Lehigh University
Lehigh Preserve

Fritz Laboratory Reports

Civil and Environmental Engineering

1974

Field evaluation of tie plate geometry, Nov. 1974

J. H. Daniels

J. W. Fisher

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION

BUREAU OF MATERIALS, TESTING AND RESEARCH
RESEARCH REPORT

FIELD EVALUATION OF TIE PLATE GEOMETRY

BY

J. Hartley Daniels
John W. Fisher

Research Project No. 72-3

High Cycle Fatigue of Welded Bridge Details

LEHIGH UNIVERSITY
Office of Research

Fritz Engineering Laboratory Report No. 386.4

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.

4. Title and Subtitle	5. Report Date
FIELD EVALUATION OF TIE PLATE GEOMETRY	November 1974
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
J. Hartley Daniels, John W. Fisher	F.E.L. Report 386.4
9. Performing Organization Name and Address	10. Work Unit No.
Fritz Engineering Laboratory #13 Lehigh University Bethlehem, Pennsylvania 18015	11. Contract or Grant No. PennDOT No. 72-3
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Pennsylvania Department of Transportation P. O. Box 2926 Harrisburg, Pennsylvania 17120	Interim Report for March 1974 to November 1974
	14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the Pennsylvania Department of Transportation and the U. S. Department of Transportation, Federal Highway Administration.

16. Abstract

This investigation demonstrates that changing tie plate thickness has negligible effect on the displacement induced stresses resulting from in-plane bending of the tie plate. Releasing the tie plate to girder connection and reducing tie plate width lowers the in-plane bending stress resulting in increased fatigue life of the tie plate. These conclusions were reached after testing six tie plates having three different configurations and two plate thicknesses at two test locations on the Lehigh Canal Bridge.

17. Key Words	18. Distribution Statement
Bridges, Fatigue, Field Testing, Structural Behavior	

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	23	

Form DOT F 1700.7 (8-69)

COMMONWEALTH OF PENNSYLVANIA

Department of Transportation

Bureau of Materials, Testing and Research

Leo D. Sandvig - Director
Wade L. Gramling - Research Engineer
Kenneth L. Heilman - Research Coordinator

Project 72-3: High Cycle Fatigue of Welded Bridge Details

FIELD EVALUATION OF

TIE PLATE GEOMETRY

by

J. Hartley Daniels

John W. Fisher

Prepared in cooperation with the Pennsylvania Department of Transportation and the U. S. Department of Transportation, Federal Highway Administration.

LEHIGH UNIVERSITY

Office of Research

Bethlehem, Pennsylvania

November 1974

Fritz Engineering Laboratory Report No. 386.4

TABLE OF CONTENTS

1. INTRODUCTION

In order to explore various schemes for repairing or replacing the existing cracked tie plates of the Lehigh River and Lehigh Canal bridges on Rt. 22 near Allentown, Pennsylvania, a special study was undertaken at two tie plate locations on the eastbound portion of the Lehigh Canal bridge. Field tests at these two locations were made on April 17 and 18, 1974 under normal vehicular traffic which included the periodic passage of a two-axle test truck of known weight. The objective of the study was to ascertain the in-plane stress range response of six different tie plates at the two test locations. Each of the tie plates had a different geometrical configuration in order to evaluate the influence of variations in tie plate geometry and thickness. In addition each of the tie plates which connect the top flanges of the floor beam and floor beam bracket across the top flange of the main longitudinal girders was either bolted to the girder flange or left unbolted to evaluate this influence on the tie plate strains.

The stress history pilot study on the Lehigh Canal Bridge had shown that large cyclic in-plane stress ranges were being experienced in the tie plates of this bridge structure[1]. A preliminary analysis indicated that the primary cause of the in-plane bending stresses were certain displacements and other conditions not accounted for in the design of the structural components. In order to evaluate the influence of changing the tie plate size in both width and thickness as well as the influence of the restraining connection to the main girder, stresses and deflections were measured under normal vehicular traffic and under a known test load. This report summarizes the results of this special study.

2. DESCRIPTION OF TESTS

Two test locations were selected near the southwest corner of the eastbound portion of the Lehigh Canal bridge as shown in Fig. 1. Location 1 was on the south girder adjacent to Abutment C. Location 2 was on the south girder at the first interior floor beam. The cracked tie plates existing at these two locations were first removed by driving out the rivets connecting the tie plates to the flanges of the floor beam, floor beam bracket and main girder. The extent of the cracking in these and other tie plates prior to the test is shown in Fig. 1

Six tie plates having three different configurations and two plate thicknesses were fabricated for this study. Each configuration is described by the width of the center portion of the plate (10 in., 8 in. of 6 in.) as shown in Fig. 2. For each configuration one plate had a thickness of 1/2 in. The other plate had a thickness of 1 in. For the 10 in. and 8 in. tie plates, 1-1/4 in. x 2-1/2 in. slotted holes were used at the connection to the main girders as shown in Fig. 2. These holes were not provided in the 6 in. tie plates. Seven-eighth inch diameter high strength A325 bolts with washers were used in 1-1/4 in. diameter holes to connect the plates to the floor beam, bracket and main girder. Oversized holes were used to enable fitting up of the plates in the field. All bolts were coated with beeswax and tightened to their maximum rated capacity using a calibrated torque wrench to provide a friction type bolted joint.

The test series is outlined in Table 1. Strain variation in the tie plates as well as strains in the web of the bracket at location 1 and horizontal displacements near locations 1 and 2 were recorded on an analog trace. Up to twelve of the total of twenty-two electrical readings were taken simultaneously. Each test therefore consisted of simultaneous recording from up to twelve selected gages for a length of time sufficient to include approximately 30 minutes of normal truck traffic and up to two passages of the test truck. For example, Tests Nos. 1 and 2 include readings from all electrical gages for one position of the two 10 in. tie plates for approximately one hour of truck traffic and four passages of the test truck.

In all but one of the tests the one inch thick tie plate was placed at location 1. However in Test No. 11 the 10 in. tie plates were alternated from their position in Test Nos. 1 and 2 so that the one inch plate was placed at location 2. This was done to ascertain the effect of changing plate thicknesses at the two locations. The objective of Tests Nos. 3 to 9 was to determine the effect of plate width and girder connection on the tie plate strains. Test 10 was used to evaluate the effect of tie plate restraint on girder displacements.

Electrical resistance strain gages were placed at mid-depth on the edges of the tie plates as shown in Fig. 3. The gages used were one-quarter inch long foil gages oriented to measure in-plane bending strains of the tie plates. Electrical resistance gages specifically designed to measure displacements were located at positions 1 and 7 as shown in Fig. 4.

These gages were oriented to measure the relative longitudinal horizontal displacement between the top flange of the main longitudinal girder and the bottom flange of the outside stringer as shown in Fig. 4. In addition two electrical resistance strain gages were placed on the web of the floor beam bracket at location 1 as shown in Fig. 5. These gages were placed directly opposite the uppermost rivet near the exterior edge of the bearing stiffener at that location and oriented to measure web (plate) bending strains.

Five Ames mechanical displacement gages were mounted as shown in Fig. 4. Each dial measured horizontal displacement of steel members in an east-west direction relative to the abutment back wall. Gages 2 and 5 were mounted at the bottom flanges of the two stringers. Gages 4 and 6 were positioned at the top and bottom flanges respectively of the main girder. Gage 3 was mounted on the top flange of the bracket and adjacent to the main girder.

The test truck used in the study was a two-axle truck provided by the District 5-0 office of the Pennsylvania Department of Transportation (PennDOT) in Allentown, Pennsylvania. The axle spacing was 152 inches. The front and rear axle loads measured 10,600 and 23,200 pounds respectively.

3. TEST RESULTS AND DISCUSSION

The stresses observed in the tie plates at locations 1 and 2 were found to be comparable to those measured earlier during the stress history study of the Lehigh Canal Bridge[1]. Relatively large stress ranges were detected in the tie plates under passage of the test truck and under the normal random vehicular traffic. The results of the study are summarized in Tables 2 to 5. Tables 2 and 3 present the stress range results for the 6 in., 8 in. and 10 in. tie plates. Each of the values shown in these tables was computed as the average of the maximum values of stress range obtained from the two strain gages located on opposite edges of a tie plate, at a particular gaged section, during all the tests for that particular tie plate configuration, plate thickness and girder connection (bolted or unbolted). Each value therefore usually represents the average of 4 to 6 maximum stress range values. The gaged section shown in Tables 2 and 3 refers to the location of a pair of gages on a tie plate. Pairs of gages, as shown in Fig. 3 were located over the bracket adjacent to the main girder, over the centerline of the girder and over the floor beam adjacent to the girder. These results indicate that the measurements under random traffic, although not observed under known truck loads, do provide a relative measure of tie plate response. Even though the loads producing the maximum stress ranges may have varied from test to test during the intervals of normal random traffic, the trends observed were nearly identical to those obtained under the known test truck loading.

Measurements on the gaged sections of the 8 in. tie plates on both sides of the main girder were influenced by the close proxomity of the slotted bolt holes connecting the tie plate to the girder flange as well as the stress concentration effect of the radius transition from the 8 to 10 in. width (Fig. 3). Because of this, a direct comparison at these gaged sections is not possible between the 8 and 10 in. tie plates. However, the results presented in Tables 2 and 3 show clearly that releasing (unbolting) the tie plates from the main girders does generally result in a substantial decrease in stress range in the tie plates. This range was further decreased by replacing the unbolted 10 in. tie plates with unbolted 6 in. tie plates. The reductions in stress range were observed at the bracket connection, over the centerline of the girder, and at the floor beam connection. The study also indicates that variations in plate thickness does not significantly alter the magnitude of stress range. Approximately the same stress ranges were observed in both the 1 in. and 1/2 in. tie plates confirming the fact that displacements are primarily responsible for the stresses that are being introduced into the tie plates. This is true at all three measured cross sections.

Table 4 shows the average of the maximum values of the range of relative horizontal east-west displacement, under normal random traffic, between the main girder and the outside stringer near locations 1 and 2. Within the range of expected experimental error, the values indicate that a significant relative displacement does occur which increases slightly as the restraint due to the tie plate connection is reduced. In addition it does not appear that tie plate thickness has any appreciable effect upon the displacements at the two tie plate locations.

The results of the mechanical deflection measurements at gage Nos. 2 to 6 shown in Fig. 4 are presented in Table 5. The table presents the average of the maximum values of the range of horizontal east-west deflection, under normal traffic, at each gage point as well as the maximum observed departure of the point from the equilibrium position in the east and west directions. The results shown in the table indicate that the deflections at the stringers and brackets were not much different across the width whereas the deflection at the top of the girder is about twice as great as experienced at the end of the stringers.

Since releasing the tie plate connection to the main girders did increase the relative tie plate to girder displacement slightly (Table 4), strain measurements were made on the web of the bracket at location 1. Electrical resistance strain gages were placed on both sides of the web as shown in Fig. 5. These gages were mounted to measure plate bending strains opposite the uppermost exterior rivet in the bearing stiffener where the bending restraint would likely be maximum. When the tie plate was bolted to the girder, no measureable strain was recorded at the web gages. Unbolting the tie plate to girder connection resulted in a maximum bending stress range of 4 ksi in the web. This low value can be expected since the web is very flexible. This indicates that unbolting the tie plates will have a negligible effect on the bracket, since the resulting web bending stresses are too low to result in fatigue damage.

4. CONCLUSIONS AND RECOMMENDATIONS

This investigation has demonstrated that changing the tie plate thickness has a negligible effect on the displacement induced stresses that result from in-plane bending of the tie plates. Installing new 10 in. wide tie plates of thickness equal to or greater than the existing tie plates and maintaining their connection to the girder will not prevent fatigue failure at a future date. The stress ranges under normal random traffic are certainly large enough to result in fatigue crack growth from bolt holes in twenty or more years.

Since the original structural design did not consider the structural action of the tie plates acting together with the girders, releasing the connection at the girders makes the structure act more in accordance with the original design assumptions. The slight increase in flexibility at the connection does not result in appreciable web bending stresses in the bracket. It is likely that the tie plates near the supports cracked many years ago. Since a cracked tie plate provides even more flexibility at the bracket to girder connection, and no fatigue damage of the bracket web is evident after this length of time, no deleterious effect on the structural integrity of the bracket or the bridge as a whole is expected if the tie plates are not bolted to the main girder.

On the basis of this study, as well as field observations and prior analytical calculations, it is recommended that 6 in. tie plates similar to those used in this study be installed at selected floor beam locations where the greatest fatigue damage has occurred or may be expected to occur. Referring to Fig. 6 it is recommended that 6 in. tie plates be installed at locations 1, 2, 3, 6, 7, 8, 9, 10 and 11 and similar locations on the other half span of the Lehigh Canal bridge as well as similar tie plate locations on the Lehigh River bridge. It is further recommended that the 6 in. tie plates not be bolted to the main girder. At the remaining tie plate locations where stresses are low 10 in. tie plates, similar to the existing tie plates, may be used. These tie plates may be bolted to the girder if desired.

Any solution that attempts to resist the displacements described herein that are occurring in the compound structure will result in large tie plate stresses. The design of this structure assumed a simple independent acting structural system. The interaction that has been built into the structure is not supposed to occur. Releasing the tie plates at the main girder will conform to the original design assumptions and still permit continuity to exist between the bracket and floor beam. It will also substantially eliminate the observed interacting effect between tie plate and girder and minimize the large tie plate stresses that result from rotation of the girder cross section.

5. ACKNOWLEDGMENTS

This investigation is part of a research project being conducted at Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania. The project is sponsored by the Pennsylvania Department of Transportation (PennDOT) and the U. S. Department of Transportation, Federal Highway Administration (FHWA).

Thanks are due Mr. Hugh T. Sutherland for his expertise and assistance in assembling the data acquisition system and instrumenting the bridge, Dr. B. T. Yen for his help in acquiring the data and to the project staff for their participation. The assistance of the Fritz Engineering Laboratory staff is acknowledged. The advice and assistance from the PennDOT and FHWA personnel is sincerely appreciated.

6. TABLES AND FIGURES

TABLE 1 - TEST SERIES

Tie Plates	Location	Test No.	Girder Connection
1 x 10 1/2 x 10	1 2	1, 2	Both tie plates bolted
1 x 10 1/2 x 10	1 2	3	Both tie plates unbolted
1 x 6 1/2 x 6	1 2	4, 5	Both tie plates unbolted
1 x 8 1/2 x 8	1 2	6, 7	Both tie plates bolted
1 x 8 1/2 x 8	1 2	8	1 in. tie plate bolted 1/2 in. tie plate unbolted
1 x 8 1/2 x 8	1 2	9	Both tie plates unbolted
No tie plates		10	
1 x 10 1/2 x 10	2 1	11	Both tie plates unbolted

TABLE 2 - TIE PLATE STRESS RANGES UNDER TEST TRUCK LOADING

Gaged Section	Plate Thickness	10 in. Width		8 in. Width		6 in. Width
		Bolted	Unbolted	Bolted	Unbolted	Unbolted
	in.	ksi	ksi	ksi	ksi	ksi
Near Bracket	1	10.5	9.5	24.0	4.5	3.6
	1/2	10.5	8.5	18.5	5.0	3.6
Center of Girder	1	5.0	7.0	7.0	4.5	2.5
	1/2	5.0	5.0	8.5	4.5	3.7
Near Floor Beam	1	7.0	7.0	16.5	5.0	2.0
	1/2	9.3	5.5	22.5	5.0	3.1

TABLE 3 - TIE PLATE STRESS RANGES UNDER RANDOM TRAFFIC LOADING

Gaged Section	Plate Thickness	10 in. Width		8 in. Width		6 in. Width
		Bolted	Unbolted	Bolted	Unbolted	Unbolted
	in.	ksi	ksi	ksi	ksi	ksi
New Bracket	1	20.5	18.0	32.0	9.0	8.5
	1/2	21.5	12.0	20.0	9.0	6.5
Centerline of Girder	1	10.0	15.5	8.0	6.5	5.0
	1/2	11.0	9.0	11.0	7.7	5.5
New Floor Beam	1	11.0	12.0	18.0	---	4.0
	1/2	12.0	8.5	26.0	8.0	4.5

TABLE 4 - RELATIVE GIRDER - OUTSIDE STRINGER DISPLACEMENTS

Location	Gage No. (Fig. 4)	10 in. Width		6 in. Width
		Bolted in.	Unbolted in.	Unbolted in.
1	1	0.07	0.078	0.094
2	7	0.07	0.063	0.094

TABLE 5 - DEFLECTION MEASUREMENTS AT LOCATION 1

UNDER RANDOM TRUCK TRAFFIC

Gage No. (Fig. 4)	Maximum Deflection Range in.	Departure	
		East in.	West in.
2	0.028	0.019	0.009
3	0.020	0.015	0.005
4	0.056	0.045	0.011
5	0.017	0.016	0.001
6	0.048	0.009	0.039

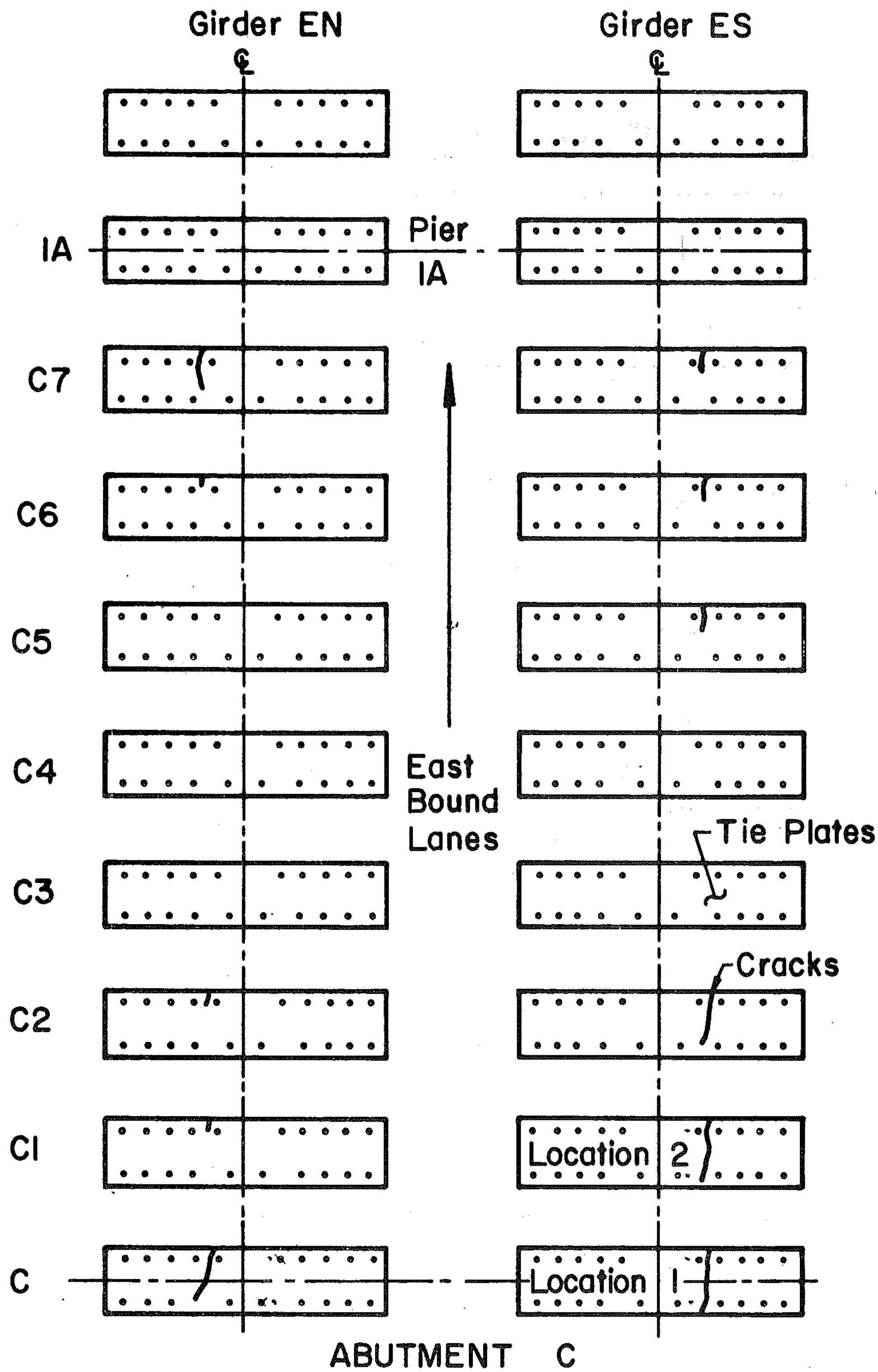


Fig. 1 Tie Plate Test Locations 1 and 2

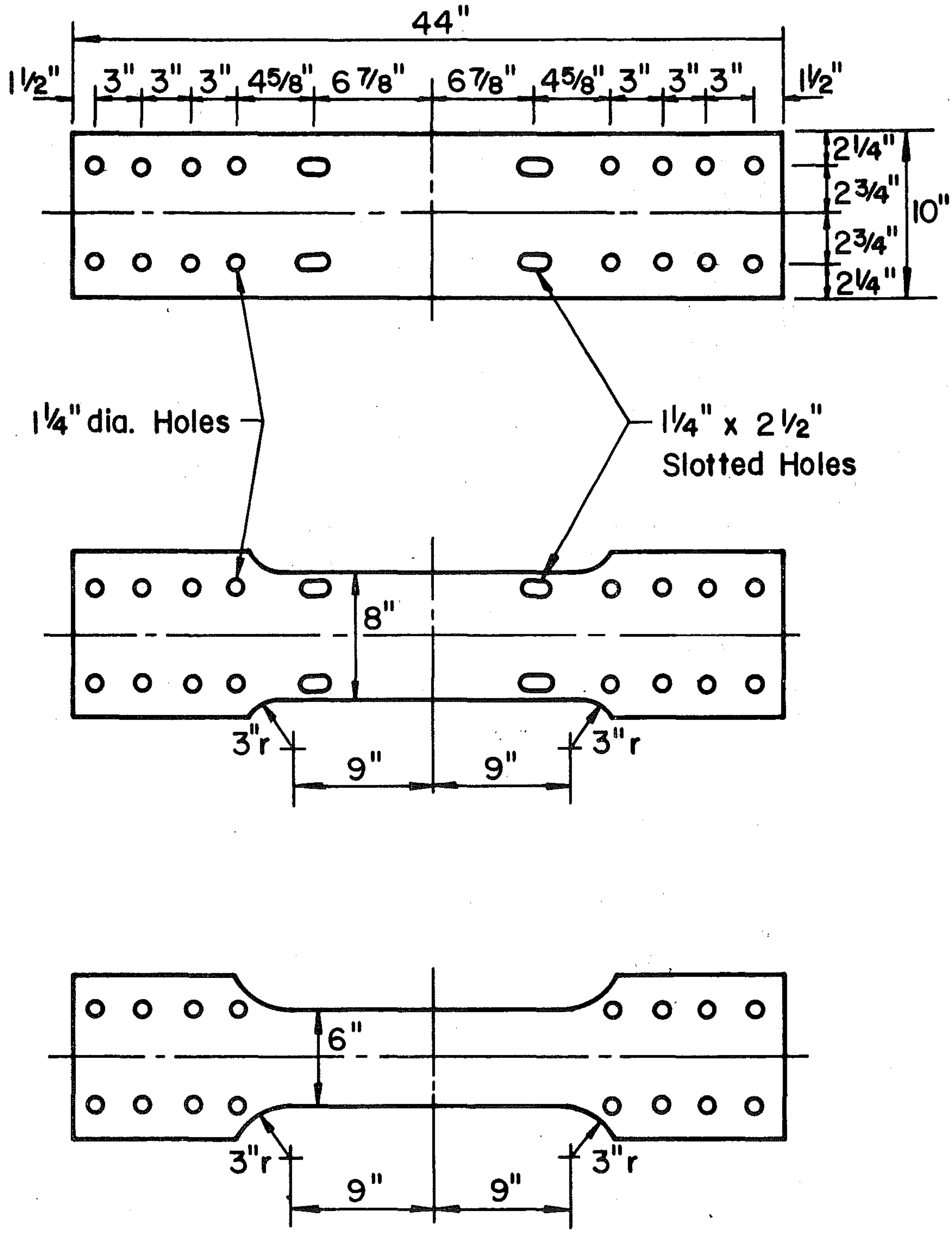


Fig. 2 Tie Plate Configurations

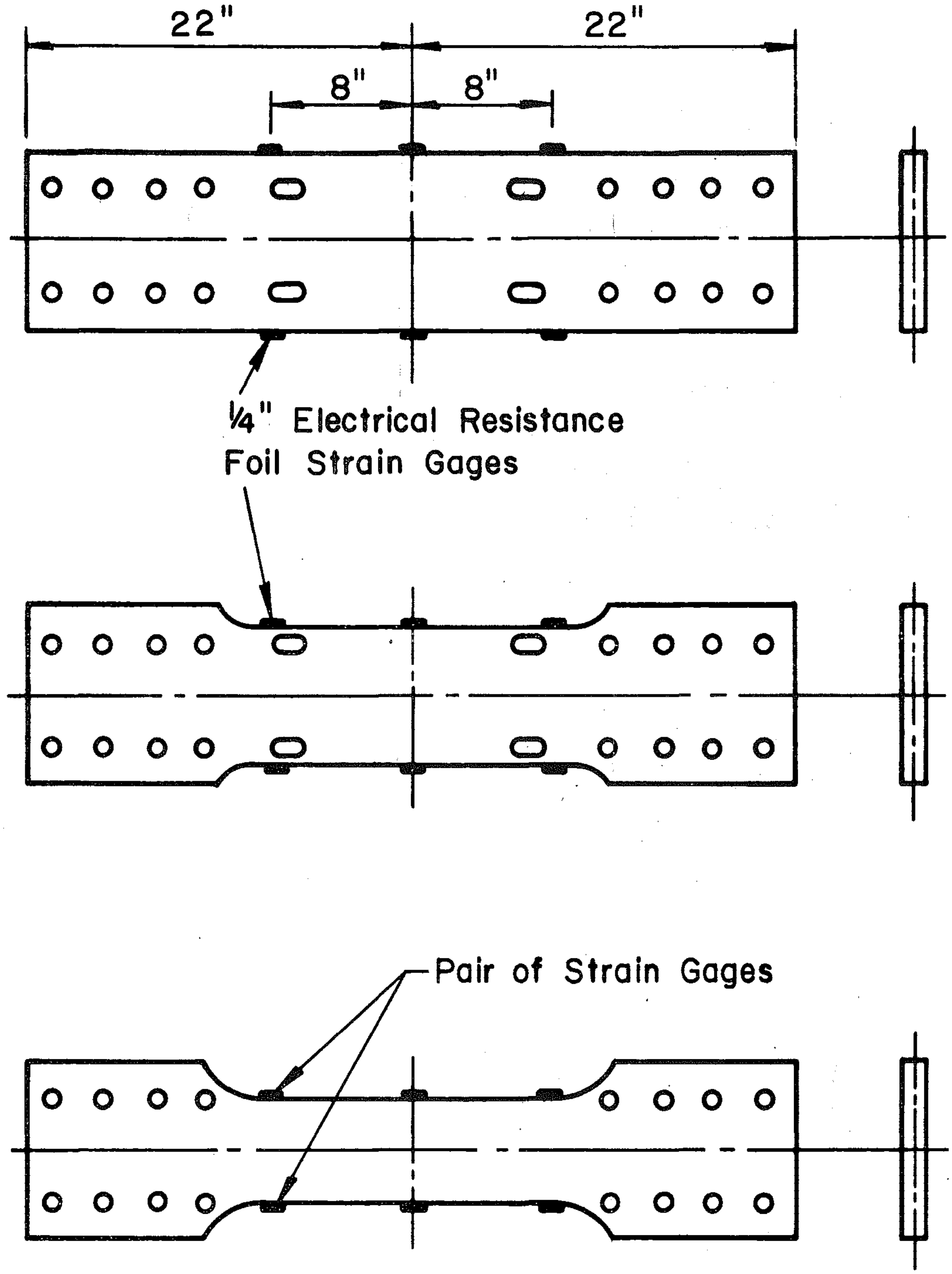


Fig. 3 Tie Plate Instrumentation

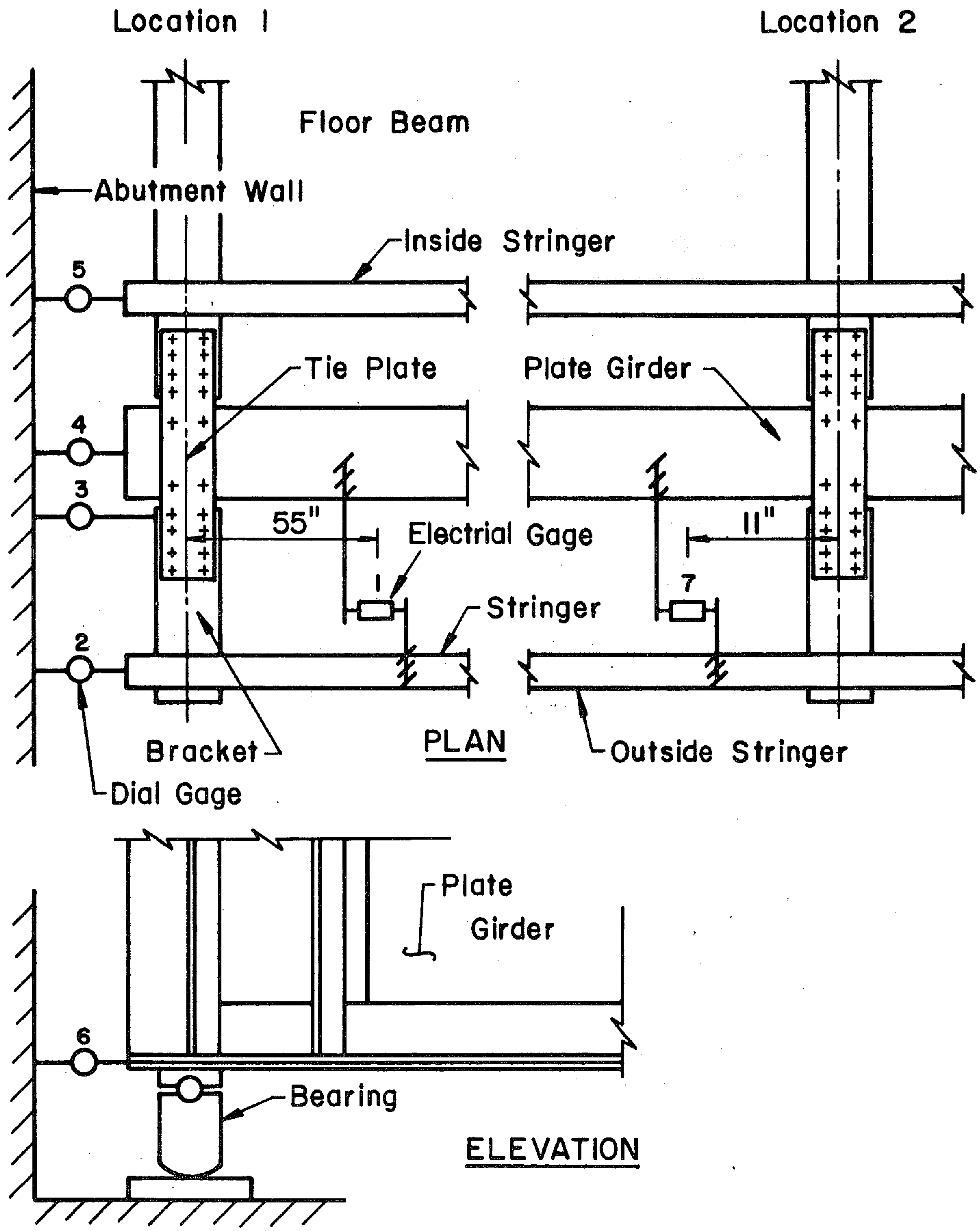
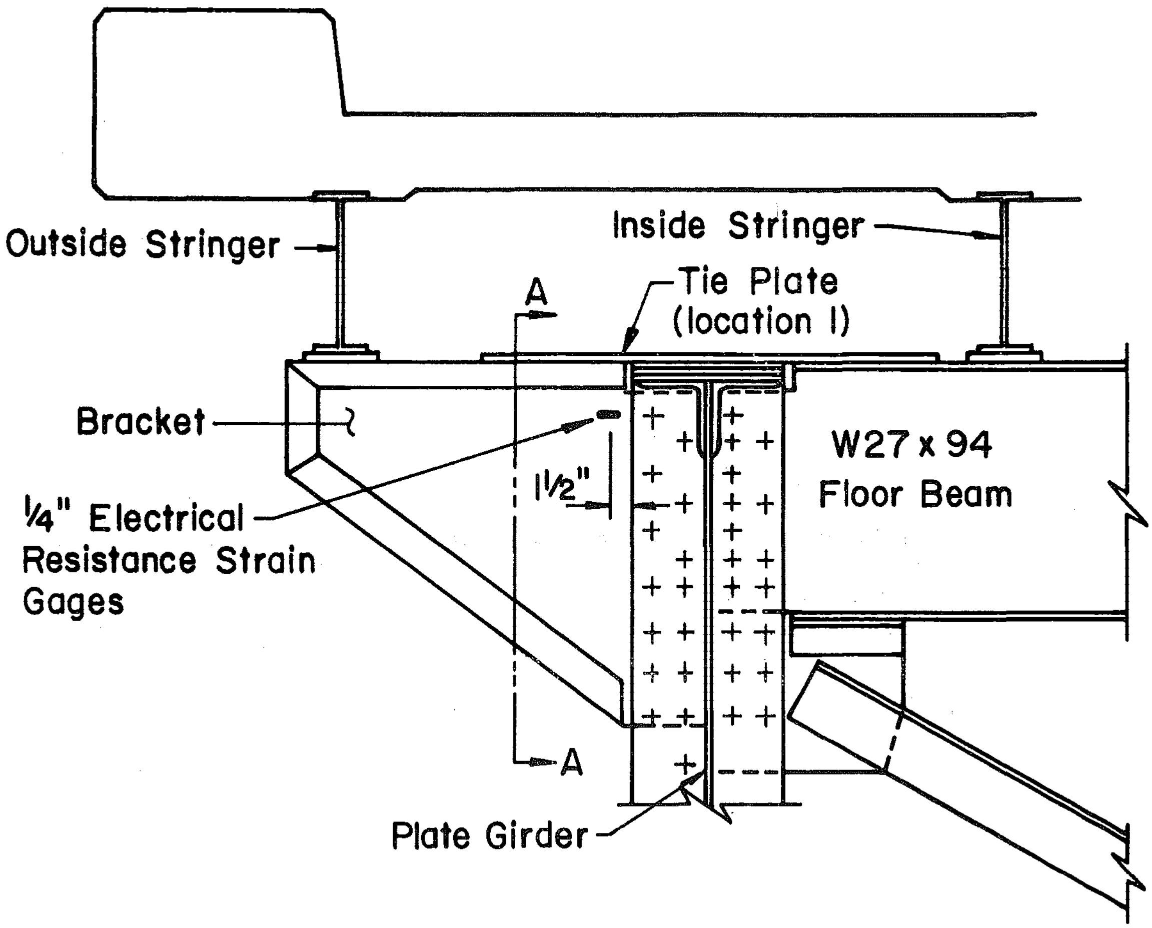


Fig. 4 Location of Deflection Gages



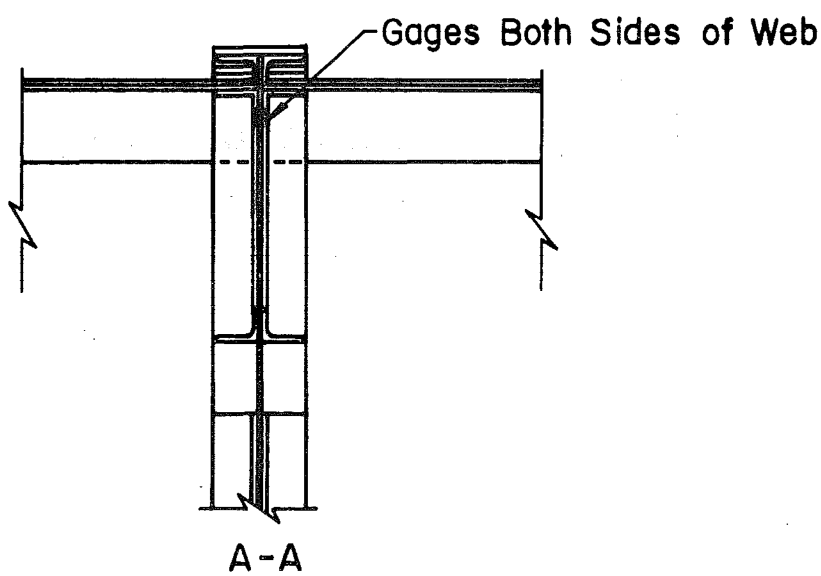


Fig. 5 Bracket Web Gages at Location 1

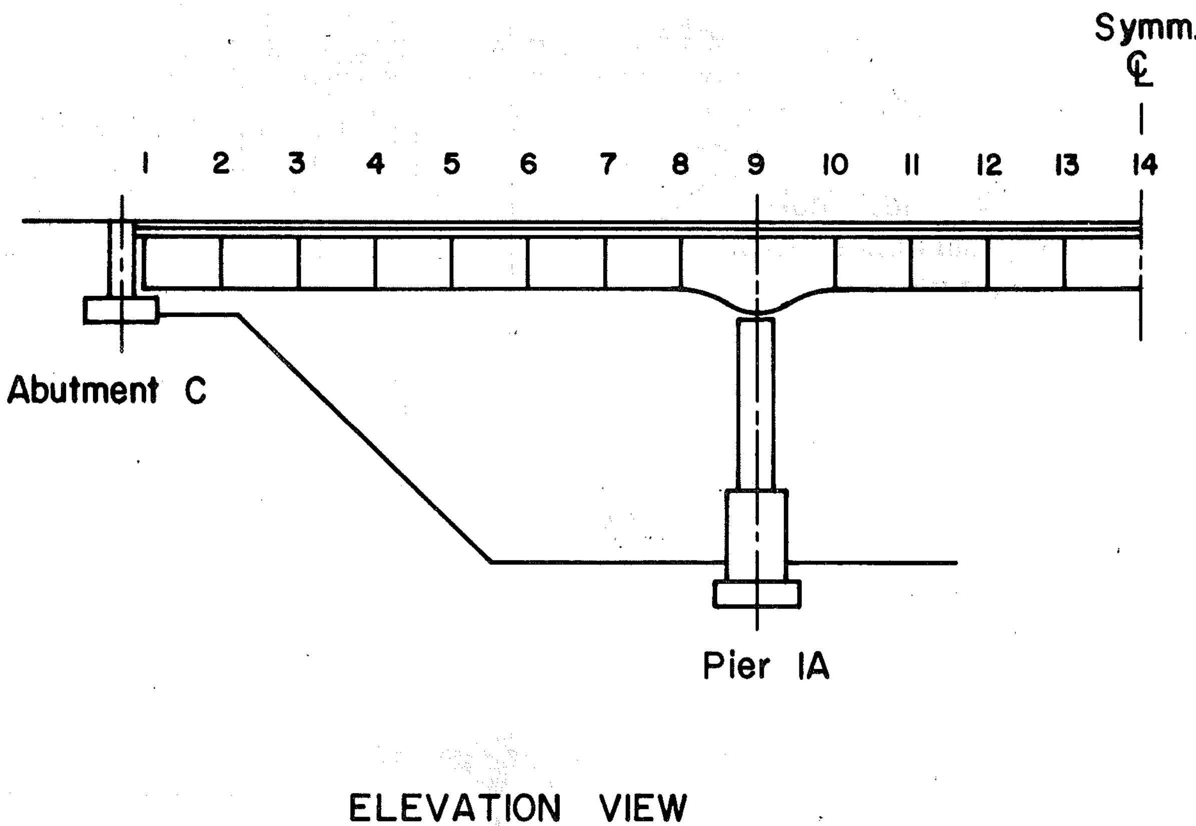


Fig. 6 Tie Plate Locations on the Lehigh Canal Bridge

7. REFERENCES

1. Fisher, John W., Yen, Ben T. and Marchica, Nicholas V.
FATIGUE DAMAGE IN THE LEHIGH CANAL BRIDGE, Fritz Engineering Laboratory, Report No. 386.1, Lehigh University, November 1974.