NATIONAL TRANSPORTATION SAFETY BOARD
Office of Research and Engineering
Materials Laboratory Division
Washington, D.C. 20594

June 19, 2008

MODELING GROUP STUDY — Report No.08-062

A. ACCIDENT

Place	:	Minneapolis, Minnesota
Date	:	August 1, 2007
Vehicle	:	I-35W Highway Bridge
NTSB No.	:	HWY07MH024
Investigator	:	Mark Bagnard

B. TOPIC ADDRESSED

Reassessment of the design of gusset plates from nodes U10 and L11 of Minnesota bridge No. 9340 (I-35W over the Mississippi River) following the approach used by Richland Engineering Limited for Ohio bridge LAK-90-2342.

C. DETAILS OF THE STUDY

1. Introduction

Richland Engineering Limited reassessed the design of gusset plates on Ohio bridge LAK-90-2342 following a 1996 partial buckling collapse of gusset plates at the L8 nodes of the eastbound bridge. As shown in the contour plots in Appendix 1, the L8 gusset plates exhibited significant section loss as a result of corrosion. In order to assess the state of the bridge for repair and continued operation, Richland recomputed stresses in the gusset plates and compared them to allowable stresses calculated using several different design criteria. The loads they used in their calculations were 5 to 30 percent higher than the design loads shown on the original design drawings (which date from 1958), based on a more recent calculation of the member loads from a 1987 structural analysis (also performed by Richland); these increases reflect a change in the live load specification and a change in the multiplier for the dead load. Further, the analysis reflects methods from 1996, and includes several methods for assessing buckling. Corrosion was not considered in the computations, which are summarized in Ref. 1. Richland Engineering Limited found that, using the modern load assessment and analysis methods, stresses in some of the gusset plates on the bridge exceeded the allowable design stresses.

In response to a query from the Safety Board asking if they considered that the partial collapse of the L8 gusset plates on Ohio bridge LAK-90-2342 was primarily caused by poor design, Richland replied "No, the failure was primarily due to corrosion and section loss in the existing plates. However, there were some poor design elements that contributed to the failure." Ohio

Department of Transportation officials have also stated that they consider corrosion to be the primary cause of that accident.

Holt and Hartmann (Ref. 2) reassessed the design of the gusset plates on Minnesota bridge No. 9340 following the 2007 collapse of the bridge. Design drawings for that bridge date from 1965. In contrast to the approach by Richland for Ohio bridge LAK-90-2342, which was aimed at continued operation, Holt and Hartmann attempted to reconstruct the original 1960's design methods used for the gusset plates on Minneapolis bridge No. 9340, so as to evaluate both the structure and the design process itself. Although the original design calculations for the main truss gusset plates are not available, documents showing original calculations for the welded floor truss gusset plates were found in the records, and these floor truss calculations were used as a guide to determine the original design methodology. Holt and Hartmann recomputed stresses in the gusset plates and compared the stresses to design allowables using the original bridge design loads; those original design loads were verified by a review of the design calculations found in the records. The stresses in the gusset plates at nodes U10 and L11 were found to exceed the design allowables by a substantial margin, indicating a significant error in the design.

There are some differences between the approaches used in Ref. 1 and Ref. 2 to reassess the designs of the gusset plates. In this report, the gusset plates at nodes U10 and L11 on Minnesota bridge No. 9340 are reassessed using the approach of Ref. 1 in order to directly compare and contrast the design deficiencies identified in the two bridges. In general, the 1996 analysis methods used by Richland Engineering for Ohio bridge LAK-90-2342 in Ref. 1 are more comprehensive and somewhat more conservative than the reconstructed 1960's analysis methods used for Minnesota bridge No. 9340 in Ref. 2. In the Ref. 2 reassessment and in this reassessment, the loading conditions are those associated with the original design of bridge No. 9340, and do not take into account any changes to the bridge after it was constructed. In Ref. 1, the loading conditions were taken from a more recent 1987 analysis.

2. Differences between the Approaches of Ref. 1 and Ref. 2

Ref. 2 computes the bending stresses (f_b), average shear stresses (f_{v-avg}), principal tension stresses (f_{ten}) and principal compression stresses (f_{comp}) in the plates and compares them to AASHO allowable stresses. The steel used in the gusset plates at nodes U10 and L11 of Minnesota bridge No. 9340 had a minimum specified yield stress in tension of 50 ksi. The allowable stress for bending and tension of the 50 ksi steel is set to 27,000 psi, which is about 55 percent of the yield stress in tension. The allowable compression stress is set to 22,000 psi. The allowable shear stress is set to 15,000 psi, which is 56 percent of the tension allowable, consistent with yielding governed by a von Mises stress criterion. In the computation of principal stresses, the shear stress used is the maximum shear stress appropriate for bending of beams of rectangular cross sections, given by $1.5f_{v-avg}$. Bending stress is also computed and is compared to the allowable bending stress, independently of all other stresses. As noted in Ref. 3, gusset plate calculations such as these are based on beam theory approximations, but gusset plate geometries are well outside the regime where beam theory is valid.

Additionally, Ref. 2 checks whether the unsupported edge lengths of gusset plates exceed 48 times their thickness. If they do and their edges are not stiffened, the plates are considered inadequate. This design check is intended to insure the stability of slender plates in compression.

Ref. 1 does not consider principal stresses. It computes the bending stresses (σ_b), average shear stresses (σ_v) and compression/tension stresses (σ_c). It then computes combined stresses $\sigma_c+|\sigma_b|$ and $\sigma_c-|\sigma_b|$ to get the largest tension and compression stresses. Note that in some cases both stresses can be tension or both can be compression. In these cases, the largest compression stress or the largest tension stress, respectively, is zero. These largest stresses are then compared to the allowable tension stresses and the allowable compression stresses.

The steel used in the gusset plates of Ohio bridge LAK-90-2342 had a minimum specified yield stress in tension of 33 ksi. The allowable tension stress in Ref. 1, for the Ohio bridge, is 18,000 psi, which is again 55 percent of the yield stress. Ref. 1 lists the allowable shear stress as 11,000 psi (61 percent of the tension allowable, again consistent with a von Mises yield criterion), but reduces that stress to a shear allowable of 8,250 psi (75 percent of 11,000 psi) for comparison with the computed gusset plate shear stress. The allowable gusset plate compression stress test in Ref. 1 includes a test for buckling of the plate. The allowable compression stress is the less negative of -18,000 psi and the critical stress based on the Euler column buckling formula, which is given by $-\pi^2E/(2.12(L/r)^2)$, where E = 29,000 ksi is the modulus of elasticity, L is the unbraced length of the plate and r is the radius of gyration of the plate, given by $r^2=t^2/12$ where t is the plate thickness. The unbraced length L is the distance from the last row of rivets in the compression member to the first row of rivets in the chord member, measured along the centerline of the compression member. The number 2.12 represents a factor of safety. The Euler buckling stress would be valid for a column of length L loaded through pins that allow free rotation at the ends; as with the note above regarding beam theory, gusset plate loading conditions are significantly different from the conditions assumed in deriving the Euler buckling stress. This difference can be accounted for by replacing length L with an effective length kL, where k is an effective length factor with a value between 0.5 and 2. A value of k that is smaller than 1 results in a higher allowable critical compression stress for cases where buckling is the mode of failure. A value of k that is greater than 1 results in a lower allowable critical compression stress. Ref. 1 uses the value of k=1, which would be consistent for a member with frictionless pinned ends.

Ref. 1 then computes two additional stresses that are not considered in Ref. 2. These are the Whitmore stress and the plate edge stress as outlined by Brown in Ref. 4. The calculations for these two stresses are shown in the next two sections.

2.1 Whitmore Stress Computation

Whitmore stress is the stress in a gusset plate that is computed under the assumption that the Whitmore area is supporting the member load. The Whitmore area is illustrated in Fig.1.

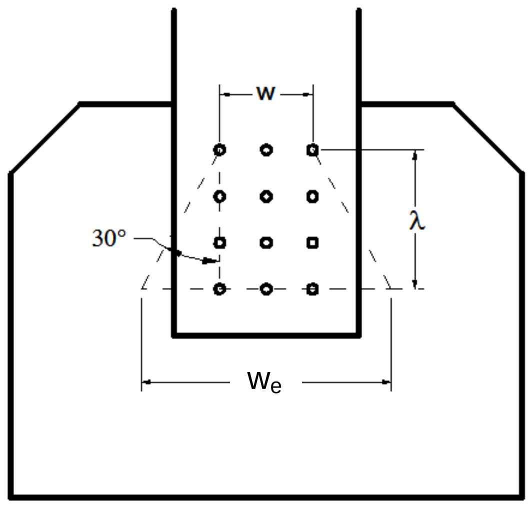

Fig. 1 Definition of Whitmore effective width

The figure shows a plate supporting a member that is riveted to it. The rivets are arranged in a rectangle of width w and length λ. The Whitmore effective width w_e is defined as

$$w_e = w + 2\lambda\tan(30°) \qquad (1)$$

and the Whitmore area is given by $t \cdot w_e$, where t is the thickness of the plate. Whitmore stress (or end stress) is given by $F/(t \cdot w_e)$ where F is the member force supported by the plate. Most truss bridge gusset connections use two plates and in such cases F is equal to half of the member force.

2.2 Edge Stress Computation (Brown method, Ref. 4)

This method compares the Brown compression stress in a plate to the allowable stress computed based on the analysis of buckling of unsupported edges of gusset plates, as described in detail in Ref. 4. The Brown compression stress is computed by dividing the member force (or half of it if there are two plates) by the area of the critical section through the plate. The critical section is located by passing a line through the midpoint of the longer free edge of the plate and perpendicular to the member. The allowable stress formula is:

$$\sigma_{allow} = -\,[\,F_y\,(1-(4a/t)^2/(2C_c^{\,2})]/FS \quad \text{if } 4a/t < C_c$$
$$\sigma_{allow} = -\,[\pi^2 E/(4a/t)^2]/FS \quad \text{if } 4a/t \geq C_c \qquad (2)$$

where

$C_c = (2\pi^2 E/\sigma_y)1/2$
σ_{allow} = allowed stress in compression
a = unsupported edge length
t = thickness of plate
E = modulus of elasticity
σ_y = yield stress
FS = factor of safety (FS=2.12)

3. Gusset Plate Stresses on Ohio Bridge LAK-90-2342

These stresses were computed and tabulated in Ref. 1. A subset of the table from Ref. 1 is reproduced below in Table 1. Table 1 lists only those joints that exceeded the allowable shear, tension, compression or buckling stresses based on the assumptions and analysis methods employed. Table 2 lists the demand-to-capacity ratios of the joints in Table 1. Bold font is used to mark ratios that exceed 1.00, indicating that the computed stress in the gusset plate exceeds the allowable. Note that L6, L8, U5 and U7 exceed the allowable shear and/or Whitmore stresses by a relatively small margin. L0, L4, L6, L8, U5 and U10 exceed buckling-related criteria by a significant margin. L8 exceeds the allowable compression stress (with buckling taken into account) by a factor of 5.43. The L8 gusset plates on the eastbound bridge partially collapsed in buckling after suffering significant section loss due to corrosion.

	Thickness (in)	Shear (KSI)	Tens. (KSI)	Comp. (KSI)	F_c	Whitmore Stress (KSI)	Brown Method (KSI)	
Allowable Stress (KSI)		8.25	18.00	F_c	Euler Buckling	18.00	Edge Stress	Allow. Stress
L0	1/2	7.24	11.54	-15.52	-18.00	-15.94 (L0-U1)	-5.43 (L0-U1)	-3.99 (L0-U1)
L4	9/16	6.88	12.00	-15.39	-8.90	15.44 (L4-U5)	-7.88 (L4-U3)	-10.12 (L4-U3)
L6	7/16	8.41	6.62	-15.67	-12.74	16.26 (L6-U7)	-7.20 (L6-U5)	-1.99 (L6-U5)
L8	7/16	8.93	8.44	-18.75	-3.45	-19.51 (L8-U9)	-9.23 ((L8-U9)	-3.05 (L8-U9)
U3	1/2	5.77	7.39	-14.90	-14.35	-14.65 (L4-U3)	-7.32 (L8-U9)	-6.51 (L8-U9)
U5	1/2	8.93	18.08	-18.93	-14.35	16.10 (L4-U5)	-6.09 (L6-U5)	-3.12 (L6-U5)
U7	7/16	4.42	11.77	-1.29	-4.88	21.15 (L8-U7)		
U10	9/16	7.95	8.61	-17.21	-18.00	-14.88 (L9-U10)	-5.67 (L9-U10)	-2.31 (L9-U10)

Table 1 Stresses in Gusset Plates on Ohio Bridge LAK-90-2342, taken from Ref. 1.

	Thickness (in)	Shear Stress	Tens. Stress	Comp. Stress	Whitmore Stress	Brown Method
L0	1/2	0.88	0.64	0.86	0.89	**1.36**
L4	9/16	0.83	0.67	**1.73**	0.86	0.78
L6	7/16	**1.02**	0.37	**1.23**	0.90	**3.62**
L8	7/16	**1.08**	0.47	**5.43**	**1.08**	**3.03**
U3	1/2	0.70	0.41	**1.04**	0.81	**1.12**
U5	1/2	**1.08**	1.00	**1.32**	0.89	**1.95**
U7	7/16	0.54	0.65	0.26	**1.18**	
U10	9/16	0.96	0.48	0.96	0.83	**2.45**

Table 2 Demand-to-Capacity Ratios for Gusset Plates on Ohio Bridge LAK-90-2342, based on data in Table 1.

4. Computation of Gusset Plate Stresses on Minnesota Bridge No. 9340

We used gusset plate sections, forces, moments and dimensions for U10 exactly as computed and documented in Ref. 2. The corresponding values for L11 were computed similarly based on bridge data. Ref. 1 does not use vertical sections such as section B-B in Ref. 2. Therefore, we used only the horizontal sections labeled A-A in Ref. 2. We then applied the stress computation approach from Ref. 1 to gusset plates U10 and L11. The resulting stresses are shown in Table 3 for the original design with ½ inch plates. Note that only the most critical Whitmore stresses and the most critical Brown method stresses in each gusset plate are shown. Also note that the allowable compression stress is Fc, a computed value listed in the Fc column. To be consistent with the approach of Ref. 1, we use allowable tension and compression stress of 27,000 psi and allowable shear stress of 11,250 psi (75% of 15,000 psi).

	Thickness (in)	Shear (KSI)	Tens. (KSI)	Comp. (KSI)	F_c	Whitmore Stress (KSI)	Brown Method (KSI)	
Allowable Stress (KSI)		11.25	27.00	F_c	Euler Buckling	27.00	Edge Stress	Allow. Stress
U10	1/2	27.23	24.19	-21.59	-9.73	-37.46 (L9-U10)	-17.78 (L9-U10)	-2.13 (L9-U10)
L11	1/2	26.01	20.81	-25.33	-14.35	-32.68 (L11-U12)	-11.20 (L11-U12)	-6.89 (L11-U12)

Table 3 Stresses in Gusset Plates U10 and L11 (Original ½ inch Plate Thickness)

	Thickness (in)	Shear Stress	Tens. Stress	Comp. Stress	Whitmore Stress	Brown Method
U10	1/2	**2.42**	0.90	**2.22**	**1.39**	**8.35**
L11	1/2	**2.31**	0.77	**1.77**	**1.21**	**1.63**

Table 4 Demand-to-Capacity Ratios for Gusset Plates U10 and L11, based on data in table 3 (Original ½ inch Plate Thickness)

The demand-to-capacity ratios based on the values in Table 3 are listed in Table 4. Of all the entries in Table 4, only the tension stresses do not exceed demand-to-capacity ratio of 1.00. If these tension stresses were replaced with principal tension stresses, as done in Ref. 2, their demand-to-capacity ratios would have exceeded 1.00 because of the contribution from shear stresses.

Note that the two buckling-related stresses that were not considered in Ref. 2, i.e., compression stresses that take buckling into account and Brown edge stresses have demand-to-capacity ratios greater than 1.00.

We then computed the stresses for the hypothetical scenario where plates U10 and L11 were 1 inch thick, which significantly reduces the computed stress in the gusset plates and significantly increases the allowable stresses for Euler buckling and the Brown method. The stresses are listed in Table 5 and the demand-to-capacity ratios are listed in Table 6.

	Thickness (in)	Shear (KSI)	Tens. (KSI)	Comp. (KSI)	F_c	Whitmore Stress (KSI)	Brown Method (KSI)	
Allowable Stress (KSI)		11.25	27.00	F_c	Euler Buckling	27.00	Edge Stress	Allow. Stress
U10	1	13.62	12.09	-10.79	-27.00	-18.73 (L9-U10)	-8.89 (L9-U10)	-8.50 (L9-U10)
L11	1	13.00	10.40	-12.67	-27.00	-16.34 (L11-U12)	-5.60 (L11-U12)	-18.94 (L11-U12)

Table 5 Stresses in Gusset Plates U10 and L11 with (<u>Hypothetical 1 inch Plate Thickness</u>)

	Thickness (in)	Shear Stress	Tens. Stress	Comp. Stress	Whitmore Stress	Brown Method
U10	1	**1.21**	0.45	0.49	0.69	**1.05**
L11	1	**1.16**	0.39	0.58	0.61	0.30

Table 6 Demand-to-Capacity Ratios for Gusset Plates U10 and L11, based on data in table 5.
(<u>Hypothetical 1 inch Plate Thickness</u>)

Finally, we evaluated the hypothetical scenario where plates U10 and L11 were 1/2 inch thick but made of steel with a yield stress of 100 ksi. Note that in the design plans for Minnesota bridge No. 9340, the allowable tension stress for steel with a minimum specified yield stress of 100 ksi is listed as 45 ksi. We therefore defined this steel as having allowable stresses that were 1.67 times those of the steel actually used in plates U10 and L11 for tension, compression and shear. The Euler buckling and Brown allowable stresses depend only on plate thickness and elastic modulus, and are independent of yield stress. The stresses are listed in Table 7 and the demand-to-capacity ratios are listed in Table 8.

	Thickness (in)	Shear (KSI)	Tens. (KSI)	Comp. (KSI)	F_c	Whitmore Stress (KSI)	Brown Method (KSI)	
Allowable Stress (KSI)		18.79	45.09	F_c	Euler Buckling	45.09	Edge Stress	Allow. Stress
U10	1/2	27.23	24.19	-21.59	-9.73	-37.46 (L9-U10)	-17.78 (L9-U10)	-2.13 (L9-U10)
L11	1/2	26.01	20.81	-25.33	-14.35	-32.68 (L11-U12)	-11.20 (L11-U12)	-6.89 (L11-U12)

Table 7 Stresses in Gusset Plates U10 and L11
(<u>Original ½ inch Plate Thickness, 100 ksi steel</u>)

	Thickness (in)	Shear Stress	Tens. Stress	Comp. Stress	Whitmore Stress	Brown Method
U10	½	**1.45**	0.54	**2.22**	0.83	**8.35**
L11	½	**1.38**	0.46	**1.77**	0.72	**1.63**

Table 8 Demand-to-Capacity Ratios for Gusset Plates U10 and L11, based on data in table 7. (Original ½ inch Plate Thickness, 100 ksi steel)

Note that the computed stresses in Table 7 are identical to those in Table 3. However, the allowable stresses related to the yield stress in Table 7 are 1.67 times those in Table 3. Consequently, those demand-to-capacity ratios in Table 8 that relate to the yield stress are 1.67 times lower than those in Table 4. The demand-to-capacity ratios in Table 8 that are related to buckling are identical to those in Table 4 because they do not depend on the yield stress.

5. Discussion of Results

This report applies the stress computation approach and design criteria used in Ref. 1 to gusset plates U10 and L11 on Minnesota bridge No. 9340. There are differences between these computations and criteria and those used in Ref. 2 for the same plates, as detailed above in Section 2.

Although the approaches are different, both clearly detect the inadequate thickness of plates U10 and L11. Ref. 2 detects it by demand-to-capacity ratios of shear, principal tension and principal compression, all well above 1.00. Additionally, it detects that plate U10 exceeds the length to thickness ratio allowed for slender edges.

The methodology of Ref. 1 detects the inadequate thickness of plates U10 and L11 by demand-to-capacity ratios of shear, compression stress, Whitmore stress and Brown edge stress that exceed 1.00 by large margins.

We also computed the stresses for hypothetical plate thickness of 1 inch. The shear stresses in U10 and L11 exceeded the allowable values by comparatively small margins. The demand-to-capacity ratios were 1.21 and 1.16, respectively. The Brown edge stress in U10 also exceeded its allowed value, but by only 5%.

Finally, we considered the hypothetical case of ½ inch plates made of 100 ksi steel. Because the allowable stress for such plates is less than twice the allowable stress for 50 ksi steel, these plates have slightly higher demand-to-capacity ratios for shear, tension and Whitmore stresses as compared to the 1 inch plates made of 50 ksi steel. The shear stresses in U10 and L11 exceeded their allowable values, and the demand-to-capacity ratios were 1.45 and 1.38, respectively. These hypothetical plates also have the same demand-to-capacity ratios for compression stresses that take buckling into account and for Brown edge stresses as the original ½ inch, 50 ksi plates. These ratios are all significantly greater than 1.00, indicating that the hypothetical ½ inch, 100 ksi plates do not meet the Richland Engineering buckling criteria.

Of the three choices considered in this report, gusset plates of 1 inch thick 50 ksi steel would have been the most appropriate for nodes U10 and L11 on Minnesota bridge No. 9340. However, even such plates did not meet all of the criteria considered in this report. It would take 1¼ inch plates made of 50 ksi steel to meet all the Richland Engineering criteria for plates U10 and L11 on Minnesota bridge No. 9340.

In all of these design assessments, the loads assumed are the maximum loads (dead loads plus live loads) that the structure would be expected to carry, while the allowable stresses (based on material and/or geometry) are specified to be much less than the expected actual capacity of the structure. The design methods are therefore conservative, and exceeding the allowable stress for one or more of these design criteria does not imply imminent collapse, but it does suggest that the structure could be vulnerable.

The gusset plates must transfer forces among the truss members they connect. Along the typical sections used for gusset plate design, the force transfer is primarily through shear stress (Ref. 2 notes that the gusset plate analyses are dominated by the shear stress). The direct stresses are generally small compared to the shear stress, so the shear stress dominates any calculation of principal stress. Further, the bending stresses evaluated in the analyses can be calculated from the moment created by the shear stress acting along the section with the moment arm being the distance from the center of the joint. Ref. 5 provides a detailed list of steps for the design of gusset plate joints for truss bridges. In that design procedure, the first step in determining the gusset plate thickness is to evaluate the shear stress on a design section against an allowable stress. In applying the allowable shear stress criterion using the method of Ref. 1, it can be seen that the as-built U10 and L11 gusset plates of Minnesota bridge No. 9340 exceed the allowable by a factor of 2.31 or more, while the gusset plates of Ohio bridge LAK-90-2342 exceeded the allowable by a factor of 1.08 or less. Also, within the limited sample of gusset plates from two bridges considered in this report, any gusset plate design that exceeded the allowable shear stress criterion also exceeded at least one of the other allowable stresses in the design criteria used in Ref 1.

The Euler buckling and Brown method criteria used in Ref. 1 are both based on buckling, but they employ different assumptions. The Euler buckling criterion used in Ref. 1 determines an allowable compression stress based on the buckling of a rectangular plate along the line of the center of a compression member. However, that allowable stress is compared with the maximum compression stress from direct compression and bending, which occurs at the edge of the gusset plate. The allowable stress is therefore calculated for one condition and then compared with a stress calculated for a different condition. For Ohio bridge LAK-90-2342, the L8 gusset plate had the longest distance along the center of a compression member and L8 was one of the joints with the thinnest gusset plates; the calculated Euler buckling load for L8 was therefore the smallest for that bridge. The maximum calculated compressive stress for L8 exceeded the Euler buckling criterion by a factor of 5.43; the L8 gusset plates of the eastbound LAK-90-2342 bridge suffered a partial buckling collapse after significant section loss due to corrosion. The compressive stresses in the as-built U10 and L11 gusset plates of Minnesota bridge No. 9340 both exceeded the allowable stresses determined using the Euler buckling criterion of Ref. 1 by a factor of approximately 2.

The Brown criterion appears to be more consistent in matching a compressive stress to an appropriate allowable. The Brown method addresses potential buckling of the unsupported edge of the gusset plate adjacent to a compression member. Using this criterion, the maximum ratio of demand to capacity for the LAK-90-2342 gusset plates was 3.62 at node L6. For Minnesota bridge No. 9340, the ratio of demand to capacity for the as-built U10 gusset plates was calculated to be 8.35, which is consistent with the fact that the U10 gusset plate unbraced length to thickness ratio exceeded that allowed in the relevant 1961 and 1962 AASHO specifications cited in Ref. 2. In addition, photographs show distortion along the edges of the U10 joints on the Minnesota bridge, which could be consistent with the initial edge buckling failure mechanism identified by Brown.

For some geometries, the Whitmore section from one member overlaps other members, which would suggest that the stress across the Whitmore section would differ from the constant stress assumed. However, as noted in Ref. 6, checking the stress along the Whitmore area is similar to evaluating the joint for block shear, in which there is tensile or compressive failure along the rivets at the end of the member and shear failure in the lines of rivets along the lateral edges of the member. This similarity results because the areas along the lateral edges of the member are projected to the Whitmore section by an angle of 30°, so these areas are reduced by a factor of tan(30°) = 0.577, which is equal to the reduction of the allowable tension or compression stress to calculate an allowable shear stress for yielding governed by a von Mises stress criterion.

6. References

1. ‘Analysis of Selected Gusset Plates, Bridge No. LAK-90-2342 (L&R), I-90 over Grand River Valley, Lake County, Ohio,’ Richland Engineering Limited, Mansfield, Ohio, July 1996.
2. Holt, R. and Hartmann, J., ‘Adequacy of the U10 & L11 gusset Plate Designs for the Minnesota Bridge No. 9340 (I-35W over the Mississippi River),’ Interim Report, Federal Highway Administration, January 11, 2008.
3. Gaylord, E. H. Jr. and Gaylord, C. N., ‘Design of Steel Structures,’ Second Edition, McGraw-Hill Book Company, New York, 1972.
4. Brown, V. L., ‘Stability Design Criteria for Gusseted Connections in Steel-Framed Structures,’ Proceedings of the 1990 Annual Technical Session, Steel Structures Research Council, 1990.
5. ‘Structural Steel Designer’s Handbook,’ R. L. Brockenbrough and F. S. Merritt, editors, 4th Edition, McGraw-Hill, New York, 2005.
6. ‘Steel Designers’ Manual,’ Sixth Edition, B. Davison and G. W. Owens, editors, The Steel Construction Institute, Blackwell Science Ltd, Oxford, 2003.

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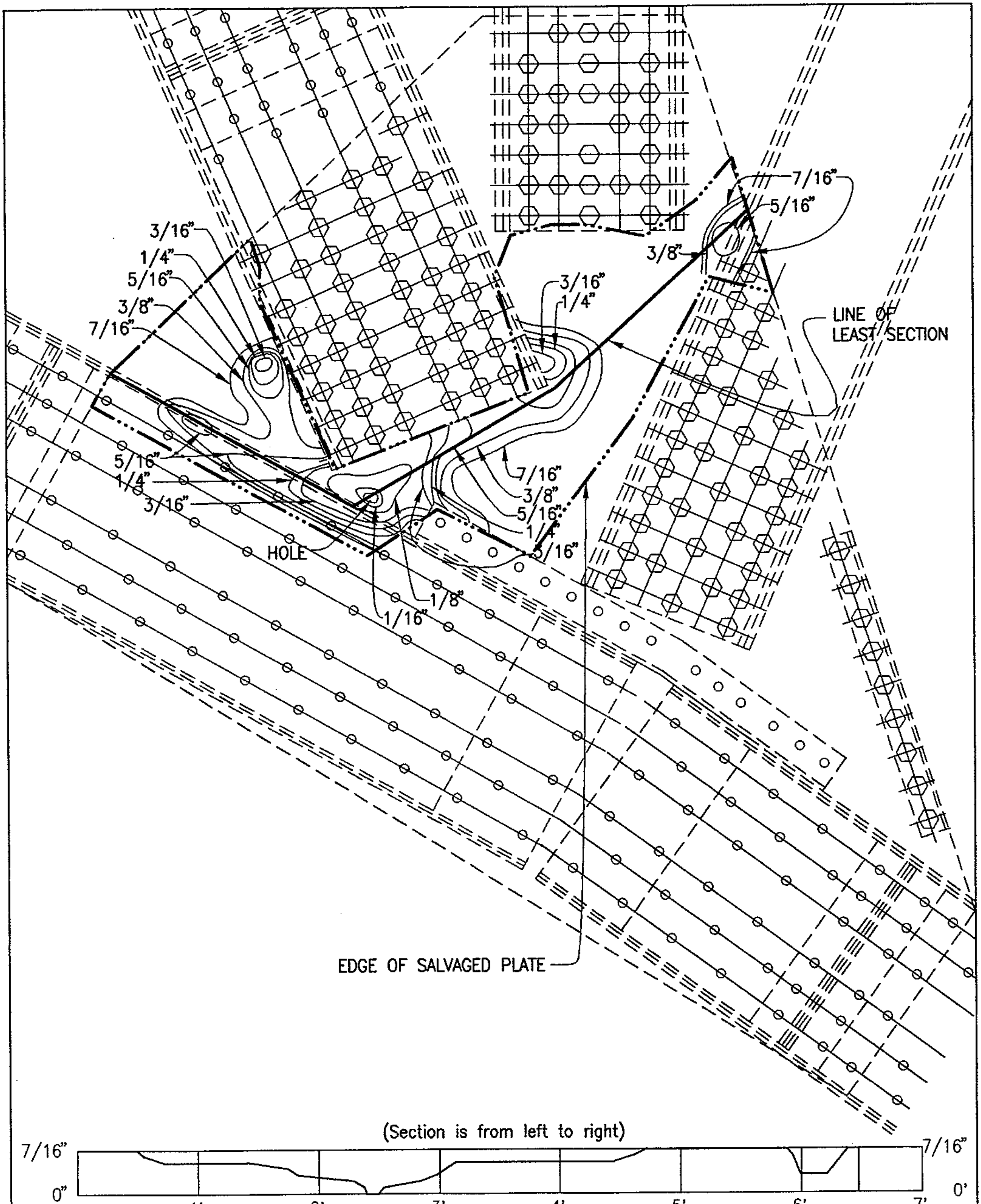
Appendix 1

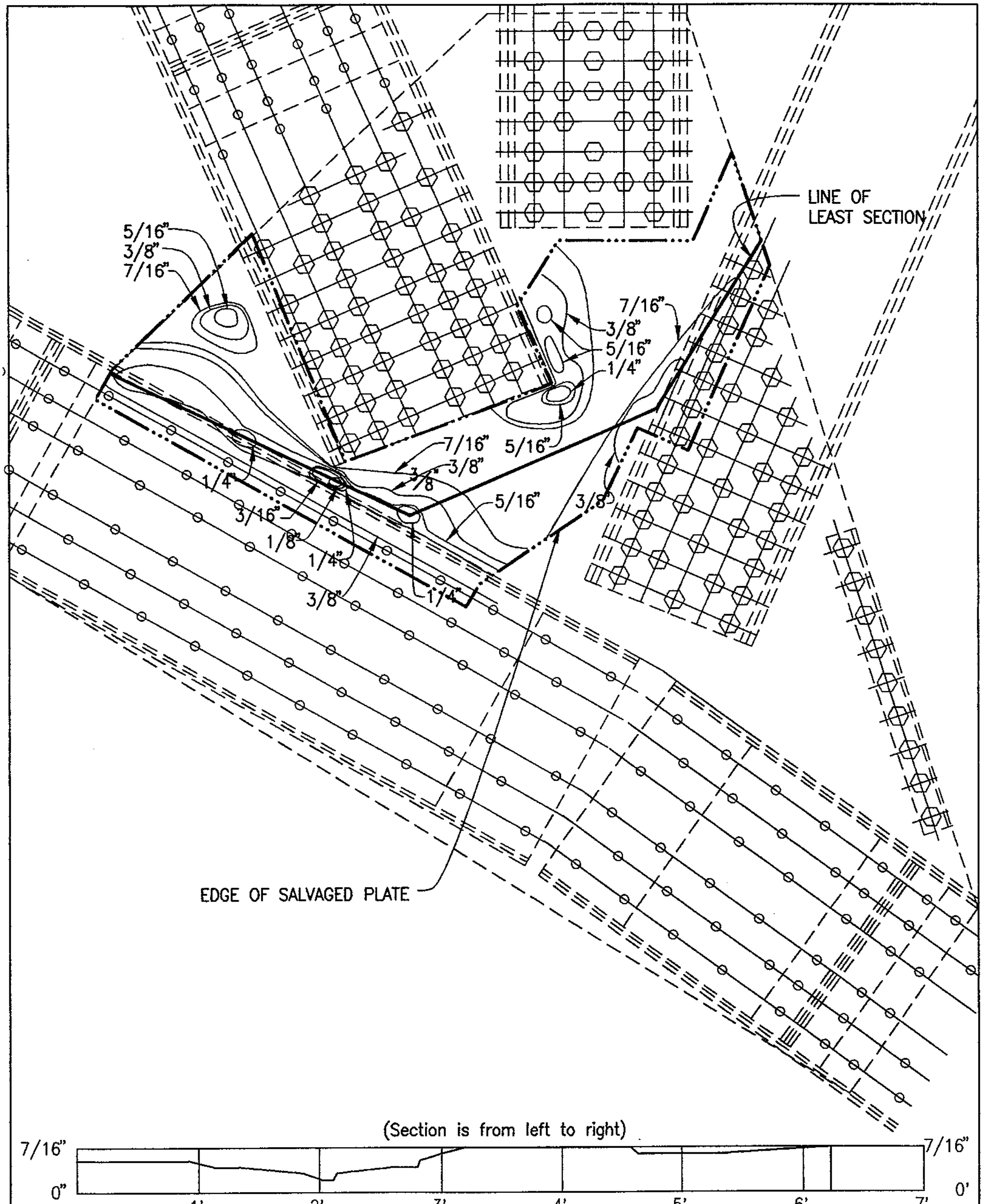
Contour plots of corrosion on the L8 gusset plates of Ohio bridge LAK-90-2342, which partially collapsed on May 24, 1996.

Documents provided by Ohio Department of Transportation.



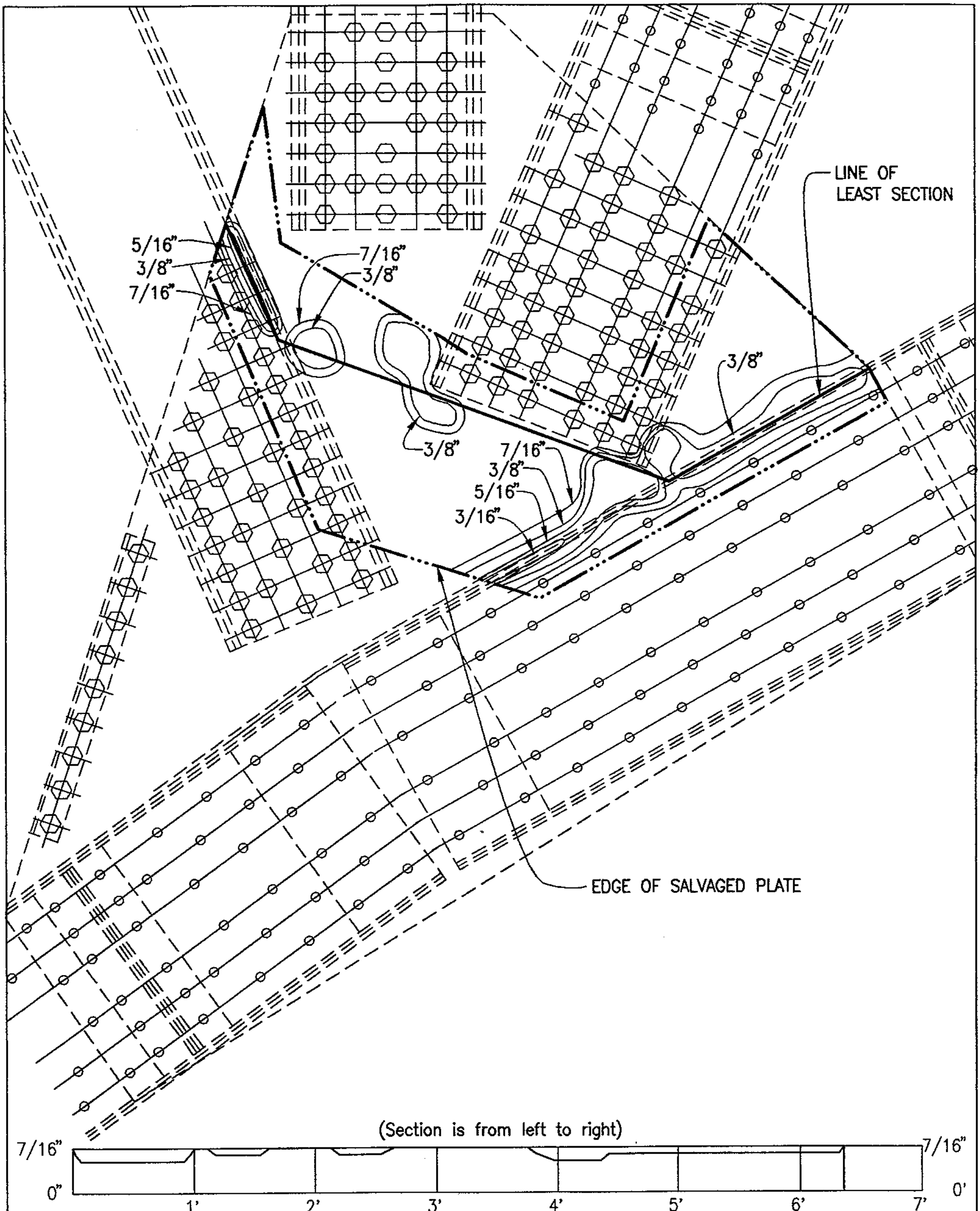
NORTH TRUSS INTERIOR

30% Section loss along line of least section



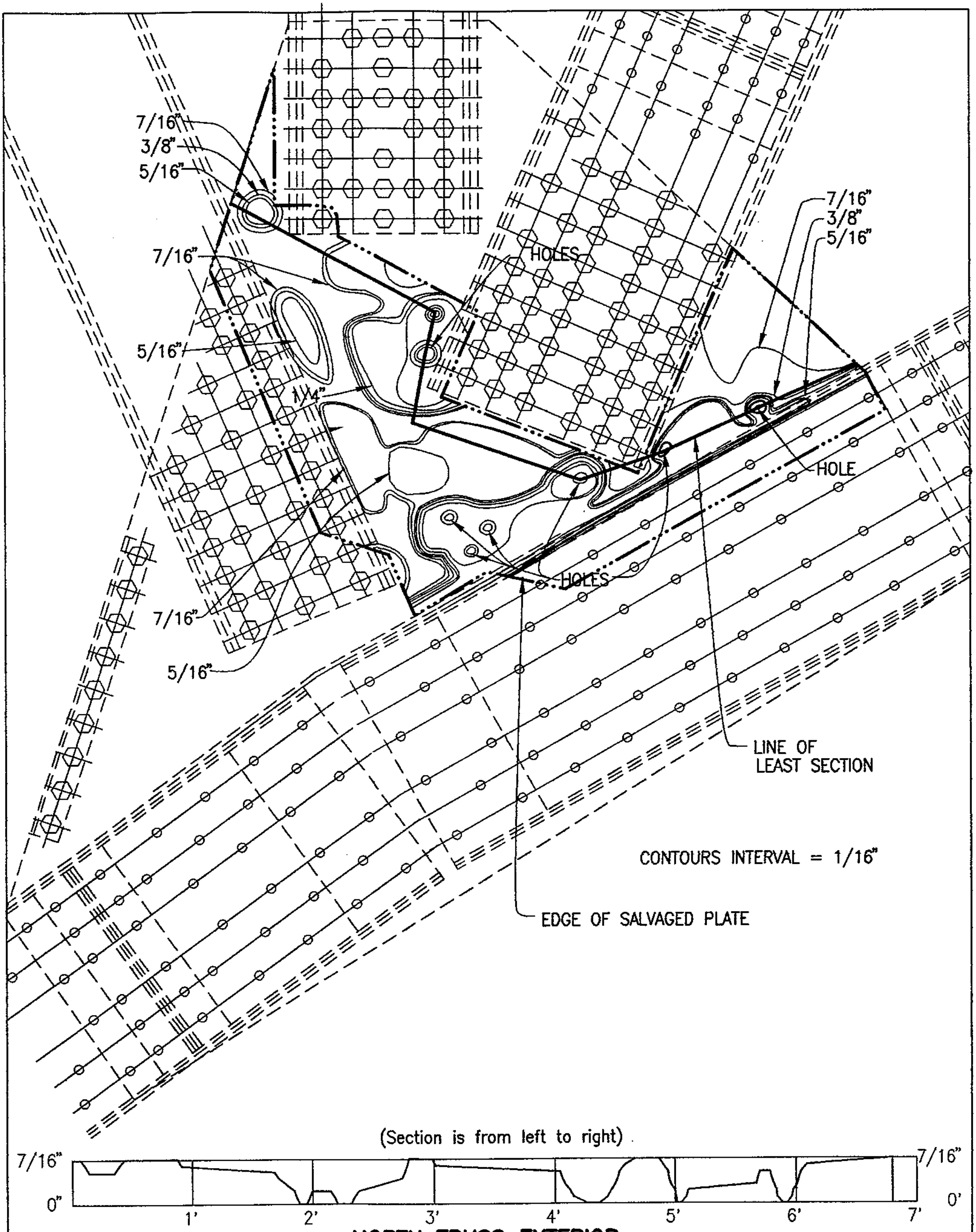
SOUTH TRUSS EXTERIOR

23% Section loss along line of least section



SOUTH TRUSS INTERIOR

13% Section loss along line of least section



NORTH TRUSS EXTERIOR

36% Section loss along line of least section