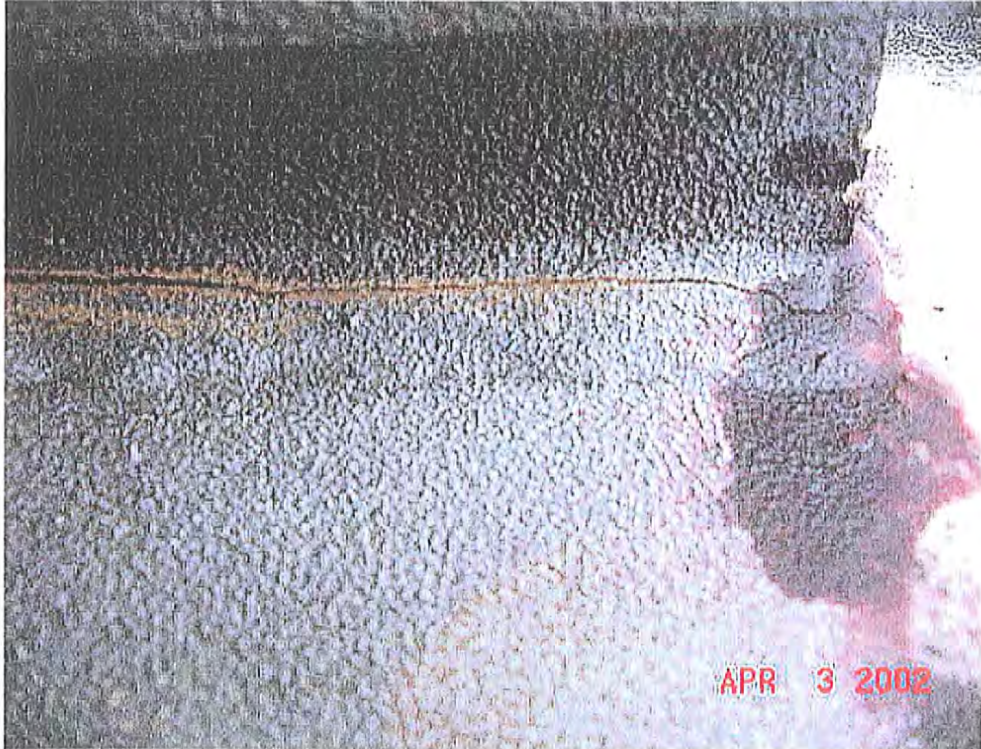


**Figure 5** - Bolted field splices connect the girders to the pier cap. Crack in the face of the bottom fillet weld



**Figure 6** - Crack in the face of the bottom fillet weld





**Figure 7 - Crack in the face of the bottom fillet weld**



**Figure 9 - Dye penetrant applied to the crack area in the web**

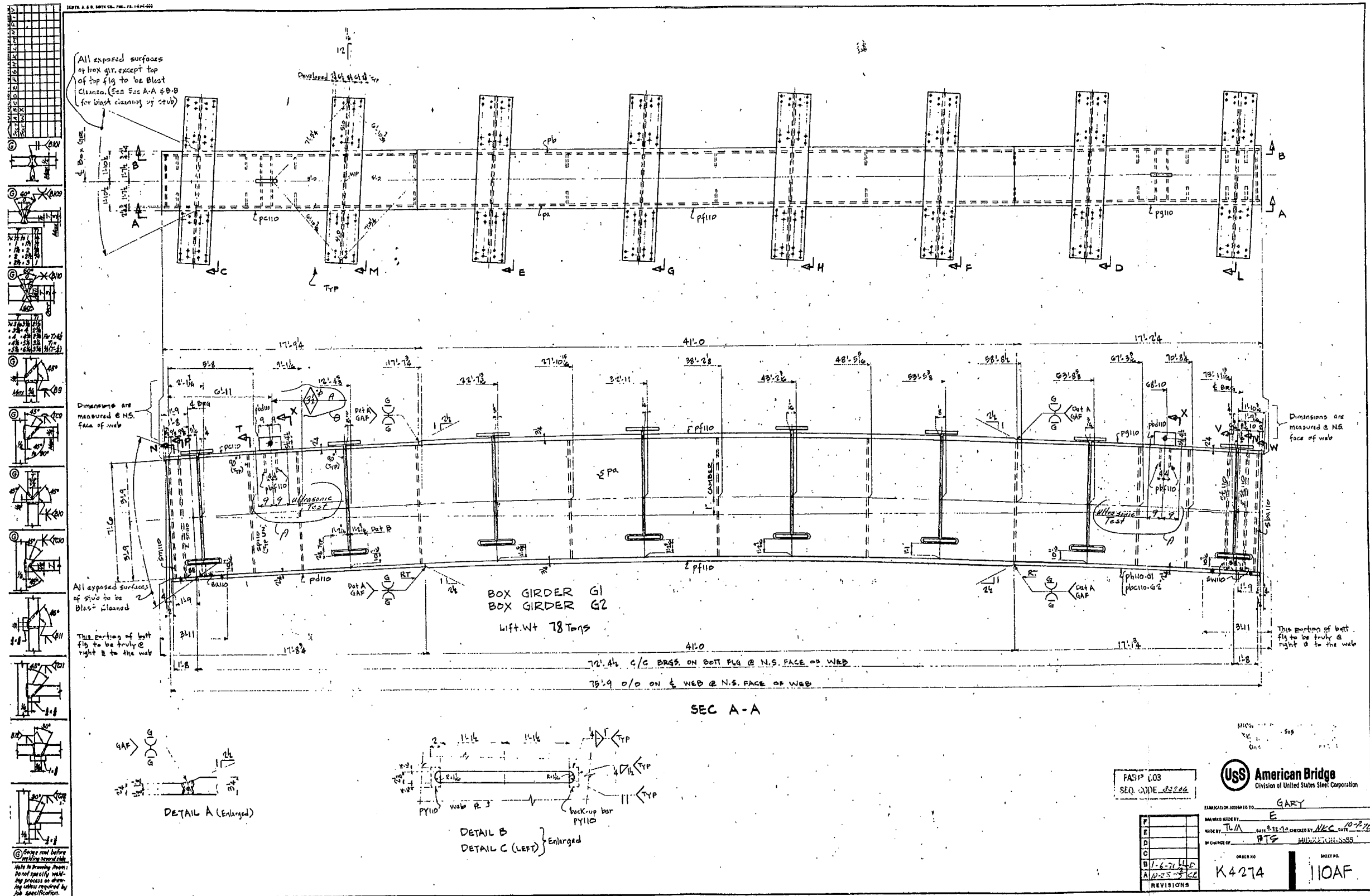
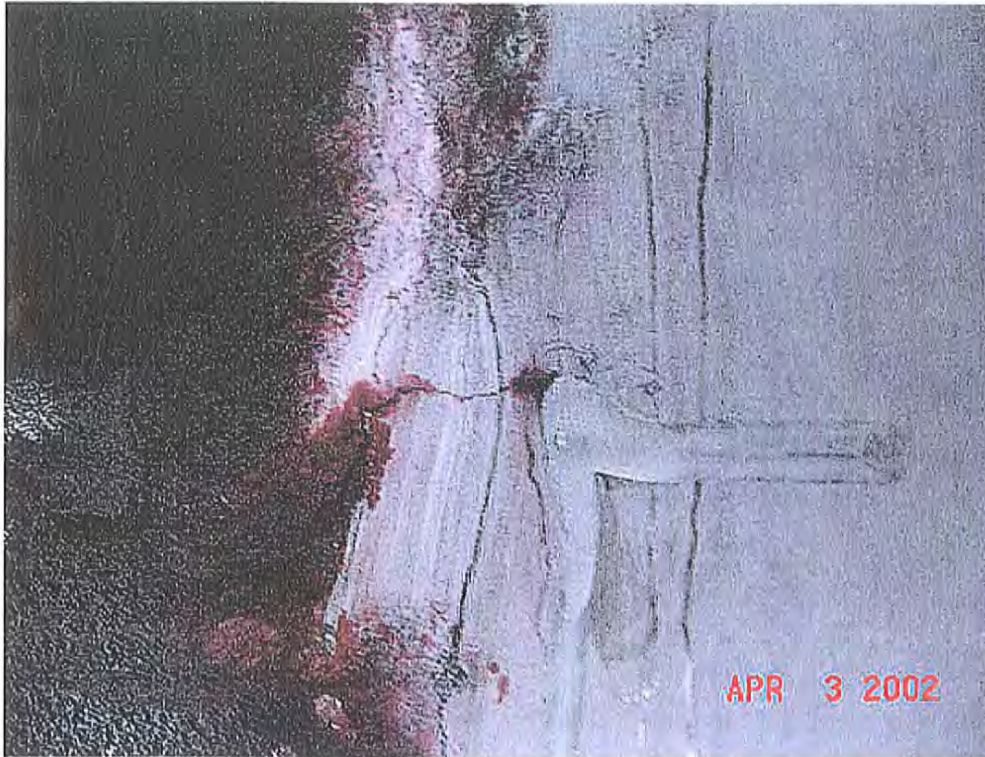


Figure 8 - Steel box girder details





**Figure 10 - Dye penetrant applied to the crack area in the web**

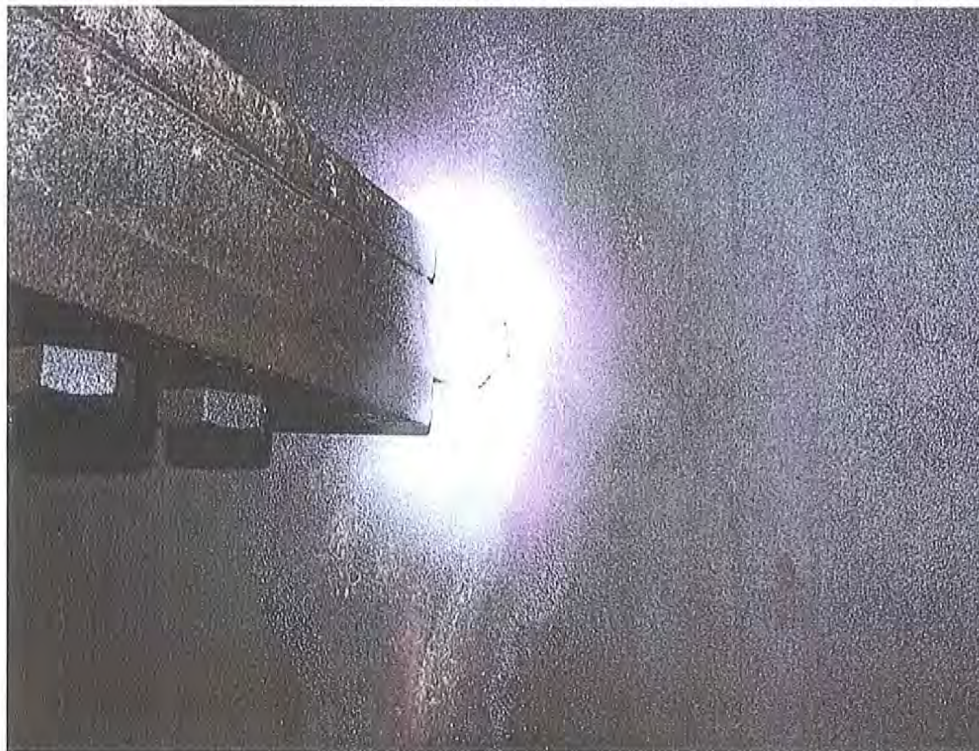


**Figure 11 - Opposite end of flange ground and no cracks were found**





**Figure 12** - Crack in the face/toe of the bottom fillet weld



**Figure 13** - Crack in the plug weld at the end of the flange



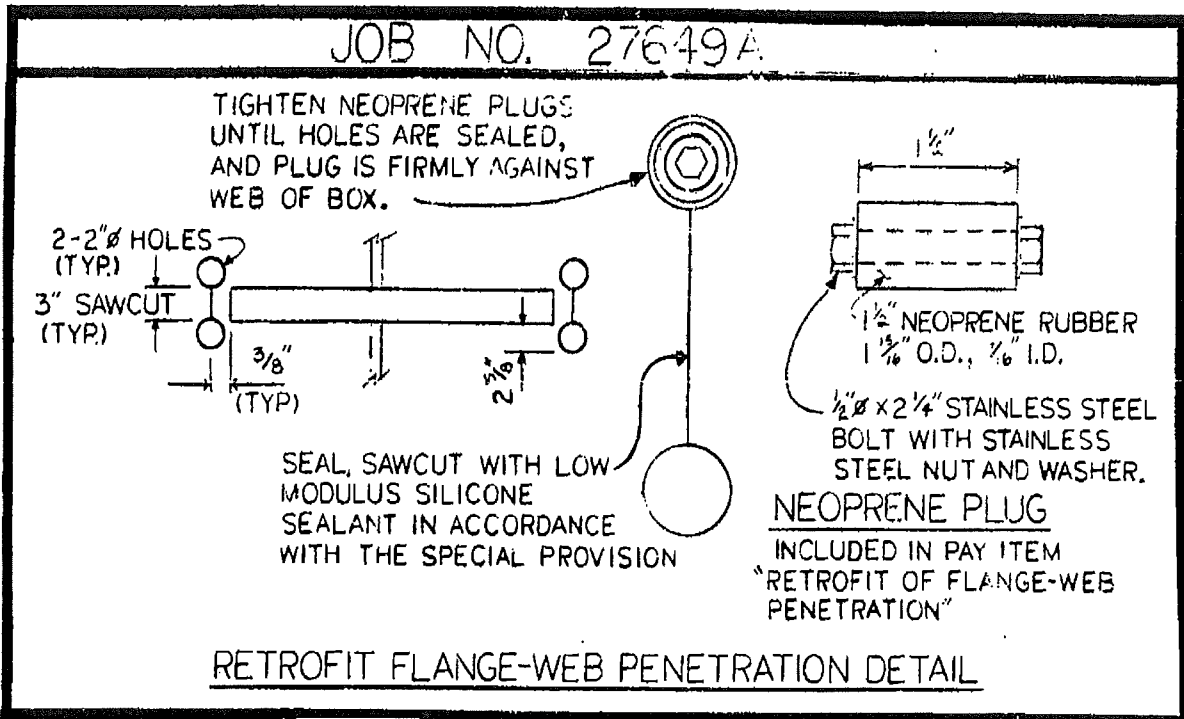


Figure 15 - Retrofit detail used on S02 of 82123

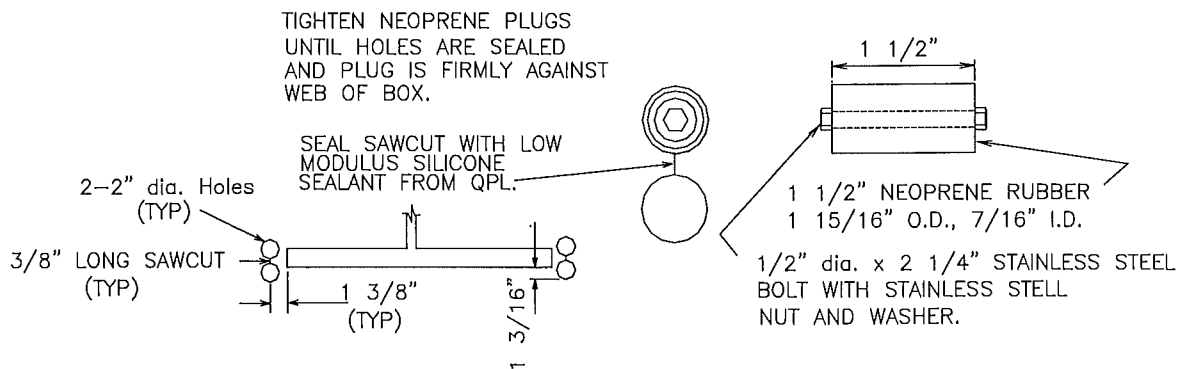


Figure 16 - Retrofit flange-web detail