INSPECTION AND RATING OF TEN BRIDGES

Final Report
Project Number ST 2019-15

by

J. M. Stallings
C.H. Yoo

Auburn University Highway Research Center
Auburn University, Alabama

sponsored by

The State of Alabama Highway Department
Montgomery, Alabama

August 1992

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of Alabama Highway Department or Auburn University. The report does not constitute a standard, specification, or regulation.

ACKNOWLEDGEMENT

The material contained herein was obtained in connection with a research project, "Inspection and Rating of Ten Bridges, ST 2019-15," conducted by the Department of Civil Engineering through the Highway Research Center at Auburn University in association with A.G. Lichtenstein & Associates. Auburn University personnel provided structural analyses and load ratings for twelve bridges in Alabama. A.G. Lichtenstein & Associates provided the subconsultant engineering services of inspection and field data collection for each of the bridges.

The research project was sponsored by the State of Alabama Highway Department. The traffic control, bridge inspection vehicles and operators were provided by the Alabama Highway Department's Division and District offices. The cooperation and assistance of personnel from the Alabama Highway Department is gratefully acknowledged. Particular appreciation is expressed to Mr. Mike Harper, Chief of the maintenance Bureau, and Mr. Fred Conway, Chief of the Bridge Bureau, for their interest in and support of this project. Mr. Terry McDuffie who was appointed as a state liaison was extremely helpful as he provided prompt and able assistance throughout the project.

Contributions to the analytical rating work and report preparation were made by a number of personnel from Auburn University. Mr. Robert A. Fulton, Mr. Peter Wang, and Mr. Gang Lin, graduate students in the Civil Engineering Department, contributed to the project under direction of Professors J. Michael Stallings and Chai H. (Jay) Yoo. Special appreciation is expressed to Mr. Paul Blair and Mr. Charles Minervino of A.G. Lichtenstein & Associates for their prompt and kind cooperation in interpreting the field data and reviewing interim reports.

ABSTRACT

A total of twelve truss bridges in Alabama were inspected and load rated in this project. The Federal Highway Administration and the Alabama Highway Department have established guidelines and requirements related to highway bridges to ensure the safety of the monitoring public. To remain in compliance with these requirements each highway bridge must be periodically inspected and the safe load capacity of the bridge must be established through a load rating. Load ratings of a number of major bridges in Alabama have never been established because adequate information about the bridge structures is not available.

The primary objective of this project was to provide a bridge inspection and load rating for twelve bridges for which a load rating had never been established. The specific project objectives were (1) to perform a thorough field inspection of the bridges, (2) to collect all pertinent field data on the geometry and details of the bridges, and (3) to perform a structural analysis and load rating of each bridge based on the superstructure load carrying capacity.

An examination of the field inspection and load rating results for all the bridges leads to the following concluding remarks.

1. The load capacity of the truss spans was typically limited by the capacity of the stringers or floorbeams instead of the capacities of the truss members or truss connections.
2. All twelve bridges exhibit signs of deterioration to varying degrees. The major causes of deterioration are: corrosion (loss of member cross sectional area), vehicle impacts due to overheight vehicles and narrow lanes, and fatigue fracture of members and rivets. Most of the bridges exhibited deterioration from all of these.

3. Corrosion of a number of stringers, floorbeams, and bearing assemblies was found to result from leakage of water through deck joints or from drainage through deck drains onto structural members below.
4. Fatigue sensitive details were created in a number of fracture critical members by field welding performed to repair corrosion or impact damage.

TABLE OF CONTENTS

DISCLAIMER i

ACKNOWLEDGEMENT ii

ABSTRACT iii

CHAPTER 1 Introduction 1

CHAPTER 2 Rating Methodology 3

CHAPTER 3 St. Clair County Structure No. 004-58-021.9 11

CHAPTER 4 Morgan County Structure No. 003-52-021.9A 30

CHAPTER 5 Jackson County Structure No. 035-36-009.8A 52

CHAPTER 6 Morgan/Madison County Structure No. 053-52-008.7B 76

CHAPTER 7 Mobile County Structure No. 013-49-003.2 102

CHAPTER 8 Montgomery County Structure No. 009-51-040.7B 117

CHAPTER 9 Wilcox County Structure No. 021-66-023.2 131

CHAPTER 10 Mobile/Baldwin County Structure No. 016-49-037.6B 148

CHAPTER 11 Lauderdale County Structure No. 002-39-008.7A 167

CHAPTER 12 Coffee County Structure No. 125-16-000.2 179

CHAPTER 13 Truss Span of Cherokee County Structure No. 068-10-19.9 197

CHAPTER 14 Truss Span of Wilcox County Structure No. 28-66-12.0 211

CHAPTER 15 Concluding Remarks 241

REFERENCES 242

CHAPTER ONE
INTRODUCTION

PROBLEM STATEMENT

The Federal Highway Administration and the Alabama Highway Department have established guidelines and requirements related to highway bridges to ensure the safety of the motoring public. To remain in compliance with these requirements each highway bridge must be periodically inspected and the safe load capacity of the bridge must be established through a load rating. Load ratings of a number of major bridges in Alabama have never been established because adequate information about the bridge structures is not available.

PROJECT OBJECTIVES

The overall goal of the project reported here was to provide a bridge inspection and load rating for twelve bridges (extended from ten bridges covered by original project) for which a load rating had never been established. The twelve bridges are listed in Table 1.1. The specific project objectives were 1) to perform a thorough field inspection of the bridges, 2) to collect all pertinent field data on the geometry and details of the bridges, and 3) to perform a structural analysis and load rating of each bridge. These objectives were achieved in accordance with all applicable Federal Highway Administration requirements and current methods of engineering practice.

FORMAT OF REPORT

A large amount of field data, rating calculations, computer output and other information was generated in meeting the project objectives stated above. To produce a coherent report of reasonable length it was necessary to limit the amount of information presented here.

The objective of the report presented here is to provide a description of the load rating calculations and results for each of the ten bridges. Condition Reports documenting the field inspection results for each bridge will be submitted separately. Field notes, drawings, and photographs documenting the inspections were submitted with each of the Condition Reports. These items were submitted separately so they can be stored with other Maintenance Bureau records for the individual bridges.

Table 1.1 Bridges for Inspection and Load Rating

County	Structure No.	Location
Coffee	125-016-000.2 Simple Thru Truss	City of Elba on Alabama 125 over White Water Creek at MP 0.1
Mobile	016-49-037.6B Simple Thru Truss with Lift Span	Mobile/Baldwin County line on US 90 over Tensaw River at MP 37.53
Jackson	035-36-009.8A Continuous & Simple Thru Truss	0.5 Mile North of Junction AL 40 on AL 35 over Tennessee River at MP 47.42
Morgan	003-52-021.9A Bascule Lift Spans Concrete Arch	City of Decatur on US 31 over Tennessee River, L&N & Southern Railroad at MP 360.50
Morgan	053-52-008.7B Thru Truss	Morgan/Madison County line on US 231 over Tennessee River at MP 307.30
Lauderdale	002-39-008.7A Simple Thru Truss	0.72 Miles West of Junction US 43 over Shoal Creek at MP 40.20
Montgomery	009-51-040.7B Simple Thru Truss Tallapoosa River MP 115.70	Montgomery/Elmore Co. Line on US 231 over
St. Clair	004-58-021.9 Simple Thru Truss	Riverside on US 78 over Coosa River at MP 142.68
Wilcox	021-66-023.2 Simple Thru Truss	4.6 Miles North of Furman on S.R. 21 over Cedar Creek at MP 96.70
Mobile	013-49-003.2 Continuous Movable Swing Span	2.6 Miles North of Bankhead Tunnel on US 43 over Three MIle Creek at MP 3.21
Wilcox	028-66-012.0 Thru Truss Span Deck Truss Spans	Over Alabama River on Highway 28
Cherokee	68-10-19.9 Thru Truss Span	Over Chattooga River Near Gaylesville

CHAPTER TWO
RATING METHODOLOGY

A structural analysis and load rating was carried out for each bridge according to accepted engineering practice and applicable AASHTO standards. As with any evaluation of an existing structure, it was necessary to make some assumptions in the analyses and ratings. For example, judgments had to be made about support conditions, whether or not connections were rigid or pinned, the effects of damage and deterioration on the load capacity of individual members, etc. Assumptions of this type were made based on normal standards of engineering practice. Descriptions of the normal analysis assumptions made for the various types of bridge construction are given later in this chapter. Special assumptions required for an individual bridge are discussed in the chapter related to that bridge.

The ratings were performed under the guidelines described by AASHTO (Manual 1983). Two types of rating methodologies are described by AASHTO (Manual 1983): the allowable stress method, and the load and resistance factor method. The allowable stress method is currently used by the Alabama Highway Department for establishing ratings for steel bridges. Hence, the allowable stress method was used in all rating calculations performed for steel bridge members under this project. The load and resistance factor method was used for reinforced concrete members where reinforcement details were available.

Quantitative bridge ratings were not calculated for substructure elements. Such calculations were outside the project scope. Quantitative calculations of ratings for substructure elements are not commonly performed when the details of the original construction are unknown. Typically because it is impossible to establish details such as amounts and locations of concrete reinforcing by visual inspection. Qualitative information on the condition of the substructure elements are given in the condition reports for each bridge.

INVENTORY AND OPERATING RATINGS

The rating results for each bridge are given in terms of Inventory and Operating ratings as defined by AASHTO (Manual 1983). Standard truck loadings shown in Figure 2.1. were provided by the Alabama Highway Department fur use in the rating calculations. The type 3S2 (AL) loading was added after the project started and was used in rating eight of the twelve bridges. Ratings were established by considering two types of loadings. For spans shorter than 200 ft., loadings created by a single standard truck in each lane were used to calculate the ratings as required by AASHTO (Manual 1983). Lane loadings were used to calculate the ratings for long spans

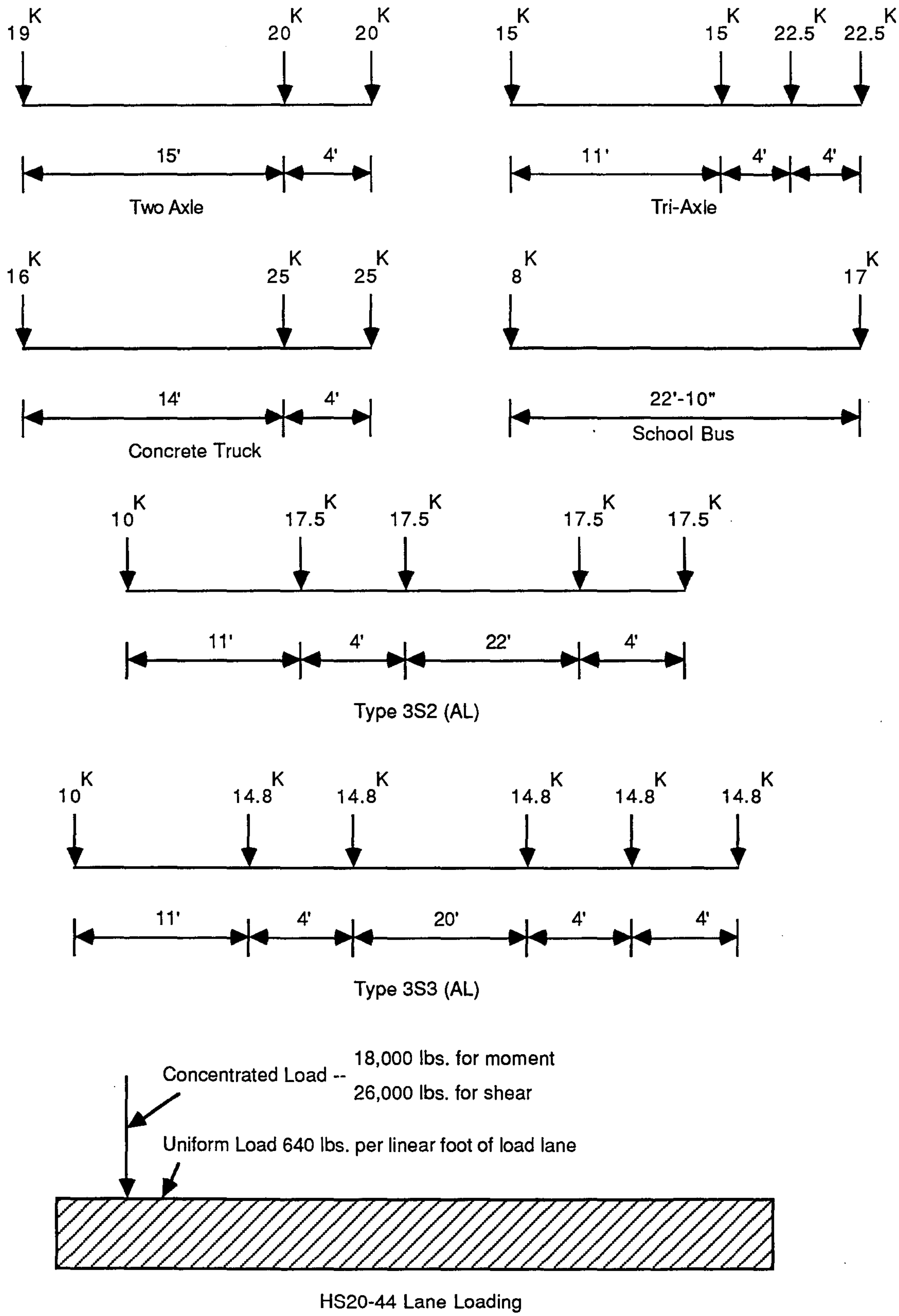


Figure 2.1 Standard Truck Loadings

(greater than 200 ft.) to account for the possibility of multiple vehicles in each lane. The AASHTO (Standard 1989) HS20-44 lane loading is also shown in Figure 2.1.

Lane loadings for the individual truck types of Figure 1.1 are not defined by AASHTO. Hence, it was necessary to use a rational means of calculating ratings for each of the standard truck types for long spans where lane loading conditions were critical. This was done by multiplying the rating determined for the HS20-44 lane loading by the ratio of the HS20 truck weight (36 tons) to the standard truck weight. The rating is expressed by

$$\text{Rating} = \frac{36T(\text{Rating for HS20-44 Lane Loading})}{\text{Standard Truck Weight}} \qquad (2.1)$$

The justification for this approach is simple. AASHTO defines the HS20-44 lane loading as a reasonable loading for roadways where a series of HS trucks, each with a gross weight of 36 tons, is possible. It appears reasonable to adjust the magnitude of the lane loading by using a ratio of the heaviest HS truck to the weight of the standard truck of interest. This approach does not specifically address truck spacings along the bridge, but appears to be reasonable since extensive traffic data for the bridge locations is not available.

Ratings for each bridge are given for two different cases referred to here as "As-Designed" and "Present-Condition". The As-Designed ratings were calculated without regard for damage and deterioration. The original member sizes and properties were determined as closely as possible and used in the bridge rating calculations. The As-Designed ratings provide an upper bound on the bridge capacity that can be used in making maintenance decisions concerning future repairs and/or strengthening. The terminology As-Designed does not specifically mean that the ratings were based on design loadings or allowable stresses used in the actual design of the structures. The Present-Condition ratings reflect the effects that damage and deterioration identified by the field inspections have on the load capacity of the bridge.

STRUCTURAL ANALYSIS AND RATING

Trusses

Structural analyses were performed to determine the dead load and live load stresses in each of the truss members. Except where specifically noted, the truss analyses were carried out using the common assumptions for analysis of planar pin jointed trusses. These analyses were carried out using a general structural analysis program BRIDGE developed by the principal investigator, Dr. J. Michael Stallings. Three dimensional analyses for a limited number of cases were performed using the general purpose finite element analysis program ABAQUS (Hibbitt et al. 1989) for verification purposes.

A series of test structures were analyzed to insure the validity of the BRIDGE program. The test structures are small structures that test all the program capabilities. In addition, comparison analyses were performed using the truss from span 6 of the St. Clair County Bridge 004-58-021.9 (discussed in Chapter Three). The truss was analyzed using both the BRIDGE program and ABAQUS. The truss was first analyzed using the dead loads only. A comparison of the results is given in Table 2.1. The BRIDGE program was then used to determine the maximum combination of dead load plus live load stress produced by the standard tri-axle truck. The truck locations along the span that produced the maximum member stresses were output. These locations were then used to perform comparison analyses with ABAQUS. The results are given in Table 2.2. The results given in Tables 2.1 and 2.2 indicate excellent agreement between the two analyses programs. This provides a high level of confidence in the BRIDGE program results.

Dead loads supported by the trusses were estimated from the field measurements of the bridge geometry and member sizes provided by A.G. Lichtenstein & Associates as a part of this project. The dead loads of the top wind bracing and floor system were applied to the trusses as joint loads. The dead loads were assumed to be distributed equally between the two main bridge trusses. The self-weights of the truss members were included in the truss analyses. The member self-weights per unit length were calculated as the product of the cross sectional area and the unit weight of steel (0.284 lb/in^3). A 30 percent increase in the self-weights of each of the truss members was used to account for the weights of the gusset plates, battens, lacing and rivets.

The maximum live load stress produced in each of the truss members was determined by using influence lines. An influence line was generated for each truss member. The influence line was then used to determine the critical position of each of the standard trucks. The influence line was also used to determine the appropriate locations for the lane loading.

Truss Connections

The load capacity of the truss connections and member splices were investigated to insure that the connections could transfer the member loads. The connection analyses were performed using standard methods for riveted connections. Corrosion and damage identified by the field inspections were incorporated in the determinations of the connections capacities. It should be noted that there is the potential for reduced connection capacity as a result of corrosion and rivet fatigue that cannot be identified by visual inspection and

Table 2.1 Comparison of Dead Load Stresses (St. Clair County Bridge 004-58-021.9, Span 6)

Member No.	Member	Stress from BRIDGE (ksi)	Stress from ABAQUS (ksi)	Percent Difference
1	L0U1	-6.3106	-6.3105	0
2	L0L1	6.9320	6.9320	0
3	L1U1	3.4528	3.4528	0
4	U1U2	-7.4307	-7.4307	0
5	L2U1	7.0963	7.0963	0
6	L1L2	6.9320	6.9320	0
7	L2U2	-0.5110	-0.5110	0
8	U2U3	-7.4307	-7.4307	0
9	L2U3	-2.2713	-2.2713	0
10	L2L3	8.1827	8.1827	0
11	L3U3	3.5545	3.5545	0
12	U3U4	-7.6327	-7.6327	0
13	L4U3	2.0039	2.0039	0
14	L3L4	8.1827	8.1827	0
15	L4U4	-0.5636	-0.5636	0
16	U4U5	-7.6327	-7.6327	0
17	L4U5	2.0039	2.0039	0
18	L4L5	8.1827	8.1827	0
19	L5U5	3.5545	3.5545	0
20	U5U6	-7.4307	-7.4307	0
21	L6U5	-2.2713	-2.2713	0
22	L5L6	8.1827	8.1827	0
23	L6U6	-0.5110	-0.5110	0
24	U6U7	-7.4307	-7.4307	0
25	L6U7	7.0963	7.0963	0
26	L6L7	6.9320	6.9320	0
27	L7U7	3.4528	3.4528	0
28	L8U7	-6.3106	-6.3105	0
29	L7L8	6.9320	6.9320	0

Table 2.2 **Comparison of Maximum Stresses Due to Dead Load Plus Tri-Axle Truck Loading (St. Clair County Bridge 004-58-021.9, Span 6)**

Member No.	Member	Stress from BRIDGE (ksi)	Stress from ABAQUS (ksi)	Percent Difference
1	L0U1	-9.797	-9.797	0
2	L0L1	10.762	10.762	0
3	L1U1	11.303	11.303	0
4	U1U2	-11.492	-11.492	0
5	L2U1	12.174	12.174	0
6	L1L2	10.762	10.762	0
7	L2U2	-0.511	-0.511	0
8	U2U3	-11.492	-11.492	0
9	L2U3	-5.913	-5.920	0.12
10	L2L3	12.634	12.634	0
11	L3U3	11.405	11.405	0
12	U3U4	-11.765	-11.765	0
13	L4U3	6.201	6.209	0.13
14	L3L4	12.634	12.634	0
15	L4U4	-0.564	-0.564	0

hammer sounding of the rivets. Such unseen conditions could not be incorporated in the rating calculations.

AASHTO (Manual 1983) provides allowable stresses for two types of rivets, Grade 1 and Grade 2. The allowable shear stress for Grade 1 rivets is less than the allowable stress for Grade 2 rivets. The grade of rivets actually used was not known for any of the twelve bridges. Conservative connection ratings were obtained by assuming that Grade 1 rivets were used. However, in some cases these conservative connection capacities governed the overall bridge ratings. According to past experience of A.G. Lichtenstein and Associates, Grade 1 rivets were seldom used in bridge construction. Hence, it was recommended in the ratings provided here that ratings based on the allowable stresses for Grade 2 rivets be used unless the field inspection identified widespread signs of overload or fatigue distress in the connections.

Truss Floor System

The truss floor systems were commonly made up of a noncomposite concrete deck on steel stringers. The steel stringers span between transverse floor beams at the truss panel points. The span lengths of the floor system members were relatively short so that the floor system ratings were not controlled by lane loadings. The structural analyses of the floor systems were carried out using the AASHTO (Standard 1989) wheel load distribution factors.

The floor system connections were commonly riveted connections between member webs. In such cases, the stringer and floor beam connections were assumed to be pinned connections. Under this assumption, no beam end moments were developed and no torsional moments were transferred to the floor beams by the stringers. Calculations of the connection capacities were performed using methods such as those described by McCormac (1971). The eccentricities of connection loads created by the gage distances of the connection angles were accounted for.

Concrete Girders and Decks

Details of the amounts and locations of the steel reinforcement in the concrete members were generally not available. It was impossible to calculate quantitative load ratings in such cases. Judgements about the capacities of concrete members are discussed for each bridge. Where reinforcement details were available, ratings were calculated according to the load and resistance factor method described by AASHTO (Manual 1983).

Fatigue Considerations

The remaining fatigue lives of the bridges were not considered in the calculations of the load ratings. Calculation of remaining bridge fatigue life and/or using fatigue requirements to establish allowable stress is not a part of the normal bridge rating process used by the Alabama Highway Department. However, comments are provided for some bridges where details especially prone to fatigue problems were identified during the field inspections.

CHAPTER THREE
ST. CLAIR COUNTY
STRUCTURE NO. 004-58-021.9

The St. Clair County bridge (004.58-021.9) spans the Coosa River near Riverside, Alabama on U.S. Highway 78 at Mile Post 142.68. A summary of the load rating results for the bridge is presented in this chapter. The results of the bridge inspection and condition survey are summarized by A.G. Lichtenstein & Associates (St. Clair County 1991).

The bridge was constructed in 1932 and underwent a major lift operation in 1972. The original bridge was made of three steel truss spans and eight reinforced concrete approach spans. The three truss spans are still in service. The approach spans were replaced by 12 reinforced concrete tee beam spans in 1972.

TRUSS RATINGS

Two of the three trusses (referred to here as span 6 and span 8) are simply supported spans of 200 ft. The dimensions, member sizes and details of spans 6 and 8 are identical. The simple spans are at each end of a 232 ft center bearing swing span truss (referred to here as span 7). Elevations showing the overall geometry, support conditions and joint numbering for trusses are given in Figures 3.1 and 3.2. Cross sectional properties for the members are given in Table 3.1 and 3.2.

Structural analyses of the trusses were performed to determine the dead load and live load stresses in each of the members. A summary of the dead load stresses is given in Table 3.3. Examples of the live load stress results are given for the tri-axle truck and the HS20-44 lane loading in Tables 3.4 and 3.5. Because span 7 is a swing span, dead load analyses were performed using two different sets of support conditions. The three supports as shown in Figure 3.2 were used in one dead load analysis. Another dead load analysis was performed using only the center support. Ratings for both dead load cases indicated that using the supports as shown in Figure 3.2 created the lowest ratings. The dead load stresses in Table 3.2 for span 7 resulted from using the three supports shown in Figure 3.2.

Summaries of the minimum inventory and operating rating factors calculated for the truss members are given in Tables 3.6 and 3.7. These ratings are based on the axial stresses created in the truss members by the standard loadings. Three types of ratings are shown in Tables 3.6 and 3.7. As-Designed ratings are given for loadings by single trucks and the equivalent

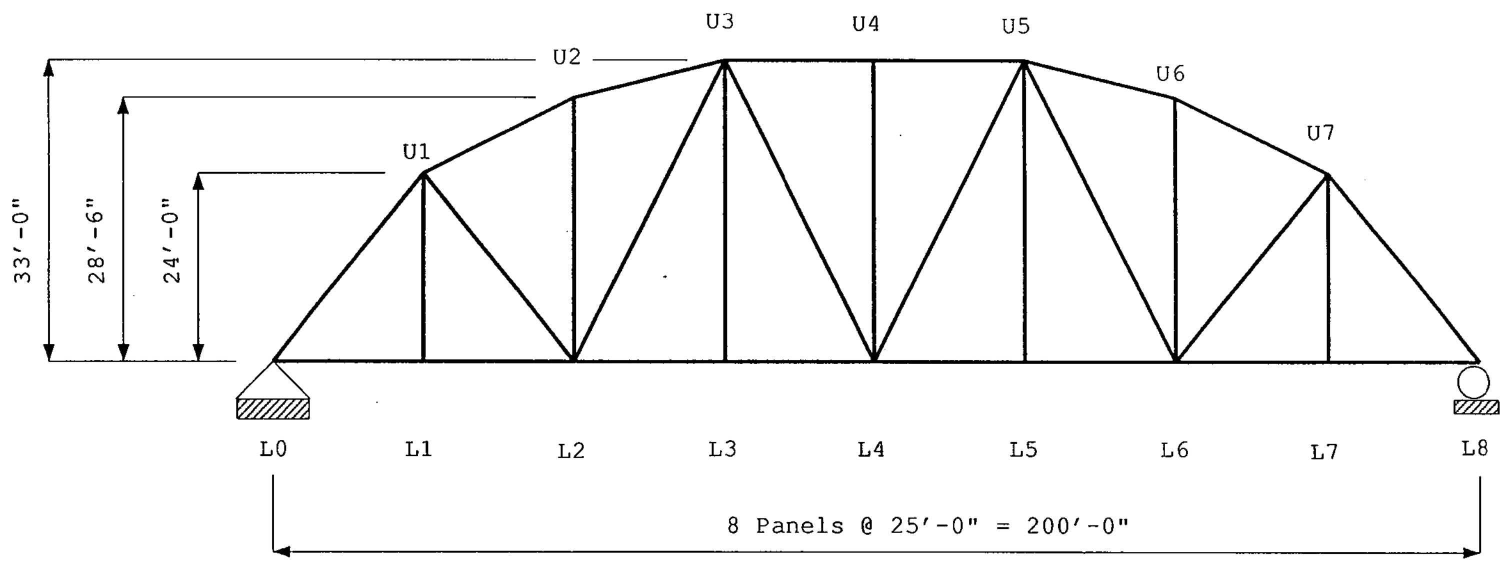


Figure 3.1 Geometry of Spans 6 & 8

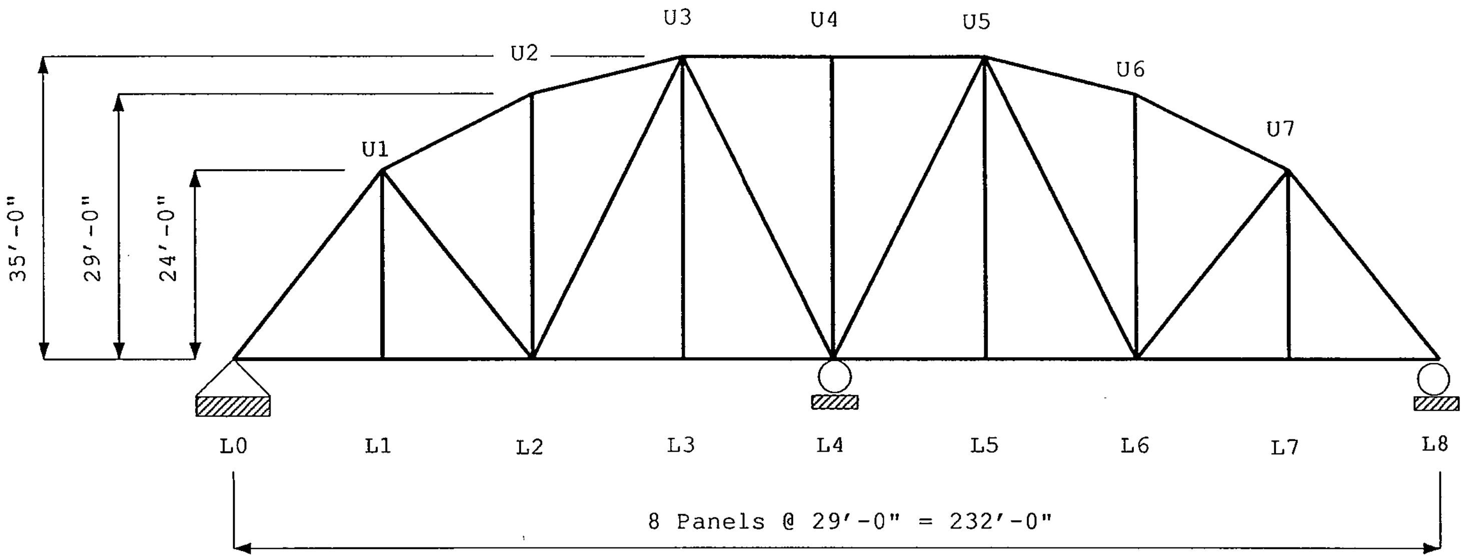


Figure 3.2 Geometry of Span 7

Table 3.1 Member Cross Section Properties for Spans 6 and 8 of St. Clair County Bridge 004-58-021.9

Member	Area ($inch^2$)	I_x ($inch^4$)	I_y ($inch^4$)
L0U1	30.2	1002.2	1099.6
L1U1			
L2U2	8.8	159.1	32.0
L3U3			
L4U4			
L2U1	12.1	256.2	452.5
L2U3	12.1	256.2	443.4
L4U3	12.1	256.2	452.5
U1U2			
U2U3	27.2	937.4	1012.5
U3U4	30.2	1002.2	1063.7
L0L1			
L1L2	19.8	625.2	793.6
L2L3			
L3L4	26.3	747.8	751.0

Table 3.2 Member Cross Section Properties for Span 7 of St. Clair County Bridge 004-58-021.9

Member	Area (inch^2)	I_x (inch^4)	I_y (inch^4)
L0U1	14.0	219.1	444.2
L1U1			
L2U2	8.8	159.1	32.0
L3U3			
L4U4			
L2U1	14.6	287.0	526.5
L2U3	14.6	287.0	526.5
L4U3	30.6	1254.0	1326.6
U1U2			
U2U3	11.7	157.0	431.4
U3U4	29.1	565.9	1064.3
L0L1			
L1L2	9.0	133.80	330.8
L2L3			
L3L4	19.80	625.20	809.1

Table 3.3 Dead Load Stresses

Member No.	Member	Dead Load Stress Span 6 & 8 (ksi)	Dead Load Stress Span 7 (ksi)
1	L0U1	-6.31	-3.69
2	L0L1	6.93	4.43
3	L1U1	3.45	3.75
4	U1U2	-7.43	-2.38
5	L2U1	7.10	-1.08
6	L1L2	6.93	4.43
7	L2U2	-0.51	-0.34
8	U2U3	-7.43	-2.38
9	L2U3	-2.27	4.41
10	L2L3	8.18	-0.69
11	L3U3	3.55	3.93
12	U3U4	-7.63	2.98
13	L4U3	2.00	-3.74
14	L3L4	8.18	-0.69
15	L4U4	-0.56	-0.61
16	U4U5	-7.63	2.98
17	L4U5	2.00	-3.74
18	L4L5	8.18	-0.69
19	L5U5	3.55	3.93
20	U5U6	-7.43	-2.38
21	L6U5	-2.27	4.41
22	L5L6	8.18	-0.69
23	L6U6	-0.51	-0.34
24	U6U7	-7.43	-2.38
25	L6U7	7.10	-1.08
26	L6L7	6.93	4.43
27	L7U7	3.45	3.75
28	L8U7	-6.31	-3.69
29	L7L8	6.93	4.43

Table 3.4 Maximum Live Load Stresses Due to Tri-Axle Truck

Member No.	Member	Maximum Live Load Stress Span 6 & 8 (ksi)	Maximum Live Load Stress Span 7 (ksi)
1	L0U1	-3.49	-5.65
2	L0L1	3.83	6.79
3	L1U1	7.85	8.10
4	U1U2	-4.06	-4.90
5	L2U1	5.08	-2.88
6	L1L2	3.83	6.79
7	L2U2	0	0
8	U2U3	-4.06	-4.90
9	L2U3	-3.64	5.05
10	L2L3	4.45	1.46
11	L3U3	7.85	8.10
12	U3U4	-4.13	1.24
13	L4U3	4.20	-2.75
14	L3L4	4.45	1.46
15	L4U4	0	0
16	U4U5	-4.13	1.24
17	L4U5	3.94	-2.90
18	L4L5	4.39	1.50
19	L5U5	7.85	8.12
20	U5U6	-3.96	-4.88
21	L6U5	3.79	5.27
22	L5L6	4.39	1.50
23	L6U6	0	0
24	U6U7	-3.96	-4.89
25	L6U7	4.88	-3.05
26	L6L7	3.71	6.55
27	L7U7	7.85	6.55
28	L8U7	-3.37	-5.45
29	L7L8	3.70	6.55

Table 3.5 Maximum Live Load Stresses Due to HS20-44 Lane Loading

Member No.	Member	Maximum Live Load Stress Span 6 & 8 (ksi)	Span 7 (ksi)
1	L0U1	-3.94	-3.93
2	L0L1	4.33	4.72
3	L1U1	5.21	5.49
4	U1U2	-4.62	-2.98
5	L2U1	4.82	-1.87
6	L1L2	4.33	4.72
7	L2U2	0	0
8	U2U3	-4.62	-2.98
9	L2U3	-2.25	4.14
10	L2L3	5.07	-0.85
11	L3U3	5.21	5.47
12	U3U4	-4.72	1.82
13	L4U3	2.39	-2.76
14	L3L4	5.07	-0.85
15	L4U4	0	0
16	U4U5	-4.72	1.82
17	L4U5	2.39	-2.76
18	L4L5	5.07	-0.85
19	L5U5	5.21	5.50
20	U5U6	-4.62	-2.98
21	L6U5	-2.25	4.15
22	L5L6	5.07	-0.85
23	L6U6	0	0
24	U6U7	-4.62	-2.98
25	L6U7	4.82	-1.87
26	L6L7	4.33	4.71
27	L7U7	5.21	5.47
28	L8U7	-3.94	-3.92
29	L7L8	4.33	4.71

Table 3.6 Inventory Rating Factors Based on Truss Member Capacities

Truck Type	As Designed Single Truck	As Designed Eq. Lane Loading	Present-Condition Eq. Lane Loading
Two Axle	1.77	1.54	1.23
Tri-Axle	1.37	1.21	0.97
Concrete Truck	1.54	1.38	1.10
3S2 (AL)	1.48	1.14	0.91
School Bus	4.18	3.63	2.89
HS20-44 Lane Loading	---	1.26	1.01

Table 3.7 Operating Rating Factors Based on Truss Member Capacities

Truck Type	As Designed Single Truck	As Designed Eq. Lane Loading	Present-Condition Eq. Lane Loading
Two Axle	2.42	2.41	1.96
Tri-Axle	1.87	1.89	1.54
Concrete Truck	2.11	2.15	1.75
3S2 (AL)	2.08	1.77	1.44
School Bus	5.70	5.68	4.62
HS20-44 Lane Loading	---	1.97	1.60

lane loadings defined by Equation 2.1. The As-Designed ratings for single trucks were controlled by the capacity of the end posts of span 7 (L0U1 and L8U7) except for one case. The one exception, the inventory rating for the 3S2 (AL) truck, was controlled by the upper chords (U3U4 and U4U5) of span 6. The As-Designed ratings for the AASHTO HS20-44 lane loading were controlled by the capacities of the upper chords (U3U4 and U4U5) of span 6. Hence, those upper chord members controlled all the As-Designed equivalent lane loading ratings.

The corrosion and impact damage identified during the field inspection was used to determine the Present-Condition ratings of the trusses. There was a great deal of moderate corrosion and impact damage to the trusses. Overall, it appears that the members of spans 6 and 8 show more damage than the members of span 7.

None of the truss members were found to have as much as 10 percent loss of cross sectional area due to corrosion. It is concluded that the corrosion damage has not significantly reduced the member capacities. A number of localized bent areas were found in the truss members. These were evaluated by recalculating the cross section of properties by the methods described by Yoo (1986). None of these local bends had a significant effect on the truss ratings.

The most significant truss damage was to the end post L8U7 on the south side of span 6. The member has been bent outward 15/16 inch (perpendicular to the centerline of the roadway) at a section six feet above the deck. The damaged area has been stiffened by repair plates, but the member is still bent outward. The effects of the bend were evaluated by considering the combination of axial load plus bending created by the bend. The rating factor for the member based on the HS20-44 lane loading was found to decrease from 2.54 to 1.60. It was also found that this member (L8U7 of span 6) controlled the Present-Condition ratings of the trusses.

Capacities of the truss connections were also evaluated. The connections are riveted with 3/4 inch diameter rivets. The rivets were assumed to be ASTM A502 Grade 1 rivets. The rivet diameter was taken from shop drawings of the trusses that were located by A.G. Lichtenstein & Associates. The rivets were assumed to be Grade 1 rivets because Grade 1 rivets were commonly used at the time the trusses were constructed and because the Grade 1 rivets are assigned the lowest allowable stresses by AASHTO (Manual 1983).

The load capacities of the connections were found to be larger than the member capacities. A summary of As-Designed ratings determined for the connections is given in Table 3.8. The ratings for the single truck loadings and equivalent lane loadings were controlled by the connection of member L3U3

Table 3.8 Inventory and Operating Rating Factors Based on Truss Connection Capacities

	As-Designed/Present Condition			
	Inventory		Operating	
Truck Type	Single Truck	Eq. Lane Loading	Single Truck	Eq. Lane Loading
Two Axle	2.04	2.39	2.93	3.85
Tri-Axle	1.52	1.88	2.18	3.03
Concrete Truck	1.72	2.14	2.47	3.44
3S2 (AL)	1.92	1.76	2.81	2.84
School Bus	5.06	5.64	7.28	9.08
HS20-44 Lane Loading	---	1.96	---	3.15

to joint U3 for all trucks except the 3S2 (AL) truck. The 3S2 (AL) truck created the most critical condition at the connection of member L1L2 to joint L2. It was judged that the visible effects of corrosion on the connection capacities was not significant. It is concluded that the As-Designed rating factors given for the connections in Table 3.8 are sufficiently accurate for use as Present-Condition ratings.

TRUSS FLOOR SYSTEM RATINGS

The floor system framing for all three trusses was very similar. A typical cross section through the floor system is shown in Figure 3.3. The floor system was made up of a 6.5 inch thick concrete deck on four stringers spaced at 5 feet 6 inches. The stringers span 25 feet between floorbeams at span 6 and 8. The stringers span 29 feet between floorbeams at span 7. A listing of cross section characteristics for the stringers and floorbeams is shown in Table 3.9.

The bending moments and shearing forces in the stringers and floorbeams were calculated by considering the riveted connections at the member ends acted as simple supports. The compression flanges of the stringers were embedded in the underside of the concrete deck over most of the central part of their span. It was judged that the concrete deck provided sufficient lateral restraint so that the stringer compression flanges are continuously braced. The compression flanges of the floorbeams were considered to be laterally braced at the stringer to floorbeam connections.

Summaries of the rating factors calculated for the stringers and floorbeams are given in Tables 3.10 and 3.11. These rating factors were based on the flexural capacities of the members. It was found that rating factors based on the shear capacities of the members were at least as large as those shown in Tables 3.10 and 3.11. Rating factors for floorbeams are shown only for span 7. It was found that rating factors for the floorbeams of spans 6 and 8 are larger than those shown for span 7. Similarly, the rating factors shown for the stringers of spans 6 and 8 were lower than those determined for the stringers of span 7.

No significant section losses were found along the stringers, so the stringer ratings shown in Table 3.10 serve as both As-Designed and Present-Condition ratings. The Present-Condition ratings shown in Table 3.11 for the floorbeams are somewhat lower than the As-Designed ratings. This results from corrosion damage found during the bridge inspection. The field inspection revealed 1/16 inch deep corrosion pitting along the top of the bottom flange of many of the floorbeams. This loss of section was accounted for in calculating the Present-Condition ratings.

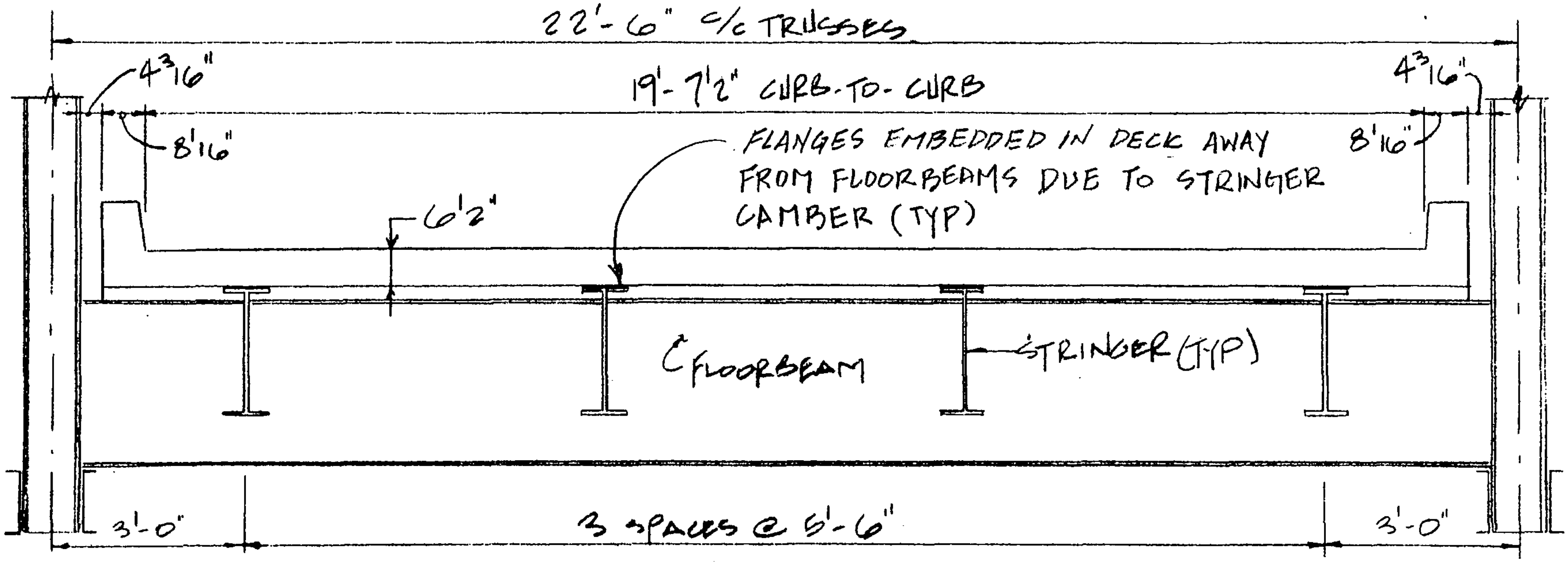


Figure 3.3 Typical Cross Section Through Truss Floor System for St. Clair County Structure No. 004-58-021.9

Table 3.9 Cross Section Properties for Stringers and Floorbeams

Member	Type	d (in)	b_f (in)	A (in^2)	I (in^4)
Floorbeams					
Span 6, 7 & 8	30 CB 115	30	10.5	33.81	4985
Stringers					
Span 6 & 8					
Outside	21 CB 58	21	8.0	17.05	1263
Inside	21 CB 60	21.03	8.02	17.64	1305
Span 7	24 CB 70	24	8.50	20.58	1954

Table 3.10 Rating Factors for Stringer of Spans 6 & 8

Truck Type	Dead Load Moment (ft-k)	Live Load Moment (ft-k)	Inventory	Operating
Two Axle	41	138	0.90	1.34
Tri-Axle	41	195	0.64	0.95
Concrete Truck	41	172	0.72	1.07
3S2 (AL)	41	125	1.00	1.48
School Bus	41	69	1.80	2.67

Table 3.11 Rating Factors for Floorbeams of Span 7

Truck	Dead Load	Live Load	As-Designed		Present-Condition	
Type	Moment (ft-k)	Moment (ft-k)	Inventory	Operating	Inventory	Operating
Two Axle	186	347	0.77	1.24	0.73	1.19
Tri-Axle	186	464	0.57	0.93	0.54	0.89
Concrete Truck	186	410	0.65	1.05	0.62	1.01
3S2 (AL)	186	325	0.82	1.32	0.78	1.21
School Bus	186	140	1.90	3.07	1.81	2.82

Rating factors calculated for the stringer to floorbeam and the floorbeam to truss connections are shown in Tables 3.12 and 3.13. No significant corrosion damage was found during the field inspections of the connections. Hence, the ratings given in Tables 3.12 and 3.13 serve as both As-Designed and Present-Condition ratings. The field inspections did identify three stringer to floorbeam connections with one missing rivet. One connection was in span 6, and two of the three connections were in span 8. It is assumed here that these missing rivets will be replaced in the near future by high strength bolts. For completeness, a rating factor was determined for the most critical of the three connections with a missing rivet. The Inventory rating factor determined for the tri-axle truck was 0.98.

APPROACH SPANS AND BRIDGE DECK

There are no structural drawings available that show the reinforcing details used in the reinforced concrete bridge decks or approach span tee beams. It was not possible to calculate quantitative rating factors for these members.

The field inspection results indicate that the concrete approach span beams appear to be in fair condition. The beams exhibit minor vertical hairline cracks and also a few localized areas of honeycombing. It is believed that the hairline cracks resulted from shrinkage stresses or flash set at the time the beams were constructed. No visible signs of overload distress were found in the approach spans that would require a reduction of the load ratings for the bridge.

The bridge deck was also found to be in fair condition. The top of the deck typically exhibited light to moderate scaling and some areas of hairline map cracking. Transverse cracks up to 1/32 inch wide in the top of the deck were common. One transverse crack up to 1/4 inch wide was found near panel point 6 in span 7. The underside of the deck generally exhibited small surface spalls, honeycombed areas and a few hollow areas. Transverse crack up to 1/16 inch wide were found. These signs of deterioration are reasonable for a bridge deck after almost 60 years in service. It is likely that reducing the weights of the trucks that cross the bridge would extend the service life of the deck. However, the condition of the deck does not require a reduction of the load ratings of the bridge.

CONCLUSIONS AND RECOMMENDATIONS

Structural analyses and load ratings of the bridge indicate that the flexural capacity of the floorbeams of span 7 control the bridge ratings. A summary of the Present-Condition ratings for the bridge are given in Table 3.14. The field inspections also identified the following maintenance and repairs that should be carried out:

Table 3.12 Rating Factors for Floor System Connections in Spans 6 & 8

Truck Type	Stringer to Floorbeam		Floorbeam to Truss	
	Inventory	Operating	Inventory	Operating
Two Axle	1.53	2.20	1.13	1.67
Tri-Axle	1.14	1.63	0.84	1.24
Concrete Truck	1.28	1.84	0.95	1.40
3S2 (AL)	1.78	2.55	1.31	1.93
School Bus	3.85	5.51	2.84	4.19

Table 3.13 Rating Factors for Floor System Connections in Spans 7

Truck Type	Stringer to Floorbeam		Floorbeam to Truss	
	Inventory	Operating	Inventory	Operating
Two Axle	1.38	2.00	1.21	1.79
Tri-Axle	1.03	1.50	0.90	1.34
Concrete Truck	1.16	1.70	1.02	1.51
3S2 (AL)	1.50	2.14	1.29	1.91
School Bus	3.42	4.98	2.99	4.43

Table 3.14 **Summary of Present-Condition Ratings for St. Clair County Structure No. 004-58-021.9**

Truck Type	Standard Weight (tons)	Inventory Rating Factor	Inventory Rating (tons)	Operating Rating Factor	Operating Rating (tons)
Two Axle	29.5	0.73	22	1.19	>29.5
Tri-Axle	37.5	0.54	20	0.89	33
Concrete Truck	33	0.62	20	1.01	>40
3S2 (AL)	40	0.78	31	1.21	>40
School Bus	12.5	1.81	>12.5	2.82	>12.5

1. The missing rivets in the stringer to floorbeam connections should be replaced by 3/4 inch diameter high strength bolts.

2. The truss members have sustained a great deal of impact damage through the years. The most frequent cause of this damage should be determined from accident records if possible and the Highway Department should study methods to prevent further damage if a pattern exists in the accident data. It may be possible to design an independently supported railing system if a slight reduction in lateral clearance is feasible, but this is obviously very difficult to implement.

   All damaged truss members should be repaired by heat straightening the lesser damaged members and heat straightening with the addition of reinforcement plates to the more severely damaged members.

3. Repair the stringer with the 1-1/2 inch long tear at the 90 degree top flange cope at the connection to the floorbeam (span 7, stringer 4, floorbeam 5) by locating and drilling a 1/2 inch diameter hole at the end of the tear.

4. The compression seals should be replaced at the abutments and new compression seals be placed at all other open joints to prevent water and debris from falling on surfaces below the joint.

5. Remove and replace the cracked pin nuts located at pier 5.

6. Deteriorated approach railings should be repaired.

7. On the approach spans it was not possible to determine which supports were fixed and which were expansion. It was also noted that the approach roadway joints varied in width from 9/16 inch to 2-3/4 inches. Judging from similar span lengths the joint widths were assumed to be similar. The Highway Department should monitor the joint openings at various temperatures to determine if the joints are properly working and that no unusual patterns are occurring.

8. Impact damage to many of the truss members has been repaired by use of field welding. Field welding was also performed at the floorbeam to truss connections at the ends of spans 6 and 8 during the 1972 lift operation. The welds at these locations have created conditions more prone to fatigue cracking than the original construction details. The copes at the ends of the stringers were flame cut in a manner that produced a 90 degree reentrant corner. It is suggested that careful inspections for fatigue cracks be made at these details during future bridge inspections.

CHAPTER FOUR
MORGAN COUNTY
STRUCTURE NO. 003-52-021.9A

Morgan County Structure No. 003-52-021.9A is commonly known as the Keller Memorial Bridge. It was originally constructed in 1927 and currently carries U.S. Route 31 traffic over the Tennessee River and the L&N and Southern Railroad at Decatur, Alabama. The bridge consists of steel stringer spans, southern arch spans, a bascule span, and northern arch spans. An elevation of the bridge illustrating the arrangement of the spans and span numbering is shown in Figure 4.1. A summary of the structural analyses and load rating results for the bridge are presented in this chapter. Results from the field inspection and condition survey are summarized by A. G. Lichtenstein & Associates (Keller Memorial Bridge 1991).

STRINGER SPAN RATINGS

There are a total of five steel stringer spans at the south end of the bridge. Records and drawings from these spans indicate that they were designed in 1964. A typical cross section through the stringer spans is shown in Figure 4.2. The sizes of the steel stringers and cover plates are indicated in the figure. Based on the time of construction the steel members are assumed to have a yield strength of 36 ksi as suggested by AASHTO (Manual 1983). The design drawings for the stringer spans indicate that shear studs were provided between the steel stringers and the concrete deck. Hence, composite section properties were used to determine the positive moment capacity of the stringers. The composite section properties were calculated by assuming a concrete strength of 3300 psi and a modular ratio of nine.

The steel stringer span lengths are under 200 ft , so the ratings were calculated for the standard truck loadings instead of lane loadings. Ratings were determined for the stringers of the simple spans (spans 1 through 4) and the span with a cantilever (span 5). It was found that the ratings for the simple span stringers were the lowest. A summary of the rating factors for the simply supported steel stringer spans is given in Table 4.1. The field inspection results indicate that there is no damage to or deterioration of the stringer spans that significantly affect their load capacities at the present. Hence, the rating factors of Table 4.1 are both the As Designed and Present Condition ratings. It should be noted that fatigue cracks were found at two diaphragm to girder connections. These cracks will eventually create a significant reduction in the load capacities of the affected stringers if they are not repaired.

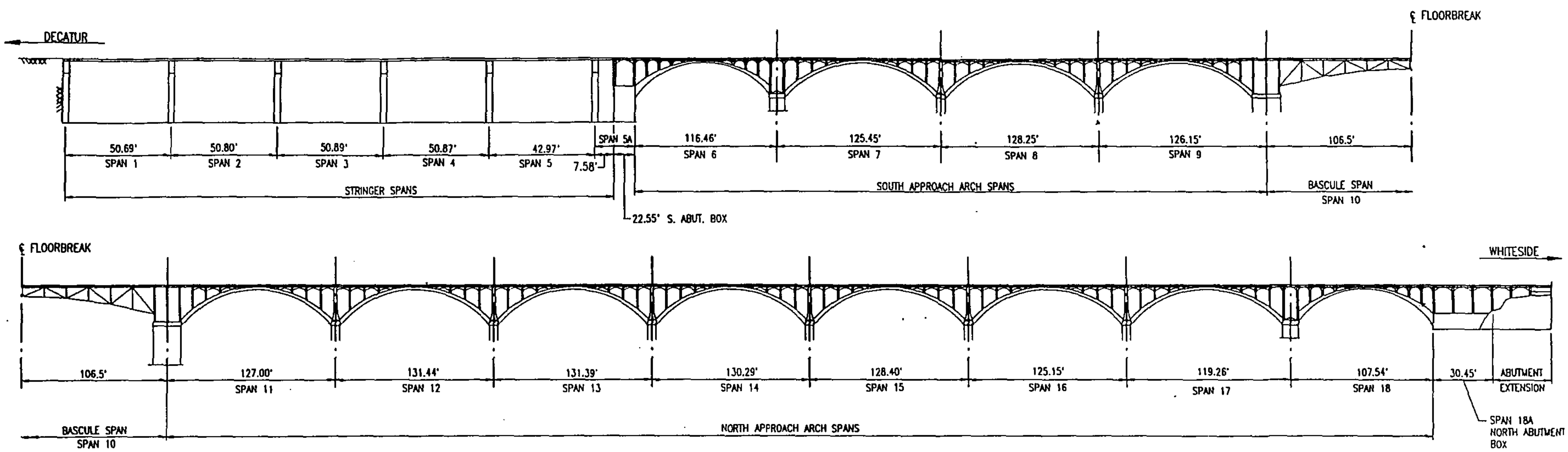


Figure 4.1. Elevation of Keller Memorial Bridge (003-52-021.9A)

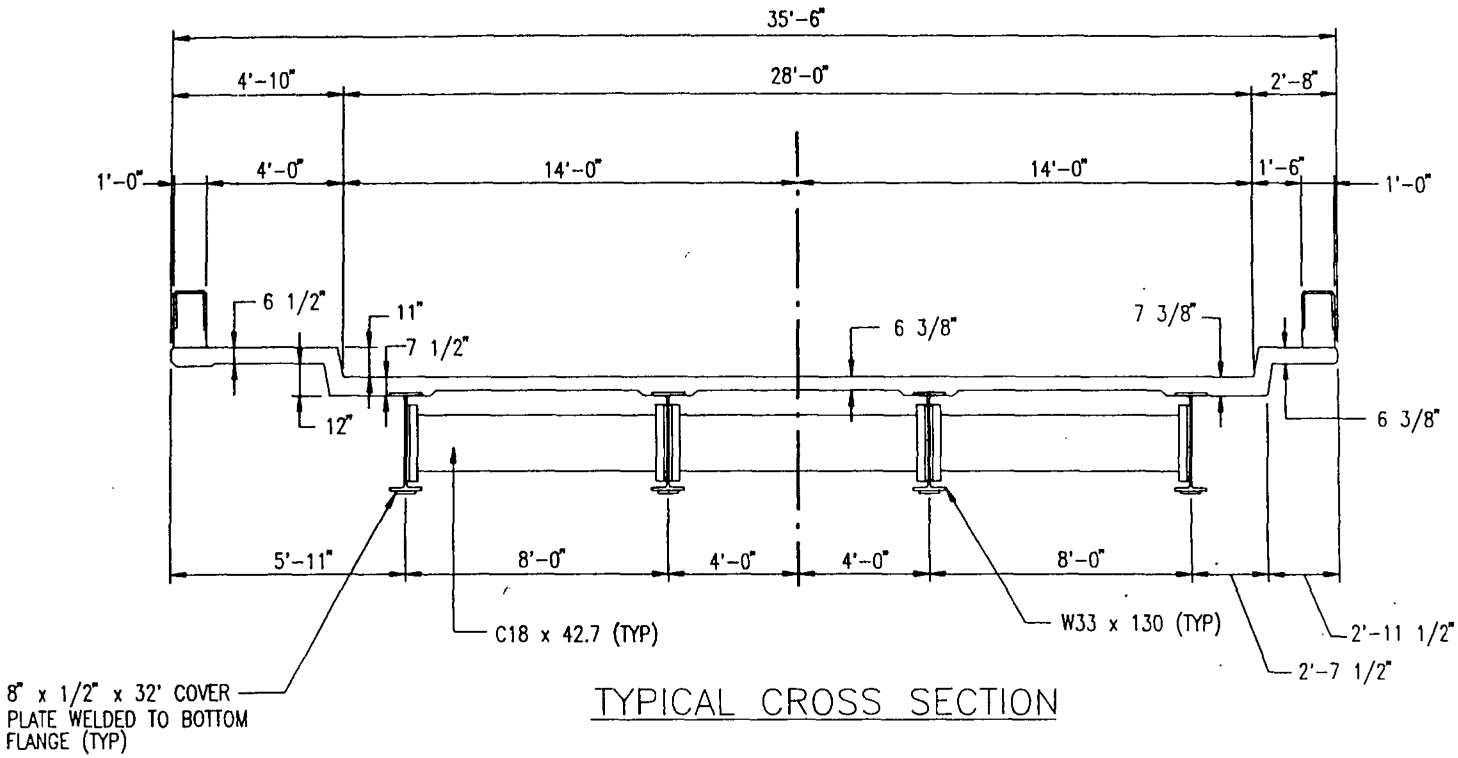


Figure 4.2. Cross Section at Stringer Spans

Table 4.1 Inventory and Operating Rating Factors for Stringer Spans

Truck Type	Inventory	Operating
Two Axle	1.12	1.79
Tri-Axle	0.83	1.33
Concrete Truck	0.94	1.50
3S2 (AL)	1.25	1.99
School Bus	2.73	4.34

The design drawings of the stringer spans provided sufficient information for performance of rating calculations for the bridge deck. The deck rating was carried out using the approach described by AASHTO (Manual 1983). To check the ratings for the standard tri-axle truck all the loading of the rear three axles was considered to be carried by tandem axles. This conservative assumption resulted in inventory and operating rating factors of 0.97 and 1.63, respectively. The ratings for the other standard trucks are higher. Hence, the deck ratings do not control the ratings of the stringer spans.

BASCULE SPAN TRUSS RATINGS

Span 10 is a double leaf bascule. Each bascule leaf consists of two underdeck trusses that cantilever 106.5 ft. The arrangement of the truss members and joint designations are illustrated in Figure 4.3. Cross section properties for the truss members are summarized in Table 4.2.

Each of the trusses is supported (in the closed position) by a combination of restraint provided by a trunnion, a concrete counterweight, a live load shoe, and a shear lock. The trunnion is a 17 inch diameter steel pin that is supported on each side of the truss at joint E1m. The total weight of the counterweight was estimated to be 1000 kips based on an assumed concrete unit weight of 150 lb./cu. ft. The center of gravity of the counterweight was estimated to be 15 ft. from the trunnion. The live load shoe is made of a beam transverse to the roadway that frames into the truss member U0L0 just below joint U0. The shear lock provides a means of transferring some amount of live load shear between the bascules by locking joint U6 of each span together. The shear lock consists of a 3.5 inch by 5.5 inch steel pin.

Structural analyses of the trusses for dead and live loads were performed with two sets of idealized support conditions. Pin supports as shown in Figure 4.3 were used for the dead load analysis. It was found by a summation of moments around the trunnion (joint E1m) that the bascule is slightly span heavy under the action of dead load alone. A summary of the dead load stresses determined for each truss member is given in Table 4.3.

Live load analyses were performed with only a pin support representing the trunnion (joint E1m) and a roller support at joint U0 modeling the live load shoe. The restraint provided by the shear lock was conservatively neglected. Live load stresses were calculated for each of the standard truck loadings and for lane loading. It was found that the standard truck loadings controlled. Because the bascule truss is a cantilever span, separate analyses were performed with each standard truck oriented in both possible traffic directions. Examples of the maximum live load stresses created in each truss member are given in Table 4.3. The live load stresses in Table 4.3 are for the tri-axle truck loading oriented toward the tip of the cantilever.

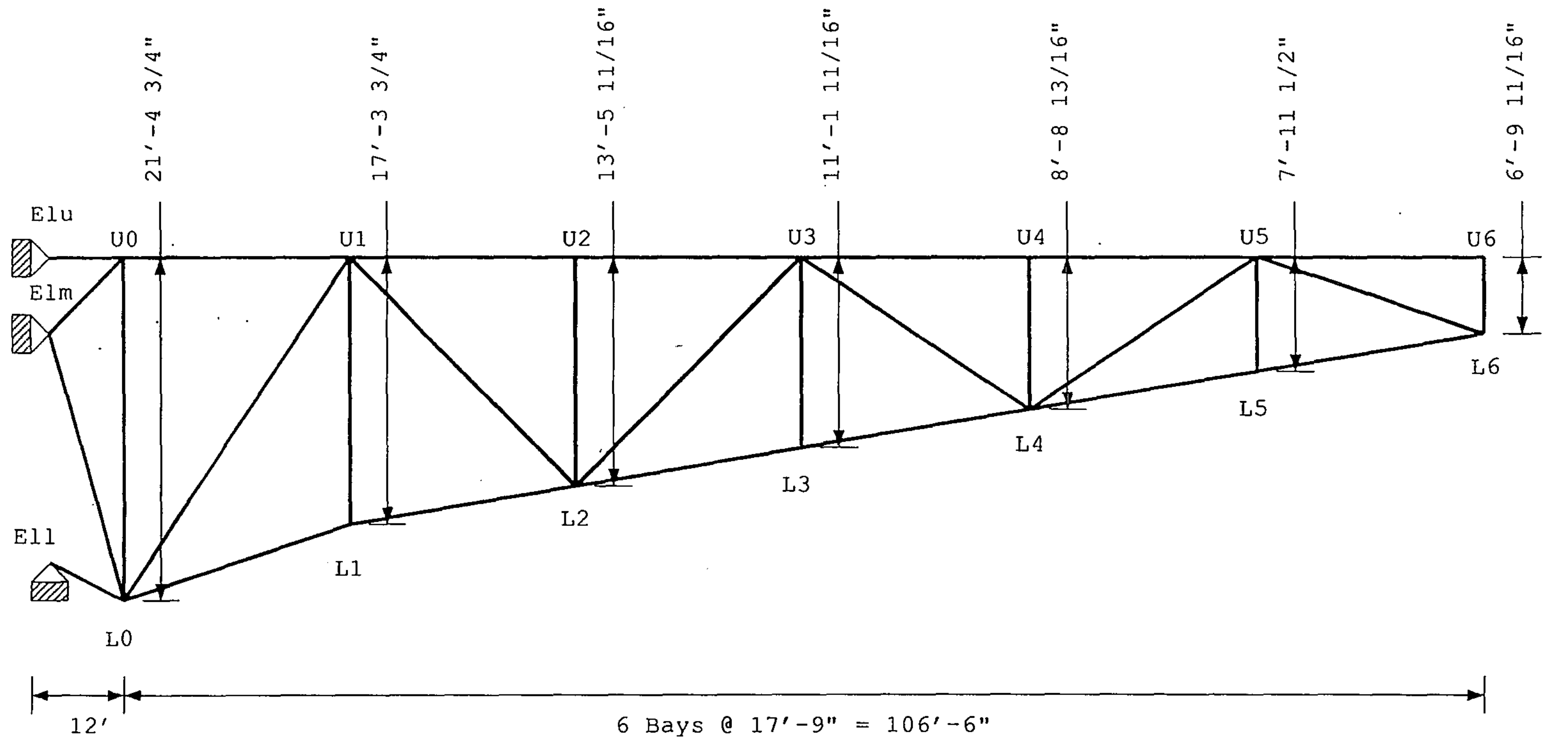


Figure 4.3. Geometry of Bascule Truss

Table 4.2 Member Cross Section Properties for Bascule Span Trusses

Member	Area ($inch^2$)	I_x ($inch^4$)	I_y ($inch^4$)
L0L1	48.0	568	2090
L1L2	48.0	568	2090
L2L3	34.1	187	1444
L3L4	34.1	187	1444
L4L5	11.4	78.7	666
L5L6	11.4	78.7	666
L0U1	18.0	198	975
L2U1	13.7	131	749
L2U3	18.0	175	991
L4U3	13.7	131	749
L4U5	16.0	106	835
L6U5	8.36	35.9	452
U0U1	39.8	596	1092
U1U2	33.4	450	1620
U2U3	33.4	450	1620
U3U4	18.0	159	991
U4U5	18.0	159	991
U5U6	14.7	181	833
L0U0	27.0	415	1305
L1U1	7.12	25.1	396
L2U2	7.12	26.5	403
L3U3	7.12	21.3	403
L4U4	7.12	25.1	403
L5U5	7.12	21.3	403
L6U6	11.72	121	681
EluU0	52.1	596	2183
ElmU0	52.1	596	2183
ElmL0	52.1	596	2183
EllL0	77.3	1272	4214

Table 4.3 Dead Load and Live Load Stress for Bascule Span Truss Members

Member	Dead Load Stress (ksi)	Live Load Stress From Tri-Axle Truck (ksi)
EluU0	9.0	4.5
ElmU0	-3.4	3.2
ElmL0	5.9	1.3
EllL0	-7.1	-5.8
L0U0	2.8	6.5
U0U1	8.1	9.5
L0U1	-4.4	-5.6
L0L1	-5.8	-8.1
L1U1	0.2	-0.7
U1U2	6.5	11.2
L2U1	4.7	7.2
L1L2	-5.8	-8.1
L2U2	-2.5	-8.7
U2U3	6.5	11.2
L2U3	-5.0	-7.3
L2L3	-4.4	-9.3
L3U3	0.6	0.2
U3U4	4.6	12.6
L4U3	5.2	10.0
L3L4	-4.4	-9.3
L4U4	-2.4	-8.6
U4U5	4.6	12.6
L4U5	-4.2	-11.5
L4L5	-2.1	-5.3
L5U5	0.3	0.2
L6U5	3.1	7.8
L5L6	-2.1	-5.3
L6U6	-0.8	-2.3

The results of the live load analyses and rating calculations indicated that the truss ratings for the various truck types were controlled by four different members. The most critical traffic direction varied. A summary of Operating and Inventory rating factors based on the truss member capacities is given in Table 4.4. The critical members that controlled each of the ratings are also listed. No significant damage to the truss members was found during the field inspection. Hence, the rating factors given in Table 4.4 are both the As Designed and Present Condition ratings.

Structural analyses were also performed to determine the dead and live load shear stresses on the trunnion pins. It was determined that the live load shear acts to relieve the dead load shear on the pins. Hence, trunnion pins do not control the ratings of the bascule span.

The trunnion stands were checked for uplift. It was found that under live load plus impact, small uplift forces are created on the trunnion stands when the effects of the shear lock are neglected. Supplementary truss analyses were performed that included the shear lock because of concern about the detrimental effects of uplift. Because the shear lock consists of a pin in an oversize hole, a relative displacement between the ends of the bascules at the shear lock was also considered. A relative displacement of up to 1.33 inch was assumed to be possible. It was found that when the shear lock is operating so that 1.33 inch, or less, relative vertical movement can occur between the tips of the bascules then no net uplift force is developed at the trunnion stands. At a relative displacement of the 1.33 inch, each shear lock will transfer approximately 30 kips. It does not appear that the conditions at the trunnion stands and shear locks should control the ratings of the bascule span at this time. However, it is imperative that the shear locks and live load shoes be well maintained, and skimmed as needed to maintain support.

A representative number of the riveted truss connections were analyzed so that connection ratings could be established. The diameter of the rivets was assumed based on tabulated rivet dimensions given by AISC (Manual 1927). Field measurements indicated that the rivet heads were 1-3/8 inch diameter, so the rivet diameter was assumed to be 7/8 inch. The connection ratings were performed using live load and dead load bar forces determined in the same manner as described for the member ratings (by neglecting the shear lock). It was found that the connection of the top chord tension member U2U3 to joint U3 controlled the connection ratings. Ratings were calculated for both common rivet types, Grade 1 and Grade 2, for which AASHTO (Manual 1983) give allowable stresses. Rating factors for the truss connections are summarized in Table 4.5. The ratings shown are based on the shear capacity of the rivets. Ratings based on bearing stresses were found to be higher than those shown in Table 4.5.

Table 4.4 Rating Factors for Bascule Span Truss Members

Truck Type	Inventory		Operating	
	Rating Factor	Critical Member	Rating Factor	Critical Member
Two Axle	0.92	L4U5	1.26	L4U5
Tri-Axle	0.71	L4U5	0.98	L4U5
Concrete Truck	0.80	L4U5	1.10	L4U5
3S2 (AL)	0.84	U0U1	1.33	L3L4
School Bus	2.24	U4U5	3.21	L3L4

Table 4.5 Rating Factors for Bascule Truss Connections

Truck Type	Grade 1 Rivets		Grade 2 Rivets	
	Inventory	Operating	Inventory	Operating
Two Axle	0.45	0.86	1.04	1.66
Tri-Axle	0.35	0.69	0.83	1.33
Concrete Truck	0.40	0.78	0.94	1.50
3S2 (AL)	0.37	0.79	0.95	1.51
School Bus	1.10	2.18	2.63	4.21

It is prudent to consider the rivets to be the lower strength Grade 1 because their actual strength is not known. It is seen from Table 4.5 that most of the rating factors for the Grade 1 rivets are less than one. However, the ratings are higher than some that will be presented later. It should also be noted that the ratings for Grade 1 rivets are considered to be conservative estimates of the connection ratings. If the support provided to the truss by the shear lock was considered, all the ratings of Table 4.5 would increase. It is also likely that the rivets used in the connections are Grade 2 instead of Grade 1. Past experiences of A.G. Lichtenstein & Associates suggest that most riveted truss bridges were constructed with Grade 2 rivets. It is seen from Table 4.5 that ratings based on Grade 2 rivets are significantly higher.

The field inspection results indicate that there is some corrosion pitting and minor section loss at some of the truss connection gusset plates. The effects of the corrosion on the connection ratings was found not to be significant. It is concluded that the overall rating of the bridge is not controlled by the truss connection capacities.

BASCULE SPAN DECK SYSTEM RATINGS

The deck system of the bascule span consisted of metal grating on top of longitudinal stringers that span between floorbeams. The stringers span 17.75 ft and are spaced transversely at 4 ft. The typical stringer is a S12 x 45. The stringers are connected to the floorbeams by a riveted connection. The typical floorbeams are either B26 x 91 or B28 x 97. The floorbeams are attached to truss gusset plates by a riveted connection.

The field inspection results indicate that the primary members of the deck system are in good condition. No reduction of the As-Designed capacity of the primary members is considered necessary for establishing Present-Condition ratings. Summaries of the rating factors calculated for the stringers and floorbeams are given in Table 4.6. The rating factors listed are based on the bending moment capacities of the stringers and floorbeams. Rating factors based on the shear capacities of the members were found to be significantly higher. Ratings based on the metal grating were also found to be higher than those of Table 4.6.

The only significant damage or deterioration of the deck system is fatigue failure of a number of the rivets in stringer to floorbeam connections. Rating factors were calculated for the stringer to floorbeam and floorbeam to truss connections for the As-Designed condition. It was found that these connections do not control the deck system ratings. At the present, the connections with broken rivets have sufficient capacity to support normal traffic loading. However, it is likely that more rivets in the same connections will fail. It is important that the broken rivets be replaced with high strength bolts. After the bolts are installed, the connection angles should be monitored closely for

Table 4.6 Bascule Span Deck System Rating Factors

Truck Type	Stringers		Floorbeams	
	Inventory	Operating	Inventory	Operating
Two Axle	1.83	2.56	1.58	2.58
Tri-Axle	1.33	1.86	1.13	1.84
Concrete Truck	1.46	2.04	1.28	2.08
3S2 (AL)	2.08	2.91	1.75	2.85
School Bus	3.35	4.69	3.58	5.83

fatigue cracks in future inspections.

DESCRIPTION OF CONCRETE ARCH SPANS

A total of 12 open spandrel arches make up the southern and northern arch spans. The span lengths of each of the arches vary from 107.54 ft to 131.44 ft. The shapes of the arches also vary. Some are circular, and some are parabolic. Each span is made of two arches side by side. Each arch supports two lines of 12 inch square spandrel columns. The columns support the deck system. The deck is reinforced concrete supported by transverse floorbeams that span between the spandrel columns. A typical cross section through the bridge at a floorbeam is shown in Figure 4.4.

Structural drawings showing many generic details of the geometry and reinforcement for the arches were provided by the Alabama Highway Department. However, specific details for each of the spans was not available. The field inspections confirmed that the as-built geometry of the arch members agreed reasonably well with drawings. Where exposed reinforcement was found, it generally agreed with details shown on the drawings. It was concluded that use of the reinforcement details shown on the drawings would provide a reasonable means of calculating the arch ratings. The ratings were performed by assuming a concrete compressive strength of 3300 psi and a yield strength for the reinforcement of 33,000 psi as suggested by AASHTO (Manual 1983).

ARCH DECK SYSTEM RATINGS

The bridge deck is 10.5 in. thick and spans longitudinally between floorbeams that are typically spaced at just over 12 ft. Transversely the deck spans between narrow longitudinal beams along the outside line of spandrel columns. The geometry of the typical panel suggests that the deck ratings would conventionally be based on two-way slab action. However, the reinforcing details indicate that primary reinforcement was provided only in the longitudinal direction. As-Designed ratings were calculated by assuming the deck to be a continuous one-way slab spanning over the floorbeams. A summary of the deck ratings is given in Table 4.7.

The inspection of the deck indicated that some cracking and efflorescence was present. However, it is not believed that the condition of the deck warrants any significant reduction in the ratings listed in Table 4.7.

Three types of transverse floorbeams are present in the arch deck system. The typical interior floorbeams has a T-beam cross section with a web width of 12 inches. An inverted L-beam with web width of 15 inches supports the deck at the interior expansion joints. An 18 inch wide rectangular beam supports the deck at the expansion joints at the ends of each arch span. It

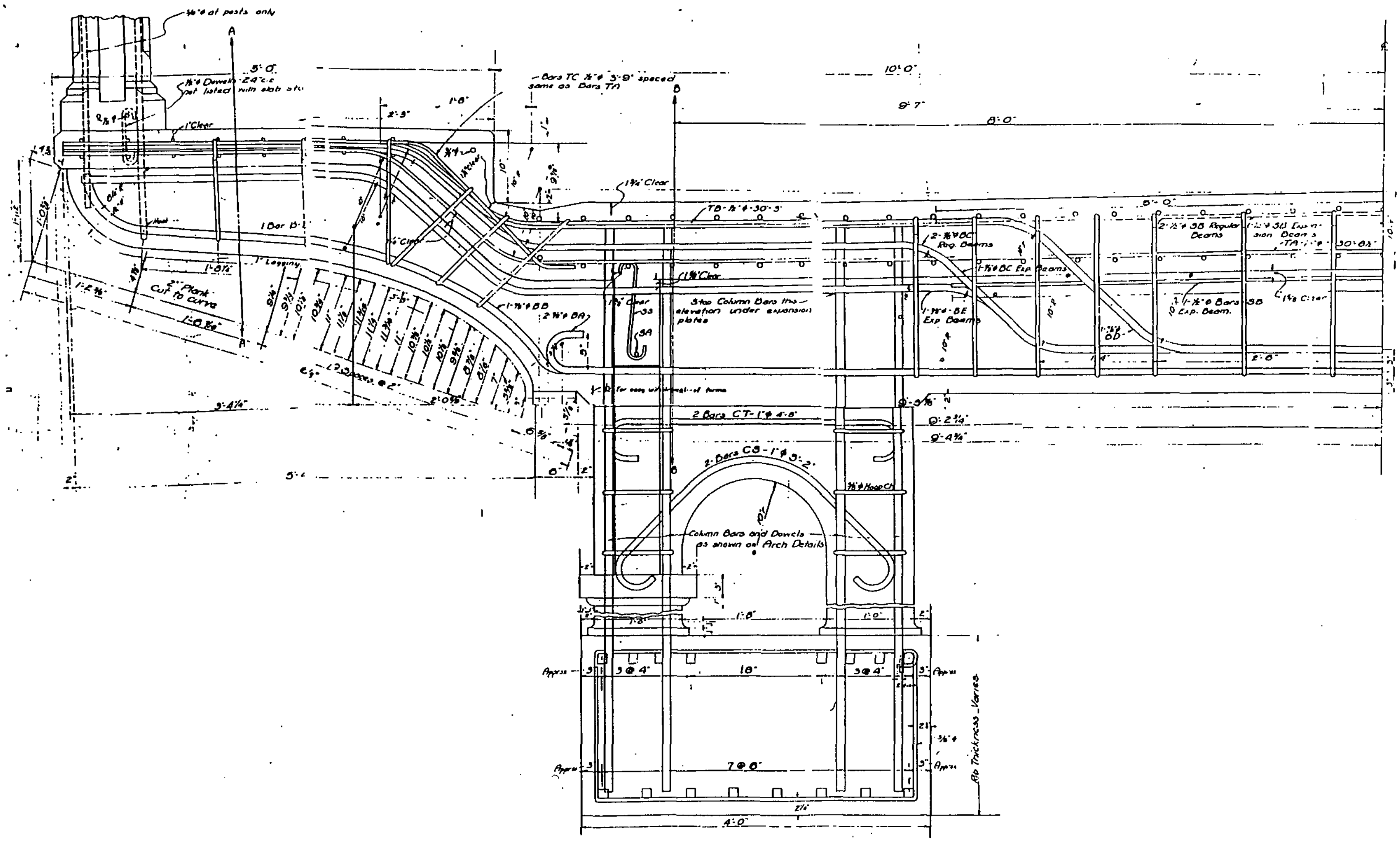


Figure 4.4. Cross Section Through Arch at Floorbeam

Table 4.7 Rating Factors for Arch Span Deck

Truck Type	Inventory	Operating
Two Axle	0.68	1.13
Tri-Axle	0.58	0.97
Concrete Truck	0.55	0.91
3S2 (AL)	0.81	1.35
School Bus	1.07	1.78

Table 4.8 Rating Factors for Arch Span Floorbeams

Truck Type	Inventory	Operating
Two Axle	0.83	1.38
Tri-Axle	0.59	0.98
Concrete Truck	0.67	1.11
3S2 (AL)	0.93	1.55
School Bus	1.65	2.75

was found that the shear capacity of the floorbeams at the ends of the arch spans controlled ratings. The floorbeam ratings are listed in Table 4.8.

A number of judgments were made in calculation of the floorbeam ratings. Based on common methods of analysis the floorbeam bending moment capacities were found to produce ratings significantly less than those shown in Table 4.8. The low ratings result from the unusual details of the floorbeam reinforcement (see Figure 4.4). The reinforcement details are not typical of modern design practices. The top reinforcement at the inside face of the spandrel columns is bent down to the bottom of the beam very near the face of the column. The amount of top reinforcement is also very low and there is no evidence of significant transverse reinforcement in the deck. Because of these details, the negative moment capacity of the floorbeams would limit the floorbeam ratings. However, the floorbeams have been carrying normal traffic and show no signs of distress in the negative moment regions. Improved ratings for the floorbeams were obtained by assuming that the floorbeams spanned between hinges at the inside faces of the columns. Restraining moments equal to the negative moment strength of the floorbeam were applied at the hinges to enhance the positive moment capacity of the beams. The ratings based on this approach were found to be higher than the ratings based on the floorbeam shear capacity shown in Table 4.8. It is believed that use of the ratings of Table 4.8 is reasonable as long as the floorbeams do not show signs of flexural distress. If future inspections reveal signs of distress in the floorbeams, the ratings should be reevaluated.

The ends of the floorbeams cantilever beyond the spandrel columns to support a sidewalk along each side of the bridge as illustrated by Figure 4.4. Bridge ratings are not normally controlled by the structural capacity of sidewalks. However, it should be noted that preliminary calculations indicate that the sidewalks do not meet the current AASHTO strength requirements and are incapable of supporting a truck wheel load. The field inspection results also indicate that the cantilever beams have sustained a significant amount of damage in the past. It is suggested that measures to prevent incidental truck loading on the sidewalks should be implemented.

SPANDREL COLUMN RATINGS

Axial loads and moments for use in rating the spandrel columns and arches were determined by planar frame analyses performed with the BRIDGE program described in Chapter Two. Frame models were developed for spans 12, 15, and 18. Spans 12 and 18 are the longest and shortest of the arch spans. Analyses of these spans provided reasonable bounds on the magnitudes of the internal forces resisted by the arch members. The arch drawings indicated that two sizes of bars were used to reinforce the arch ribs. The ribs of the shorter spans, such as span 18, were reinforced with 1 inch square bars. The longer spans were reinforced with 1.25 inch square bars.

Span 15 was chosen for analysis because it is the longest span for which 1 inch square bars were used. Span 15 is also practically the same as span 8 which was found to have the most significant damage to the arch ribs.

The frame models were developed to represent one of the two arches of each span. Loadings for one traffic lane were applied to the model. The models for each of the spans was developed using gross section moments of inertia for each of the members. Because there are two lines of spandrel columns on each arch, the moments of inertia of the side by side columns were simply added together. One-half of the resulting forces were considered to be resisted by each of the two columns. There is a line of narrow longitudinal beams along the exterior column line. It was recognized that the presence of the beams will cause the exterior columns to resist somewhat more than one-half the total forces. However, it was impractical and unnecessary to attempt a more sophisticated allocation of the forces.

The results of the rating calculations indicated that the shortest spandrel columns of each span controlled the column ratings for the span. Overall, the column ratings were found to be controlled by the shortest columns of span 18. There are four expansion joints along each of the arch spans. The expansion joints produce conditions so that the spandrel columns are not braced against side sway. Hence, bending moment amplification to account for side sway was considered for each span. No moment amplification was required by AASHTO (Standard 1989) for the shortest columns of span 18. For span 12, the most critical columns were sufficiently slender so that amplification of the bending moments was necessary. The moment amplification was first attempted using the most recent AASHTO Specifications (Standard 1989). It was found that the rib shortening effects present in an open spandrel arch structure conflict with some of the simplifying assumptions made in the development of the codified procedure. The analysis problem was overcome by using a previously acceptable moment amplification method described by ACI (Building Code 1977).

Ratings for the spandrel columns are listed in Table 4.9. Separate ratings are given for As-Designed and Present-Condition. the field inspection revealed that the exterior short spandrel columns on all arch spans exhibited diagonal cracking. The Present-condition ratings account for the observed cracking. The Present-Condition ratings were obtained by reanalysis of the structure assuming that the outside column of the two shortest columns had been removed. The resulting forces were then considered to be resisted by the remaining single column. It is seen from Table 4.9 that the result is a 10 to 15 percent decrease in the column ratings.

Table 4.9 Rating Factors for Spandrel Columns

Truck Type	As-Designed Inventory	As-Designed Operating	Present-Condition Inventory	Present-Condition Operating
Two Axle	0.57	0.95	0.50	0.84
Tri-Axle	0.40	0.67	0.34	0.57
Concrete Truck	0.44	0.74	0.39	0.66
3S2 (AL)	0.61	1.02	0.51	0.86
School Bus	1.53	2.55	1.41	2.35

ARCH RATINGS

Rating factors for the arch ribs were calculated by using frame analysis results as discussed in the previous section. It was found that the most critical location along each of the arches was at the base where the arch meets the piers. As-Designed rating factors for the arches of spans 12, 15, and 18 are listed in Table 4.10. It is seen that the lowest ratings were found for span 15.

A significant amount of map cracking was found at the base of the arches during the field inspection. Almost all the damage apparently resulted from barge impacts. the most significant damage was found at spans 6 through 9. spans 8 and 9 exhibited map cracking at their base with longitudinal cracks visible along the top of the ribs. The observed damage was accounted for by adjustments to the ratings for span 15 which has very similar geometric details to most heavily damaged arch, span 8. The effects of the map cracking were approximated by considering all the concrete cover over the arch rib ties to be removed. The longitudinal cracks along the tops of the ribs suggest that some corrosion of the primary reinforcement has occurred. The effects of the corrosion were approximated by reducing the total area of longitudinal reinforcement in the rib by 5 percent. The resulting ratings are listed in Table 4.11. It is seen that the estimated effects of the damage results in a 20 to 25 percent reduction in the arch ratings.

CONCLUSIONS AND RECOMMENDATIONS

Structural analyses and load ratings of the various spans of the Keller Memorial bridge indicate that the load capacity of the short spandrel columns of the arch spans control the ratings. A summary of the Present-Condition ratings based on the spandrel columns is given in Table 4.12. Consideration should also be given to the following maintenance and repairs that were identified by the field inspection.

1. The extensive cracking of the arches and piers at Piers 7, 8, and 13 should be repaired. The longitudinal cracks in the arch ribs should be injected and map cracking should be sealed. These areas should be monitored closely during subsequent inspections.

2. The two cracked welds in the stringer span diaphragm connection should be ground smooth, tested and repaired.

3. The railing on the stringer spans does not conform with current AASHTO requirements. The railing is approximately three inches too short. Improvements to the railing should be considered.

4. The northern most floorbeam and surrounding concrete in the north abutment should be repaired.

Table 4.10 As-Designed Rating Factors for Arch Ribs

Truck Type	Span 12		Span 15		Span 18	
	Inventory	Operating	Inventory	Operating	Inventory	Operating
Two Axle	0.84	1.40	0.65	1.09	0.69	1.15
Tri-Axle	0.64	1.06	0.49	0.82	0.51	0.85
Concrete Truck	0.73	1.21	0.56	0.94	0.59	0.98
3S2 (AL)	0.86	1.43	0.70	1.17	0.82	1.36
School Bus	2.06	3.43	1.67	2.79	1.86	3.09

Table 4.11 Arch Rib Rating Factors

Truck Type	As-Designed		Present-Condition	
	Inventory	Operating	Inventory	Operating
Two Axle	0.65	1.09	0.51	0.85
Tri-Axle	0.49	0.82	0.39	0.64
Concrete Truck	0.56	0.94	0.44	0.73
3S2 (AL)	0.70	1.17	0.56	0.93
School Bus	1.67	2.79	1.31	2.19

Table 4.12 Summary of Present-Condition Ratings for Morgan County Structure No. 003-52-021.9A

Truck Type	Standard Weight (tons)	Inventory Rating Factor	Inventory Rating (tons)	Operating Rating Factor	Operating Rating (tons)
Two Axle	29.5	0.50	15	0.84	25
Tri-Axle	37.5	0.34	13	0.57	21
Concrete Truck	33	0.39	13	0.66	22
3S2 (AL)	40	0.51	20	0.86	34
School Bus	12.5	1.41	>12.5	2.35	>12.5

5. All cracked short spandrel columns should be repaired.
6. The deck joints in the arch spans should be sealed to prevent leakage.
7. In general, major spalls found in the concrete arches, abutments and piers should be patched.
8. The live load shoes on the bascules should be reinforced and provisions made for periodic visual inspection of the live load shoes and bearing plates to assure adequate bearing contact in the closed position.
9. Holes should be drilled at the end of the cracks in the copes of the stringers in the bascule span to arrest further crack growth. These cracks are in fatigue sensitive details and should be monitored closely during subsequent inspections.
10. All loose, damage, or broken rivets should be replaced with high strength bolts. After the bolts are installed, the connection angles should be monitored closely for fatigue cracks in future inspections.
11. The cantilevered sidewalks of the arch spans do not meet current AASHTO requirements. Measures should be taken to prevent incidental truck loading on the sidewalks.

CHAPTER FIVE
JACKSON COUNTY
STRUCTURE NO. 035–36–009.8A

Jackson County Structure No. 035–36–009.8A is commonly known as the B.B. Comer Bridge. It was constructed in 1930 and currently carries Alabama Route 35 traffic over the Tennessee River from Section to Scottsboro. The bridge consists of main truss spans (four simple span through trusses and one three–span continuous through truss), thirteen (13) steel I–beam approach spans, and one reinforced concrete tee beam approach span. An elevation of the bridge illustrating the arrangement of the spans and span numbering is shown in Figure 5.1. A summary of the structural analyses and load rating results are presented in this chapter. Results from the field inspection and condition survey are summarized by A.G. Lichtenstein & Associates (B.B. Comer Bridge 1991).

STRINGER SPAN RATINGS

There is a total of thirteen steel stringer spans at the Scottsboro end of the bridge. Based on the time of construction the steel members are assumed to have a yield strength of 30 ksi as suggested by AASHTO (Manual 1983). Most steel stringer bridges with a concrete deck constructed in the early 1930's did not use any mechanical shear studs. Lacking evidence of shear studs between the steel stringer and the concrete deck, non–composite section properties were used in rating.

Each steel stringer approach span is basically a simple span with one or two cantilever overhangs. An expansion joint separates cantilever tips. A typical cross section of the steel stringer approach span is shown in Figure 5.2. Four B33x125 stringers are used in span 8 and a half of span 9. B30x108 stringers are used in the rest of the approach spans. Cross sectional properties were taken from AISC (Iron and Steel Beams 1985). Span 8 and span 16 were used to calculate the As–Designed ratings. Span 8 has the longest span length of 54 feet 4 inches and span 16 is typical of several repeating spans. The resulting rating factors given in Table 5.1 were based on the flexural capacities (Positive moment controlled) of the stringers.

Present–Condition ratings shown in Table 5.1 are lower than the As–Designed ratings. The field inspection (B.B. Comer Bridge 1991) revealed that the arch shape concrete diaphragms are cracked and broken so that they appear to have no positive connection to the steel stringers, particularly in spans 8,9, and 15. The loss of lateral bracing was accounted for in calculating the Present–Condition ratings.

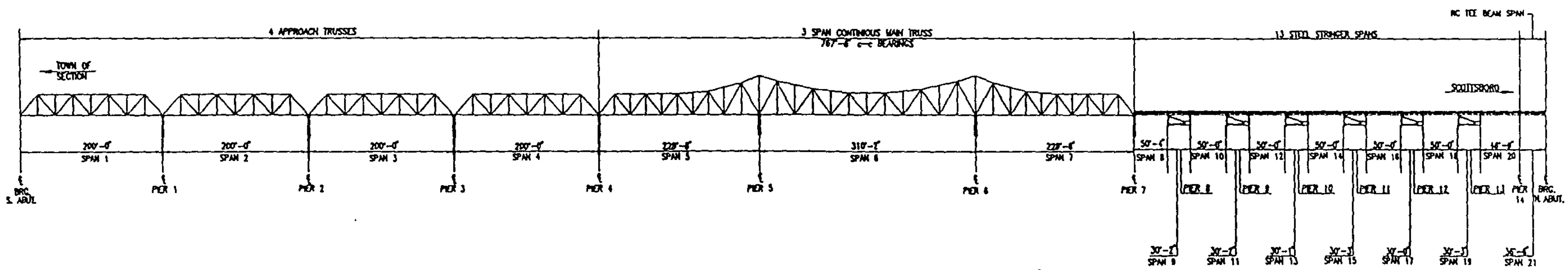


Figure 5.1. Elevation of Scottsboro Bridge (035-36-009.8A)

BEAM TABLE	
LOCATION	STRINGER SIZE
SPANS 8 TO ℄ 9	B33 x 125
SPANS ℄ 9 TO 20	B30 x 108

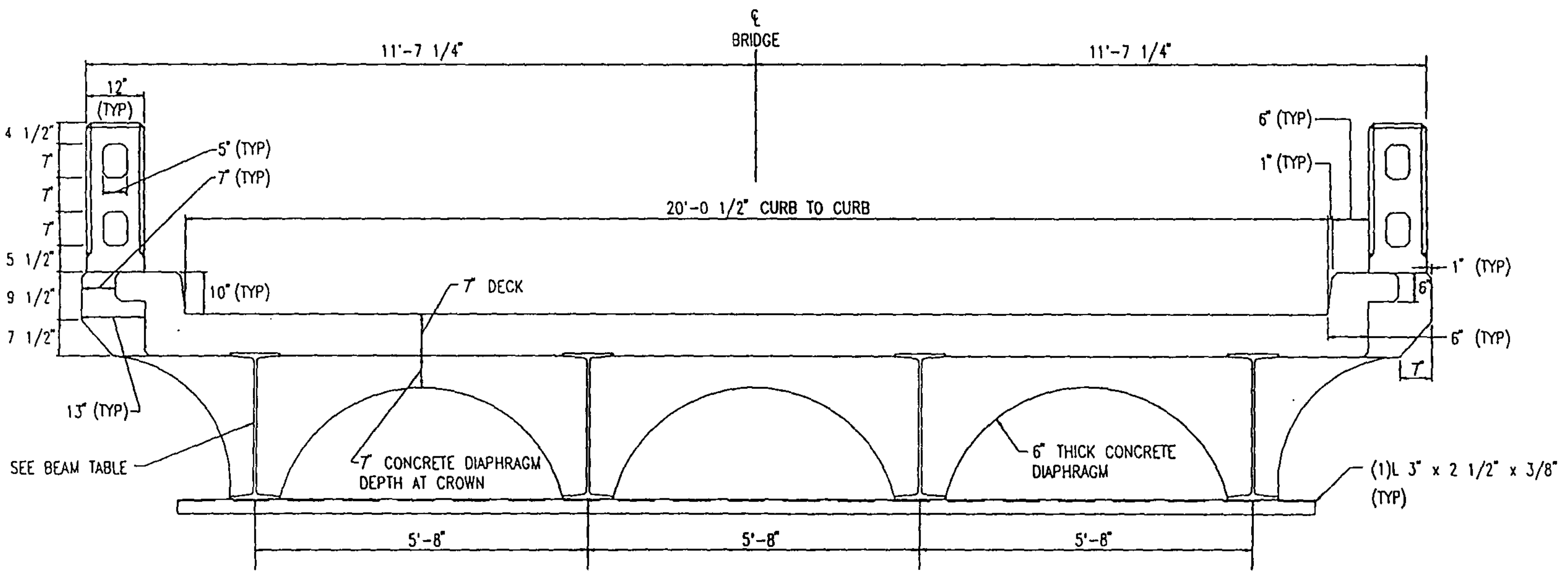


Figure 5.2. Cross Section at Stringer Spans

Table 5.1 **Rating Factors for Stringer of Spans 8 & 16**

Truck Type	As-Designed		Present-Condition	
	Inventory	Operating	Inventory	Operating
Two Axle	0.68	1.08	0.61	1.05
Tri-Axle	0.51	0.80	0.45	0.74
Concrete Truck	0.58	0.90	0.51	0.84
3S2 (AL)	0.74	1.21	0.65	1.13
School Bus	1.68	2.65	1.48	2.46

The field inspection has identified that the concrete railing has been hit and has completely failed in several locations on the north approach. It is assumed here that these railings will be repaired in the near future. The field inspection report also indicated that excessive live load deflections were observed at the cantilever ends of the stringer spans. Live load deflections were calculated at the cantilever ends as suggested in the inspection report. Deflections were calculated at the ends of spans 8 and 16 for tri-axle dump truck loading. The computed deflections were approximately three times as large as the AASHTO (Standard 1989) limiting value of L/300(15 x 12/300 = 0.6 inch). The calculated deflections and inspection results indicate that measures should be taken to reduce the live load deflections at the cantilever ends.

The use of shear locks at the cantilever tips is suggested in the inspection report as a possible means of reducing the live load deflections. Deflections were recalculated so that the effectiveness of shear locks could be evaluated. The maximum calculated deflection was 1.01 inch which is 68% larger than the AASHTO limiting value. However, if the deflection of 1.01 inch is adjusted for the final operating ratings presented at the end of this report, the AASHTO limit is satisfied. It is concluded that shear locks at the cantilever tips would provide a sufficient reduction of live load deflections.

It should be noted that shear locks at the cantilever ends are not required for adequate strength of the stringer spans. The live load deflection limit described above is related to serviceability and maintenance of the spans. According to an authoritative publication on AASHTO deflection limits by AISI (Bulletin No. 19, 1971), limits on live load deflections provide highway bridges with service lives uninterrupted by fatigue damage, durable riding surfaces, and comfortable crossings for pedestrians and occupants of moving vehicles.

APPROACH TRUSS RATINGS

Four identical through trusses of 200 feet span are referred to here as approach trusses (spans 1 through 4). An elevation showing the overall geometry, support conditions and joint numbering for the trusses is given in Figure 5.3. Typical cross section of the trusses is shown in Figure 5.4. Cross sectional properties for the truss members are given in Table 5.2.

Structural analyses of the truss were performed using ABAQUS (1989) and the in–house computer program BRIDGE to determine the dead load and live load stresses in each of the members. Analyses were performed using standard truck axle loadings and equivalent lane loadings. Equivalent lane loads were used to account for the possibility of having multiple trucks on the 200 feet span. A summary of the dead load and maximum live load stresses is given in Table 5.3. Because of the somewhat long span length of the truss, top and bottom chords develop the maximum stress under an equivalent lane

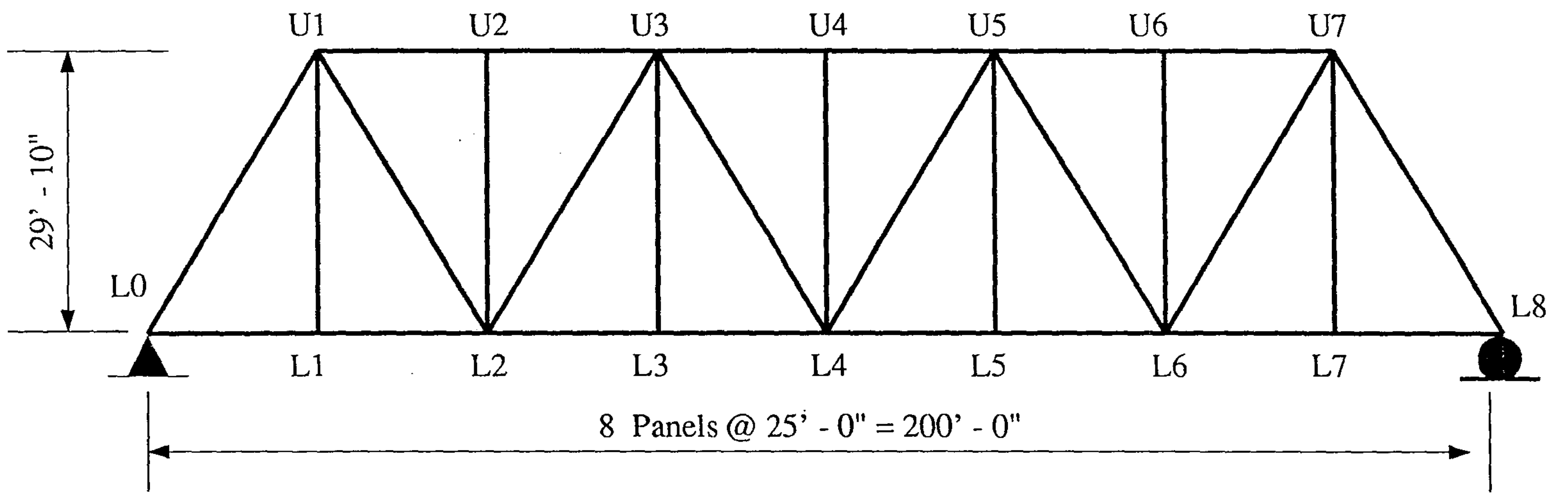


Figure 5.3. Elevation of Approach Trusses

Table 5.2 Member Cross Section Properties, Approach Truss

Member	Area ($inch^2$)	I_x ($inch^4$)	I_y ($inch^4$)
L0U1	27.7	961	1,014
L1U1	9.1	163	33
L2U2	9.1	163	33
L3U3	9.1	163	33
L4U4	9.1	163	33
L2U1	17.6	322	686
L2U3	17.6	322	686
L4U3	11.7	157	436
U1U2	25.5	883	949
U2U3	25.5	883	949
U3U4	31.3	1,044	1,185
L0L1	19.8	625	794
L1L2	19.8	625	794
L2L3	29.3	803	1,173
L3L4	29.3	803	1,173

Table 5.3 Dead Load Stresses and Maximum Live Load Stresses (Approach Truss Spans, in ksi)

Member	Dead Load	Maximum Live Load
L0U1	-6.45	-3.88
L0L1	5.79	3.48
L1U1	3.48	7.60
U1U2	-7.75	-4.64
L2U1	7.33	4.64
L1L2	5.79	3.48
L2U2	-0.46	-0.00
U2U3	-7.75	-4.64
L2U3	-4.41	-3.80
L2L3	8.44	5.04
L3U3	3.59	7.60
U3U4	-8.44	-5.04
L4U3	-2.20	4.49
L3L4	8.44	5.04
L4U4	0.53	0.00

load while the tri–axle dump truck produces the maximum stresses in most verticals and diagonals whose influence lines are confined within the adjacent panels.

Summaries of the minimum inventory and operating rating factors calculated for the approach truss are given in Table 5.4. These ratings are based on the axial stresses created in the truss members by the standard truck loadings. As–Designed ratings are given for loadings by single trucks and the equivalent lane loadings defined by Equation 2.1.

The corrosion and impact damage identified during the field inspection was used to determine Present–Condition ratings for the approach truss. There was a great deal of minor to moderate corrosion and impact damage to the truss, particularly to members L2U2 and L8U7. None of the truss members had a sufficient amount of damage to affect the final truss ratings, although some individual member capacities were reduced. For both As–Designed and Present–Condition ratings, member U3U4 controls the truss rating.

Capacities of the approach truss connections were also evaluated. The truss analyses revealed that members L0L1, L1L2, L2L3, L3L4, L1U1,L2U1, L3U3, and L4U3 are in tension. The connections at the ends of L0L1, L2L3, L2U1, and L3U3 were selected as the most critical ones. The field notes indicate that the head diameter of rivets is 1 1/2 inches. A shank diameter of 7/8 inch was chosen from tabulated rivet dimensions given by AISC (Manual 1927). The rivets were conservatively assumed to be ASTM A502 Grade 1 rivets simply because the Grade 1 rivets are assigned the lowest allowable stresses by AASHTO (Manual 1983).

The load capacities of the connections were found to be smaller than the truss member capacities indicating that the rivets are likely to be Grade 2 rivets. A summary of As–Designed ratings determined for the connections is given in Table 5.5. The ratings for the single truck loadings and equivalent lane loadings were controlled by the shear capacity of the connection of member L2L3 to joint L2 for all loading cases. It was judged that the minor corrosion of the connection was insignificant. It is concluded that the As–Designed rating factors given for the connection capacities in Table 5.5 are sufficiently accurate for use as Present–Condition ratings.

APPROACH TRUSS FLOOR SYSTEM RATINGS

The floor system framing for all four trusses is identical. A typical section through the floor system at midspan is shown in Figure 5.4. The floor system is made up of 6 3/4 inch thick concrete deck on four stringers (21 CB 60) spaced at 5 feet 6 inches. The stringers span 25 feet between floorbeams (30 CB 115).

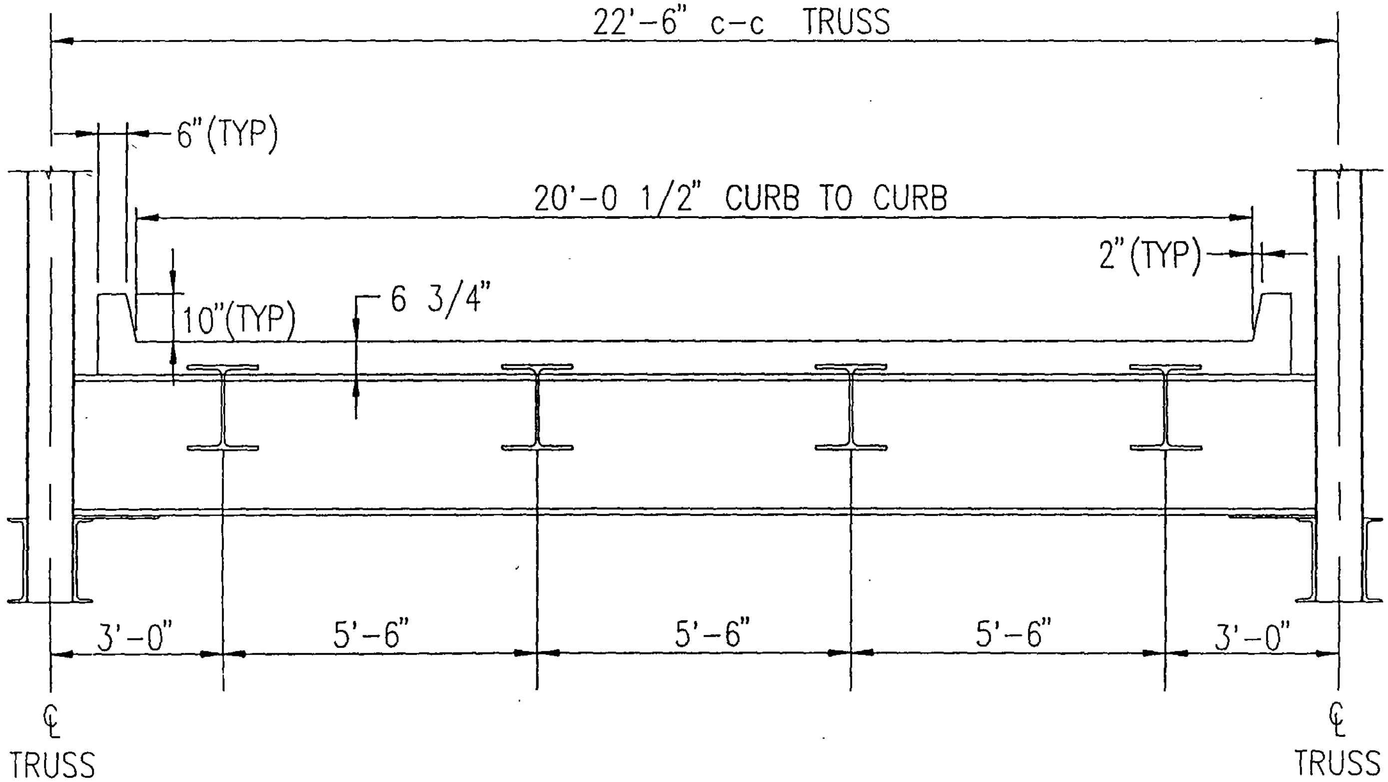


Figure 5.4. Cross Section Through Approach Trusses Floor System

Table 5.4 Rating Factors on Approach Truss Members

	As-Designed/Present Condition			
	Inventory		Operating	
Truck Type	Single Truck	Eq. Lane Loading	Single Truck	Eq. Lane Loading
Two Axle	1.51	1.25	2.49	2.06
Tri-Axle	1.17	0.98	1.93	1.62
Concrete Truck	1.33	1.12	2.19	1.84
3S2 (AL)	1.20	0.92	1.98	1.52
School Bus	3.60	2.95	5.59	4.85
HS20-44 Lane Loading	-	1.02	-	1.69

Table 5.5 Rating Factors Based on Approach Truss Connection Capacities

	As-Designed/Present Condition			
	Inventory		Operating	
Truck Type	Single Truck	Eq. Lane Loading	Single Truck	Eq. Lane Loading
Two Axle	0.45	0.37	1.41	1.18
Tri-Axle	0.35	0.29	1.10	0.93
Concrete Truck	0.40	0.33	1.25	1.05
3S2 (AL)	0.36	0.28	1.13	0.87
School Bus	1.06	0.88	3.35	2.78
HS20-44 Lane Loading	-	0.31	-	0.97

The bending moments and shearing forces in the stringers and floorbeams were calculated by considering the riveted connections at the member ends act as simple supports. The compression flanges of the stringers were embedded in the underside of the concrete deck. It was judged that the concrete deck provided sufficient lateral restraint so that the stringer compression flanges are continuously braced. The compression flanges of the floorbeams were considered to be laterally braced at the stringer to floorbeam connections.

Summaries of the rating factors calculated for the stringers and floorbeams are given in Tables 5.6 and 5.7. These rating factors were based on the flexural capacities of the members. It was found that rating factors based on the shear capacities of the members and the stringer to floorbeam and the floorbeam to truss connections are on the order of two to three times higher than those presented in Tables 5.6 and 5.7. Hence, those rating factors are not presented in this report.

No significant section losses were found along the stringers, so the stringer ratings shown in Table 5.6 serve as both As–Designed and Present–Condition ratings. It is assumed here that broken rivets and loose rivets on the stringer to floorbeam connections identified by A.G. Lichtenstein & Associates (B.B. Commer Bridge 1991) will be replaced in the near future by high strength bolts. The Present–Condition rating factors given in Table 5.7 for the floorbeams are slightly lower than the As–Designed ratings. This results from corrosion damage found during the field inspection. The bridge inspection revealed up to 3/16 inch deep corrosion pitting loss at the top of the bottom flange and web on the floorbeam No. 8 of span 3. This loss of section was accounted for in calculating the Present–Condition ratings.

MAIN TRUSS RATINGS

The three–span continuous truss (228'8"–310'2"–228'8") is referred to here as main truss (spans 5 through 7). An elevation showing the overall geometry, support conditions and joint numbering for the truss is shown in Figure 5.5. A typical cross section of the truss is given in Figure 5.6. Cross sectional properties for the truss members are given in Table 5.8.

Structural analyses of the truss were performed using ABAQUS (1989) and the in–house computer program BRIDGE to determine the dead load and live load stresses in each of the members. A summary of the dead load and maximum live load stresses is given in Table 5.9. Because of the long span length of the truss, top and bottom chords develop the maximum stress under an equivalent lane load while the tri–axle dump truck produces the maximum stresses in most verticals and diagonals whose influence lines are confined within the adjacent panels.

Summaries of the minimum inventory and operating rating factors calculated for the main truss are given in Table 5.10. These rating factors are based on the axial stresses created in the truss members by the standard truck loadings. As–Designed ratings are given for loadings by single trucks and the equivalent lane loadings defined by Equation 2.1.

Although a great deal of minor to moderate corrosion and impact damage was identified during the field inspection, none of the truss members had a sufficient loss of section to affect the final truss ratings. For both As-Designed and Present-Condition ratings, members U3U4 and U4U5 controlled the truss rating.

Capacities of the main truss connections were also evaluated. After reviewing the field notes and a series of photographs provided by A.G. Lichtenstein & Associate, connections at both ends of L3U3, L4U5, L4U4, U10U11, and L13U13 were selected as the most critical ones. The truss analyses also confirmed that these were indeed highly stressed connections. The rivets were conservatively assumed to be ASTM A502 Grade 1 rivets with a shank diameter of 7/8 inch as was the case in the rating the approach trusses.

The load capacities of the connections were found to be higher than the truss member capacities except for the operating ratings for the equivalent lane loadings. A summary of As–Designed ratings determined for the connections is given in Table 5.11. The ratings for the single truck loadings and equivalent lane loadings were controlled by the shear capacity of the connection of the member U10U11 to joint U10 for all loading cases. It was judged that the minor corrosion on the connection was insignificant, and the rating factors given in Table 5.11 were sufficiently accurate for use as Present–Condition ratings.

MAIN TRUSS FLOOR SYSTEM RATINGS

The floor system framing is symmetrical with respect to the centerline of the continuous truss. A typical section through the floor system at midspan is shown in Figure 5.6. The floor system is made up of 6 3/4 inch thick concrete deck on four stringers. The stringer span ranges from 22 feet 4 inches to 25 feet 11 inches. Two outside stringers of 18 CB 52 next to the truss and two interior stringers of 21 CB 58 were used on the shorter spans, and all four 21 CB 60 stringers were used on the larger spans. All floorbeams were fabricated from 30 CB 115.

The bending moment and shearing forces in the stringers and floorbeams were evaluated by assuming the riveted connections at the member ends act as simple supports. The compression flanges of the stringers were embedded in the underside of the concrete deck as shown in

Table 5.6 Rating Factors on Approach Truss Stringer

Truck Type	As-Designed/Present-Condition Inventory	As-Designed/Present-Condition Operating
Two Axle	0.85	1.32
Tri-Axle	0.60	0.93
Concrete Truck	0.68	1.06
3S2 (AL)	0.94	1.46
School Bus	1.69	2.63
HS20-44 Lane Load	0.85	1.32

Table 5.7 Rating Factors on Approach Truss Floorbeams

Truck	As-Designed		Present-Condition	
Type	Inventory	Operating	Inventory	Operating
Two Axle	0.81	1.29	0.80	1.27
Tri-Axle	0.60	0.96	0.59	0.94
Concrete Truck	0.68	1.08	0.67	1.06
3S2 (AL)	0.94	1.50	0.93	1.47
School Bus	2.05	3.25	2.02	3.19
HS20-44 Lane Load	0.85	1.35	0.84	1.33

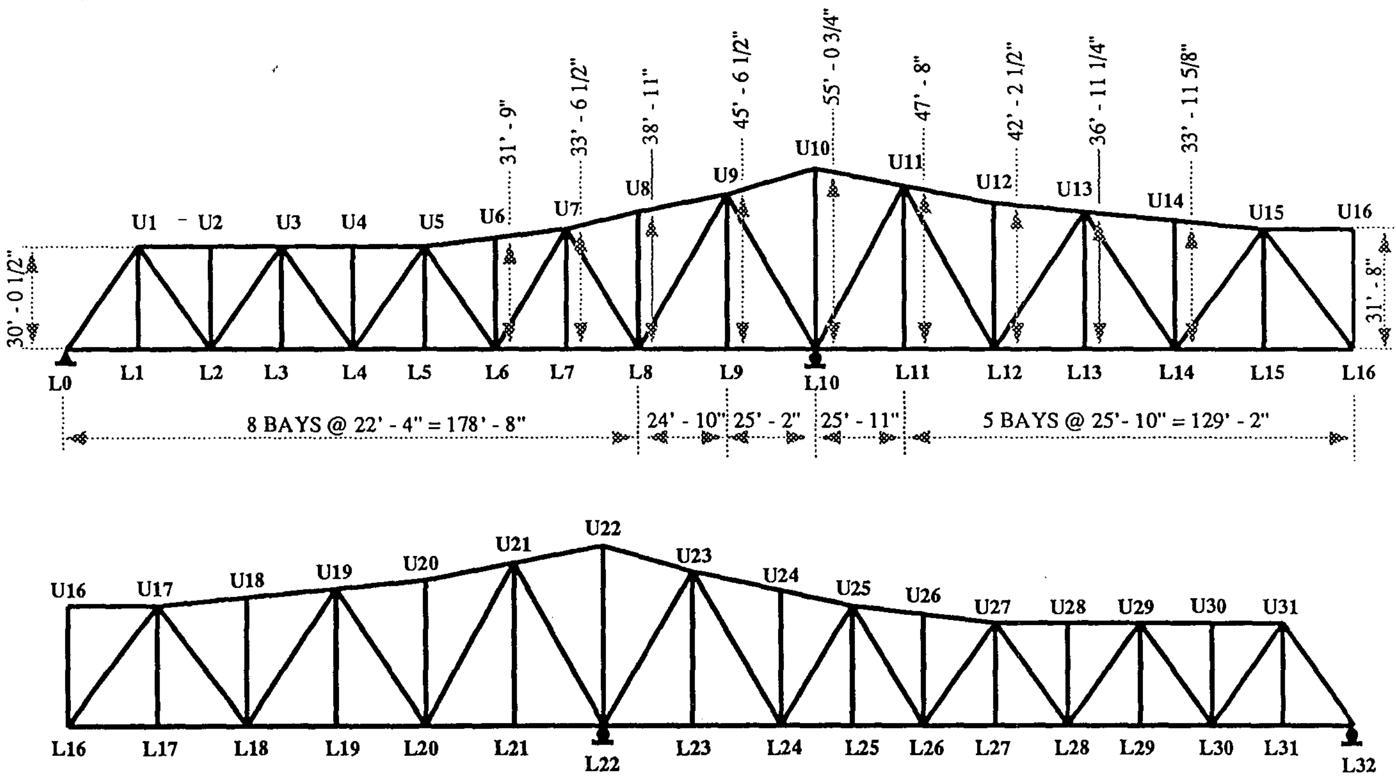


Figure 5.5. Elevation of Main Truss

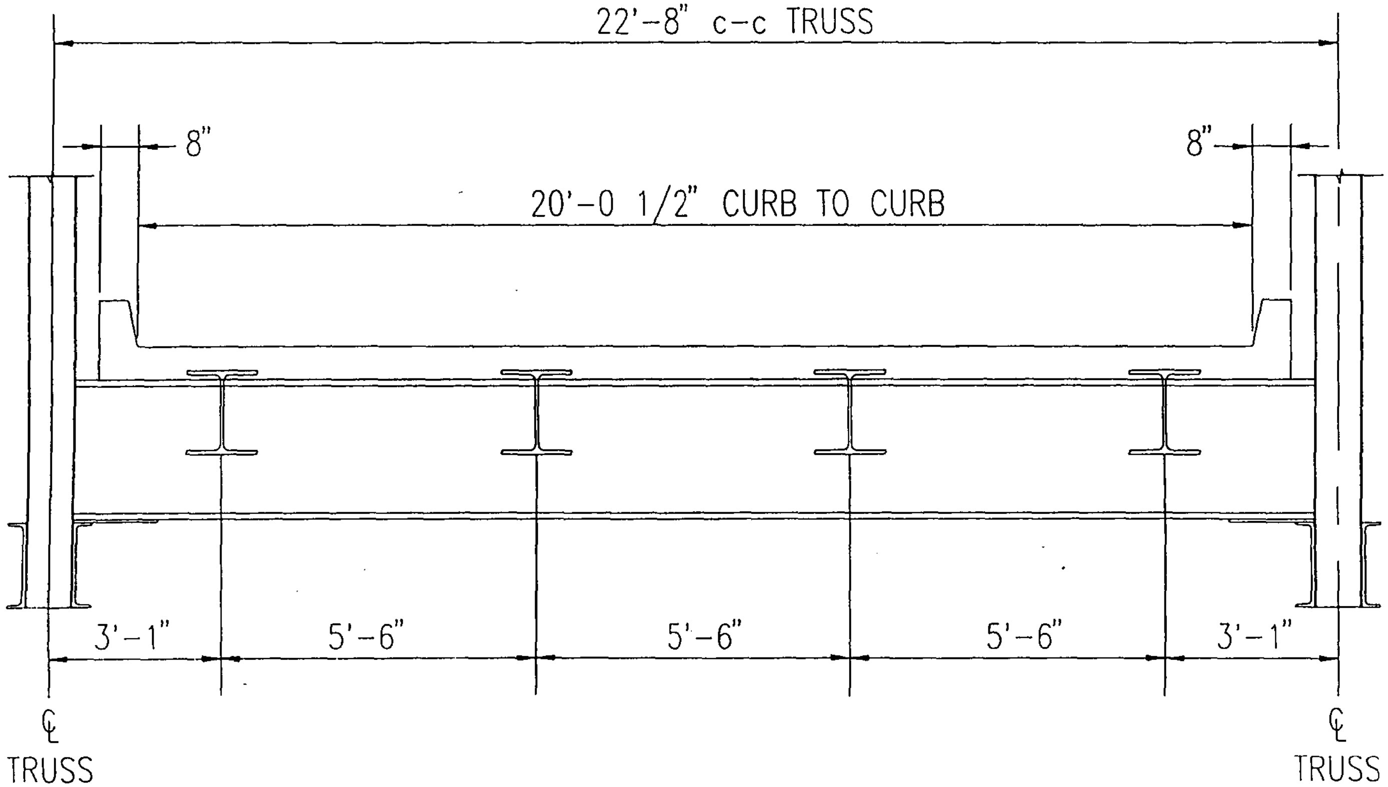


Figure 5.6. Cross Section Through Main Truss Floor System

Table 5.8 Member Cross Section Properties, Main Truss

Member	Area $(inch^2)$	I_x $(inch^4)$	I_y $(inch^4)$
L0L1,L1L2	12.06	256.2	452.5
L2L3,L3L4	17.58	322.4	685.6
L4L5,L5L6	17.58	322.4	672.1
L6L7,L7L8	12.06	256.2	471.0
L8L9,L9L10	23.4	692.6	972.6
L10L11,L11L12	23.4	692.6	954.0
L12L13,L13L14	12.06	256.2	471.0
L14L15,L15L16	17.58	322.40	672.1
U1U2,U2U3, U3U4,U4U5	19.80	625.2	793.6
U5U6,U6U7, U7U8,U8U9	19.80	625.2	793.6
U9U10,U10U11	29.28	802.8	1,242.4
U11U12,U12U13	19.80	625.2	793.6
U13U14,U14U15, U15U16	19.80	625.2	793.6
L10U10[b]	29.28	802.8	1,326.5
L1U1 - L15U15	9.71	163.4	32.5
L0U1	23.40	692.6	917.4
L2U1	12.06	256.2	461.7
L2U3	12.06	256.2	471.0
L4U3	8.94	133.8	337.5
L4U5	8.78	101.4	317.3
L6U5	19.80	625.2	793.6
L6U7	17.58	322.4	672.1
L8U7	23.40	692.6	972.6
L8U9	23.40	692.6	963.3
L10U9[a]	24.96	1,098.4	1,048.4
L10U11[a]	24.96	1,098.4	1,088.6
L12U11	23.40	692.6	991.3
L12U13[b]	29.28	802.8	1,338.7
L14U13	19.80	625.2	778.1
L14U15	19.80	625.2	809.1
L16U15	12.06	256.2	461.7

[a]: section properties taken from AISC 6th edition

[b]: section properties for C 15x50 in lieu of C 15x51.9

Table 5.9 **Dead Load Stresses and Maximum Live Load Stresses (Main Truss Spans, in ksi)**

Member	Dead Load	Maximum Live Load
L0U1	-4.72	-4.87
L0L1	5.39	5.64
L1U1	2.79	8.08
U1U2	-5.39	-6.31
L2U1	5.89	8.64
L1L2	5.39	5.64
L2U2	-0.33	0.00
U2U3	-5.39	-6.31
L2U3	-2.66	-7.07
L2L3	7.14	8.93
L3U3	2.85	8.12
U3U4	-6.10	-8.74
L4U3	-0.94	7.82
L3L4	7.14	8.93
L4U4	-0.33	0.00
U4U5	-6.10	-8.74
L4U5	5.36	8.91
L4L5	5.30	9.98
L5U5	2.85	8.15
U5U6	-2.00	-7.88
L6U5	-4.62	-4.88
L5L6	5.30	9.98
L6U6	-0.36	-0.06
U6U7	-2.00	-7.88
L6U7	7.25	6.32
L6L7	-2.53	10.64
L7U7	2.81	8.18
U7U8	5.40	-4.65
L8U7	-5.71	-4.57
L7L8	-2.54	10.64
L8U8	0.11	-0.24
U8U9	5.44	-4.68
L8U9	7.09	5.17
L8L9	-7.78	-5.01
L9U9	3.29	8.32
U9U10	9.05	5.11
L10U9	-5.50	-4.21
L9L10	-7.78	-5.01
L10U10	-5.97	-3.17
U10U11	8.80	4.97
L10U11	-6.58	-4.28
L10L11	-7.26	-4.89
L11U11	3.40	8.37
L11U12	4.52	4.48

Table 5.9 Dead Load Stresses and Maximum Live Load Stresses (Main Truss Spans, in ksi) . . . *continued*

Member	Dead Load	Maximum Live Load
L12U11	7.51	5.13
L11L12	-7.26	-4.49
L12U12	-0.36	0.07
U12U13	4.52	4.47
L12U13	-4.79	-3.80
L12L13	-0.70	7.52
L13U13	3.22	8.23
U13U14	-3.11	-6.33
L14U13	6.20	6.01
L13L14	-0.70	7.52
L14U14	-0.55	-0.33
U14U15	-3.10	-6.31
L14U15	-4.00	-5.33
L14L15	6.27	8.69
L15U15	3.26	8.32
U15U16	-6.31	-8.13
L16U15	2.00	6.79
L15L16	6.27	8.69
L16U16	-0.37	0.00

Table 5.10 Rating Factors on Main Truss Members

Truck Type	As-Designed/Present Condition			
	Inventory		Operating	
	Single Truck	Eq. Lane Loading	Single Truck	Eq. Lane Loading
Two Axle	1.44	1.17	2.09	1.69
Tri-Axle	1.12	0.92	1.62	1.33
Concrete Truck	1.27	1.00	1.84	1.51
3S2 (AL)	1.10	0.86	1.60	1.25
School Bus	3.41	2.76	4.94	4.00
HS20-44 Lane Loading	-	0.96	-	1.39

Table 5.11 Rating Factors on Main Truss Connection Capacities

Truck Type	As-Designed/Present Condition			
	Inventory		Operating	
	Single Truck	Eq. Lane Loading	Single Truck	Eq. Lane Loading
Two Axle	0.96	0.85	1.73	1.75
Tri-Axle	0.76	0.67	1.36	1.37
Concrete Truck	0.86	0.76	1.55	1.56
3S2 (AL)	0.78	0.63	1.40	1.29
School Bus	2.35	2.00	4.22	4.12
HS20-44 Lane Loading	-	0.69	-	1.43

Table 5.12 Rating Factors on Main Truss Stringers

Truck Type	As-Designed/Present-Condition	
	Inventory	Operating
Two Axle	0.64	1.04
Tri-Axle	0.45	0.73
Concrete Truck	0.51	0.83
3S2 (AL)	0.71	1.15
School Bus	1.28	2.07
HS20-44 Lane Load	0.65	1.06

Table 5.13 Rating Factors on Main Truss Floorbeams

Truck Type	As-Designed/Present-Condition	
	Inventory	Operating
Two Axle	0.78	1.24
Tri-Axle	0.57	0.92
Concrete Truck	0.65	1.04
3S2 (AL)	0.89	1.42
School Bus	1.94	3.10
HS20-44 Lane Load	0.81	1.30

Figure 5.6. It was judged that the concrete deck provided sufficient lateral restraint so that the stringer compression flanges are continuously braced. The compression flanges of the floorbeams were considered to be laterally braced at the stringer to floorbeam connections.

Calculations indicated that ratings for the exterior stringers were lower than those for the interior stringers. Summaries of the rating factors calculated for the exterior stringers and floorbeams are given in Tables 5.12 and 5.13. These rating factors were based on the flexural capacities of the members. It was found that rating factors based on the shear capacities of the members and the stringer to floorbeam and floorbeam to truss connections (including the combined shear and tension analysis) are on the order of two to five times higher than those presented in Tables 5.12 and 5.13. Hence, those rating factors are not presented in this report.

No significant section losses were found along the stringers and floorbeams, so the rating factors given in Tables 5.12 and 5.13 serve as both As–Designed and Present–Condition ratings. The field inspection identified two stringer to floorbeam connections with a single lost rivet. It was assumed in the rating calculations that the lost rivets will be replaced by high strength bolts in the near future.

CONCRETE APPROACH SPAN AND BRIDGE DECK

There are no structural drawings available that show the reinforcing details used in the reinforced concrete bridge decks or approach span tee beams. It was not possible to calculate quantitative rating factors for these members.

The field inspection results indicate that the concrete approach span (span 21, 36 feet 6 inches) appear to be in fair condition. The bridge deck was also found to be in fair condition. The top of the deck typically exhibited light to moderate scaling and some areas of hairline map cracking. Transverse cracks up to 1/8 inch wide in the top of the deck were common on top of piers 8 – 13 adjacent to steel stringer cantilevers. Transverse cracks at the middle of span 6 (middle of main truss center span where the floorbeams were rated low) were typically up to 1/8 inch wide. The underside of the deck generally exhibited small surface spalls and minor cracking. A few shallow pop–outs with exposed reinforcing bars were also found. The condition of the deck, however, does not appear to require a reduction of the load ratings of the bridge.

CONCLUSIONS AND RECOMMENDATIONS

Structural analyses and load ratings of the bridge indicate that the flexural capacity of the exterior stringers of the main truss in panel L8L9 (stringer span of 24 feet 10 inches) control the bridge ratings. A summary of the Present–Condition ratings for the bridge is given in table 5.14. The field inspections also identified the following maintenance and repairs that should be carried out:

1. Rivets missing from the stringer to floorbeam connections should be replaced with A325 high strength bolts and hardened washers. The connections are located at the floorbeam numbers 12 and 19 in the center span (span 6) of the main truss.

2 The cracks in the concrete deck which are 1/8 inch wide or wider should be sealed by epoxy injection.

3. The broken railings in spans 11,13,14, and 21 should be replaced.

4. The broken rail posts on the northern approach in spans 16,17, and 21 should be replaced.

5. The diaphragms adjacent to the expansion joints on the north approach which have broken up are ineffective and should be replaced.

6. All damaged truss members, end portals, and sway frames should be repaired by heat straightening.

7. The rocker bearings on pier 1 should be placed into proper position and should receive new bolts.

8. Installation of shear locks at the cantilever joints in the north approach span stringers is suggested as an adequate measure for controlling live load deflections at the cantilever tips.

9. Large spalls on the substructure should be repaired.

10. The horizontal crack in the column leg of pier 14 should be injected with an epoxy resin compound and monitored closely.

Table 5.14 Summary of Present-Condition Ratings for Jackson County Structure No. 035-36-009.8A

Truck Type	Standard Weight (tons)	Inventory		Operating	
		Rating Factor	Rating (tons)	Rating Factor	Rating (tons)
Two Axle	29.5	0.37	11	1.04	>29.5
Tri-Axle	37.5	0.29	11	0.73	27
Concrete Truck	33	0.33	11	0.83	27
3S2 (AL)	40	0.28	11	0.87	35
School Bus	12.5	0.88	11	2.07	>12.5

CHAPTER SIX
MORGAN/MADISON COUNTY
STRUCTURE NO. 053–52–008.7B

Morgan/Madison County Structure No. 053–52–008.7B was constructed in 1933 and currently carries U.S. Route 231 and Alabama Route 53 traffic over the Tennessee River from Laceys Spring to Huntsville. The bridge consists of six steel I–beam approach spans and seven truss spans. The truss spans consists of four 200 feet long simple span through trusses and one three–span continuous through truss. An elevation of the bridge illustrating the arrangement of the spans and span numbering is shown in Figure 6.1. A summary of the structural analyses and load rating results are presented in this chapter. Results from the field inspection and condition survey are summarized by A.G. Lichtenstein & Associates (Morgan/Madison County 1991).

STRINGER SPAN RATINGS

There is a total of six steel stringer spans (five at Laceys Spring side and one at Huntsville side). Based on the time of construction the steel members are assumed to have a yield strength of 30 ksi as suggested by AASHTO (Manual 1983). Most steel stringer bridges with a concrete deck constructed in the early 1930's did not use any mechanical shear studs. Lacking evidence of shear studs between the steel stringer and the concrete deck, non–composite section properties were used in the rating calculations. A.G. Lichtenstein & Associates did find shop drawings of the truss spans and standard deck girder details which correspond to the approach spans (spans 1 through 5, and 13). The availability of reinforcement details of the concrete deck made it possible to rate the slab also. It was assumed according to AASHTO (Manual 1983) that the concrete strength was 3300 psi and the yield strength of the reinforcing steel was 33 ksi.

Each steel stringer approach span is basically a simple span with one or two cantilever overhangs. An expansion joint separates cantilever tips. A typical cross section of the steel stringer approach span is shown in Figure 6.2. Spans 3,5, and 13 were used to determine the As–Designed ratings. Span 3 is a 50 feet long simple span (B30x108 stringers) with a 15 feet overhang on each end. Span 5 is a 54 feet long simple span (B33x125 stringers) with one 15 feet overhang on the Laceys Spring side. Span 13 consists of four 62 feet long simple steel stringers (B36x150). The rating factors given in Table 6.1 were based on the flexural capacities (positive bending moment controlled) of the stringers.

The field inspection (Morgan/Madison County 1991) revealed that the arch shape concrete diaphragms are cracked and broken so that they appear

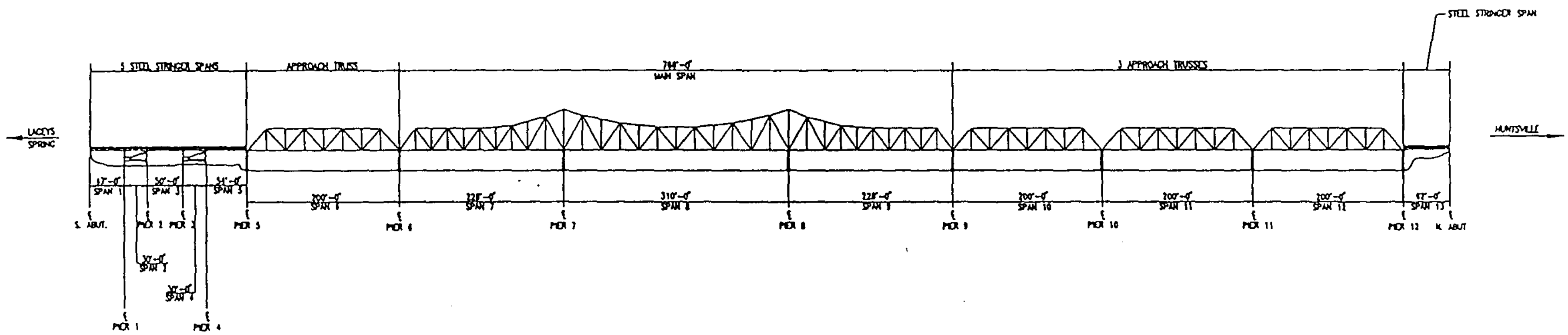


Figure 6.1. East Elevation of Huntsville Bridge (053-52-008.7B)

BEAM TABLE	
LOCATION	STRINGER SIZE
SPAN 1	B30 x 108
SPAN 3	B30 x 108
SPAN 5	B33 x 125
SPAN 13	B36 x 150

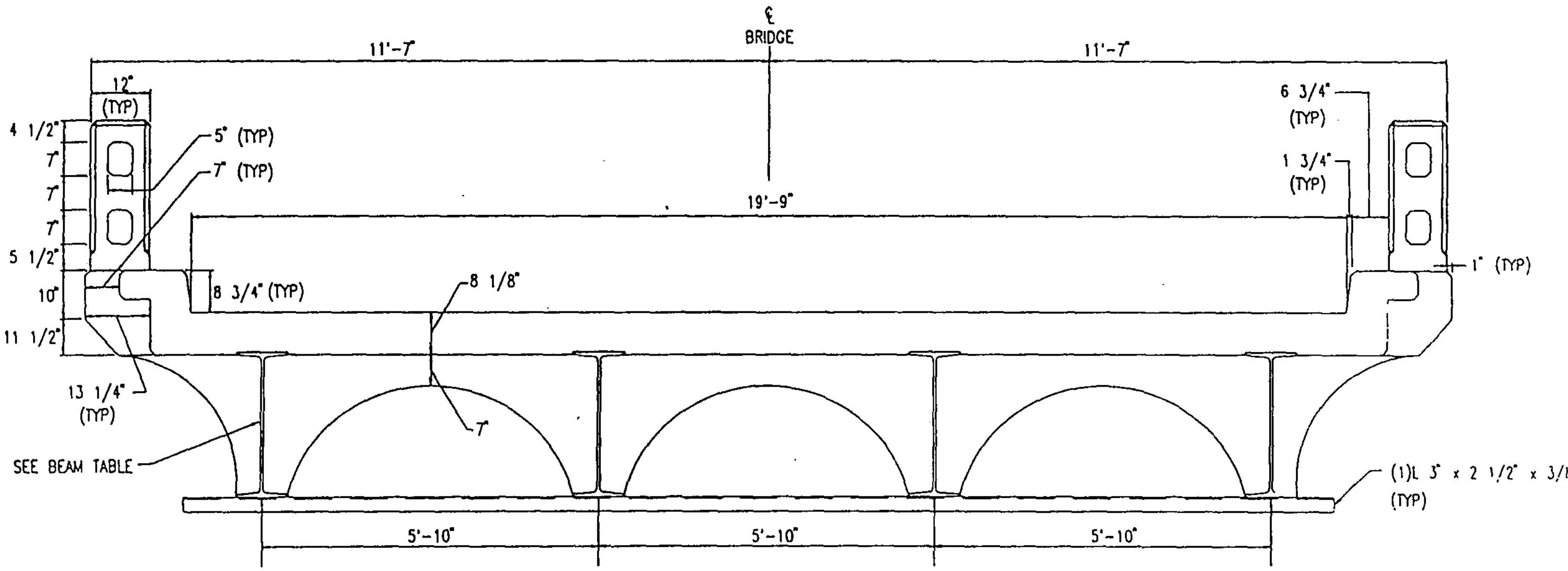


Figure 6.2. Typical Cross Section at Stringer Approach Spans

Table 6.1 Rating Factors for Approach Stringer Spans

Truck Type	As-Designed/Present-Condition	
	Inventory	Operating
Two Axle	0.57	1.00
Tri-Axle	0.42	0.77
Concrete Truck	0.47	0.84
3S2 (AL)	0.56	1.13
School Bus	1.36	2.26

to have no positive connection to the steel stringers, particularly in spans 1 through 5. The resulting loss of lateral bracing was accounted for by reducing the allowable flexural stresses in calculating the Present–Condition ratings. However, this loss of lateral bracing did not reduce the final ratings of the stringer spans since they were controlled by the flexural capacities of the stringers in span 13. The arch diaphragms were not broken in span 13. Rating factors for the stringer span decks are given in Table 6.2. In the Present–Condition rating of the concrete deck, calculation of a quantitative reduction of the load carrying capacity to account for the deterioration was impractical. It was judged that the condition of the deck would create no more than a 30 percent decrease in the deck capacity. Present-Condition ratings were estimated by reducing the deck capacity by 30 percent as shown in Table 6.2. It was found that even these conservative estimates of the deck ratings do not control the overall bridge ratings.

The field inspection has identified that the concrete railing has been hit and has completely failed in several locations on the west side next to abutment 1. It is assumed here that these railings will be repaired in the near future.

The field inspection report also indicated that excessive live load deflections were observed at the cantilever ends of the stringer spans. Live load deflections were calculated at the cantilever ends as suggested in the inspection reports. Deflections were calculated at the ends of span 3 and 5 for tri–axle dump truck loading. The computed deflections were approximately three times as large as the AASHTO (Standard 1989) limiting value of L/300 (15x12/300=0.6 inch). The calculated deflections and inspection results indicate that measures should be taken to reduce the live load deflections at the cantilever tips.

The use of shear locks at the cantilever ends is suggested in the inspection report as a possible means of reducing the live load deflections. Deflections were recalculated so that the effectiveness of shear locks could be evaluated. The maximum calculated deflection for full truck loading was 1.01 inch which is 68 per cent larger than the AASHTO limiting value. For truck loadings corresponding to the final operating ratings presented at the end of this report, the AASHTO deflection limit is satisfied. It is concluded that shear locks at the cantilever tips would provide a sufficient reduction of live load deflections.

It should be noted that shear locks at the cantilever ends are not required for adequate strength of the stringer spans. The live load deflection limit described above is related to serviceability and maintenance of the spans. According to an authoritative publication on AASHTO deflection limits by AISI (Bulletin No. 19, 1971), limits on live load deflections provide highway steel

Table 6.2 Rating Factors for Concrete Deck

Truck Type	As-Designed Inventory	As-Designed Operating	Present-Condition Inventory	Present-Condition Operating
Two Axle	2.39	3.98	1.64	2.77
Tri-Axle	2.12	3.54	1.48	2.49
Concrete Truck	1.91	3.18	1.34	2.23
3S2 (AL)	2.73	4.55	1.91	3.19
School Bus	3.56	5.94	2.49	4.16

bridges with service lives uninterrupted by fatigue damage, durable surfaces, and comfortable crossings for pedestrians and occupants of moving vehicles.

APPROACH TRUSS RATINGS

Four identical through trusses of 200 feet span are referred to here as approach trusses (span 1 and spans 10 through 12). An elevation showing the overall geometry, support conditions and joint numbering for the trusses is given in Figure 6.3. A typical cross section of the trusses is shown in Figure 6.4. Cross sectional properties for the truss members are given in Table 6.3.

Structural analyses of the truss were performed using ABAQUS (1989) and the in–house computer program BRIDGE to determine the dead load and live load stresses in each of the members. A summary of the dead load and maximum live load stresses is given in Table 6.4. Because of the somewhat long span length of the truss, the top and bottom chords develop the maximum stress under an equivalent lane load while the tri–axle dump truck produces the maximum stresses in most verticals and diagonals whose influence lines are confined within the adjacent panels.

Summaries of the minimum inventory and operating rating factors calculated for the approach truss are given in Table 6.5. These ratings are based on the axial stresses created in the truss members by the standard truck loadings. As–Designed ratings are given for loadings by single trucks and the equivalent lane loadings defined by Equation 2.1.

The corrosion and impact damage identified during the filed inspection was considered in determining the Present–Condition ratings for the approach truss. There was a great deal of minor to moderate corrosion and impact damage to the truss, particularly to members L2U2 and L8U7. None of the truss members had a sufficient loss of section to affect the truss ratings, although some individual member capacities were reduced. For both As–Designed and Present–Condition ratings, member U3U4 controls the truss rating.

Capacities of the approach truss connections were also evaluated. The truss analyses revealed that members L0L1, L1L2, L2L3, L3L4, L1U1, L2U1, L3U3, and L4U3 are in tension. The connections at the ends of L1L2, L2L3, L2U1, and L3U3 were selected as the most critical ones. The shop drawings (17/19 and 18/19) by Virginia Bridge and Iron Co. indicate that the diameter of rivets is 3/4 inch. It was found that the splicing of the bottom chord tension member L2L3 at the middle of the member controlled the connection ratings. Ratings were calculated for both common rivet types, Grade 1 and Grade 2, for which AASHTO (Manual 1983) give allowable stresses. Rating factors for the truss connections are summarized in Table 6.6. The ratings shown are based

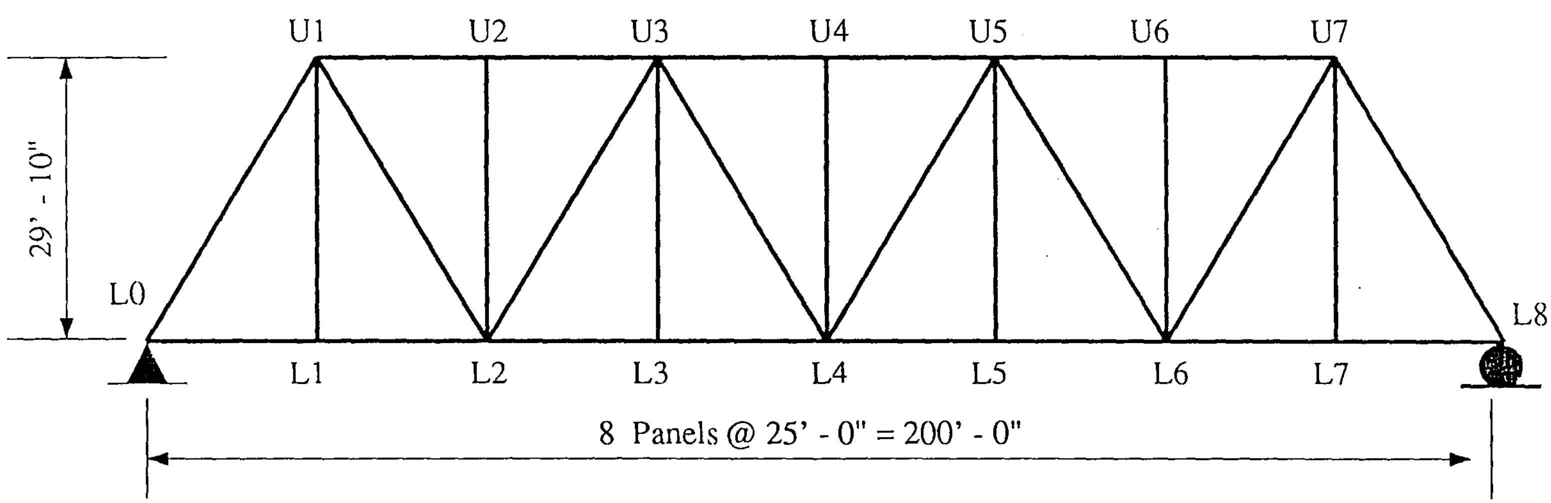


Figure 6.3. Elevation of Approach Trusses

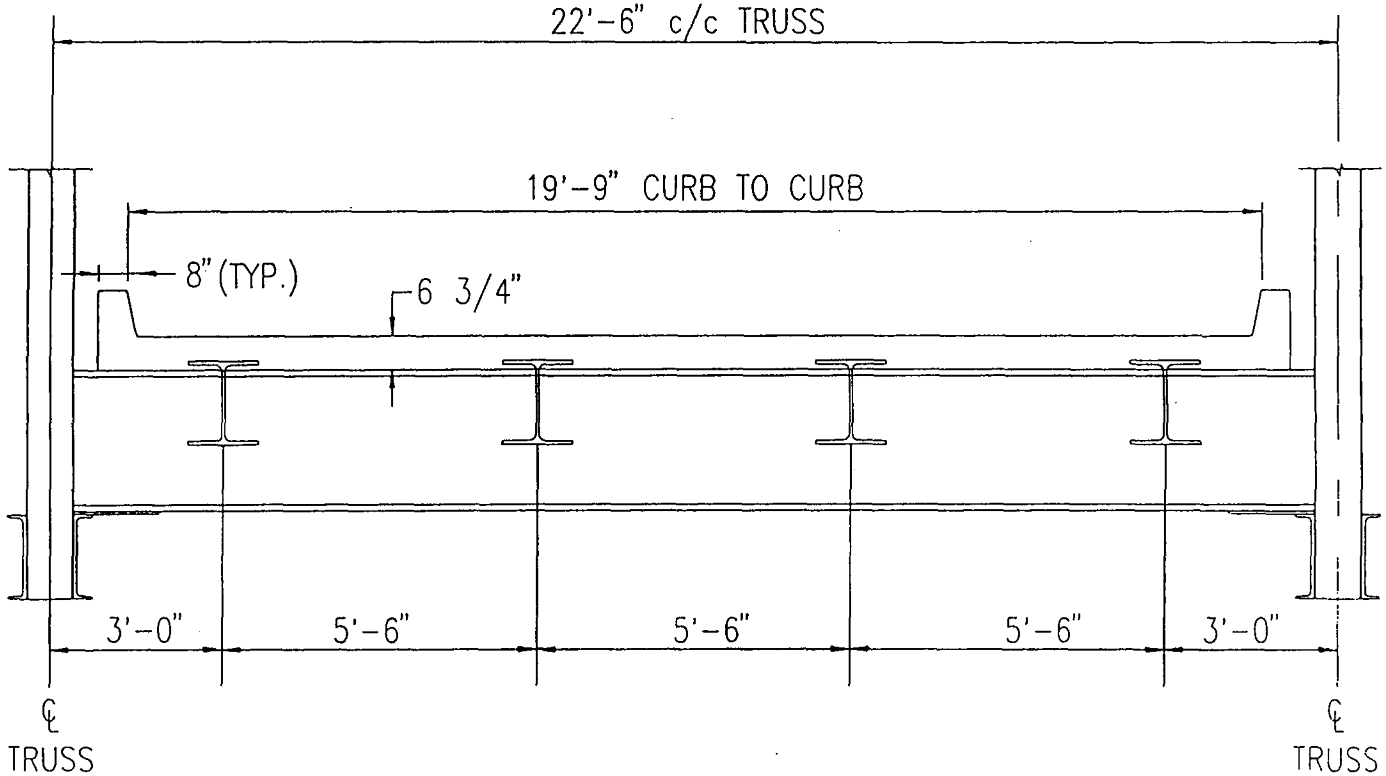


Figure 6.4. Cross Section Through Approach Trusses Floor System

Table 6.3 Member Cross Section Properties, Approach Truss

Member	Area ($inch^2$)	I_x ($inch^4$)	I_y ($inch^4$)
L0U1	28.3	976	1,011
L1U1	9.1	163	33
L2U2	9.1	163	33
L3U3	9.1	163	33
L4U4	9.1	163	33
L2U1	17.6	322	686
L2U3	17.6	322	672
L4U3	11.7	157	414
U1U2	25.4	881	950
U2U3	25.4	881	950
U3U4	34.2	1,109	1,246
L0L1	19.8	625	794
L1L2	19.8	625	794
L2L3	29.3	803	1,150
L3L4	29.3	803	1,150

Table 6.4 Dead Load Stresses and Maximum Live Load Stresses In Approach Truss Members

Member	Dead Load Stress (ksi)	Maximum Live Load Stress (ksi)
L0U1	-6.19	-3.79
L0L1	5.69	3.48
L1U1	3.41	7.60
U1U2	-7.65	-4.65
L2U1	7.21	4.64
L1L2	5.69	3.48
L2U2	-0.43	-0.00
U2U3	-7.65	-4.65
L2U3	-4.35	-3.80
L2L3	8.31	5.04
L3U3	3.53	7.60
U3U4	-7.59	-4.60
L4U3	2.17	4.49
L3L4	8.31	5.04
L4U4	-0.54	0.00

Table 6.5 Rating Factors for Approach Truss Members

Truck Type	As-Designed/Present Condition			
	Inventory		Operating	
	Single Truck	Eq. Lane Loading	Single Truck	Eq. Lane Loading
Two Axle	1.86	1.56	2.92	2.45
Tri-Axle	1.46	1.23	2.28	1.92
Concrete Truck	1.65	1.40	2.58	2.19
3S2 (AL)	1.48	1.15	2.32	1.80
School Bus	4.40	3.69	6.89	5.77
HS20-44 Lane Loading	–	1.28	–	2.00

Table 6.6 Rating Factors for Approach Truss Connection Capacities

Truck Type	As-Designed/Present Condition			
	Grade 1 Rivets		Grade 2 Rivets	
	Inventory	Operating	Invengory	Operating
Two Axle	0.16	0.88	1.20	2.33
Tri-Axle	0.13	0.69	0.95	1.83
Concrete Truck	0.14	0.79	1.08	2.08
3S2 (AL)	0.12	0.65	0.89	1.72
School Bus	0.37	2.08	2.84	5.50
HS20-44 Lane Loading	0.13	0.72	0.99	1.91

on the shear capacity of the rivets. Ratings based on bearing stresses were found to be much higher than those shown in Table 6.6.

It is prudent to consider the rivets to be the lower strength Grade 1 because their actual strength is not known. It is seen from Table 6.6 that most of the rating factors for the Grade 1 rivets are less than those based on the truss member capacity. It should be noted that the ratings for Grade 1 rivets are considered to be conservative estimates of the connection ratings. It is likely that the rivets used in the connections are Grade 2 instead of Grade 1. Past experiences of A.G. Lichtenstein & Associates suggest that most riveted truss bridges were constructed with Grade 2 rivets. It is seen from Table 6.6 that ratings based on Grade 2 rivets are significantly higher.

The field inspection results indicate that there is minor corrosion on most of the connections. The effects of the corrosion on the connections was found not to be significant.

APPROACH TRUSS FLOOR SYSTEM RATINGS

The floor system framing for all four trusses is identical. A typical section through the floor system at midspan is shown in Figure 6.3. The floor system is made up of a 6 3/4 inch thick concrete deck on four stringers (21 CB 63) spaced at 5 feet 6 inches. The stringers span 25 feet between floorbeams (30 CB 115). Properties for stringers and floorbeams were taken from AISC (Iron and Steel Beams 1985).

The bending moments and shearing forces in the stringers and floorbeams were calculated by considering the riveted connections at the member ends to act as simple supports. The compression flanges of the stringers were embedded in the underside of the concrete deck. It was judged that the concrete deck provided sufficient lateral restraint so that the stringer compression flanges are continuously braced. The compression flanges of the floorbeams were considered to be laterally braced at the stringer to floorbeam connections.

Summaries of the rating factors calculated for the stringers and floorbeams are given in Tables 6.7 and 6.8. These rating factors were based on the flexural capacities of the members. It was found that rating factors based on the shear capacities of the members and the stringer to floorbeam and the floorbeam to truss connections are on the order of two to three times higher than those presented in Tables 6.7 and 6.8. Hence, those rating factors are not presented in this report. It is assumed here that broken rivets and loose rivets on the stringer to floorbeam connections identified by A.G. Lichtenstein & Associates (Morgan/Madison County Bridge 1991) will be replaced in the near future by high strength bolts. The exact locations with

Table 6.7 Rating Factors for Approach Truss Stringer

Truck Type	As-Designed/Present-Condition Inventory	As-Designed/Present-Condition Operating
Two Axle	0.94	1.44
Tri-Axle	0.67	1.02
Concrete Truck	0.75	1.15
3S2 (AL)	1.04	1.59
School Bus	1.87	2.87
HS20-44 Lane Load	0.97	1.49

Table 6.8 Rating Factors for Approach Truss Floorbeams

Truck	As-Designed		Present-Condition	
Type	Inventory	Operating	Inventory	Operating
Two Axle	0.82	1.30	0.78	1.25
Tri-Axle	0.61	0.96	0.58	0.92
Concrete Truck	0.69	1.09	0.66	1.04
3S2 (AL)	0.95	1.50	0.91	1.45
School Bus	2.06	3.26	1.97	3.14
HS20-44 Lane Load	0.86	1.36	0.82	1.31

damaged or loose fasteners are given in Table 1 of the above mentioned field inspection report.

No significant section losses were found along the stringers, so the stringer ratings shown in Table 6.7 serve as both As–Designed and Present–Condition ratings. The Present–Condition rating factors given in Table 6.8 for the floorbeams are slightly lower than the As–Designed ratings. This results from corrosion damage found during the field inspection. The bridge inspection revealed up to 1/4 inch deep corrosion pitting loss at the top of the bottom flange and web on the floorbeam No. FB0 of span 12. This loss of section was accounted for in calculating the Present–Condition ratings.

MAIN TRUSS RATINGS

The three–span continuous truss (228'8"–310'2"–228'8") is referred to here as the main truss (spans 7 through 9). An elevation showing the overall geometry, support conditions and joint numbering for the truss is shown in Figure 6.5. A typical cross section of the truss is given in Figure 6.6. Cross sectional properties for the truss members are given in Table 6.9.

Structural analyses of the truss were performed using ABAQUS (1989) and the in–house computer program BRIDGE to determine the dead load and live load stresses in each of the members. A summary of the dead load and maximum live load stresses is given in Table 6.10. Because of the long span length of the truss, the top and bottom chords develop the maximum stress under an equivalent lane load while the tri–axle dump truck produces the maximum stresses in most verticals and diagonals whose influence lines are confined within the adjacent panels. Summaries of the minimum inventory and operating rating factors calculated for the main truss are given in Table 6.11. These rating factors are based on the axial stresses created in the truss members by the standard truck loadings. As–Designed ratings are given for loadings by single trucks and the equivalent lane loadings defined by Equation 2.1.

Although a great deal of minor to moderate corrosion and impact damage was identified during the field inspection, none of the truss members had a sufficient loss of section to affect the final truss ratings. For both As–Designed and Present–Condition ratings, members U3U4 and U4U5 controlled the truss rating.

Capacities of the main truss connections were also evaluated. After reviewing the field notes and a series of photographs provided by A.G. Lichtenstein & Associate, connections at both ends of U3U4, U4U5, L4U5, L2U1, L9U9, L3L4, and L5L6 were selected as the most critical ones. The truss analyses also confirmed that these were indeed highly stressed connections. The shop drawings (8/19 through 14/19) by Virginia Bridge and

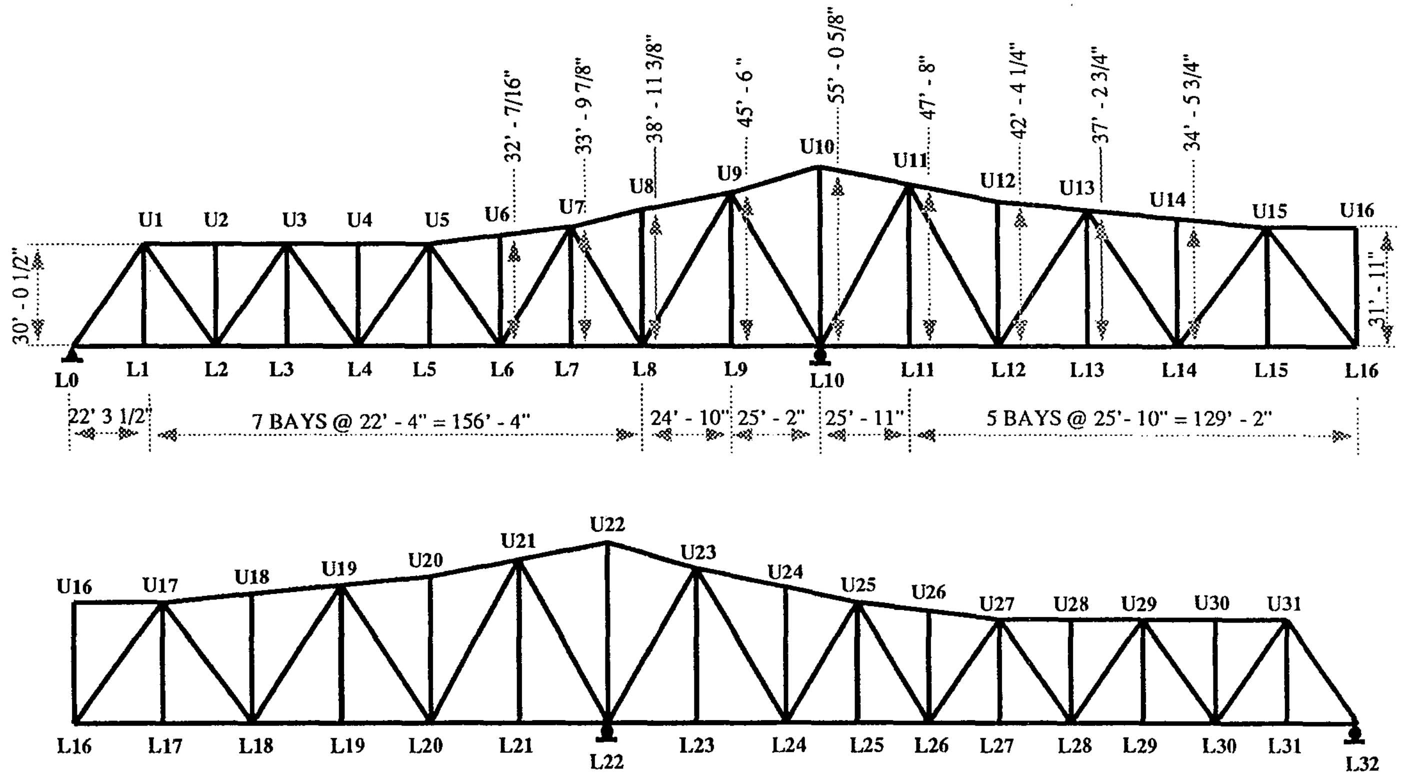


Figure 6.5. Elevation of Main Trusses

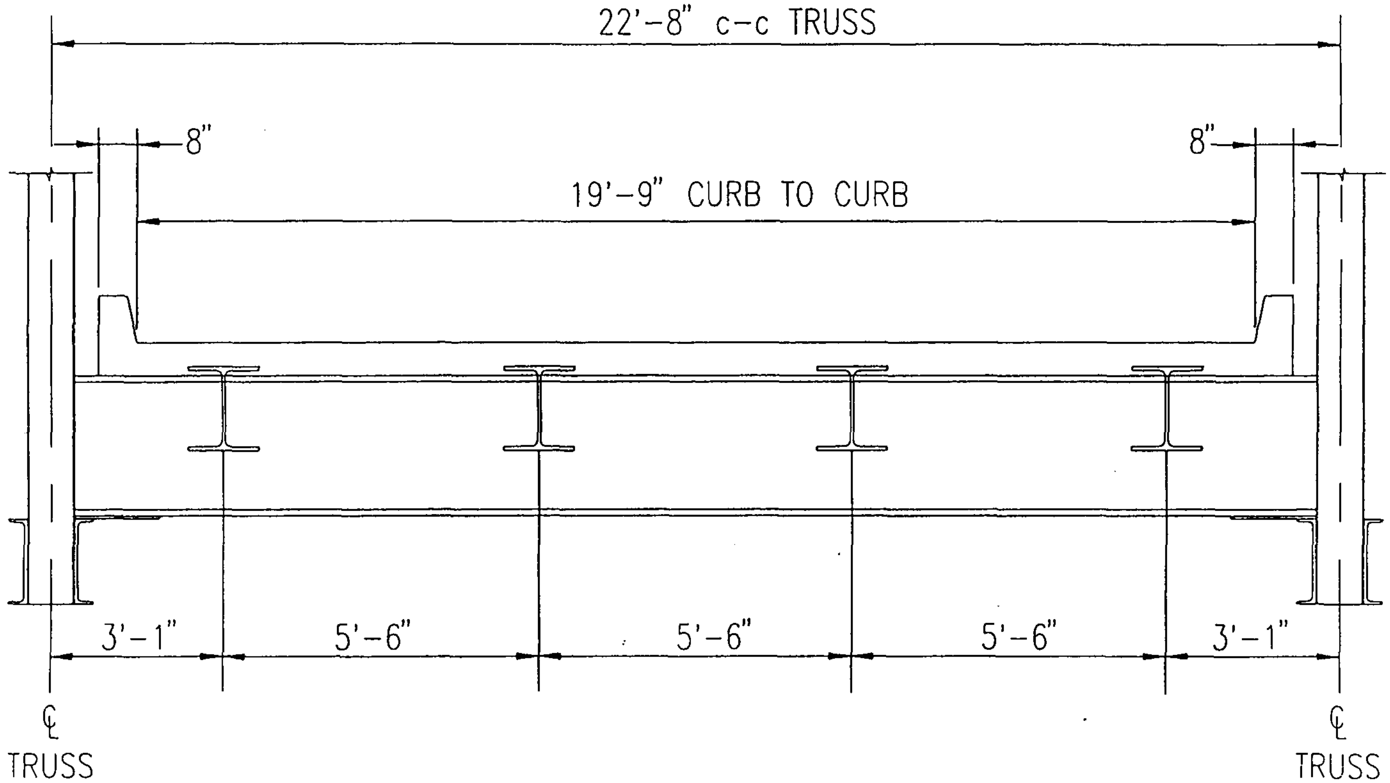


Figure 6.6. Cross Section Through Main Truss Floor System

Table 6.9 Member Cross Section Properties, Main Truss

Member	Area (inch^2)	I_x (inch^4)	I_y (inch^4)
L0L1,L1L2	12.06	256.2	452.5
L2L3,L3L4, L4L5,L5L6	17.58	322.4	699.3
L6L7,L7L8	12.06	256.2	466.4
L8L9,L9L10	29.3	802.8	1,138.4
L10L11,L11L12	23.4	692.6	954.0
L12L13,L13L14	12.06	256.2	480.4
L14L15,L15L16	17.58	322.40	699.3
U1U2,U2U3, U3U4,U4U5	19.80	625.2	778.1
U5U6,U6U7	19.80	625.2	785.8
U7U8,U8U9	19.80	625.2	793.6
U9U10,U10U11	29.28	802.8	1,242.4
U11U12,U12U13	19.80	625.2	778.1
U13U14,U14U15	19.80	625.2	778.1
U15U16	29.28	802.8	1,127.1
L10U10	29.28	802.8	1,326.5
L1U1 - L15U15	9.11	163.4	32.5
L0U1	23.40	692.6	917.4
L2U1	12.06	256.2	461.7
L2U3	12.06	256.2	471.0
L4U3	8.94	133.8	334.1
L4U5	7.78	94.6	288.8
L6U5	19.80	625.2	778.1
L6U7	17.58	322.4	672.1
L8U7	23.40	692.6	972.6
L8U9	23.40	692.6	963.3
L10U9	24.96	1,098.4	1,068.4
L10U11	26.76	1,147.0	1,124.3
L12U11	23.40	692.6	991.3
L12U13	30.36	1,244.2	1,288.0
L14U13	19.80	625.2	801.3
L14U15	19.80	625.2	809.1
L16U15	12.06	256.2	452.5

Table 6.10 Dead Load Stresses and Maximum Live Load Stresses In Main Truss Members

Member	Dead Load Stress (ksi)	Maximum Live Load Stress (ksi)
L0U1	-4.83	-4.94
L0L1	5.73	5.69
L1U1	3.05	8.61
U1U2	5.50	-6.23
L2U1	6.04	8.58
L1L2	5.73	5.69
L2U2	-0.22	0.00
U2U3	-5.50	-6.23
L2U3	2.74	-7.02
L2L3	7.45	8.82
L3U3	3.11	8.64
U3U4	-6.24	-8.62
L4U3	-0.78	7.75
L3L4	7.45	8.82
L4U4	-0.22	0.00
U4U5	-6.24	-8.62
L4U5	5.97	9.97
L4L5	5.61	9.83
L5U5	3.11	8.68
U5U6	-2.10	-7.73
L6U5	-4.55	-4.85
L5L6	5.61	9.83
L6U6	-0.22	-0.03
U6U7	-2.10	-7.73
L6U7	7.22	6.28
L6L7	-2.18	10.43
L7U7	3.07	8.71
U7U8	5.39	-4.57
L8U7	-5.73	-4.51
L7L8	-2.18	10.43
L8U8	0.16	-0.33
U8U9	5.43	-4.61
L8U9	7.07	5.14
L8L9	-6.20	-3.98
L9U9	3.66	8.86
U9U10	9.17	5.08
L10U9	-5.44	-4.19
L9L10	-6.20	-3.98
L10U10	-5.83	-3.16
U10U11	8.95	4.94
L10U11	-6.24	-3.95
L10L11	-7.27	-4.83
L11U11	3.71	8.92
L11U12	4.61	4.49

Table 6.10 **Dead Load Stresses and Maximum Live Load Stresses In Main Truss Members . . . *continued***

Member	Dead Load Stress (ksi)	Maximum Live Load Stress (ksi)
L12U11	7.63	5.13
L11L12	-7.27	-4.83
L12U12	-0.18	0.07
U12U13	4.60	4.49
L12U13	-4.67	-3.67
L12L13	-0.56	7.53
L13U13	3.52	8.77
U13U14	-2.99	-6.30
L14U13	6.17	5.96
L13L14	-0.56	7.53
L14U14	-0.30	-0.10
U14U15	-2.99	-6.29
L14U15	-3.99	-5.33
L14L15	6.30	8.75
L15U15	3.56	8.86
U15U16	-4.13	-5.55
L16U15	1.81	6.76
L15L16	6.30	8.75
L16U16	-0.37	0.00

Table 6.11 Rating Factors for Main Truss Members

Truck Type	As-Designed/Present Condition			
	Inventory		Operating	
	Single Truck	Eq. Lane Loading	Single Truck	Eq. Lane Loading
Two Axle	1.17	1.17	1.70	1.70
Tri-Axle	0.92	0.92	1.34	1.34
Concrete Truck	1.05	1.05	1.52	1.52
3S2 (AL)	0.86	0.86	1.26	1.26
School Bus	2.76	2.76	4.02	4.02
HS20-44 Lane Loading	–	0.96	–	1.40

Table 6.12 Rating Factors Based on Main Truss Connection Capacities

Truck Type	As-Designed/Present Condition			
	Inventory		Operating	
	Single Truck	Eq. Lane Loading	Single Truck	Eq. Lane Loading
Two Axle	1.62	1.34	2.71	2.25
Tri-Axle	1.26	1.05	2.11	1.77
Concrete Truck	1.42	1.20	2.39	2.01
3S2 (AL)	1.25	0.99	2.10	1.66
School Bus	3.77	3.16	6.32	5.30
HS20-44 Lane Loading	--	1.10	--	1.84

Iron Co. indicate that the diameter of rivets is 7/8 inch. The rivets were conservatively assumed to be ASTM A502 Grade 1 rivets. It was found that the splicing of the bottom chord member L3L4 at the middle of the member controlled the connection ratings.

The load capacities of the connections were found to be higher than the truss member capacities. A summary of As–Designed ratings determined for the connections is given in Table 6.12. The ratings for the single truck loadings and equivalent lane loadings were controlled by the shear capacity of the rivets. It was judged that the minor corrosion on the connection was insignificant, and the rating factors given in Table 6.12 were sufficiently accurate for use as Present–Condition ratings.

MAIN TRUSS FLOOR SYSTEM RATINGS

The floor system framing is symmetrical with respect to the centerline of the continuous truss. A typical section through the floor system at midspan is shown in Figure 6.6. The floor system is made up of a 6 3/4 inch thick concrete deck on four stringers. The stringer span length ranges from 22 feet 4 inches to 25 feet 11 inches. Two outside stringers made of 18 B 55 and two interior stringers made of 21 CB 58 were used on spans up 24 feet 10 inches. All four stringers were made of 21 B58 on the longer spans. All floorbeams were fabricated from 30 CB 115.

The bending moment and shearing forces in the stringers and floorbeams were evaluated by assuming the riveted connections at the member ends to act as simple supports. The compression flanges of the stringers were embedded in the underside of the concrete deck as shown in Figure 6.5. It was judged that the concrete deck provided sufficient lateral restraint so that the stringer compression flanges are continuously braced. The compression flanges of the floorbeams were considered to be laterally braced at the stringer to floorbeam connections.

Summaries of the rating factors calculated for the stringers and floorbeams are given in Tables 6.13 and 6.14. These rating factors were based on the flexural capacities of the members. It was found that rating factors based on the shear capacities of the members and the stringer to floorbeam and floorbeam to truss connections are on the order of two to five times higher than those presented in Tables 6.13 and 6.14. Hence, those rating factors are not presented in this report. No significant section losses were found along the stringers and floorbeams, so the rating factors given in Tables 6.13 and 6.14 serve as both As–Designed and Present–Condition ratings.

Table 6.13 Rating Factors for Main Truss Stringers

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.70	1.11
Tri-Axle	0.49	0.78
Concrete Truck	0.56	0.89
3S2 (AL)	0.77	1.23
School Bus	1.38	2.21
HS20-44 Lane Load	0.71	1.13

Table 6.14 Rating Factors for Main Truss Floorbeams

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.78	1.25
Tri-Axle	0.58	0.93
Concrete Truck	0.66	1.05
3S2 (AL)	0.90	1.43
School Bus	1.96	3.12
HS20-44 Lane Load	0.82	1.31

CONCRETE DECKS ON TRUSS SPANS

There are no structural drawings available that show the reinforcing details used in the reinforced concrete decks on truss spans. It was not possible to calculate quantitative rating factors for these members.

The field inspection results indicate that the concrete decks appear to be in fair condition. The top of the deck typically exhibited light to moderate scaling and some areas of hairline map cracking. Transverse cracks up to 1/16 inch wide were found throughout. Transverse cracks in span 8 (main truss) were typically up to 1/8 inch. One transverse crack was found to be up to 1/4 inch wide near panel point 2, span 11. One large spall in span 11 required immediate repair. The underside of the deck generally exhibited small surface spalls and minor cracking. The condition of the deck, however, does not appear to require a reduction of the load ratings of the bridge.

CONCLUSIONS AND RECOMMENDATIONS

Structural analyses and load ratings of the bridge indicate that the final bridge ratings are controlled by the flexural capacity of the exterior stringers of the main truss in panel L8L9 (stringer span of 24 feet 10 inches), the flexural capacity of the approach stringer spans, and the main truss member capacity. A summary of the Present–Condition ratings for the bridge is given in Table 6.15. The field inspections also identified the following maintenance and repairs that should be carried out:

1. The stringer to floorbeam connections which have broken rivets should be replaced with A325 bolts and hardened washers. Table 1 of A.G. Lichtenstein & Associates (Morgan/Madison 1991) report includes all locations with damaged or loose fasteners.

2. The large spall on the top of the deck in span 11, which was cold patched by Division 1 maintenance crew, should be repaired with a partial depth patch of high strength fast set concrete.

3. The cracks in the concrete deck which are 1/8 inch wide or wider should be sealed by epoxy injection or by brooming with MMA polymer.

4. The broken west railing in span 1 next to abutment 1 should be replaced.

5. The broken posts on the south side approach east railing in spans 1 and 3 should be replaced.

6. The newer railing on the south approach in spans 1,2, and 3 which was constructed without expansion provisions should be monitored for detrimental thermal effects.

Table 6.15 Summary of Present-Condition Ratings for Morgan/Madison County Structure No. 053-52-008.7B

Truck Type	Standard Weight (tons)	Inventory		Operating	
		Rating Factor	Rating (tons)	Rating Factor	Rating (tons)
Two Axle	29.5	0.57	17	1.00	29.5
Tri-Axle	37.5	0.42	16	0.77	29
Concrete Truck	33	0.47	16	0.84	28
3S2 (AL)	40	0.47	19	0.86	34
School Bus	12.5	1.36	>12.5	2.21	>12.5

7. The diaphragms adjacent to the joints on the south approach which have broken up are ineffective and should be replaced.

8. All damaged truss members, end portals, and sway frames should be repaired by heat straightening the lesser damaged members and heat straightening with the addition of reinforcement plates to the more severely damaged (torn) members.

9. The large number of spalls found on the outside face of curb/sidewalk should be monitored.

10. The heavy growth of vines on the south approach piers and underside of deck as well as adjacent to abutment 2 should be removed.

11. On the south approach spans, the roadway joints varied in width and appeared closed. The joint opening at various temperatures should be monitored to determine if the joints are working properly.

12. The clogged drains on the deck should be unclogged, and the five missing pipe extensions should be replaced.

13. New joint seals should be poured at all joints on the approach stringer spans to prevent water and debris from falling on surfaces below the joint. On the truss spans it is suggested that the installation of waterproof deck joints be considered.

14. Rehabilitation of the bearings on the north and south approach spans should be considered. The loose retaining nut on the east side of pier 11 should be tightened. The rocker bearing on the west side of per 22 should be placed into proper position and receive new anchor bolts.

15. Cracks found in the railing support verticals directly above the bearing pins should be monitored.

16. The stringer diaphragm in span 7 between FB10 and FB11 should be connected to stringer 1 with high strength bolts.

17. All welding performed on tension elements such as the bottom flanges of stringers and truss tension members should receive smooth grinding and testing by ultrasonic or other nondestructive methods.

18. Large spalls on the substructure should be repaired.

19. Efforts should be made to control the speed of traffic crossing the structure and consideration should be given to posting a lower speed limit on the bridge to alleviate the poor sight distance in span 8.

CHAPTER SEVEN
MOBILE COUNTY
STRUCTURE NO. 013–49–003.2

Mobile County Structure No. 013–49–003.2 was constructed in 1933 and currently carries U.S. Route 43 traffic over the Three Mile River. The original bridge consisted of one deck–girder swing span and one reinforced concrete T–beam approach span. The manually operated swing span was apparently converted to a fixed bridge as it does not appear to have been opened for several years. An elevation of the bridge illustrating the arrangement of the spans and span numbering is shown in Figure 7.1. A summary of the structural analyses and load rating results are presented in this chapter. Results from the field inspection and condition survey are summarized by A.G. Lichtenstein & Associates (Mobile County 1991).

REINFORCED CONCRETE T–BEAM APPROACH SPAN RATINGS

There is only one reinforced concrete T–beam approach span at the north end of the swing span as shown in Figure 7.1. The 38 feet 3 inches long approach span is made up of a 7 1/4 inches thick roadway deck supported by five reinforced concrete T–beams as shown in Figure 7.2. The curb to curb roadway width is 30 feet. The structure is rated as a two–lane bridge. Based on the time of construction it is assumed according to AASHTO (Manual 1983) that the concrete strength is 3300 psi, and the yield strength of the reinforcing steel is 33 ksi. During the field inspection, A.G. Lichtenstein & Associates found drawings of the plate girder swing span and details for the reinforced concrete T–beam and the roadway deck. All dimensions shown were checked by the field inspection crews. Dimensions which vary in the field are noted on the plans. The availability of reinforcement details of the concrete deck made it possible to rate the slab also.

A summary of the T–beam approach span ratings is given in Table 7.1. Ratings for the concrete deck are summarized in Table 7.2. The field inspection results indicated that the concrete deck and beams were in generally good condition. The only exception was a fairly large spall (10 inches wide and 1 inch deep) on the lower end of the center T–beam (beam #3) web at the north abutment. It is judged that the present condition of the deck and beams do not warrant any reduction in the ratings listed in Tables 7.1 and 7.2.

RATING FACTORS FOR SWING SPAN PLATE GIRDERS

The two main girders of the swing span are a built–up tapered plate girders. The top and bottom flanges consisted of two angles that were riveted to the top and bottom of the web plate. Cover plates were added to the

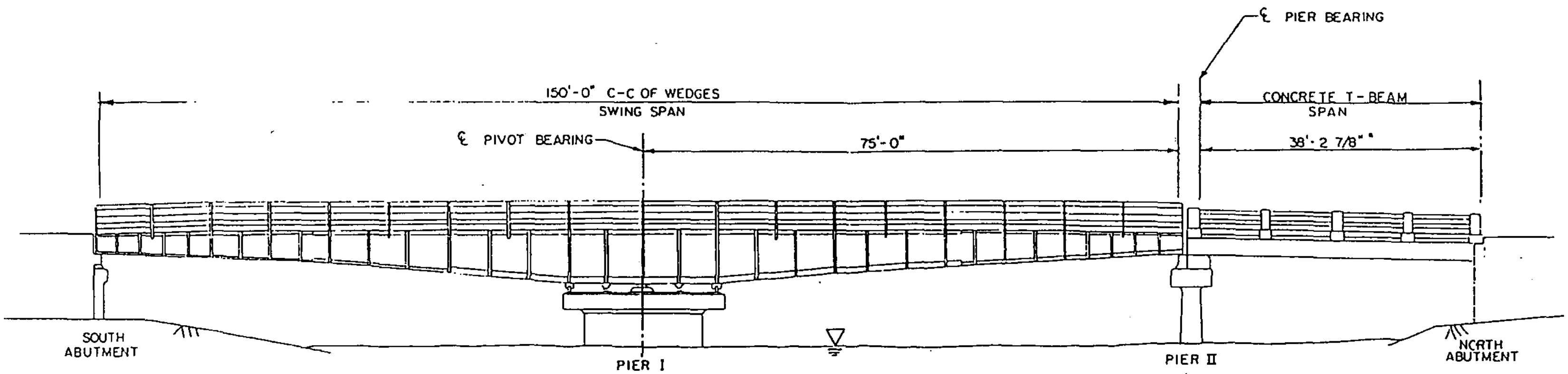


Figure 7.1. East Elevation of Mobile Bridge (013-49-003.2)

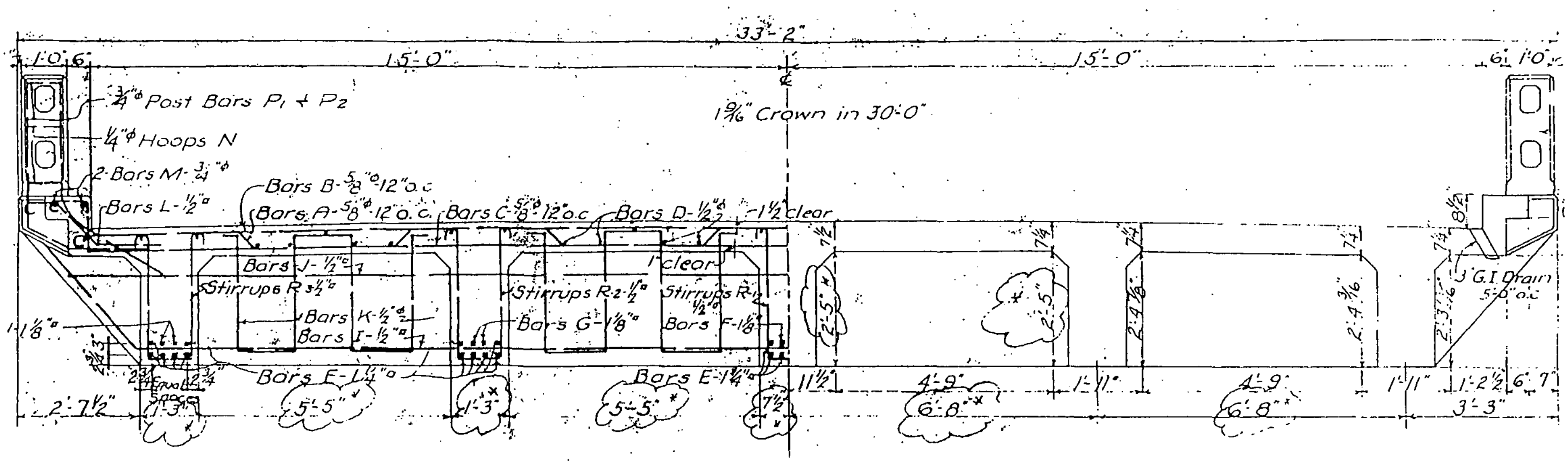


Figure 7.2. Cross Section at Concrete Tee-Beam Span

Table 7.1 Rating Factors for Approach Concrete T-Beam Span

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.93	1.55
Tri-Axle	0.67	1.12
Concrete Truck	0.76	1.27
3S3 (AL)	0.98	1.64
3S2 (AL)	1.19	1.99
School Bus	2.29	3.82

Table 7.2 Rating Factors for Concrete Deck

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	1.59	2.64
Tri-Axle	1.41	2.35
Concrete Truck	1.27	2.12
3S3 (AL)	1.81	3.02
3S2 (AL)	2.14	3.57
School Bus	2.37	3.95

outstanding legs to increase flange areas. The depth of the plate girder varies from 2 feet 6 inches at the abutments to 8 feet 8 1/2 inches at the center pier. Section properties of the tapered plate girders along the length are listed in Table 7.3. The yield strength of steel was assumed to be 30 ksi as per AASHTO (Manual 1983).

Structural analyses of the swing span were performed using the in–house computer program BRIDGE to determine the dead and live load stresses at the various locations along the girder. Two dead load analyses were performed. First, the swing span was modeled as a two–span continuous beam to simulate the condition of open to traffic closed to navigation. Secondly, dead load stresses were calculated for a cantilever configuration with a span of 75 feet to simulate the open position. The analysis for live load stresses was performed using standard truck axle loadings for the bridge in the closed position. It was found that the combined dead load and live load stresses determined for the closed position controlled the ratings.

Summaries of the minimum inventory and operating rating factors calculated for the plate girders are given in Table 7.4. These rating factors are based on the flexural capacity of the girder. The compressive flexural stress on the top cover plate at a section measured 20 feet 2 inches from the end controlled the ratings. Checks of the bearing stiffeners and intermediate transverse stiffeners were made, and the stiffeners were found to be adequate for the girder ratings shown in Table 7.4. Ratings were also calculated based on the splices in cover plates and web plates. These rating factors were found to be much higher than those shown in Table 7.4. The condition report by A.G. Lichtenstein & Associates (Mobile County 1991) indicates that the plate girders are generally in fair condition with light rust and peeling paint. It is judged that the rating factors given in Table 7.4 are sufficiently accurate for use as Present–Condition ratings.

SWING SPAN FLOOR SYSTEM RATINGS

A typical section through the floor system of the swing span is shown in Figure 7.3. There are six different floorbeam configurations that follow the varying depth of the tapered main girders. There are a total of eleven floorbeams which are numbered from FB0 to FB10 in the framing plan prepared by A.G. Lichtenstein & Associates. As the bridge is symmetrical about the centerline, only six different configurations were examined in the rating calculations. All of the floorbeams except FB3 and FB7 are fabricated plate girders with double angle flanges. An elevation showing the geometry, support conditions and joint numbering for the trussed floorbeams FB3 and FB7 is given in Figure 7.4.

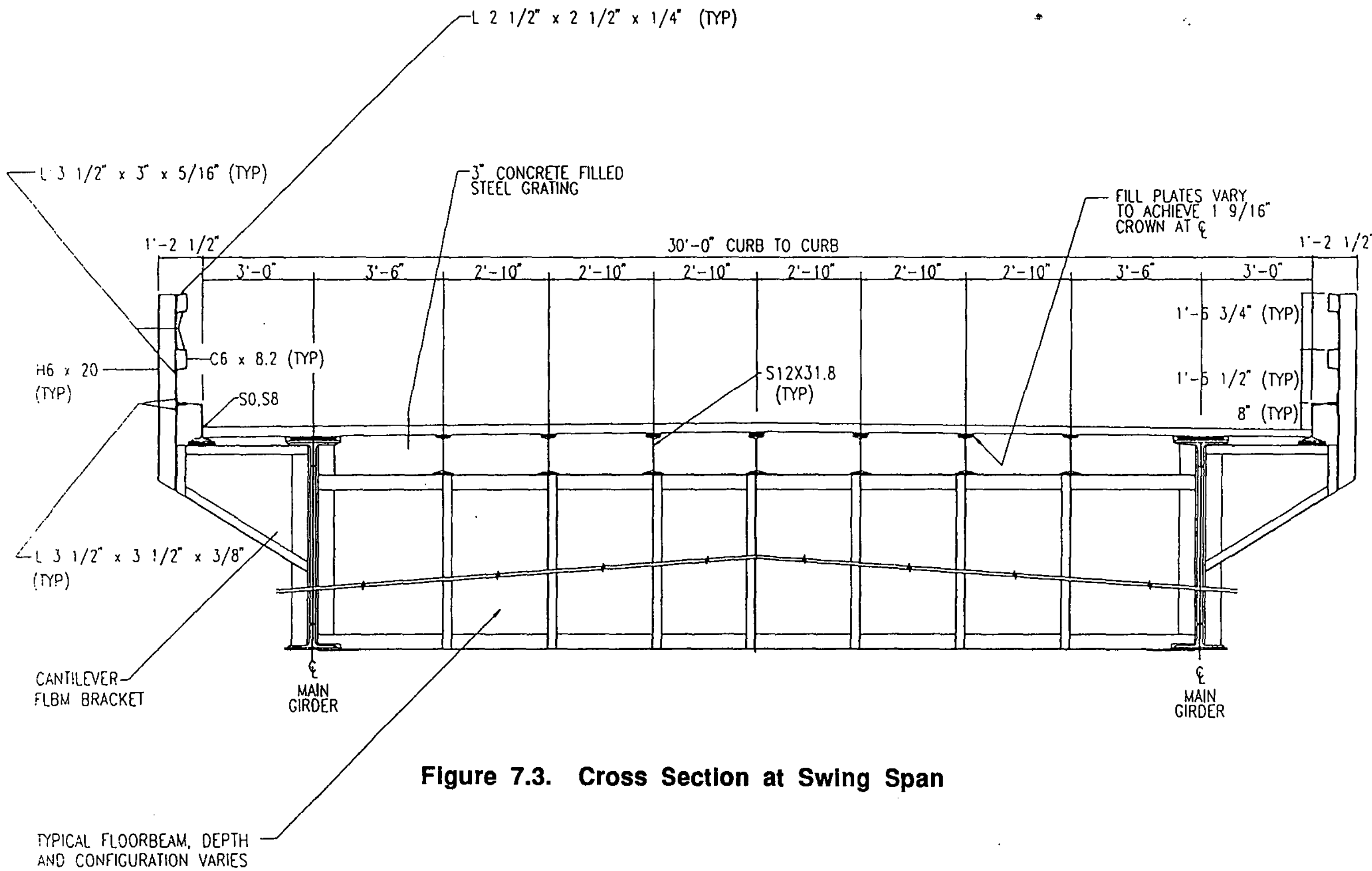


Figure 7.3. Cross Section at Swing Span

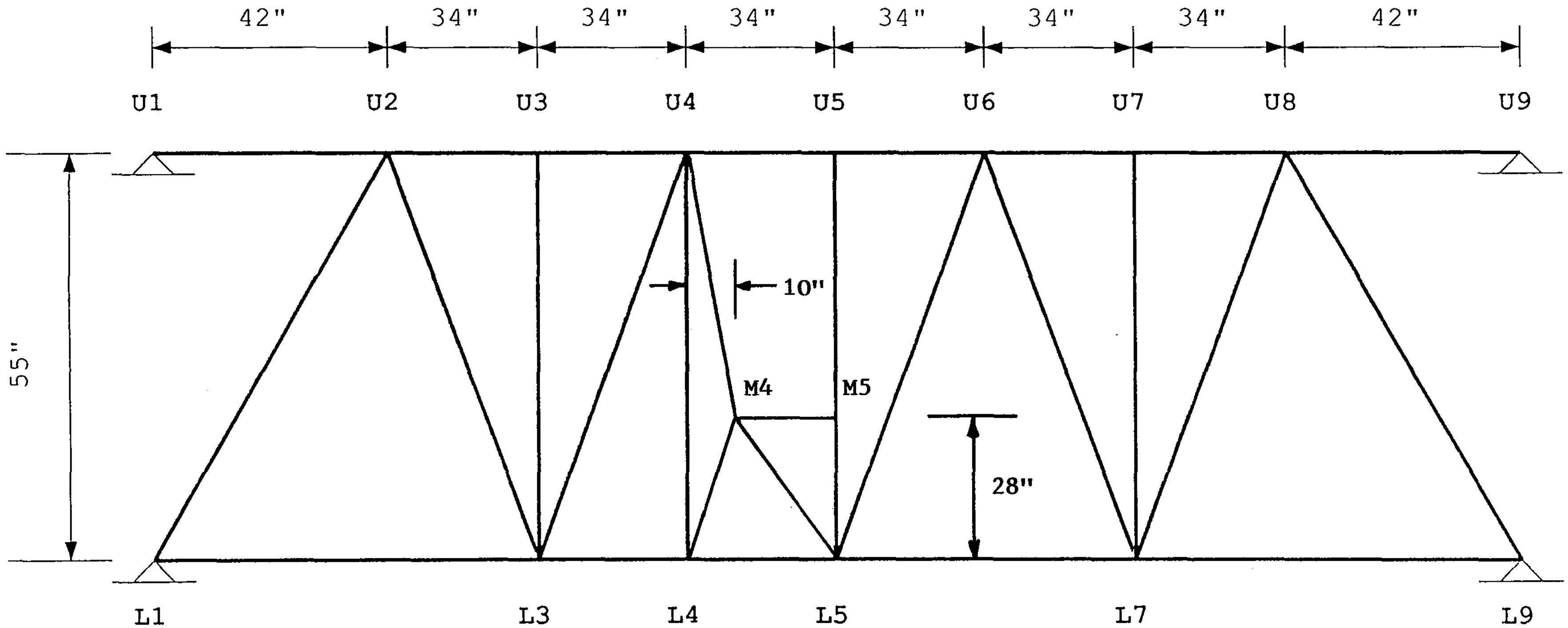


Figure 7.4. Elevation of Trussed Floorbeam

Table 7.3 Section Properties of Main Girder

Distance from End (inch)	Depth (inch)	Web Thickness (inch)	Cover Plates W (inch)	Cover Plates T (inch)	Area (inch2)	Ix (inch4)
0	30.0	0.3125	14	0.5	46.38	8,120
38	32.5	0.3125	14	0.5	47.16	9,668
77	34.5	0.3125	14	0.5	47.78	11,010
116	36.25	0.3125	14	0.5	48.33	12,270
155	38.25	0.3125	14	0.5	48.95	13,790
195	40.0	0.3125	14	0.5	49.50	15,210
242	43.00	0.375	18	0.75	66.13	24,520
291	46.0	0.375	18	0.75	67.25	28,330
340.5	48.75	0.375	18	0.75	91.08	42,800
390	52.0	0.375	18	0.75	92.30	49,220
454	56.0	0.4375	18	0.75	97.30	58,720
519	60.25	0.4375	18	0.75	99.16	68,880
584	64.75	0.4375	18	0.75	101.13	80,620
649	70.00	0.5	18	0.75	107.80	97,390
714	74.88	0.5	18	0.75	124.44	130,230
780	80.25	0.5	18	1.00	136.13	166,830
840	80.5	0.5	18	1.00	136.25	167,970
900	80.5	0.5	18	1.00	136.25	167,970

Table 7.4 Rating Factors for Main Girders

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.77	1.25
Tri-Axle	0.59	0.95
Concrete Truck	0.66	1.07
3S3 (AL)	0.69	1.12
3S2 (AL)	0.73	1.19
School Bus	1.79	2.91

Structural analyses were performed on the trussed floorbeam using the in–house computer program BRIDGE. Input data was taken from the field notes and the design drawings. A summary of the member cross section properties for the trussed floorbeams is given in Table 7.5. Summaries of the minimum inventory and operating rating factors calculated for the trussed floorbeam are given in Table 7.6. There was a great deal of minor to moderate corrosion damage to the truss members particularly caused by drainage problems. None of the truss members has a sufficient loss of section to affect the truss ratings. Compression member, L1U2, controlled the ratings. Compression members L7U6 and L9U8 also had operating rating factors less than one. Ratings were also calculated for the trussed floorbeam connections assuming Grade 1 rivets. Ratings based on the connections were found to be higher than those shown in Table 7.6.

The plate girder floorbeams (FB0,FB1,FB2,FB4,FB5) were reported to be in good condition and there was not any reason to reduce their capacities. Due to the even floorbeam spacing (16 feet 3 inches), each floorbeam (FB0 through FB3) is expected to carry approximately the same amount of induced live loads. The floorbeam numbered FB2 had the smallest section modulus and therefore controlled the ratings. Table 7.7 summarizes the rating factors for the plate girder floorbeams. Ratings were calculated for the floorbeam connections capacities and were found to be much higher than those given in Table 7.7.

The cantilever floorbeam brackets were perhaps the most severely damaged members of the bridge. The severity of the corrosion damage is documented in the A.G. Lichtenstein & Associates field notes (pp. 36 and 37) and AGLAS Condition Report (Mobile County 1991). There was as much as 1/8 inch loss of web on both sides of web and also a complete loss of the top flange at some locations Despite the severity of the corrosion damage, rating factors calculated based on the moment, shear, and connection capacities of the cantilever floorbeam brackets were found to be as high as those given in Table 7.7. One common weakness of the cantilever floorbeam brackets was the riveted connection of the cantilever to the main plate girder. However, the primary exception was the cantilever floorbeam bracket at the south end on the east side of the bridge (see Photo 8 in Condition Report, Mobile County 1991). The impact damage to that member is so severe that it does not have a calculable load capacity. The member must be replaced before the portion of the bridge supported by the cantilever floorbeam bracket is reopened to traffic.

There are seven (interior) stringers (S12x31.8) and two fascia stringers in the swing span floor system as shown in Figure 7.2. Cross sectional properties were taken from AISC (Iron and Steel Beams 1985) for the interior stringers. Fascia stringer properties were calculated based on the field inspection data. Both interior and fascia stringers were modeled as continuous beams which span the full length of the bridge as per field notes

Table 7.5 Member Cross Section Properties, Trussed Floorbeams

Member	Area ($inch^2$)	r_x (inch)	r_y (inch)
U1U2	5.72	1.61	1.27
U2U3	5.72	1.61	1.27
U3U4	5.72	1.61	1.27
U4U5	5.72	1.61	1.27
U5U6	5.72	1.61	1.27
U6U7	5.72	1.61	1.27
U7U8	5.72	1.61	1.27
U8U9	5.72	1.61	1.27
L1U2	4.97	1.26	1.36
L3U2	4.97	1.26	1.36
L3U3	3.24	0.94	1.29
L3U4	3.24	0.94	1.29
L4U4	3.24	0.94	1.29
L4M4	3.24	0.94	1.29
M4U4	3.24	0.94	1.29
M4M5	7.08	–	–
L5M4	3.24	0.94	1.29
L5M5	3.24	0.94	1.29
M5U5	3.24	0.94	1.29
L5U6	3.24	0.94	1.29
L7U6	3.24	0.94	1.29
L7U7	3.24	0.94	1.29
L7U8	4.97	1.26	1.36
L9U8	4.97	1.26	1.36
L1L3	5.72	1.61	1.27
L3L4	5.72	1.61	1.27
L4L5	5.72	1.61	1.27
L5L7	5.72	1.61	1.27
L7L9	5.72	1.61	1.27

Table 7.6 Rating Factors for Trussed Floorbeams

Truck Type	As-Designed/Present-Condition	
	Inventory	Operating
Two Axle	0.60	0.83
Tri-Axle	0.42	0.59
Concrete Truck	0.47	0.66
3S3 (AL)	0.60	0.84
3S2 (AL)	0.64	0.90
School Bus	1.28	1.79

Table 7.7 Rating Factors for Plate Girder Floorbeams

Truck Type	As-Designed/Present-Condition	
	Inventory	Operating
Two Axle	0.71	1.12
Tri-Axle	0.50	0.79
Concrete Truck	0.56	0.89
3S3 (AL)	0.70	1.10
3S2 (AL)	0.76	1.21
School Bus	1.52	2.40

(pp. 7/47). Structural analyses of the stringers were performed using the in–house computer program BRIDGE to determine the dead and live load stresses. Because of the short span lengths of the stringers (16 feet 3 inches), truck wheel loads controlled the ratings.

Summaries of the minimum inventory and operating rating factors calculated for the stringers are given in Tables 7.8 and 7.9. These ratings are based on the flexural capacities of the stringers. There was a great deal of corrosion damage, particularly on the bottom flanges of the fascia stringers as shown in the field notes (pp. 35,36,37, and 38/47). Present–Condition ratings for the stringers were calculated by using cross sectional properties that accounted for the section losses due to corrosion. The ratings were reduced by the section losses, but not by as much as originally expected.

STEEL GRATINGS

The deck of the swing span was made of 3 inch deep concrete filled steel gratings fabricated from structural tees. Sufficient details of the gratings that are necessary to calculate the load carrying capacity are not available. It was not possible to calculate quantitative rating factors for the steel gratings.

The field inspection results indicate that the concrete filled steel gratings appear to be in fair condition. It was judged that the condition of the steel gratings did not warrant a reduction of the load ratings of the bridge.

CONCLUSIONS AND RECOMMENDATIONS

Structural analyses and load ratings of the bridge indicate that the final bridge ratings are controlled by the trussed floorbeams (FB3,FB7). However, the rating factors for the other elements (plate girder floorbeams, interior stringer) of the bridge are not substantially greater than those for the trussed floorbeams. A summary of the Present–Condition ratings for the bridge is given in Table 7.10. The field inspection also identified the following maintenance and repair that should be carried out:

1. The spall at the underside of beam #3 of the concrete T–beam span should be repaired.

2. The clogged deck drains on the concrete T–beam span should be unclogged. Installation of deck drains (w/pipe extensions) and curbs along the swing span should be considered to prevent drainage from the deck onto the fascia stringers.

3. Joint seals should be placed at all open joints on the swing span to prevent water and debris from falling on the superstructure members below.

Table 7.8 Rating Factors for Interior Stringers

Truck Type	As-Designed Inventory	As-Designed Operating	Present-Condition Inventory	Present-Condition Operating
Two Axle	0.80	1.15	0.64	0.94
Tri-Axle	0.61	0.87	0.49	0.71
Concrete Truck	0.64	0.92	0.52	0.75
3S3 (AL)	0.85	1.21	0.68	0.99
3S2 (AL)	0.89	1.28	0.72	1.04
School Bus	1.41	2.02	1.13	1.64

Table 7.9 Rating Factors for Fascia Stringers

Truck Type	As-Designed Inventory	As-Designed Operating	Present-Condition Inventory	Present-Condition Operating
Two Axle	1.35	1.90	1.19	1.72
Tri-Axle	1.02	1.44	0.90	1.30
Concrete Truck	1.08	1.52	0.95	1.37
3S3 (AL)	1.42	2.01	1.26	1.81
3S2 (AL)	1.50	2.12	1.33	1.91
School Bus	2.36	3.34	2.09	3.01

Table 7.10 Summary of Present-Condition Ratings for Mobile County Structure No. 013-49-003.2

Truck Type	Standard Weight (tons)	Inventory Rating Factor	Inventory Rating (tons)	Operating Rating Factor	Operating Rating (tons)
Two Axle	29.5	0.60	18	0.83	24
Tri-Axle	37.5	0.42	16	0.59	22
Concrete Truck	33	0.47	16	0.66	22
3S3 (AL)	42	0.60	25	0.84	35
3S2 (AL)	40	0.64	26	0.90	36
School Bus	12.5	1.28	>12.5	1.79	>12.5

4. The access hatch in the swing span deck should be sealed with a pourable or gun grade elastomeric joint sealer to prevent water from draining on the superstructure members below.

5. Approach guard rails should be installed at the ends of the structure to protect against impact of the ends of the bridge railing.

6. It is recommended that the portion of the roadway supported by the cantilevered floorbeam brackets on the east side of the bridge not be reopened to traffic until after the floorbeam bracket at the southeast corner is replaced.

CHAPTER EIGHT
MONTGOMERY COUNTY
STRUCTURE NO. 009–51–040.7B

Montgomery County Structure No. 009–51–040.7B was constructed in 1920 and currently carries U.S. Route 231 traffic over the Tallapoosa River from Montgomery to Wetumpka. An elevation of the bridge illustrating the arrangement of spans and span numbering is shown in Figure 8.1. The original bridge consisted of 13 reinforced concrete stringer spans (6 on the west approach and 7 on the east approach) and four truss spans. The two center trusses (spans 9 and 10) are camel back Pratt type and the two outer trusses were standard Pratt trusses. The two outer Pratt trusses were damaged by traffic impact (in separate incidents) and replaced in 1970 (west) and 1977 (east) with two steel stringer spans in each location (one steel bent was erected in the middle of the original truss span). During the inspection, A.G. Lichtenstein & Associates obtained a general elevation, original design plans for the trusses and concrete piers, design plans for the concrete stringer spans, and design plans for the steel stringer spans 7,8,11, and 12. The field inspection crews checked these plans against the existing structure and generally found them to be representative of the current geometry and configuration. A summary of the structural analyses and load rating results are presented in this chapter. Results from the field inspection and condition survey are summarized by A.G. Lichtenstein & Associates (Montgomery County 1991).

REINFORCED CONCRETE STRINGER SPAN RATINGS

The 34 feet 8 inches long approach spans (1 through 6 and 13 through 19) are made up of a 7 inch thick roadway deck supported by four reinforced concrete T–beams as shown in Figure 8.2. The dimensions shown were checked by the field inspection crews. Dimensions which varied in the field are noted. The curb to curb roadway width is 17 feet 6 inches. The structure is rated as a two–lane bridge. Based on the time of construction it is assumed according to AASHTO (Manual 1983) that the concrete strength is 3300 psi, and the yield strength of the reinforcing steel is 33 ksi.

A summary of the T–beam approach span ratings is given in Table 8.1. The availability of reinforcement details of the concrete deck made it possible to rate the slab also. It was found that the flexural capacity of the interior stringers controlled the ratings. Rating factors for the concrete deck were found to be significantly higher than those given in Table 8.1, and therefore are not included in this report. The field inspection results indicated that the concrete deck and beams were in generally fair condition. Localized poor conditions were noted for the concrete deck throughout the spans including transverse and map cracking near joints, ruts, and several patch areas.

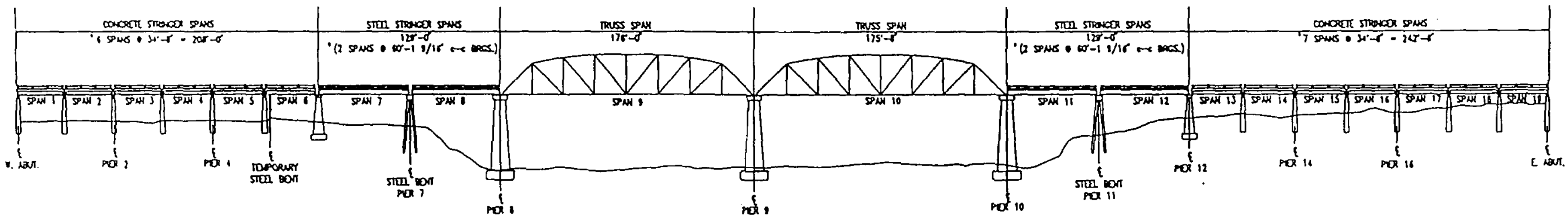


Figure 8.1. South Elevation of Montgomery Bridge (009-51-040.7B)

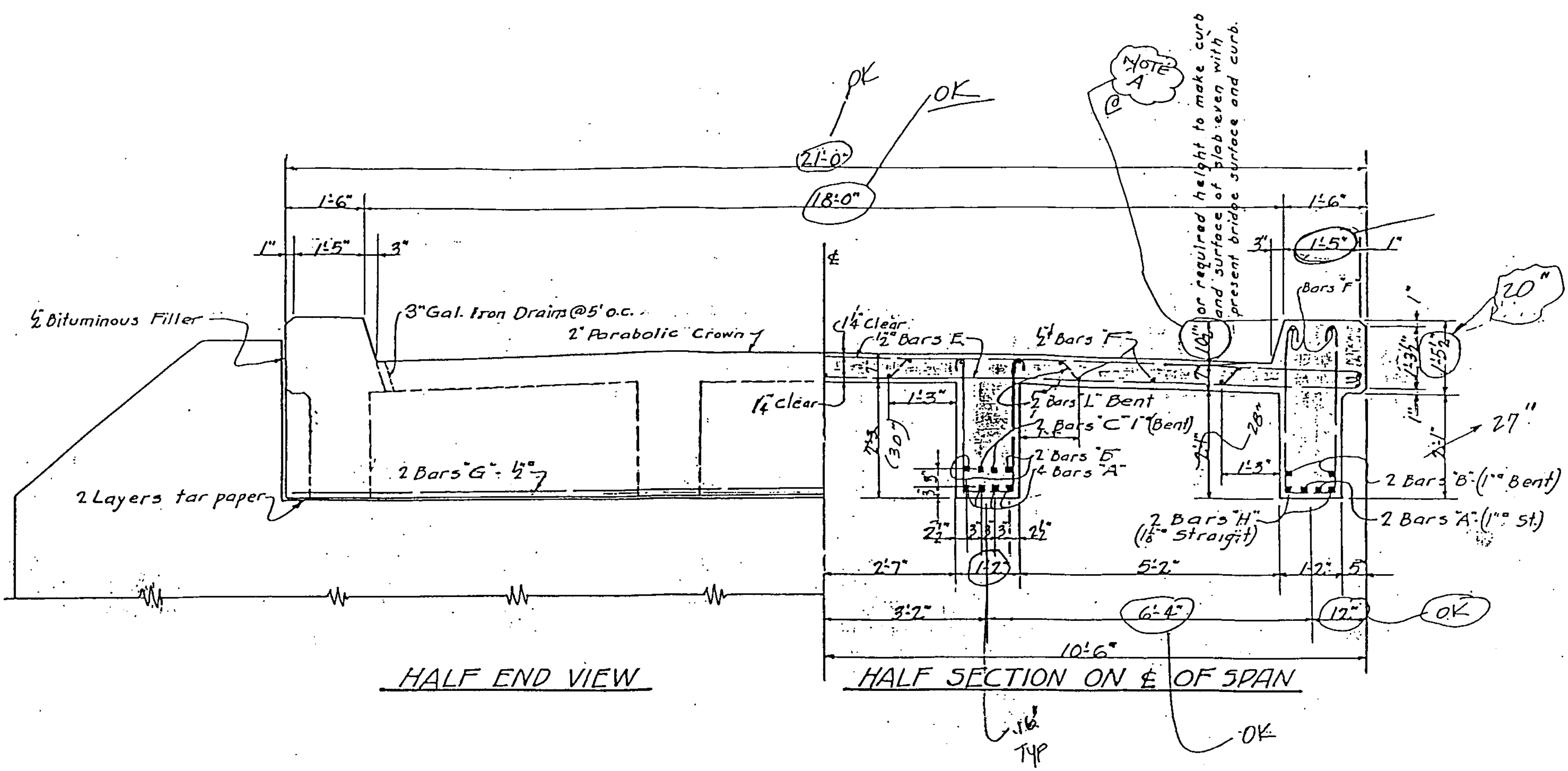


Figure 8.2. Cross Section at Concrete Stringer Spans

Table 8.1 Rating Factors for Approach Concrete T-Beam Spans Based on Flexural Capacity

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.85	1.42
Tri-Axle	0.61	1.02
Concrete Truck	0.69	1.15
3S2 (AL)	0.95	1.58
School Bus	2.15	3.58

Random hairline cracks and minor spalling with a few exposed rebar were noted on the concrete beam webs throughout the spans. However, it is judged that the present condition of the deck and beams do not warrant any reduction in the ratings listed in Table 8.1. It should be noted that the rating factors in Table 8.1 do not account for the generally poor conditions found at the bearings of the T–beams. As described by A.G. Lichtenstein & Associates there was a significant reduction in bearing area under the beam ends due to spalling of the pier cap at several locations. While it is not possible to establish a quantitative rating for the bearings, the conditions warrant corrective action.

STEEL STRINGER SPAN RATINGS

There is a total of four identical steel stringer spans (spans 7,8,11, and 12). The 60 feet 1 9/16 inches (c–c bearings) long steel stringer spans are made up of a 6 1/2 inches thick reinforced concrete roadway deck supported by three wide flange stringers (W36x170). The material is conservatively assumed to be A36 steel. These stringers are laterally connected by channel (C18x42.7) diaphragms. A typical cross section through the steel stringer spans is shown in Figure 8.3. As can be seen from Figure 8.3, there are mechanical shear connectors installed between the top flange of the steel stringer and the concrete deck. Therefore, composite section properties were used in the rating calculations using a modular ratio of nine.

Summaries of the minimum inventory and operating rating factors evaluated for the composite stringers are given in Table 8.2. These rating factors are based on the flexural capacity of the stringers. Ratings were also calculated for the concrete deck. The resulting rating factors were found to be much higher than those given in Table 8.2. Hence, they are not presented in this report. The condition report (Montgomery County 1991) indicates that the steel stringer spans were generally in good condition with no significant defects noted. It is judged that the rating factors given in Table 8.2 are sufficiently accurate for use as Present–Condition ratings.

TRUSS RATINGS

The two center trusses (span 9 and span 10) are camel back Pratt type trusses. The span lengths of span 9 and span 10 are 176 feet and 175 feet 8 inches, respectively. The two trusses are identical except that each of the end panel lengths of span 10 are 2 inches shorter than those of span 9. An elevation of span 9 showing the overall geometry, support conditions and joint numbering for the truss is shown in Figure 8.4. Cross section properties for the truss members are given in Table 8.3.

Structural analyses of the truss were performed using the in–house computer program BRIDGE to determine the dead load and live load stresses

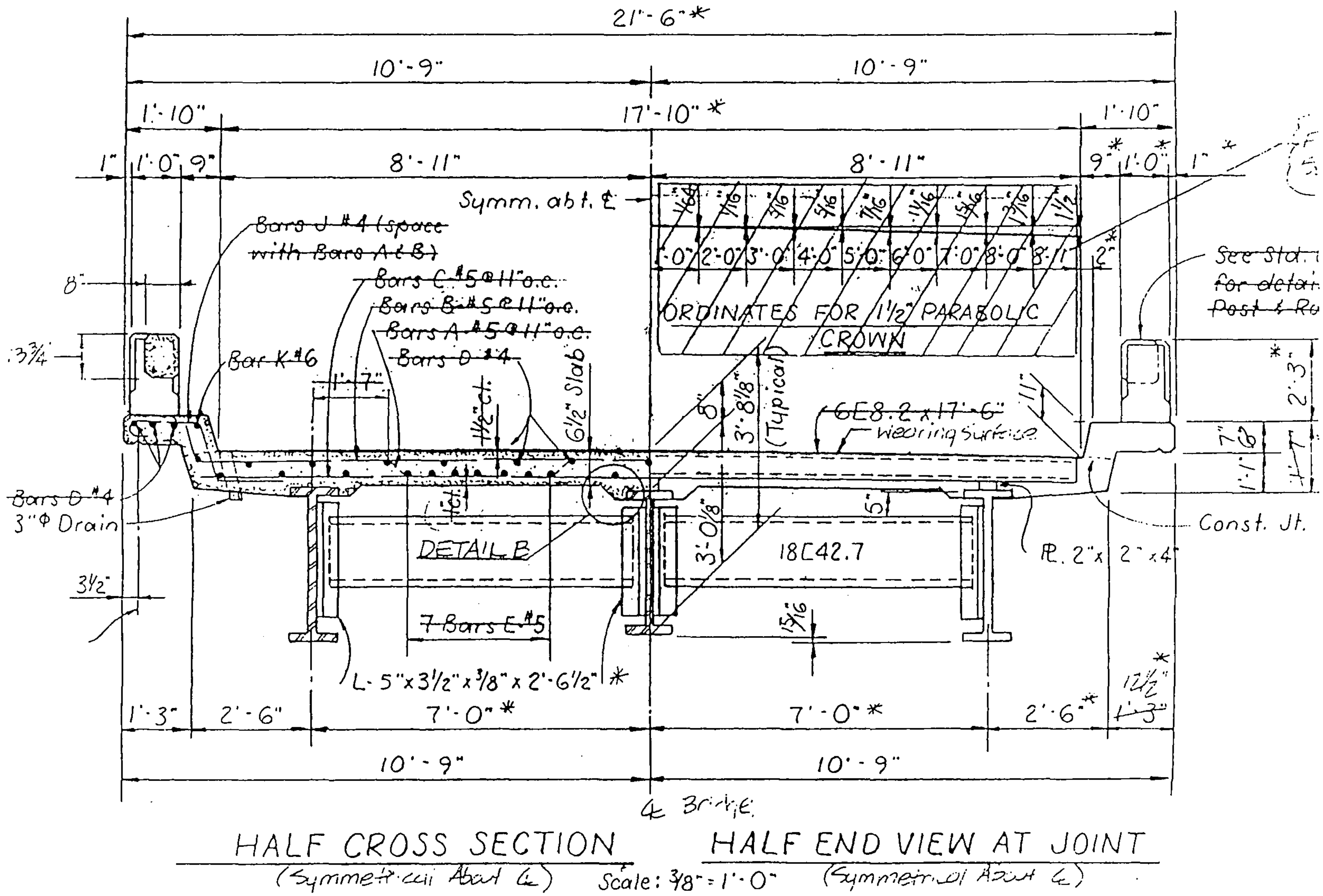


Figure 8.3. Cross Section at Steel Stringer Spans

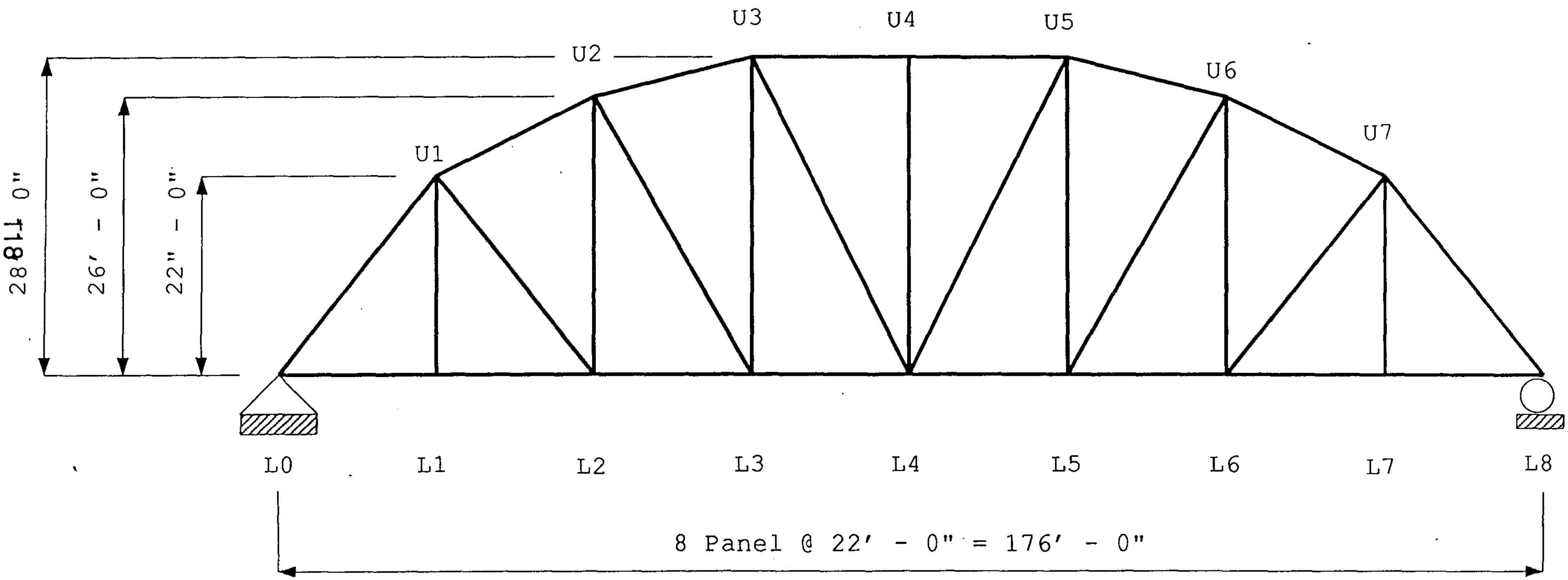


Figure 8.4. Elevation of Center Trusses (Span 9 Shown)

Table 8.2 Rating Factors for Composite Stringers

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	1.75	3.04
Tri-Axle	1.32	2.28
Concrete Truck	1.49	2.58
3S2 (AL)	1.79	3.10
School Bus	3.24	5.40

Table 8.3 Member Cross Section Properties

Member	Area (inch^2)	I_x (inch^4)	I_y (inch^4)
L0U1,U1U2 U2U3,U3U4	25.43	882.0	884.6
L0L1,L1L2	12.20	73.4	209.5
L2L3	19.38	94.6	245.2
L3L4	19.94	96.4	249.0
L1U1	7.72	21.2	134.0
L2U2	7.78	94.6	149.3
L3U3,L4U4	6.72	64.6	126.6
L2U1	8.78	101.4	317.3
L3U2,L4U3	6.72	64.6	240.1

in each of the members. A summary of the dead load and maximum live load stresses is given in Table 8.4. Because of the long span length of the truss, top and bottom chords develop the maximum stress under an equivalent lane load while the tri-axle dump truck produces the maximum stresses in most verticals and diagonals whose influence lines are confined within the adjacent panels.

Summaries of the minimum inventory and operating rating factors calculated for the trusses are given in Table 8.5. These rating factors are based on the axial stresses created in the truss members by the standard truck loadings. As-Designed ratings are given for loadings by single trucks and the equivalent lane loadings defined by Equation 2.1. It should be noted that the ratings for equivalent lane loadings are shown for general information. In rating calculations, lane loads are typically used to account for the possibility of multiple trucks in a single lane. Section 5.2.2 of AASHTO (Manual 1983) does not require the possibility of multiple trucks in a single lane to be considered except for spans longer than 200 feet.

Although a great deal of minor to moderate corrosion and impact damage including web distortion and bowing and loss of net area of vertical members was identified during the field inspection, none of the truss members had a sufficient loss of section to affect the final truss ratings. It is somewhat interesting to note that member L3L4 controlled the inventory ratings while member U3U4 controlled the operating ratings. This is due to the way AASHTO (Manual 1983) specifies the allowable stresses for the two different ratings.

Capacities of the truss connections were also evaluated. After reviewing the field notes and a series of photographs provided by A.G. Lichtenstein & Associates, connections at both ends of L1U1, L2U1, L3U2, L4U3, L1L2, and L2L3 were selected as the most critical ones. The truss analyses also confirmed that these were indeed highly stressed connections. The field notes indicated (pp. 29,31) that the head diameter of the rivets was 1 1/4 inches. A shank diameter of 3/4 inch was taken from AISC (1927) for use in the rating calculations.

A summary of As-Designed ratings determined for the connections is given in Table 8.6. As can be seen from Table 8.6, the rating factors for truss member connections are substantially lower than those for the truss member capacities although Grade 2 rivets were assumed in the rating calculations. The connection of member L2L3 at the joint L3 controlled for all cases examined. It was judged that the minor corrosion on the connections was insignificant, and the rating factors given in Table 8.6 are sufficiently accurate for use as Present-Condition ratings.

Table 8.4 Dead Load Stresses and Maximum Live Load Stresses

Member	Dead Load (ksi)	Maximum Live Load (ksi)
L10U1	-6.94	-4.21
L0L1	10.22	6.20
L1U1	4.00	8.72
U1U2	-7.25	-4.40
L2U1	9.11	6.93
L1L2	10.22	6.20
L2U2	-3.17	-4.57
U2U3	-8.32	-5.03
L3U2	6.77	7.87
L2L3	9.35	5.67
L3U3	-0.38	5.36
U3U4	-8.84	-5.35
L4U3	3.40	7.61
L3L4	10.56	6.39
L4U4	-0.47	0.00

Table 8.5 Rating Factors for Truss Members

	As-Designed/Present Condition			
	Inventory		Operating	
Truck Type	Single Truck	Eq. Lane Loading	Single Truck	Eq. Lane Loading
Two Axle	1.14	1.04	2.10	1.89
Tri-Axle	0.89	0.82	1.63	1.49
Concrete Truck	1.01	0.93	1.85	1.69
3S2 (AL)	0.93	0.77	1.69	1.40
School Bus	2.71	2.45	5.02	4.46
HS20-44 Lane Loading	–	0.85	–	1.55

Table 8.6 Rating Factors for Truss Connection Capacities for Grade 2 Rivets

	As-Designed/Present Condition			
	Inventory		Operating	
Truck Type	Single Truck	Eq. Lane Loading	Single Truck	Eq. Lane Loading
Two Axle	0.37	0.34	1.28	1.17
Tri-Axle	0.29	0.27	1.00	0.92
Concrete Truck	0.33	0.31	1.13	1.04
3S2 (AL)	0.30	0.25	1.03	0.86
School Bus	0.88	0.81	3.01	2.75
HS20-44 Lane Loading	–	0.28	–	0.96

TRUSS FLOOR SYSTEM RATINGS

The floor system framing is symmetrical with respect to the centerline of the truss. A typical section through the floor system at midspan is shown in Figure 8.5. The floor system is made up of a 7 1/2 inch thick concrete deck with 2.5 inches of asphalt wearing surface on six stringers. The stringer span is 22 feet except at the end panels of span 10. B15x41 shapes were used for all stringers (S1 through S6) and B28x104 shapes were used for all floorbeams (FB0 through FB4). Cross sectional properties for these shapes were taken from AISC (1985).

The bending moment and shearing forces in the stringers and floorbeams were evaluated by assuming the riveted connections at the member ends to act as simple supports. The compression flanges of the stringers were embedded in the underside of the concrete deck as shown in Figure 8.5. It was judged that the concrete deck provided sufficient lateral restraint so that the stringer compression flanges are continuously braced. The compression flanges of the floorbeams were considered to be laterally braced at the stringer to floorbeam connections.

Summaries of the rating factors calculated for the stringers and floorbeams are given in Tables 8.7 and 8.8. These rating factors were based on the flexural capacities of the members. It was found that rating factors based on the shear capacities of the members and the stringer to floorbeam and floorbeam to truss connections (including the combined shear and tension analysis) are on the order of two to five times higher than those presented in Tables 8.7 and 8.8. Hence, those rating factors are not presented in this report. As can be seen from Table 8.7, the Present–Condition rating factors for truss span stringers are substantially lower than those of As–Designed rating factors. This is the reflection of severe corrosion losses of the fascia stringers. Significant section losses were found near midspan of the fascia stringers. The corrosion losses were created by drainage through the deck drains along each side of the bridge. One area with heavy losses is at the top flange of fascia stringer 6 between FB1 and FB2 in span 9. That stringer has lost nearly half of its effective area as reported in the field notes (pp. 64) and inspection photographs C13 through C15. Greater cross section losses were reported in the field notes (pp. 70) for the fascia stringer 1 between FB5 and FB6 in span 10. The corrosion damage resulted in section loss at the exterior top of the bottom flange and a portion of lower web in addition to the top flange area.

The heavy section losses at the fascia stringers resulted in very low ratings as illustrated in Table 8.7. An alternate rating is available. By limiting traffic to one lane along the center of the bridge, direct loading of the fascia stringers would be avoided. It would be possible to raise the ratings to the values applicable to the interior stringers. No significant damage or corrosion

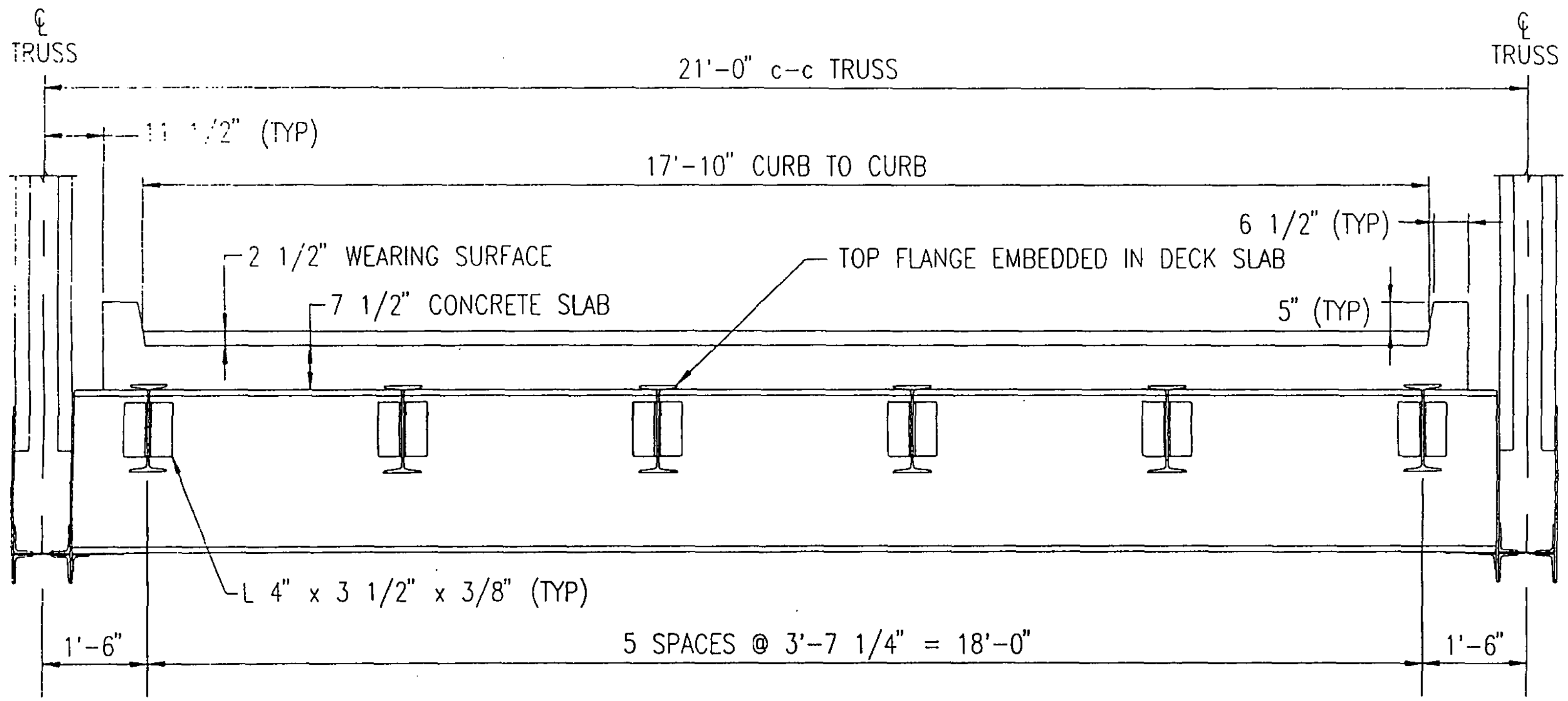


Figure 8.5. Cross Section at Center Truss Spans

Table 8.7 Rating Factors for Truss Stringers

Truck Type	As-Designed		Present-Condition	
	Inventory	Operating	Inventory	Operating
Two Axle	0.74	1.19	0.29	0.57
Tri-Axle	0.52	0.85	0.21	0.40
Concrete Truck	0.59	0.95	0.23	0.46
3S2 (AL)	0.84	1.36	0.33	0.65
School Bus	1.48	2.39	0.58	1.14

Table 8.8 Rating Factors for Truss Floorbeams

Truck Type	As-Designed		Present-Condition	
	Inventory	Operating	Inventory	Operating
Two Axle	1.12	1.82	1.07	1.76
Tri-Axle	0.81	1.33	0.78	1.28
Concrete Truck	0.92	1.50	0.88	1.45
3S2 (AL)	1.28	2.09	1.23	2.02
School Bus	2.78	4.55	2.67	4.39

was found at the interior stringers of the truss spans. Hence, the As–Designed ratings of Table 8.7 are equal to the Present–Condition ratings for the interior stringers.

CONCRETE DECKS ON TRUSS SPANS

There are no structural drawings available that show the reinforcing details used in the reinforced concrete decks on truss spans. It was not possible to calculate quantitative rating factors for these members.

The field inspection results indicate that the concrete decks appear to be in fair condition. The top of the deck typically exhibited random hairline cracks and some areas of map cracking. Random transverse hairline cracks with efflorescence were found on the underside of the deck. Otherwise, the underside of the deck was found to be in generally good condition. The condition of the deck does not appear to require a reduction of the load ratings of the bridge.

CONCLUSIONS AND RECOMMENDATIONS

The results of the field inspection, structural analyses, and load ratings indicate that there are a number of alternatives that should be considered in setting the final ratings of the bridge. First, as described by A.G. Lichtenstein & Associates (Montgomery County 1991), the bearing capacity of the ends of the concrete T–beam stringers is questionable. Due to the nature of the cracking and structural details at the bearings it is not possible to calculate quantitative ratings for the bearings. It is recommended that positive action to improve the conditions at the ends of the concrete T–beam stringers should be taken.

Rating calculations indicate that the load capacities of the riveted truss connections are very low when the allowable shear stress for Grade 1 rivets is used. The truss connections do not control the bridge ratings when the allowable shear stress for Grade 2 rivets is used. Because it is not known whether Grade 1 or Grade 2 rivets were used in the connections, it would be conservative to assume that Grade 1 rivets were used. It is suggested that such a conservative assumption is not necessary since no distress of the riveted connections was found during the bridge inspection. It is recommended that load restrictions not be placed on the bridge based on the connection capacities. If future inspections reveal distress of the connections, this recommendation should be reconsidered.

The final Present–Condition ratings for the bridge are controlled by the flexural capacities of the stringers of the truss floor systems. A summary of the ratings based on the present capacity of the fascia stringers in given in Table 8.9. The ratings given in Table 8.9 are based on the continued use of two traffic lanes on the bridge. The bridge ratings can be significantly improved by

Table 8.9 Present-Condition Ratings for Fascia Stringers of Truss Spans of Montgomery County Structure No. 009-51-040.7B

Truck Type	Standard Weight (tons)	Inventory Rating Factor	Inventory Rating (tons)	Operating Rating Factor	Operating Rating (tons)
Two Axle	29.5	0.29	9	0.57	17
Tri-Axle	37.5	0.21	8	0.40	15
Concrete Truck	33	0.23	8	0.46	15
3S2 (AL)	40	0.33	13	0.65	26
School Bus	12.5	0.58	7	1.14	>12.5

providing a positive means to restrict traffic to a single lane along the center of the deck. Ratings based on the interior stringer capacities for a single traffic lane are shown in Table 8.10. The ratings based on the interior stringer capacities would control the bridge ratings for all trucks except for one case noted in Table 8.10.

The field inspections also identified the following maintenance and repairs that should be carried out although the bridge is scheduled for removal and replacement in 1992:

1. The deck joints at the truss spans should be resealed.

2. The clogged drains on the deck should be unclogged and pipe extensions installed to divert drainage away from fascia stringers.

Table 8.10 Present-Condition Ratings for Interior Stringers of Truss Spans of Montgomery County Structure No. 009-51-040.7B

Truck Type	Standard Weight (tons)	Inventory Rating Factor	Inventory Rating (tons)	Operating Rating Factor	Operating Rating (tons)
Two Axle	29.5	0.74	22	1.19	>29.5
Tri-Axle	37.5	0.52	20	0.85	32
Concrete Truck	33	0.59	20	0.95	31
3S2 (AL)	40	0.84	34	1.36*	>40
School Bus	12.5	1.48	>12.5	2.39	>12.5

*Controlling value is 1.03 for truss connections as shown in Table 8.6.

CHAPTER NINE
WILCOX COUNTY
STRUCTURE NO. 021–66–023.2

Wilcox County Structure No. 021–66–023.2 was constructed in 1939 and currently carries AL Route 21 traffic over Cedar Creek between Furman and Braggs. The bridge consists of an 80 feet pony truss (span 7) with 3 timber stringer approach spans (1 through 3) and 10 steel stringer approach spans (4 through 6 and 8 through 14). The steel stringer approach spans are three–span continuous and two–span continuous configurations. An elevation of the bridge illustrating the arrangement of spans and span numbering is shown in Figure 9.1. A summary of the structural analyses and load rating results are presented in this chapter. Results from the field inspection and condition survey are summarized by A.G. Lichtenstein & Associates' Condition Report (Wilcox County 1991).

TIMBER STRINGER APPROACH SPAN RATINGS

The 17 feet 1 inch long timber stringer approach spans (1 through 3) are made up of a 6 1/4 inches thick roadway deck supported by 15 heavily creosoted timber beams (6"x14") as shown in Figure 9.2. The curb to curb roadway width is 23 feet 8 inches. The structure is rated as a two–lane bridge. It is further assumed that the timber stringers are Southern Pine, No.1 dense SR, Timber and Post grade. An allowable flexural stress of 1,550 psi is assumed for inventory ratings and 2060 psi for operating ratings. It was judged that the contact between the concrete deck and top surface of the timber stringers provided sufficient lateral restraint so that the stringers are continuously braced. As can be seen from Figure 9.2, the timber stringer spacings are uneven, ranging from 7 inches to 24 1/4 inches. An average stringer spacing of 1.63 feet was used to calculate a wheel load distribution factor (0.326) for use in the stringer rating calculations. Cross sectional properties used are based on the actual sizes of the stringers to reflect the size changes due to shrinkage and sawing.

A summary of the timber stringer approach span rating factors is given in Table 9.1. The rating factors are based on the flexural capacity of the stringers. The field inspection crew found the timber stringers to be in generally fair condition with small checks. Therefore, it is judged that the As–Designed rating factors given for the flexural capacities of the timber stringers in Table 9.1 are sufficiently accurate for use as Present–Condition ratings.

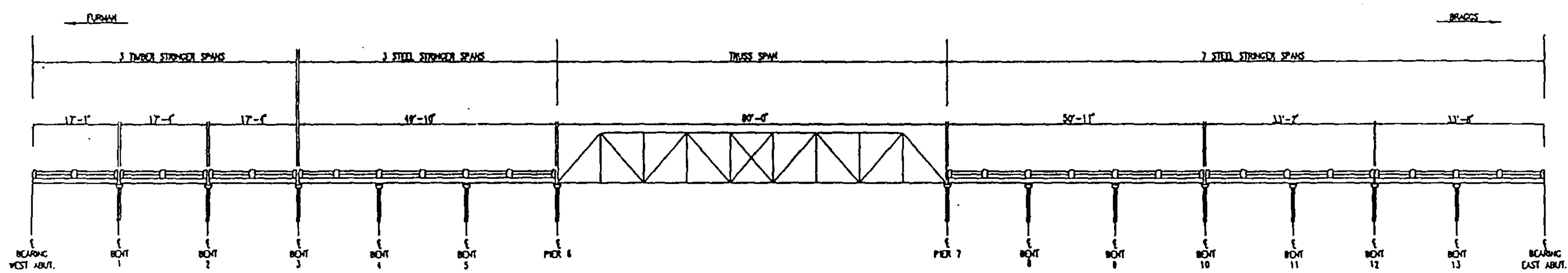


Figure 9.1. South Elevation of Furman Bridge (021-66-023.2)

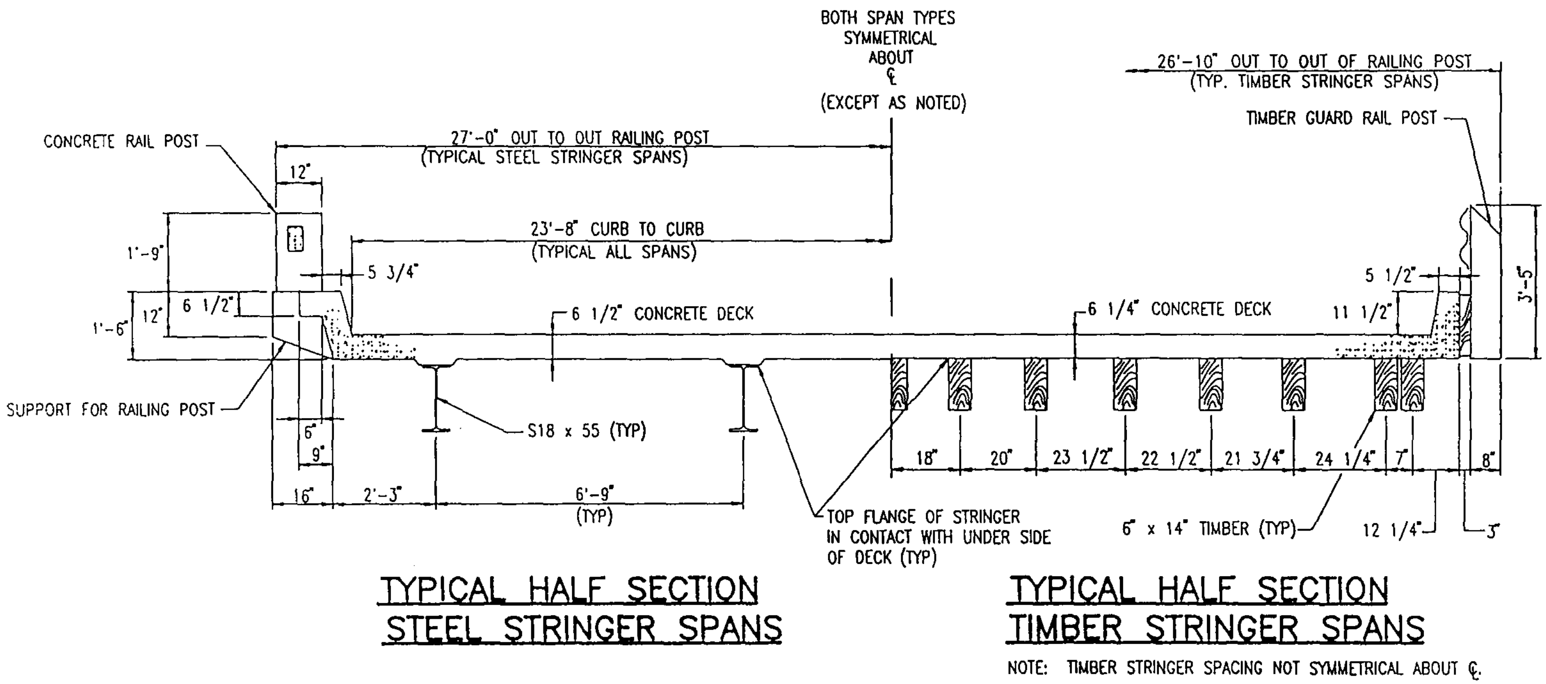


Figure 9.2. Cross Sections at Approach Spans

Table 9.1 Rating Factors for Timber Stringer Spans

Truck Type	As-Designed/Present-Condition	
	Inventory	Operating
Two Axle	0.66	0.97
Tri-Axle	0.48	0.71
Concrete Truck	0.53	0.78
3S3 (AL)	0.67	0.99
3S2 (AL)	0.75	1.11
School Bus	1.22	1.79

STEEL STRINGER SPAN RATINGS

There is a total of four steel stringer spans (spans 4 through 7 and 8 through 14). The steel stringer approach spans consist of two three–span and two two–span continuous configurations as shown in Figure 9.1. The cross section details are identical at all the stringer spans. The superstructure is made up of a 6 1/2 inches thick reinforced concrete roadway deck supported by four I–beam stringers (S18x55). The yield stress of the steel is taken to be 33 ksi according to AASHTO (Manual 1983). The stringers are laterally connected by concrete diaphragms above bents (negative moment regions, 4,5,8,9,11,13) where there is no deck joint. However, there are no diaphragms above bents where a deck joint is present. A typical cross section through the steel stringer spans is shown in Figure 9.2. The field inspection could not confirm the presence of mechanical shear connectors between the top flange of the steel stringer and the concrete deck. Therefore, the typical section is shown in Figure 9.2 without shear connectors and noncomposite section properties were used in the rating calculations. In determining the allowable bending stress in the negative moment zones the unbraced length was conservatively taken as one-half the span length.

Summaries of the minimum inventory and operating rating factors evaluated for the steel stringers are given in Table 9.2. These rating factors are based on the flexural capacity of the stringers. The Condition Report (Wilcox County 1991) indicates that the steel stringer spans were generally in fair condition. All surfaces of the stringers exhibited light rust with no significant section losses. It is judged that the As–Designed rating factors for the flexural capacities of the stringers in Table 9.2 are sufficiently accurate for use as Present–Condition ratings.

TRUSS RATINGS

The span length of the pony truss (span 7) is 80 feet. An elevation of span 7 showing the overall geometry, support conditions and joint numbering for the truss is shown in Figure 9.3. Cross section properties for the truss members are given in Table 9.3.

Structural analyses of the truss were performed using the in–house computer program BRIDGE to determine the dead load and live load stresses in each of the members. A summary of the dead load and maximum live load stresses is given in Table 9.4. Because of the relatively short span length of the truss, truss members develop the maximum live load stress under a tri–axle dump truck load. It should be noted that slightly different members were used for the members U1L2 and U3L2 of the north and south trusses. However, there are no practical differences in the member forces of the two trusses.

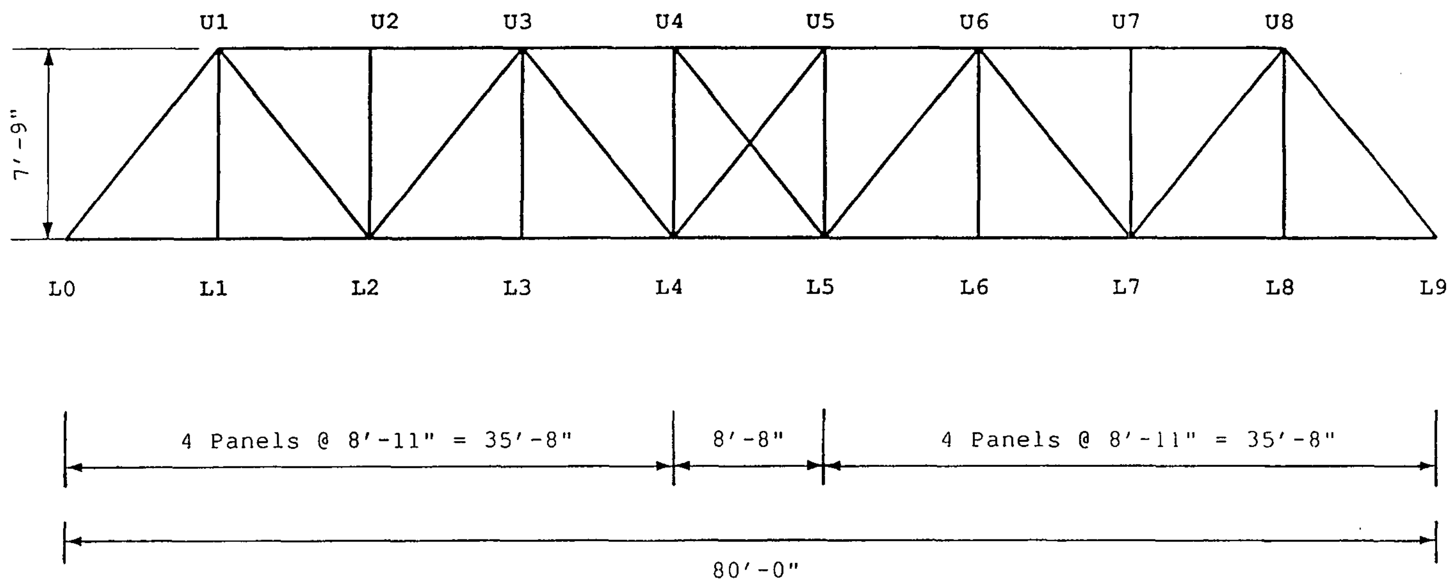


Figure 9.3. Elevation of Truss

Table 9.2 Rating Factors for Noncomposite Steel Stringers

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	1.34	1.95
Tri-Axle	1.02	1.44
Concrete Truck	1.07	1.51
3S3 (AL)	1.32	1.87
3S2 (AL)	1.41	1.99
School Bus	2.31	3.27

Table 9.3 Member Cross Section Properties

Member	Area ($inch^2$)	I_x ($inch^4$)	I_y ($inch^4$)
L0U1,U1U2 U2U3	15.98	146.75	511.24
U3U4,U4U5	18.98	170.99	575.24
L0L1,L1L2	9.00	33.18	174.52
L2L3,L3L4 L4L5	16.50	52.27	310.00
U1L1,U2L2, U3L3,U4L4	4.18	4.90	180.96
U3L4	4.96	7.92	97.48
U4L5	3.24	2.84	65.82
U1L2,U3L2	8.72	32.42	150.84 (North Truss)
U1L2,U3L2	9.00	33.18	174.52 (South Truss)

Table 9.4 Dead Load Stresses and Maximum Live Load Stresses

Member	Dead Load (ksi)	Maximum Live Load Plus Impact (ksi)
L0L1	6.89	-10.23
L1L2	6.89	9.55
L2L3	8.48	11.47
L3L4	8.48	11.47
L4L5	9.39	12.17
L0U1	-5.16	-7.13
U1U2	-6.81	-9.32
U2U3	-6.81	-9.32
U3U4	-8.19	-11.03
U4U5	-8.22	-10.70
L1U1	2.90	12.73
L2U2	-0.19	0.00
L3U3	2.97	12.77
L4U4	-0.37	-4.26
L2U1	6.87	10.92
L2U3	-4.61	-9.20
L4U3	4.14	13.41
L4U5	0.15	-4.86
L5U4	0.15	5.39

Table 9.5 Rating Factors for Truss Members

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.74	1.14
Tri-Axle	0.56	0.86
Concrete Truck	0.64	0.99
3S3 (AL)	0.66	1.01
3S2 (AL)	0.68	1.04
School Bus	1.79	2.74

Summaries of the minimum inventory and operating rating factors calculated for the truss are given in Table 9.5. These rating factors are based on the axial stresses created in the truss members by the standard truck loadings. The compressive axial force in the member U3U4 controlled both the inventory and operating ratings.

The truss members were found in generally satisfactory condition. All exposed surfaces exhibited light rust with more severe paint loss occurring on the truss web members. However, no significant section loss was noted which would affect the rating capacity. It is, therefore, concluded that the As–Designed rating factors for the truss members in Table 9.5 are sufficiently accurate for use as Present–Condition ratings.

Capacities of the truss connections were also evaluated. After reviewing the field notes and a series of photographs provided by A.G. Lichtenstein & Associate, connections at both ends of L0L1, L2L3, L4L5, L2U1, L4U3, L1U1, and L3U3 were selected as the most critical ones. The truss analyses also confirmed that these were indeed highly stressed connections. The field notes indicated (pp. 24–31) that the head diameter of the rivets was 1 1/4 inches. A shank diameter of 3/4 inch was taken from AISC (1927).

A summary of As–Designed ratings determined for the connections is given in Table 9.6. As can be seen from Tables 9.5 and 9.6, the rating factors for truss member connections are higher than those for the truss member capacities. The shear capacity of the connection of member L0L1 at the joint L0 controlled for all cases examined. The rating factors based on the bearing capacities were approximately two to three times higher than those given in Table 9.6. Hence they are not presented in this report. It was judged that the minor corrosion on the connection was insignificant, and the As–Designed rating factors given in Table 9.6 were sufficiently accurate for use as Present–Condition ratings.

TRUSS FLOOR SYSTEM RATINGS

The floor system framing is symmetrical with respect to the centerline of the truss. A typical section through the floor system at midspan is shown in Figure 9.4. The floor system is made up of a 6 1/2 inch thick concrete deck on six stringers. The stringer span is 8 feet 11 inches except at the center panel where the span is 8 feet 8 inches. S10x30 shapes were used for all stringers (S1 through S6) and S24x79.9 shapes were used for all floorbeams (FB0 through FB4). Cross sectional properties for these shapes were taken from AISC (1985). 7"x1/2" cover plates were riveted (field notes pp. 14/49) to the top and bottom flanges of the floorbeams as shown in Figure 9.4. It should be noted that the floorbeams were spliced at approximately the 1/3 point of their length. 3/8" splice plates were riveted to the web and top and bottom flanges. The actual floorbeam members (S24x79.9) were butt welded at the splice

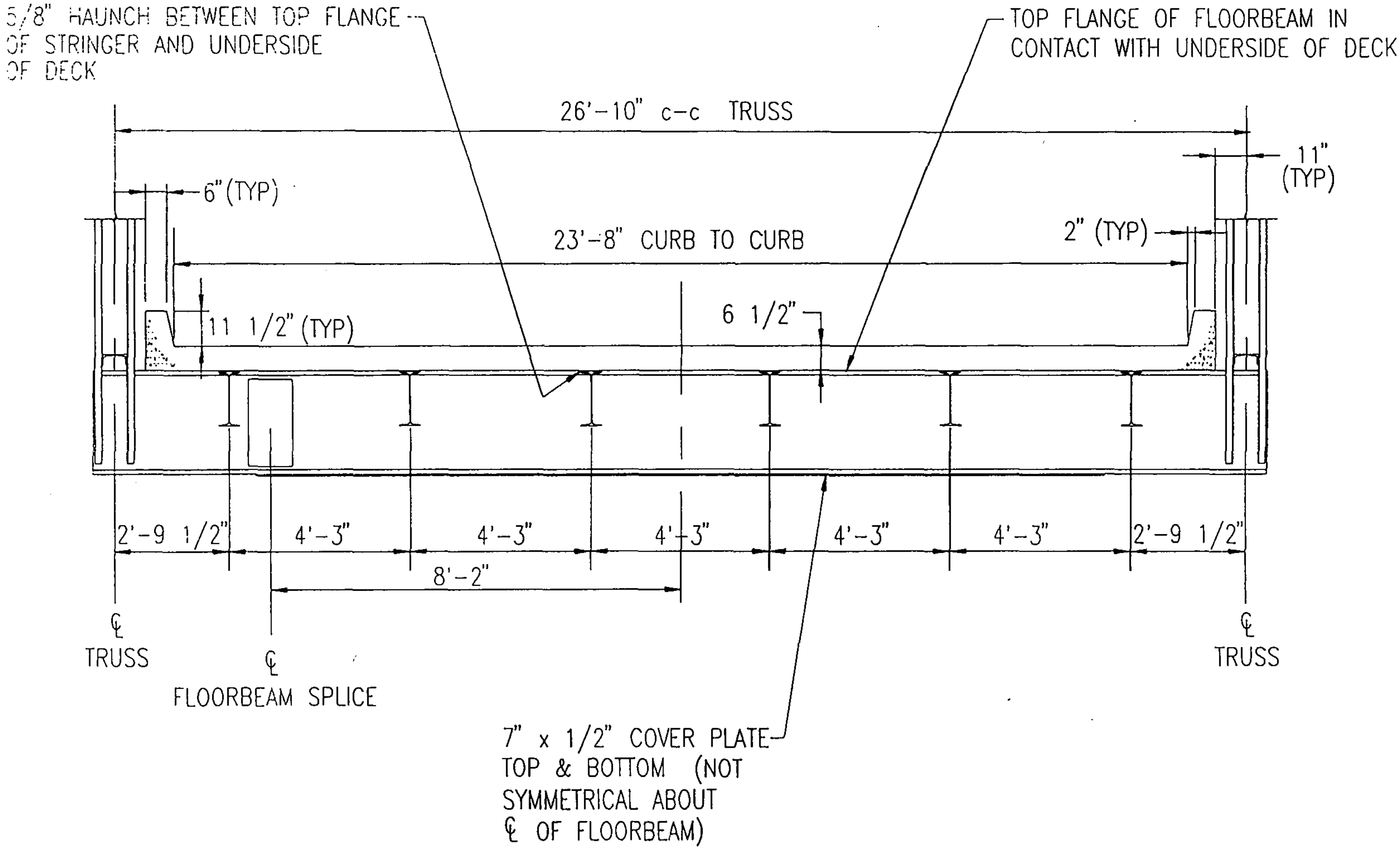


Figure 9.4. Cross Section at Truss Span

Table 9.6 Rating Factors for Truss Connection Capacities

Truck Type	As-Designed/Present-Condition	
	Inventory	Operating
Two Axle	0.92	1.49
Tri-Axle	0.71	1.15
Concrete Truck	0.80	1.30
3S3 (AL)	0.77	1.25
3S2 (AL)	0.81	1.32
School Bus	2.15	3.51

location. It was noted by the field inspection crew that all copes at the ends of the stringers were flame cut in a manner that produces a 90 degree reentrant corner. Stringer 1 (S1) at floorbeam 7 (FB7) has a flame cut gouge in the web at the cope. The gouge is nearly full thickness of the web and extends behind the clip angles (field notes pp. 41–42 and photo 9 in the condition report).

The bending moment and shearing forces in the stringers and floorbeams were evaluated by assuming the riveted connections at the member ends to act as simple supports. The compression flanges of the stringers were in contact with the underside of the concrete deck as shown in Figure 9.4. It was judged that the contact between the concrete deck and top flanges of the stringers provided sufficient lateral restraint so that the stringer compression flanges are continuously braced. The compression flanges of the floorbeams were considered to be laterally braced at the stringer to floorbeam connections although the top flanges of the floorbeams are in contact with the underside of the deck.

Summaries of the rating factors calculated for the stringers and floorbeams are given in Tables 9.7 and 9.8. These rating factors were based on the flexural capacities of the members. It was found that rating factors based on the shear capacities of the members and the stringer to floorbeam and floorbeam to truss connections (including the combined shear and tension analysis) are substantially higher than those presented in Tables 9.7 and 9.8. Hence, those rating factors are not presented in this report. The truss stringers and floorbeams were found in generally satisfactory condition with light rust typical. No significant section loss which would affect the rating capacity was found. It is judged that the As–Designed rating factors given in Tables 9.7 and 9.8 are sufficiently accurate for use as Present–Condition ratings.

CONCRETE DECKS

There are no structural drawings available that show the reinforcing details used in the reinforced concrete decks. It was not possible to calculate quantitative rating factors.

The field inspection results indicate that the concrete decks appear to be in fair condition with light to moderate scaling throughout. All spans exhibited a pattern of hairline transverse cracks ranging in length from 1/2 to full width of the deck. These cracks were located directly above each floorbeam in the truss span and over the piers (negative moment regions) in spans 4 through 6 and 8 through 14. The underside of deck was found to be in generally fair condition. Spans 4 through 14 exhibited a pattern of hairline transverse cracks with leaching ranging from 1/2 to full width of the deck. The condition of the deck does not appear to require a reduction of the load ratings of the bridge.

Table 9.7 Rating Factors for Truss Stringers

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	1.35	1.89
Tri-Axle	1.20	1.68
Concrete Truck	1.08	1.51
3S3 (AL)	1.82	2.54
3S2 (AL)	1.54	2.16
School Bus	1.91	2.67

Table 9.8 Rating Factors for Truss Floorbeams

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.85	1.24
Tri-Axle	0.61	0.89
Concrete Truck	0.68	0.99
3S3 (AL)	0.85	1.24
3S2 (AL)	0.97	1.42
School Bus	1.55	2.26

Table 9.9 Summary of Present-Condition Ratings for Wilcox County Structure No. 021-66-023.2

Truck Type	Standard Weight (tons)	Inventory Rating Factor	Inventory Rating (tons)	Operating Rating Factor	Operating Rating (tons)
Two Axle	29.5	0.66	19	0.97	29
Tri-Axle	37.5	0.48	18	0.71	27
Concrete Truck	33	0.53	17	0.78	26
3S3 (AL)	42	0.67	28	0.99	42
3S2 (AL)	40	0.75	30	1.11	>40
School Bus	12.5	1.22	>12.5	1.79	>12.5

CONCLUSIONS AND RECOMMENDATIONS

Structural analyses and load ratings of the bridge indicate that the final bridge ratings are controlled by the flexural capacities of the timber stringer approach spans. A summary of the Present–Condition ratings for the bridge is given in Table 9.9. The field inspection (Condition Report pp. 7) also identified the following maintenance and repairs that should be carried out:

1. River bank protection and river bed scour protection by the addition of stone riprap may be needed to prevent further erosion and slumping. The river bed appears to be scour prone material and this might be a scour critical bridge. AHD should schedule a scour evaluation of this bridge.

2. A timber crib assembly in the channel (consisting of piles and planks) was noted at the base of pier 7. The purpose of this assembly was not immediately apparent. If the assembly serves as a possible scour protection for the pier, its effectiveness appears questionable. In any event, it is restricting flow and may cause significant eddy currents during high stages of flow.

3. New joint seals should be placed at all open joints on the approach stringer spans, above piers 6 and 7 and bents 10 and 12, to prevent water and debris from falling on surfaces below the joint.

4. Approach guard rails should be installed at the ends of the structure to protect against impact of the bridge rail.

5. Speed reduction signs should be installed at both approaches and the Department should request assistance from local and state police in enforcement of speed and load capacity posting.

6. The flame cut copes at the ends of the truss stringers should be carefully inspected for possible fatigue cracks during future bridge inspections.

CHAPTER TEN
MOBILE/BALDWIN COUNTY
STRUCTURE NO. 016–49–037.6B

Mobile/Baldwin County Line Structure No. 016–49–037.6B was constructed in 1925 and currently carries U.S. Route 90 traffic over the Tensaw River from Mobile to Baldwin County. The original bridge consisted of 12 steel stringer spans (6 spans each approach) and 5 camel back Pratt truss spans, the center span being a lift span. The lifting machinery and towers were removed in 1978 leaving the lift span permanently in the down position. An elevation of the bridge illustrating the arrangement of the spans and span numbering is shown in Figure 10.1. A summary of the structural analyses and load rating results are presented in this chapter. Results from the field inspection and condition survey are summarized by A.G. Lichtenstein & Associates (Mobile/Baldwin County 1991).

STRINGER SPAN RATINGS

There is a total of twelve steel stringer spans (six at Mobile side and six at Baldwin side). A.G. Lichtenstein & Associates found shop drawings of the truss spans and standard deck girder details which correspond to the approach spans (spans 1 through 6 and 12 to 17). At a first glance, the six approach spans on each side of the main spans seem to consist of a central four continuous segment and one simple span on both ends separated by an expansion joint. A careful reexamination of the old plan marked as 5/11, reveals that stringers are not continuous and there is a vertical construction joint in the concrete deck and diaphragms hidden 1 inch below the top of deck.

A typical cross section of steel stringer approach spans is shown in Figure 10.2. B26x91 sections were used for stringers for all stringer approach spans. Diaphragms made of C15x33.9 were placed at every 1/3 point of each span connecting the stringers through clip angles. Concrete diaphragms were placed at each pier location. Based on the time of construction the steel members are assumed to have a yield strength of 30 ksi as suggested by AASHTO (Manual 1983). Most steel stringer bridges with a concrete deck constructed in the early 1930's did not use any mechanical shear studs. Lacking evidence of shear studs between the steel stringer and the concrete deck, non–composite section properties were used in the rating calculations. The allowable flexural stresses (inventory and operating) were determined assuming the compression flanges of the stringers are continuously braced by the concrete deck (the top flange of the stringer is continuously embedded into the concrete deck). A summary of As–Designed rating factors for the steel stringer approach spans is given in Table 10.1.

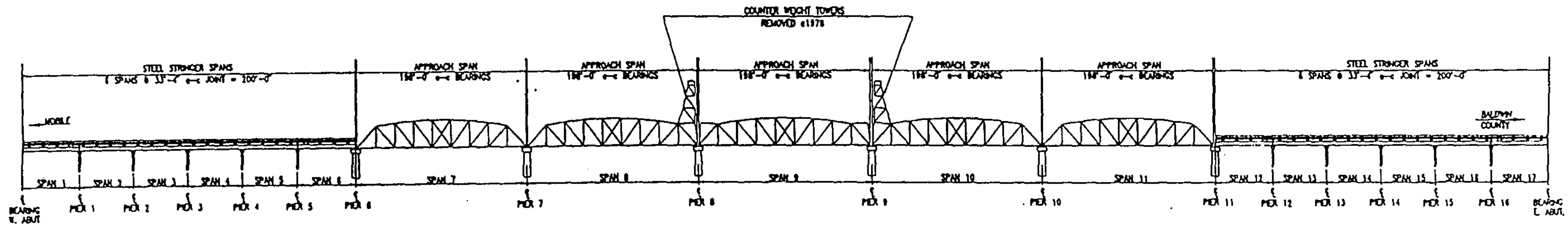
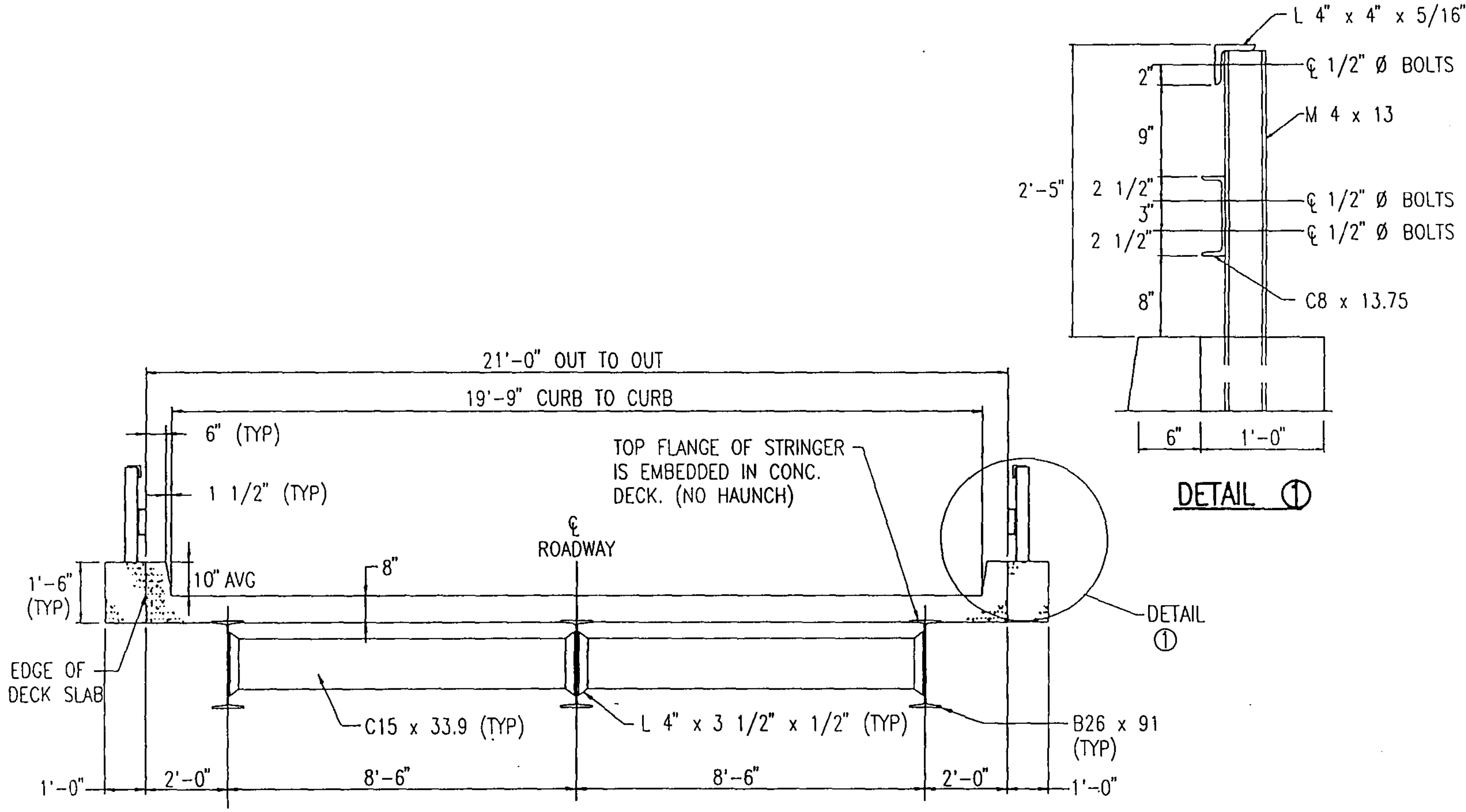


Figure 10.1. South Elevation of Mobile/Baldwin Bridge (016-49-037.6B)



NOTE: PARABOLIC ROADWAY CROSS SECTION ACHIEVED BY RAISING MIDDLE STRINGER RELATIVE TO FASCIA STRINGER.

Figure 10.2. Cross Section at Approach Stringer Spans

Table 10.1 Rating Factors for Approach Stringer Spans

Truck Type	As-Designed/Present-Condition	
	Inventory	Operating
Two Axle	0.61	1.00
Tri-Axle	0.44	0.71
Concrete Truck	0.49	0.81
3S3 (AL)	0.62	1.01
3S2 (AL)	0.75	1.22
School Bus	1.59	2.60

The availability of reinforcement details of the concrete deck made it possible to rate the slab also. It was assumed according to AASHTO (Manual 1983) that the concrete strength was 3300 psi and the yield strength of the reinforcing steel was 33 ksi. A summary of the As–Designed rating factors for the concrete deck is given in Table 10.2.

The field inspection (Mobile/Baldwin County 1991) revealed that the steel approach stringers were in generally fair condition with localized poor conditions occurring at the beam bearing areas. Three worst conditions found during the field inspection were summarized in the Condition Report (Mobile/Baldwin County 1991). Repair plates had been previously welded to the bottom flange at these locations. It is judged that these conditions are not likely to affect the load carrying capacities of the stringers. It is concluded that the As–Designed rating factors given in Table 10.1 are sufficiently accurate for use as Present–Condition ratings. The condition of the concrete deck is also judged to be fair to good so that the As–Designed rating factors given in Table 10.2 can be used as the Present–Condition rating factors.

TRUSS RATINGS

There are five Camel back through trusses that span 198 feet each (spans 7 through 11). Although there is a remnant of the counter weight towers (top end panel U8U9) at both ends of the center lift span (span 9) and the adjacent end of side spans (spans 8 and 10), all five trusses are essentially identical. An elevation showing the overall geometry, support conditions and joint numbering for the trusses is given in Figure 10.3. A typical cross section of the trusses is shown in Figure 10.4. Cross sectional properties for the truss members are given in Table 10.3.

Structural analyses of the truss were performed using the in–house computer program BRIDGE to determine the dead load and live load stresses in each of the members. Because the truss span length (198 feet) is less than 200 feet only one standard truck was placed in each lane per span as per provision 5.2.2 of AASHTO (Manual 1983). A summary of the dead load and maximum live load stresses is given in Table 10.4. Top and bottom chords develop the maximum stress under a six axle vehicle (3S3) while the tri–axle dump truck produces the maximum stresses in most verticals and diagonals whose influence lines are confined within the adjacent panels.

Summaries of the minimum As–Designed inventory and operating rating factors calculated for the trusses are given in Table 10.5. These ratings are based on the axial stresses created in the truss members by the standard truck loadings. The compression capacity of member U3U4 controlled most of the ratings. The compression capacity of U4U5 controlled the ratings for the 3S2 and 3S3 truck loadings.

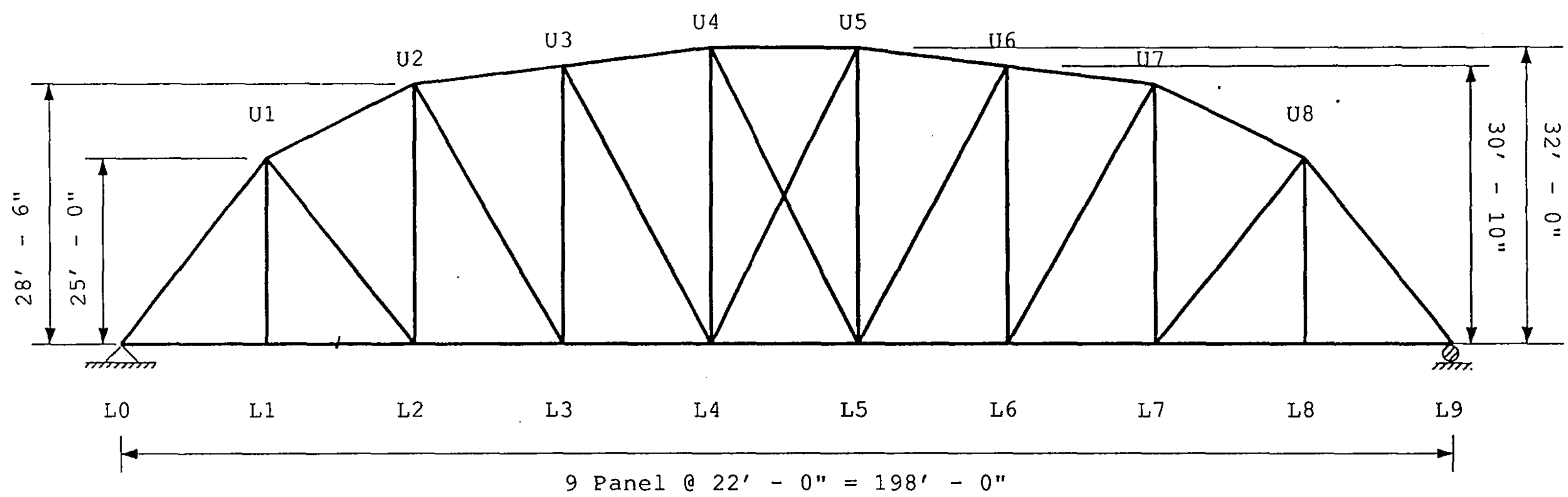


Figure 10.3. Elevation of Typical Truss Span

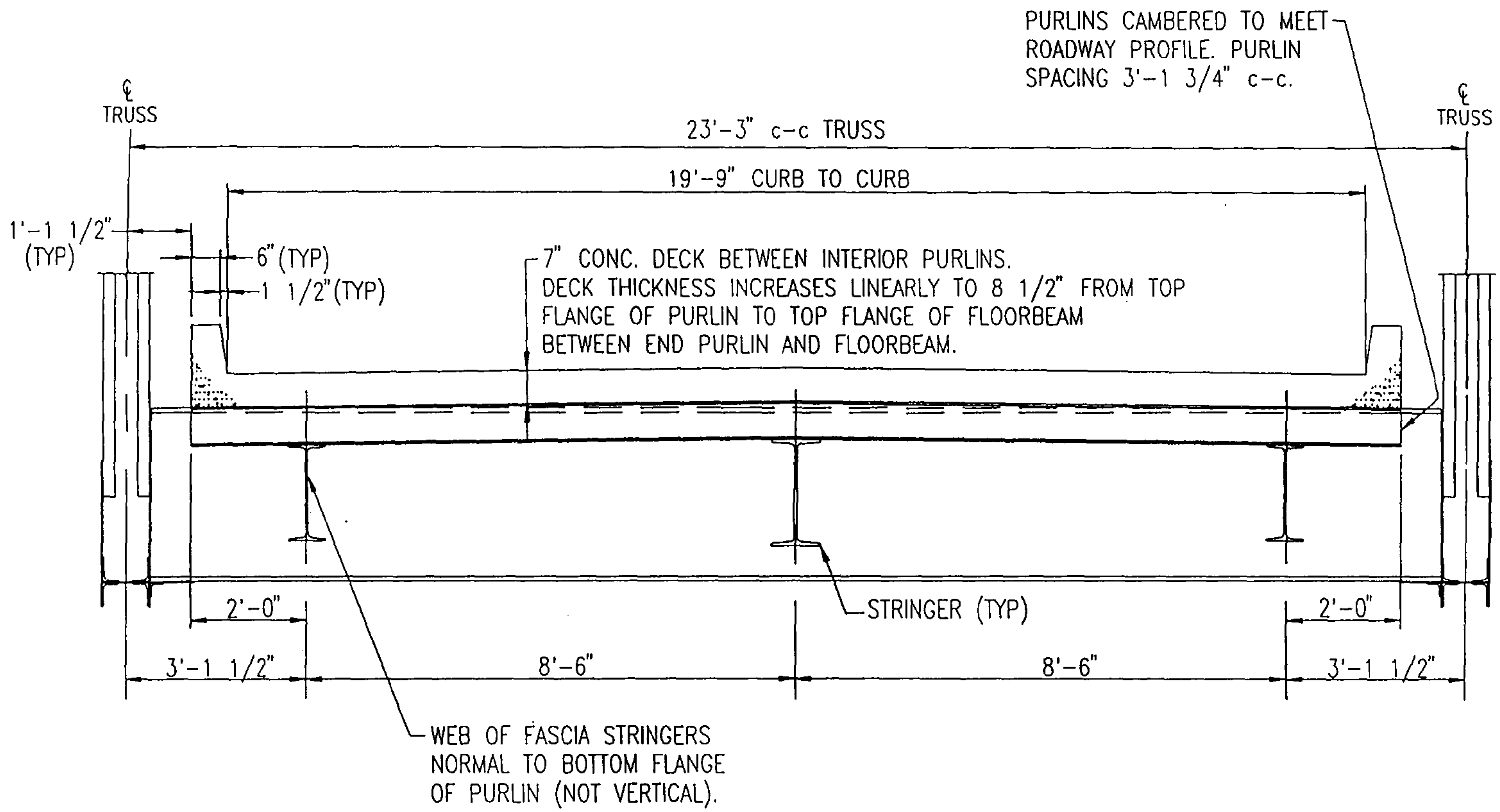


Figure 10.4. Cross Section at Truss Spans

Table 10.2 Rating Factors for Concrete Deck

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	1.31	2.18
Tri-Axle	1.16	1.94
Concrete Truck	1.05	1.74
3S3 (AL)	1.77	2.95
3S2 (AL)	1.50	2.49
School Bus	2.06	3.43

Table 10.3 Truss Member Cross Section Properties

Member	Area $(inch^2)$	I_x $(inch^4)$	I_y $(inch^4)$
L0U1	39.02	755.9	2,114.7
L1U1,U2U3 U3U4,U4U5	27.15	562.1	1,568.4
L0L1,L1L2	21.13	139.3	709.5
L2L3,L3L4	28.69	199.5	951.3
L4L5	33.01	241.1	1,093.4
L1U1	9.92	41.0	395.1
L2U2,L3U3 L4U4	11.72	157.0	488.8
U1L2	14.12	80.7	543.4
U2L3	9.92	36.3	391.5
U3L4	8.36	34.2	332.1
L4U5	7.72	20.6	302.2

Table 10.4 Dead Load Stresses and Maximum Live Load Stresses

Member	Dead Load (ksi)	Maximum Live Load Plus Impact (ksi)
L0L1	7.25	3.23
L1L2	7.25	3.23
L2L3	8.21	3.61
L3L4	9.77	4.24
L4L5	9.07	3.84
L0U1	-5.94	-2.65
U1U2	-8.78	-3.86
U2U3	-10.38	-4.51
U3U4	-11.08	-4.74
U4U5	-11.10	-4.72
L1U1	3.70	6.72
L2U2	-4.71	-3.60
L3U3	-1.66	-2.97
L4U4	0.82	1.84
L2U1	8.82	4.74
L2U3	7.38	5.50
L4U3	4.16	5.66
L5U4	0.23	2.95

Table 10.5 Rating Factors for Truss Members

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.66	1.58
Tri-Axle	0.51	1.23
Concrete Truck	0.58	1.39
3S3 (AL)	0.50	1.20
3S2 (AL)	0.52	1.26
School Bus	1.56	3.75

The corrosion and impact damage identified during the field inspection was considered in determining the Present–Condition ratings for the trusses. There was a great deal of minor to moderate corrosion and impact damage to the truss, particularly to members L2U1, L4U3, and L7U8 (Condition Report pp. 7). In addition to these three members, a total of ten additional members with moderate impact damage were identified from the Field Notes (pp. 128–156/204). Member cross sectional properties for these thirteen damaged members were recalculated. Except for the two end portals, L0U1 and L9U8, all of the damaged members were tension members. Therefore, the load carrying capacity of the diagonal member, L4U3, which was twisted and kinked about 8 1/2 inches out of line was not significantly affected. None of the truss members had a sufficient loss of section to affect the truss ratings, although some individual member capacities were reduced. It is judged that the As–Designed rating factors given for the truss members in Table 10.5 are sufficiently accurate for use as Present–Condition ratings.

Capacities of the truss connections were also evaluated. The connections at the ends of tension members, L1L2, L2L3, L4L5, L1U1, L2U1, L3U2, and L4U3 were selected as the most critical. The shop drawing (sheet No. 7/11 designated by A.G. Lichtenstein & Associates) by Harrington, Howard and Assoc. indicates that the diameter of rivets is 7/8 inch. The rivets were conservatively assumed to be ASTM A502 Grade 1 rivets. It was found that the splice of the bottom chord tension member L4L5 at the middle of the member controlled the connection ratings. Rating factors for the truss connections are summarized in Table 10.6. The ratings shown are based on the shear capacity of the rivets. Ratings based on bearing stresses were found to be much higher than those shown in Table 10.6.

The load capacities of the connections were found to be higher than the truss member capacities. It was judged that the minor corrosion on the connection was insignificant, and the rating factors given in Table 10.6 were sufficiently accurate for use as Present–Condition ratings.

TRUSS FLOOR SYSTEM RATINGS

A typical section through the truss span floor system is shown in Figure 10.4. Between floorbeams there are typically six S8x20.5 purlins spaced at 3 feet 1 3/4 inches center to center. These purlins were placed on top of stringers and are cambered to meet the roadway profile. Structural analyses were performed on the purlins using the in–house computer program BRIDGE assuming each purlin to be a two span continuous beam (stringer spacing of 8 feet 6 inches as span length) with 2 feet of overhang on each side. Summaries of the minimum inventory and operating rating factors calculated for the purlins are given in Table 10.7. These factors were controlled by the negative moment carrying capacity of the purlin at the top of interior stringer. Rating factors for the shear capacity were also evaluated. The resulting rating

Table 10.6 Rating Factors for Truss Connection Capacities

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.97	2.37
Tri-Axle	0.76	1.86
Concrete Truck	0.87	2.11
3S3 (AL)	0.65	1.57
3S2 (AL)	0.75	1.82
School Bus	2.30	5.60

factors were substantially higher than those given in Table 10.7 for moment capacity. Hence they are not presented in this report. The purlins were found to be in generally fair condition with section losses found mainly in the top flange and web of the cantilever portions beyond the fascia stringers. The most severe case of section loss occurred on the south cantilever of purlin 4 between floorbeam 3 and 4 in span 8 as reported in the Condition Report (pp. 9, Mobile/Baldwin County 1991) and Field Notes (pp. 80/204). The Present–Condition ratings in Table 10.7 were calculated using reduced section properties reflecting section losses due to corrosion.

One interior stringer (B22x68.5) and two fascia stringers (B20x59.5) span between floorbeams as shown in Figure 10.4. The bending moments and shearing forces in the stringers were evaluated by assuming the riveted connections at the member ends to act as simple supports. Cross sectional properties were taken from AISC (Iron and Steel Beams 1985). Ratings for the truss floor stringers were calculated separately for the interior stringer (S2) and the fascia stringers (S1 and S3). The wheel load distribution factor for the fascia stringers was evaluated following AASHTO provision 3.23.2.3.1.2 (Standard 1989). The wheel load distribution factor for the fascia stringers was 1.18. The distribution factor for the interior stringer was taken as S/5.5 (1.55) from AASHTO Table 3.23.1 (Standard 1989). Summaries of the minimum inventory and operating rating factors calculated for the stringers are given in Tables 10.8 and 10.9. These rating factors are based on the flexural capacities of the stringers. There was a great deal of corrosion damage to the fascia stringers at the top flange at the connection between stringer and purlin and in the stringer web at the connection between stringer and floorbeam. The most severe case of fascia stringer top flange section loss (remaining top flange thickness is only 3/8 inch of the original 3/4 inch) was found in the south fascia stringer (S3) under purlin 2 in the truss span 8 (Field Notes pp. 79). The Present–Condition rating factors given in Table 10.8 accounted for those section losses. The Field Notes and Condition Report by A.G. Lichtenstein & Associates (Mobile/Baldwin County 1991) indicate that the interior stringers are generally in good condition. It is judged that the rating factors given in Table 10.9 are sufficiently accurate for use as Present–Condition ratings for the interior stringers.

There are ten floorbeams (B30x115) in each truss span designated from west to east FB0 to FB9. Cross sectional properties of the rolled shape were taken from AISC (Iron and Steel Beams 1985).

The bending moments and shearing forces were evaluated by assuming the riveted connections at the floorbeam ends to act as simple supports. The compression flanges were assumed to be continuously supported by the concrete deck. Summaries of the minimum inventory and operating rating factors for the truss floorbeams are given in Table 10.10. These rating factors were based on the flexural capacities of the members. It was found that rating

Table 10.7 Rating Factors for Truss Floor Purlins

Truck Type	As-Designed Inventory	As-Designed Operating	Present-Condition Inventory	Present-Condition Operating
Two Axle	1.00	1.44	0.79	1.15
Tri-Axle	0.89	1.28	0.71	1.03
Concrete Truck	0.80	1.15	0.64	0.92
3S3 (AL)	1.36	1.95	1.07	1.56
3S2 (AL)	1.15	1.65	0.91	1.32
School Bus	1.18	1.70	0.93	1.36

Table 10.8 Rating Factors for Fascia Stringers

Truck Type	As-Designed Inventory	As-Designed Operating	Present-Condition Inventory	Present-Condition Operating
Two Axle	0.83	1.25	0.34	0.57
Tri-Axle	0.59	0.89	0.25	0.41
Concrete Truck	0.66	1.00	0.27	0.46
3S3 (AL)	0.81	1.23	0.34	0.57
3S2 (AL)	0.95	1.43	0.38	0.64
School Bus	1.61	2.43	0.75	1.27

Table 10.9 Rating Factors for Interior Stringer

Truck Type	As-Designed/Present-Condition Inventory	As-Designed/Present-Condition Operating
Two Axle	0.67	1.07
Tri-Axle	0.47	0.76
Concrete Truck	0.53	0.85
3S3 (AL)	0.66	1.05
3S2 (AL)	0.76	1.22
School Bus	1.30	2.08

Table 10.10 Rating Factors for Truss Floorbeams

Truck Type	As-Designed		Present-Condition	
	Inventory	Operating	Inventory	Operating
Two Axle	0.55	0.96	0.37	0.71
Tri-Axle	0.41	0.70	0.27	0.52
Concrete Truck	0.46	0.79	0.31	0.59
3S3 (AL)	0.60	1.04	0.40	0.77
3S2 (AL)	0.64	1.10	0.43	0.81
School Bus	1.38	2.39	0.92	1.76

factors based on the shear capacities of the members and the stringer to floorbeam and floorbeam to truss connections (including the combined shear and tension analysis) are substantially higher than those presented in Table 10.10.

As can be seen from Table 10.10, the Present–Condition rating factors for floorbeams are considerably lower than the As–Designed rating factors. This is the reflection of severe corrosion losses of floorbeams. The worst case was found in span 9, FB9 (end floorbeam). That floorbeam has heavy losses near midspan (losses measured 1/8 to 1/4 inch at bottom flange, Field Notes, pp. 101/204).

CONCRETE DECKS ON TRUSS SPANS

Reinforcement details for the truss span decks were available in sheet number 8/11 of the shop drawings. However, the lettering and lines are so badly faded as to make sensible reading of the details impossible.

The field inspection results indicate that the concrete deck appears to be in generally fair condition. The underside of the deck typically exhibited numerous longitudinal hairline cracks and honeycombed areas. The hairlinecracks are for the most part free of efflorescence. The concrete is typically spalled along the top flange of the purlins. The deck was sounded with a hammer and appears to be solid. The condition of the deck does not appear to require a reduction of the load ratings of the bridge.

CONCLUSIONS AND RECOMMENDATIONS

Structural analyses and load ratings of the bridge indicate that the final bridge ratings are controlled by the flexural capacity of the south fascia stringer of the truss span 8 in panel L2L3 (between FB2 and FB3). A summary of the Present–Condition ratings for the bridge is given in Table 10.11. The field inspection also identified the following maintenance and repairs that should be carried out:

1. Field measurements taken at the truss rocker bearings (at locations of apparent overstress) show the pin hole is oversized by up to 1/2 inch for the pin diameter. Repair schemes should be implemented to increase pin plate bearing area by addition of reinforcing plates. All misaligned pin nuts should be realigned and tightened.

2. All damaged truss members, end portals, and sway bracings should be repaired by heat straightening the lesser damaged members (bent but not torn) and heat straightening with the addition of reinforcement plates to the torn and damaged members.

Table 10.11 Summary of Present-Condition Ratings for Mobile/Baldwin County Structure No. 016-49-037.6B

Truck Type	Standard Weight (tons)	Inventory Rating Factor	Inventory Rating (tons)	Operating Rating Factor	Operating Rating (tons)
Two Axle	29.5	0.34	10	0.57	17
Tri-Axle	37.5	0.25	9	0.41	15
Concrete Truck	33	0.27	9	0.46	15
3S3 (AL)	42	0.34	14	0.57	24
3S2 (AL)	40	0.38	15	0.64	26
School Bus	12.5	0.75	9	1.27	>12.5

3. The stringer–to–floorbeam connections in truss spans 8 and 9 which have broken rivets (one rivet missing, span 8; one rivet head missing, span 9) should be retrofitted with A325 bolts and hardened washers (Field Notes, pp. 83 and 93 of 204).

4. Pier 5 was observed to be out of plumb. AHD should schedule underwater inspection of this and other piers in conjunction with an investigation of the cause of the out–of–plumb condition. In addition, the vertical crack in pier 11 should be injected with an epoxy resin compound and monitored closely for subsequent detrimental movement. Large spalls and cracks on the substructure should be repaired. The void beneath the east abutment breastwall should be filled.

5. All damaged/deteriorated bridge railings should be repaired.

6. Bolted reinforcement plates should be added to the deteriorated areas on the stringer webs in the bearing areas in the approach stringer spans 1–6 and 12–17 (Field Notes, pp. 55 through 77 of 204).

7. New joint seals should be placed at all open joints on the approach stringer spans, above piers 1, 5, and 12, to prevent water and debris from falling on surfaces below the joint. The top of deck should be sawcut and then filled with poured joint material to seal and control the transverse cracking at each abutment and above piers 2 through 4 and 13 through 16. The joints between the truss spans, above pier 6 through 11, also permit drainage and debris buildup on the substructure below.

8. Clean all rocker bearings of debris/rust and repaint. Replace all deteriorated anchor bolts and nuts.

CHAPTER ELEVEN
LAUDERDALE COUNTY
STRUCTURE NO. 002–39–008.7A

The Shoal Creek bridge, Lauderdale County Structure No. 002–39–008.7A, was constructed in 1924 and currently carries the westbound traffic of U.S. Route 72, U.S. Route 43, and Alabama Route 13 over Shoal Creek from Killen to Florence. The bridge consists of five identical 157.5 feet long camel back Pratt truss spans with no approach spans. An elevation of the bridge illustrating the arrangement of the spans and span numbering is shown in Figure 11.1. A summary of the structural analyses and load rating results are presented in this chapter. Results from the field inspection and condition survey are summarized by A.G. Lichtenstein & Associates (Lauderdale County 1991).

TRUSS RATINGS

An elevation showing the overall geometry, support conditions and joint numbering for the trusses is given in Figure 11.2. Cross sectional properties for the truss members are given in Table 11.1. A typical cross section through the truss deck system is shown in Figure 11.3. Following provision 5.2.2 of AASHTO (Manual 1983), the bridge was rated as a one lane bridge since the actual curb to curb roadway width of 17 feet 8 inches is less than the threshold value of 18 feet. The structure is currently used for one lane bridge.

Structural analyses of the truss were performed using the in–house computer program BRIDGE to determine the dead load and live load stresses in each of the members. Because truss span length (157.5 feet) is less than 200 feet, only one standard truck was placed in each lane per span as per provision 5.2.2 of AASHTO (Manual 1983). A summary of the dead load and maximum live load stresses is given in Table 11.2. The top and bottom chords develop the maximum stress under a six axle vehicle (3S3) while the tri–axle dump truck produces the maximum stresses in most verticals and diagonals with influence lines that are confined within the adjacent panels.

Summaries of the minimum As–Designed inventory and operating rating factors calculated for the trusses are given in Table 11.3. These ratings are based on the axial stresses created in the truss members by the standard truck loadings. The tension capacity of diagonal member L3U2 controlled ratings for all truck loadings for both inventory and operating ratings.

The trusses were found to be in generally poor condition. The significant defects found on the trusses were due to impact damage. Vertical members were bent inward due to impact damage of the lowest member (sway strut) of the attached

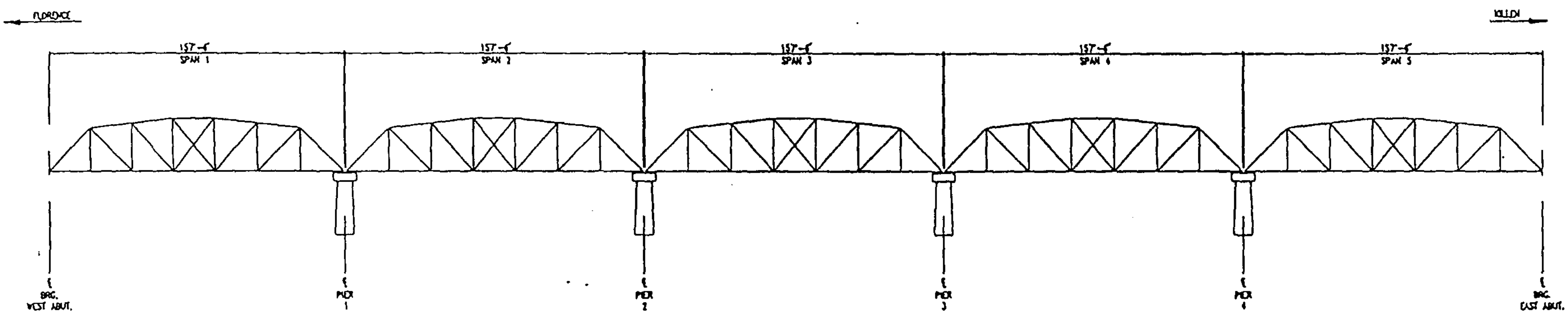


Figure 11.1. South Elevation of Shoal Creek Bridge (002-39-008.7A)

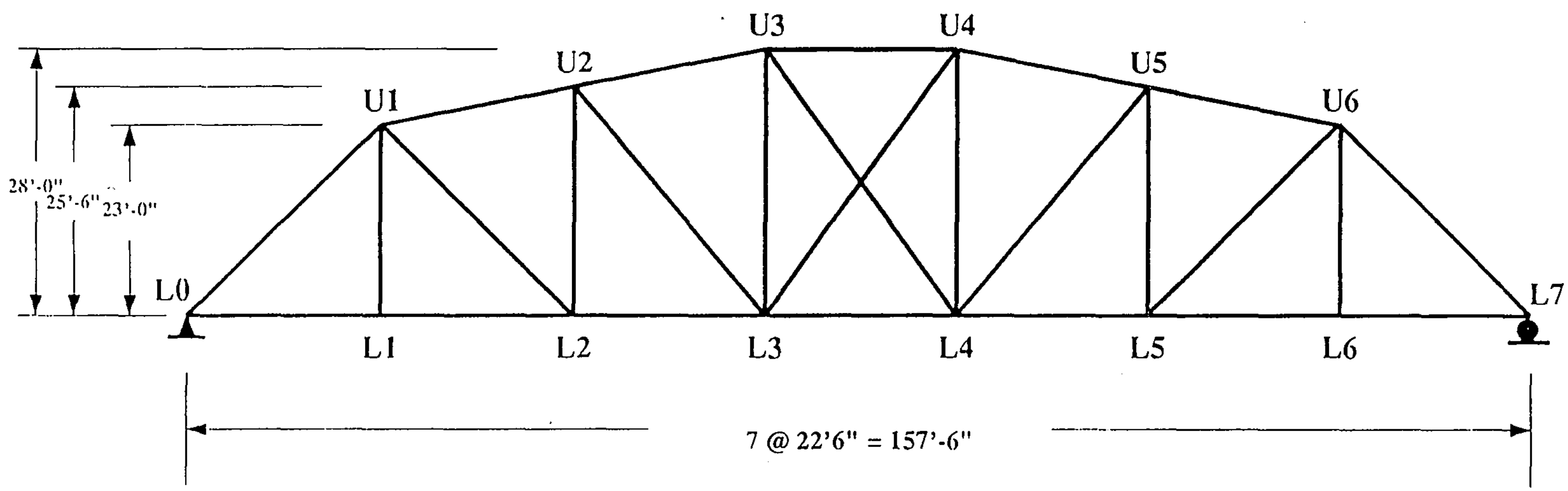


Figure 11.2. Elevation of Trusses

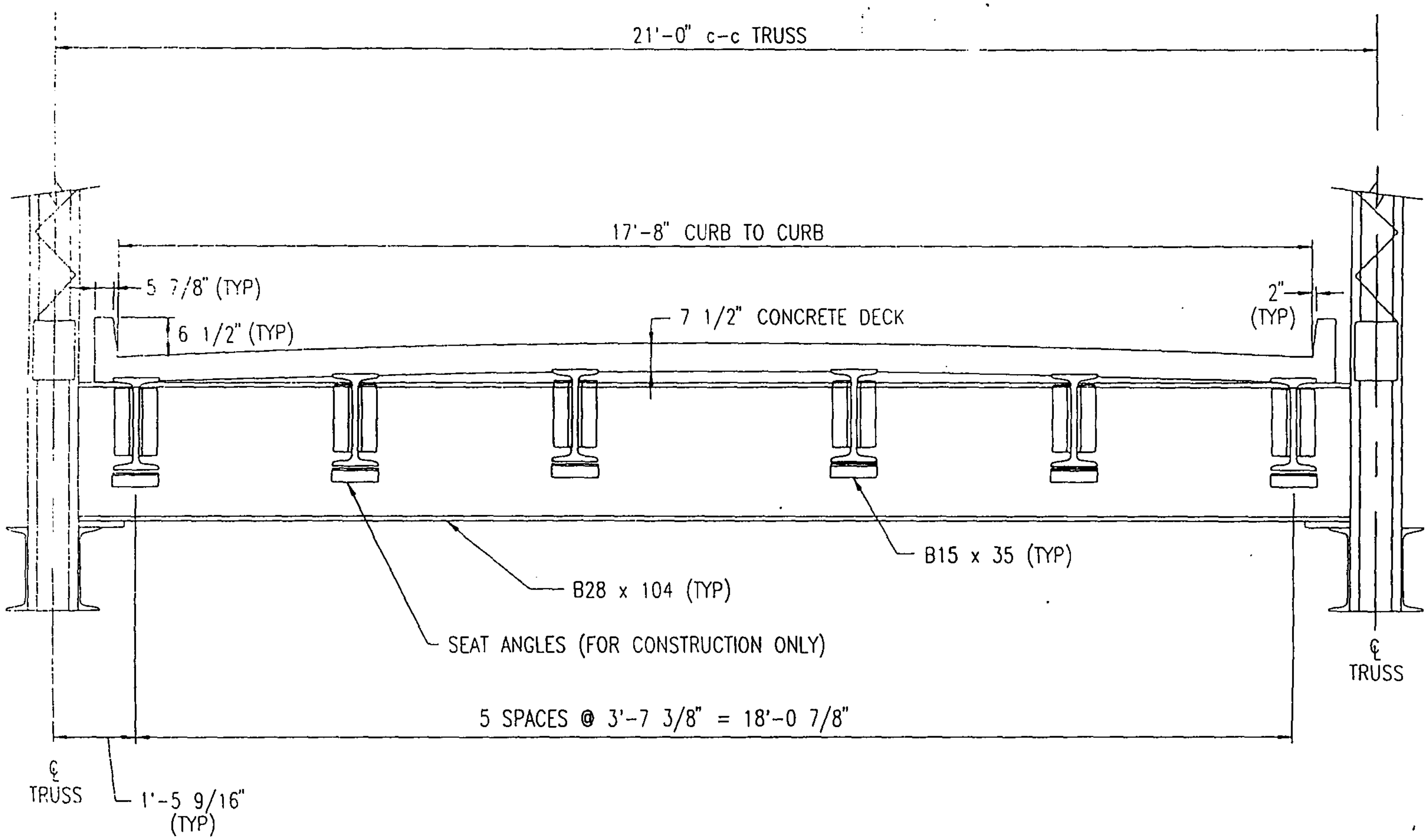


Figure 11.3. Cross Section Through Truss Floor System

Table 11.1 Truss Member Cross Section Properties

Member	Area ($inch^2$)	I_x ($inch^4$)	I_y ($inch^4$)
L0U1	24.37	510.0	896.4
U1U2,U2U3 U3U4	24.37	510.0	867.9
L0L1,L1L2	11.72	157.0	444.6
L2L3	17.60	206.0	646.9
L3L4	20.54	230.4	764.7
L1U1	7.12	22.5	147.0
L2U2,L3U3	8.04	71.6	170.6
L2U1	9.00	33.2	174.5
U2L3	4.18	6.8	82.9
U3L4,L3U4	3.24	2.8	65.8

Table 11.2 Dead Load Stresses and Maximum Live Load Stresses

Member	Dead Load (ksi)	Maximum Live Load Plus Impact (ksi)
L0L1	7.26	3.07
L1L2	7.25	3.07
L2L3	7.28	3.03
L3L4	6.74	2.70
L0U1	-5.04	-2.11
U1U2	-5.29	-2.20
U2U3	-5.79	-2.36
U3U4	-5.83	-2.35
L1U1	3.52	4.82
L2U2	-2.22	-2.33
L3U3	1.31	1.54
L2U1	6.89	3.71
L3U2	4.44	5.45
L4U3	1.00	3.76

Table 11.3 Rating Factors for Truss Members

Truck Type	As-Designed		Present-Condition	
	Inventory	Operating	Inventory	Operating
Two Axle	2.73	4.26	2.53	4.00
Tri-Axle	2.12	3.31	2.02	3.18
Concrete Truck	2.40	3.74	2.28	3.60
3S3 (AL)	2.19	3.42	2.15	3.40
3S2 (AL)	2.31	3.61	2.18	3.44
School Bus	6.44	10.07	6.30	9.94

vertical sway bracing. The sway bracing itself was in poor condition due to collision damage. All end portals displayed forms of collision damage which had torn gusset plates at some locations as shown in photo 19 of the Condition Summary Report (Lauderdale County 1991).

There were losses found in the truss members caused by localized rust, pitting, or impacted rust. The case that affected the rating the most is at member L4U5. A 1/16 inch loss of thickness on both 3 inches wide angle legs resulted in 9 percent area loss. Section losses and out of alignment of the truss members were accounted for in the calculation of the Present–Condition ratings that are given in Table 11.3.

It is quite evident from Table 11.3 that the trusses were designed for two–lane traffic. Although they are not being presented in this report, the rating factors calculated for two–lane truck loading are all greater than 1.0 for both inventory and operating ratings for both As–Designed and Present–Condition ratings.

The connections at both ends of tension members, L3U2,L1U1,L4U3, and L2U1 were selected as the most critical. The head diameter of rivets was measured as 1 1/4 inches as shown on page 8 of 100 of the Field Notes. The shank diameter of rivets was taken as 3/4 inch from page 4–135 of AISC (1980). The rivets were conservatively assumed to be ASTM A502 Grade 1 rivets. It was found that the connection capacity of tension member L3U2 at joint L3 controlled the connection ratings. Rating factors for the truss connections are summarized in Table 11.4. The ratings shown are based on the shear capacity of the rivets. Ratings based on bearing stresses were found to be much higher than those shown in Table 11.4. It was judged that the minor corrosion on the connections was insignificant, and the rating factors given in Table 11.4 are sufficiently accurate for use as Present–Condition ratings.

TRUSS FLOOR SYSTEM RATINGS

A typical section through the truss span floor system is shown in Figure 11.3. There are six B15x35 stringers spaced at 3 feet 7 3/8 inches center to center. Cross sectional properties for the stringers were taken from AISC (Iron and Steel Beams 1985). The bending moments and shearing forces in the stringers were evaluated by assuming the riveted connections at the member ends to act as simple supports. Summaries of the minimum inventory and operating rating factors calculated for the stringers are given in Table 11.5. These rating factors are based on the flexural capacities of the stringers. Rating factors based on the shear capacities were found to be much higher than those given in Table 11.5. Although minor section losses including losses in the web of the stub stringers were reported on pp. 27–33 of Field Notes, the stringers were found to be in good condition with no losses that significantly affect the ratings. It is judged that the rating factors given in Table 11.5 are sufficiently accurate for use as Present–Condition ratings.

Table 11.4 Rating Factors for Truss Connection Capacities

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	2.48	3.76
Tri-Axle	1.93	2.93
Concrete Truck	2.18	3.31
3S3 (AL)	2.00	3.03
3S2 (AL)	2.11	3.19
School Bus	5.87	8.90

Table 11.5 Rating Factors for Stringers

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.66	0.99
Tri-Axle	0.47	0.70
Concrete Truck	0.53	0.79
3S3 (AL)	0.64	0.97
3S2 (AL)	0.75	1.13
School Bus	1.28	1.94

There are eight floorbeams (B28x104) in each truss span designated from west to east as FB0 to FB7. Cross sectional properties of the floorbeam were taken from AISC (Iron and Steel Beams 1985). The bending moments and shearing forces carried by the floorbeams were evaluated by assuming the riveted connections at the member ends to act as simple supports. The compression flanges were assumed to be supported at the stringer connection points. Summaries of the minimum inventory and operating rating factors for the truss floorbeams are given in Table 11.6. These rating factors were based on the flexural capacities of the members. It was found that rating factors based on the shear capacities of the members and the stringer to floorbeam and floorbeam to truss connections (including the combined shear and tension analysis) are substantially higher than those presented in Table 11.6. Hence, those rating factors are not presented in this report.

As can be seen from Table 11.6, the Present–Condition rating factors for floorbeams are slightly lower than those of As–Designed rating factors. This is a reflection of minor corrosion losses in the top flanges of floorbeams as reported on page 27 and 31 of Field Notes. Reduced section properties of FB7 in span 4 due to corrosion pitting of the top flange (page 31 of Field Notes) controlled the Present–Condition ratings.

CONCRETE DECKS ON TRUSS SPANS

There are no structural drawings available that show the reinforcing details used in the reinforced concrete bridge deck. It was not possible to calculate quantitative rating factors for the deck.

The field inspection results indicate that the concrete bridge deck was in satisfactory condition. The top of the deck exhibited medium scale and a number of transverse 1/16 inch wide cracks about every 4 to 5 feet with a few fairly large size scaled areas found in span 4 and 5. The underside of the deck displayed minor honeycombing. Map cracking was visible on about 15 percent of span 5. Numerous transverse cracks were found that are about 1 foot long with efflorescence indicating moisture is seeping through from above. The condition of the deck, however, does not appear to require a reduction of the load ratings of the bridge.

CONCLUSIONS AND RECOMMENDATIONS

Structural analyses and load ratings of the bridge indicate that the final bridge ratings are controlled by the flexural capacity of the stringers. A summary of the Present–Condition ratings for the bridge is given in Table 11.7. The field inspections also identified the following maintenance and repairs that should be carried out:

1. The truss members have sustained significant vehicular impact damage through the years. Since the bridge now carries only a single lane of traffic, it is expected that future occurrences of impact damage due to lateral clearance will be reduced. Impact damage to the end portals and

Table 11.6 Rating Factors for Truss Floorbeams

Truck Type	As-Designed Inventory	As-Designed Operating	Present-Condition Inventory	Present-Condition Operating
Two Axle	1.25	1.91	1.18	1.82
Tri-Axle	0.91	1.40	0.86	1.33
Concrete Truck	1.03	1.58	0.98	1.51
3S3 (AL)	1.36	2.09	1.29	1.98
3S2 (AL)	1.44	2.20	1.36	2.10
School Bus	3.13	4.80	2.96	4.60

Table 11.7 Summary of Present-Condition Ratings for Lauderdale County Structure No. 002-39-008.7A

Truck Type	Standard Weight (tons)	Inventory Rating Factor	Inventory Rating (tons)	Operating Rating Factor	Operating Rating (tons)
Two Axle	29.5	0.66	19.5	0.99	29.5
Tri-Axle	37.5	0.47	18	0.70	26
Concrete Truck	33	0.53	17	0.79	26
3S3 (AL)	42	0.64	27	0.97	41
3S2 (AL)	40	0.75	30	1.13	>40
School Bus	12.5	1.28	>12.5	1.94	>12.5

sway frames due to overheight vehicles may be expected to continue. All damaged truss members, end portals, and sway frames should be repaired by heat straightening the lesser damaged members and heat straightening with the addition of reinforcement plates to the more severely damaged (torn) members. The cracked clip angle connecting the vertical sway bracing to the vertical truss member at panel point 4 of span 5 should be repaired.

2. The fascia stub cantilever stringers should be reinforced to restore capacity lost to corrosion.

3. The existing railing should be replaced to meet the current standards.

4. The cracks in the concrete deck should be sealed by epoxy injection or by flooding the deck with MMA polymer.

5. The spalls in the fascia of the deck over the floorbeams should be repaired.

6. All welding performed on truss members in tension should be monitored for onset of fatigue distress.

7. The riprap at the west abutment should be repaired and protection installed at the east abutment.

8. The debris on the bridge deck should be removed and deck drains unclogged.

9. The horizontal gusset plates that attach the lateral bracing system to the end floorbeams should be reinforced.

10. New approach guardrails should be constructed that meet the current standards of strength, transition, and termination.

CHAPTER TWELVE
COFFEE COUNTY
STRUCTURE NO. 125–16–000.2

Coffee County Structure No. 125–16–000.2 was constructed in 1940 and currently carries Alabama Route 125 over the White Water Creek. The bridge consists of 8 steel stringer approach spans (1 through 6, 8 and 9) and a one span camel back pony truss (span 7). A sidewalk runs the full length of the bridge along the west side. An elevation of the bridge illustrating the arrangement of the spans and span numbering is shown in Figure 12.1. A summary of the structural analyses and load rating results are presented in this chapter. Results from the field inspection and condition survey are summarized by A.G. Lichtenstein & Associates (Coffee County 1991).

STRINGER SPAN RATINGS

There is a total of eight steel stringer spans (six at south side and two at north side). Spans 5 and 6 are slightly skewed, and spans 8 and 9 are sharply skewed. A typical cross section of steel stringer approach spans is shown in Figure 12.2. Based on the time of construction the steel members are assumed to have a yield strength of 33 ksi as suggested by AASHTO (Manual 1983). Most steel stringer bridges with a concrete deck constructed in the early 1940's did not use any mechanical shear studs between the steel stringer and the concrete deck. Lacking evidence of shear studs, non–composite section properties were used in the rating calculations.

The two fascia stringers (designated as S1 and S5) and three inside stringers (designated as S2, S3, and S4) are I–shape girders that can not be matched with any available beam designations. The cross sectional area and moment of inertia of the fascia and inside stringers were calculated from measured dimensions. The cross section properties for the stringers are given in Table 12.1. Due to the sharply skewed deck geometry of span 9, the stringer span lengths vary from 36 feet 6 inches to 41 feet 3 3/4 inches for the north and south side fascia stringer, respectively. The rating calculations were based on the length of the longest stringer. The allowable flexural stresses (inventory and operating) were determined assuming the compression flanges of the stringers are continuously braced by the concrete deck. Although the top flange of the stringer is not embedded into the concrete deck, the friction and bond between the top flange and the concrete deck was assumed sufficient to brace the compression flange.

The flexural capacity of the south side stringer in span 9 controlled the ratings. A summary of As–Designed rating factors for the approach steel stringer spans is given in Table 12.2. The field inspection (Coffee County 1991) revealed that the steel approach stringers were found in generally fair

Table 12.1 Section Properties for Approach Span Stringers

	Stringers	
	S1, S5	S2, S3, S4
d (in)	24	23-3/4
b_f (in)	9	9
t_f (in)	13/16	11/16
t_w (in)	7/16	7/16
A (in2)	24.4	22.16
I (in4)	2,374	2,088

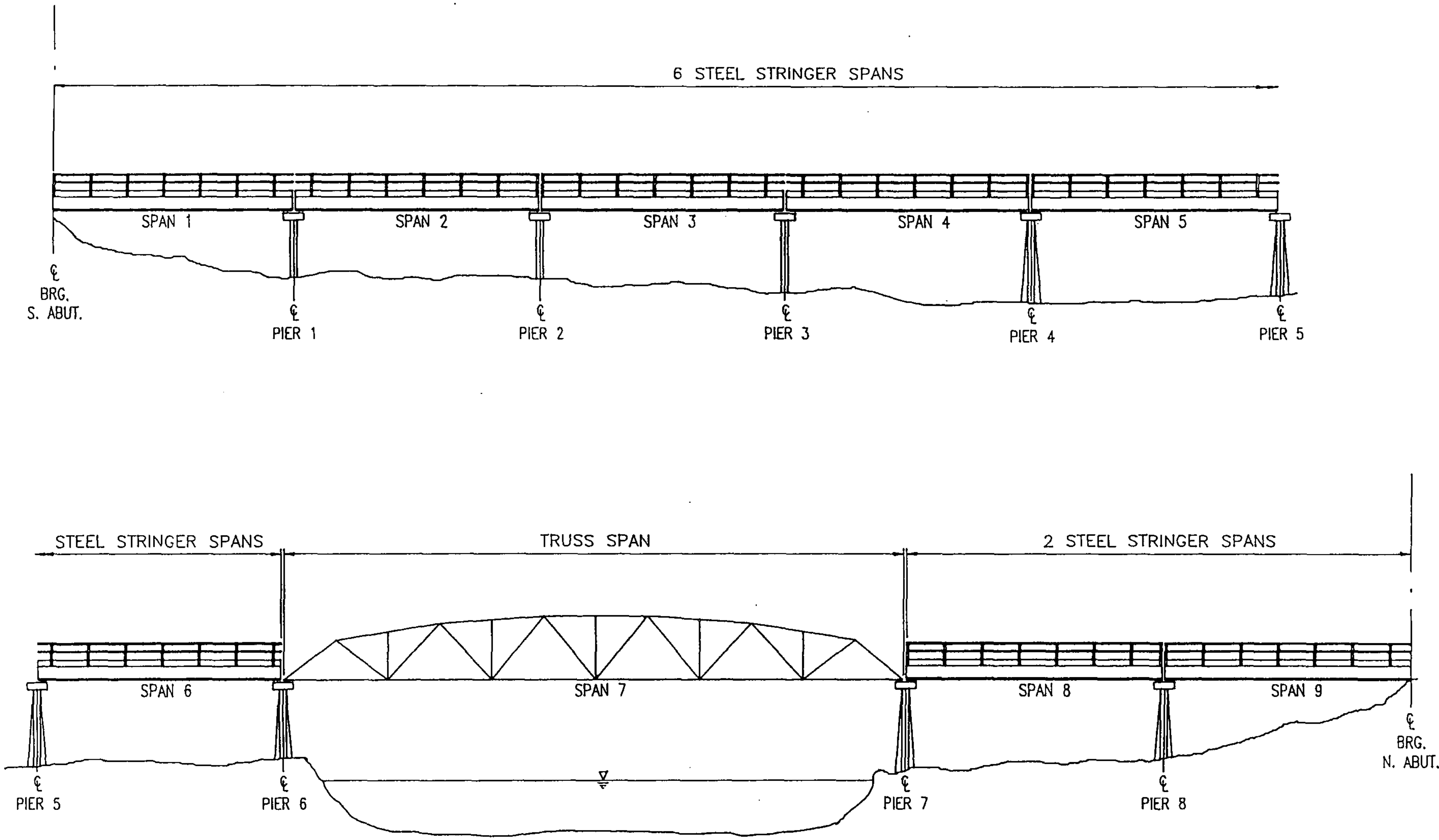
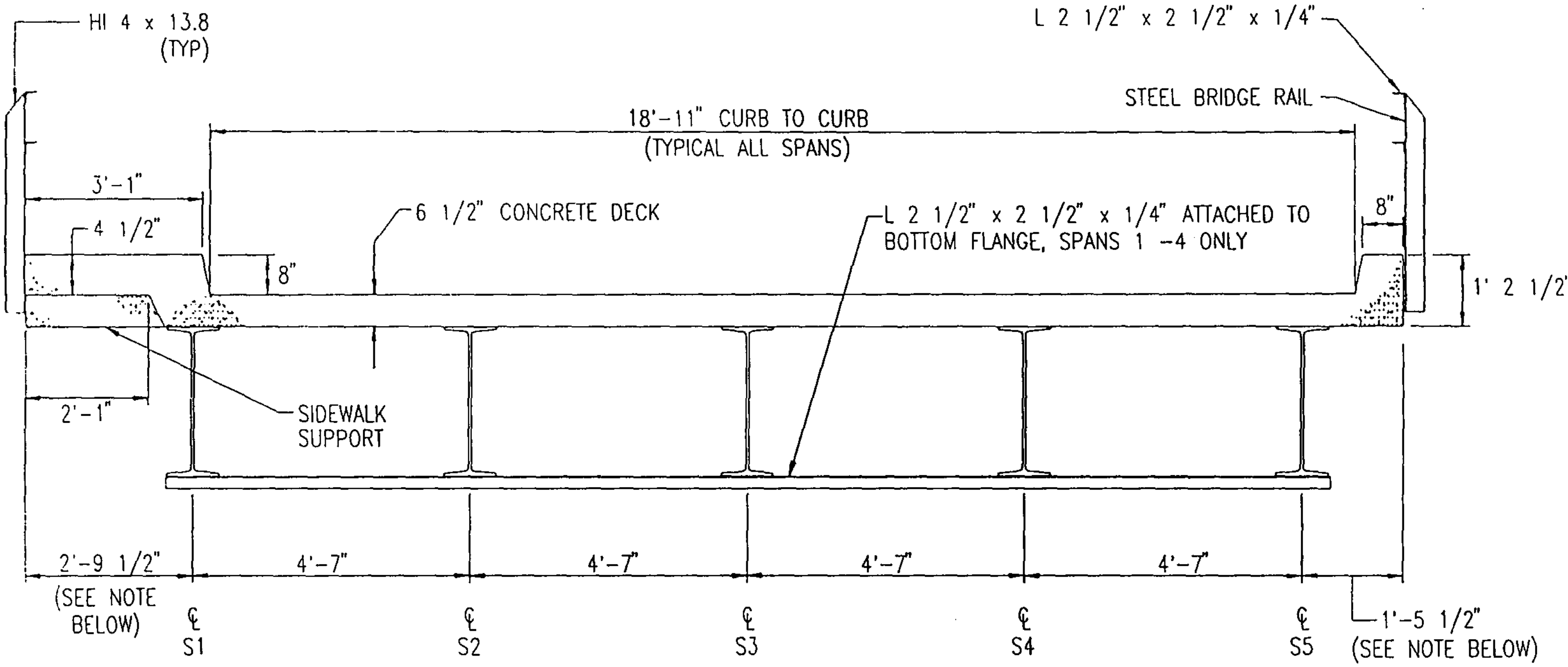


Figure 12.1. East Elevation of Elba Bridge (125-16-000.2)



NOTE: DIMENSION VARIES FOR SPANS 3,4,5,6,8 & 9 DUE TO CURVATURE OF THE DECK.

Figure 12.2. Cross Section at Steel Stringer Approach Spans

condition with light rust and no significant losses except under the open joints above each pier. The stringers under open joints typically displayed rust and pitting along the base of the web and the top of the bottom flange. The worst case of this condition occurred in span 8 at the south end of stringer 5; the web was pitted 3/16 inch deep by 5 inches wide. Reevaluation of the flexural capacities of the stringers using the damage details identified on pp. 44 and 54 of Field Notes revealed none of these minor rust conditions controlled the ratings. It was concluded that the As–Designed rating factors given in Table 12.2 are sufficiently accurate for use as Present–Condition ratings.

TRUSS RATINGS

An elevation showing the overall geometry, support conditions and joint numbering for the camel back pony truss is given in Figure 12.3. A typical cross section of the trusses is shown in Figure 12.4. Cross sectional properties for the truss members are given in Table 12.3.

Structural analyses of the truss were performed using the in–house computer program BRIDGE to determine the dead load and live load stresses in each of the members. The truss span length (100 feet 3 inches) is less than 200 feet, therefore, only one standard truck was placed in each lane per span as per provision 5.2.2 of AASHTO (Manual 1983). A summary of the dead load and maximum live load plus impact stresses is given in Table 12.4. Tri–axle dump truck loading controlled the maximum live load stresses in all members.

Summaries of the minimum As–Designed inventory and operating rating factors calculated for the truss are given in Table 12.5. These ratings are based on the axial stresses created in the truss members by the standard truck loadings. The compression capacity of member U1U2 controlled the ratings for all truck loadings.

The corrosion and impact damage identified during the field inspection was considered in determining Present–Condition ratings for the trusses. There was a great deal of minor to moderate corrosion and impact damage to the truss. The bottom chords exhibited medium to heavy pitting (1/8 to 1/4 inch) along the top portion of the members as illustrated in photo 14 of the Condition Report (Coffee County 1991). None of the truss members had a sufficient loss of section to affect the truss ratings, although some individual member capacities were reduced. The vertical, L4U4 of the east truss sustained the most severe impact damage. The west (roadway side) flange of the member was torn totally through and the web was dented one inch to the north along a length of 9 inches, centered 24 inches down from the centerline of the top chord. The attachment of a repair plate to the damaged flange as shown in photo 15 of the Condition Report appears to be adequate. As can be seen from Table 12.4, the member is subjected to very low stresses (the

Table 12.2 Rating Factors for Steel Stringer Approach Spans

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.83	1.29
Tri-Axle	0.61	0.94
Concrete Truck	0.69	1.07
3S3 (AL)	0.90	1.39
3S2 (AL)	1.10	1.70
School Bus	2.05	3.18

Table 12.3 Truss Member Cross Section Properties

Member	Area ($inch^2$)	I_x ($inch^4$)	I_y ($inch^4$)
L0L2	9.84	24.1	369.8
L2L4	15.56	52.4	481.3
L4L6	19.46	63.8	586.0
L0U1,U1U3	15.59	57.5	542.1
U3U4,U4U6	21.76	133.5	796.8
L2U2,L4U4,L6U6	8.81	17.5	289.6
U1L2	9.84	24.1	369.8
L2U3,L4U3	6.62	16.9	264.9
L4U5,L6U5	4.80	12.5	195.2

Table 12.4 Dead Load Stresses and Maximum Live Load Stresses

Member	Dead Load (ksi)	Maximum Live Load Plus Impact (ksi)
L0L2	9.15	9.50
L2L4	10.45	10.09
L4L6	9.69	9.16
L0U1	-7.24	-7.50
U1U2	-9.50	-9.92
U2U3	-9.48	-9.91
U3U4	-8.65	-8.87
U4U5	-8.65	-8.87
U5U6	-9.16	-9.32
L2U1	7.12	7.51
L2U2	0.07	0.17
L2U3	-3.72	-7.40
L4U3	5.60	8.57
L4U4	-0.12	0.01
L4U5	-0.34	-8.13
L6U5	3.59	10.71
L6U6	-0.14	0.00

Table 12.5 Rating Factors for Truss Members

Truck Type	As-Designed/Present-Condition	
	Inventory	Operating
Two Axle	0.69	1.17
Tri-Axle	0.54	0.91
Concrete Truck	0.61	1.03
3S3 (AL)	0.56	0.95
3S2 (AL)	0.60	1.01
School Bus	1.64	2.77

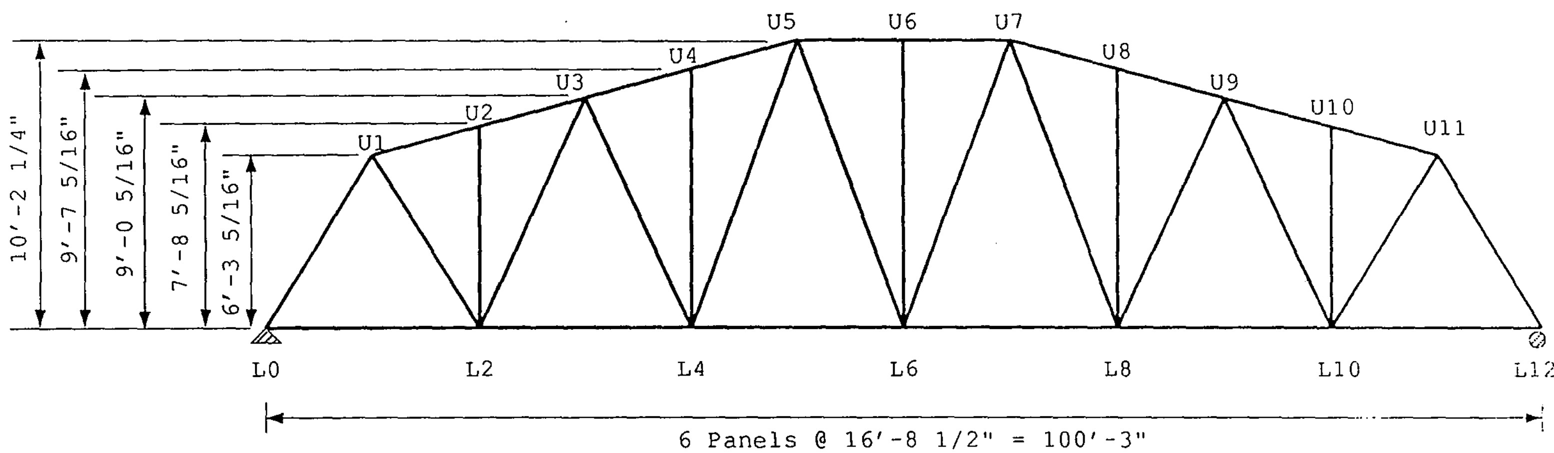


Figure 12.3. Elevation of Pony Truss

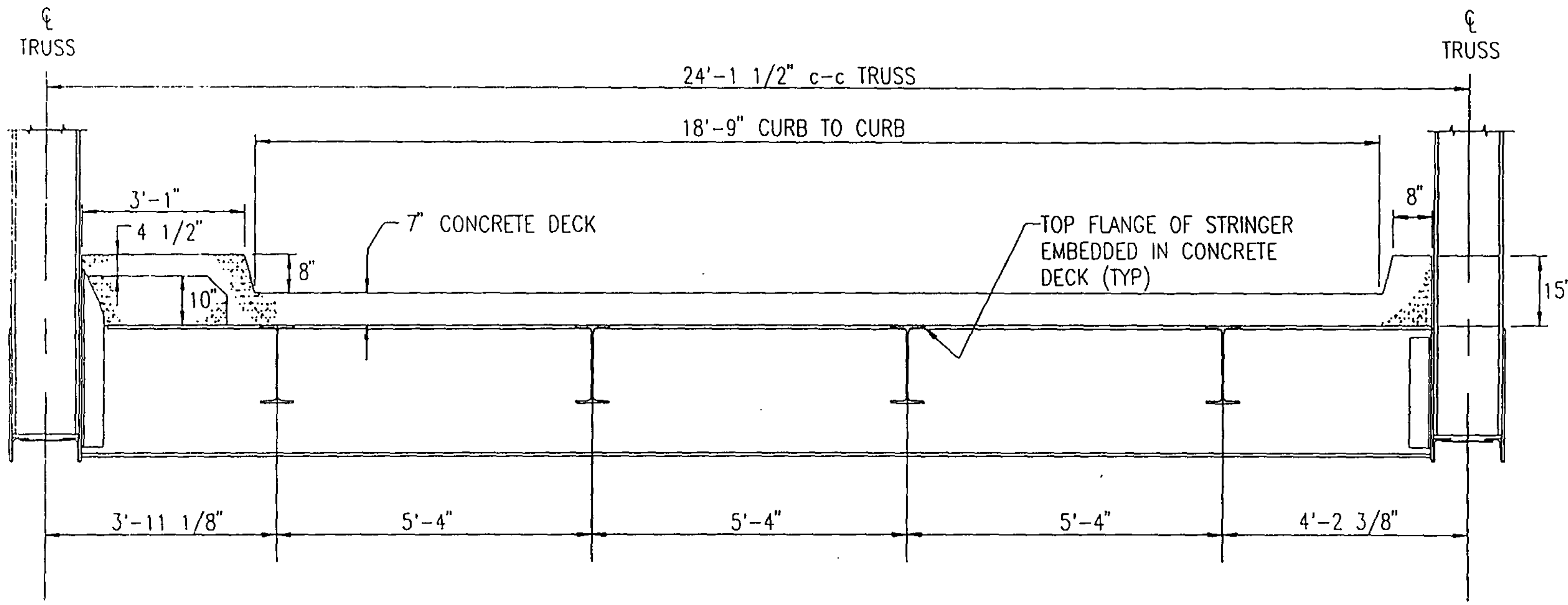


Figure 12.4. Cross Section at Pony Truss Span (Looking North)

member carries only very low dead load stress). Hence, the damage to member L4U4 does not affect the final rating of the truss. It is judged that the As–Designed rating factors given for the truss members in Table 12.5 are sufficiently accurate for use as Present–Condition ratings.

Capacities of the truss connections were also evaluated. The connections at the ends of tension members, L0L2, L2L4, L4L6, L2U1, L4U3, and L6U5 were selected as the most critical. The head diameter of rivets was measured as 1 1/4 inches (pp. 17 of Field Notes). The shank diameter of the rivets was taken as 3/4 inch from page 4–135 of AISC (1980). The number of rivets and the condition of the connections were taken from a series of field photos (B–12, B–16, B–17, B–19, B–20, C–17, E–14, E–15) and Field Notes (pp. 25 and 29). It appears that the condition of the connections is generally fair with no significant corrosion damage that would adversely affect the ratings.

It was found that the connection of the member L4L6 at the joint L4 controlled the connection ratings. As–Designed and Present–Condition rating factors for the truss connections are summarized in Table 12.6. The rating factors were calculated using AASHTO's highest allowable stresses by assuming the rivets are Grade 2 rivets. The ratings shown are based on the shear capacity of the rivets. Ratings based on bearing stresses were found to be much higher than those shown in Table 12.6. As can be seen from Table 12.6, the rating factors for truss member connections are just about even with those for the truss member capacities although Grade 2 rivets were assumed in the rating calculations. Rating factors calculated by assuming the rivets are Grade 1 are very small, near zero for some connections. Because it is not known whether Grade 1 or Grade 2 rivets were used in the connections, it would be conservative to assume that Grade 1 rivets were used. It is suggested that such a conservative assumption is not necessary since no distress of the riveted connections was found during the bridge inspection. It is recommended that load restrictions not be placed on the bridge based on the connection capacities. If future inspections reveal distress of the connections, this recommendation should be reconsidered.

TRUSS FLOOR SYSTEM RATINGS

A typical section through the truss span floor system is shown in Figure 12.4. There are four W16x36 stringers spaced at 5 feet 4 inches center to center. Structural analyses were performed on the stringers using the in–house computer program BRIDGE assuming each stringer to be a simple beam with a span length of 16 feet 8 1/2 inches. The compression flanges of the stringers were embedded in the underside of the concrete deck as shown in Figure 12.4. It was judged that the concrete deck provided sufficient lateral restraint so that the stringer compression flanges are continuously braced. Summaries of the minimum inventory and operating rating factors calculated

Table 12.6 Rating Factors for Truss Connection Capacities

Truck Type	As-Designed/Present-Condition	
	Inventory	Operating
Two Axle	0.55	1.21
Tri-Axle	0.43	0.95
Concrete Truck	0.49	1.08
3S3 (AL)	0.44	0.97
3S2 (AL)	0.46	1.02
School Bus	1.34	2.96

for the stringers are given in Table 12.7. These rating factors were based on the flexural capacities of stringers. It was found that rating factors based on the shear capacities of the members and the stringer to floorbeam connections are much higher than those presented in Table 12.7. The Condition Summary Report (Coffee County 1991) indicates that the stringers and floorbeams were in satisfactory condition with no significant rusting or paint loss. It was judged that the As–Designed rating factors given in Table 12.7 are sufficiently accurate for use as Present–Condition ratings.

There are five floorbeams in the truss span. The floorbeams span between the lower chords of the trusses at the interior panel points. Each floorbeam is a rolled shape CB27x91. Cross sectional properties of the shape were taken from AISC (Iron and Steel Beams 1985).

The bending moments and shearing forces carried by the floorbeams were evaluated by assuming the riveted connections at the member ends to act as simple supports. The compression flanges were assumed to be supported at the stringer connection. Summaries of the As–Designed minimum inventory and operating rating factors for the truss floorbeams are given in Table 12.8. These rating factors were based on the flexural capacities of the members. It was found that rating factors based on the shear capacities of the members and floorbeam to truss connections (including the combined shear and tension analysis) are substantially higher than those presented in Table 12.8. Hence, those rating factors are not presented in this report. Although minor corrosion damage is reported (pp. 43–44, Field Notes), it was judged that the rating factors given in Table 12.8 are sufficiently accurate for use as Present–Condition rating factors.

CONCRETE DECKS ON TRUSS SPANS

There are no structural drawings available that show the reinforcing details used in the reinforced concrete bridge deck. It was not possible to calculate quantitative rating factors for the deck.

The field inspection results indicate that the concrete bridge deck was in satisfactory condition. The top of the deck displayed hairline transverse cracks at 4 feet 6 inches intervals for the full length of the truss span with a hairline longitudinal crack along the east curb line. The underside of the deck was found to be in fair condition. Random areas of hairline transverse cracks were present in all spans. Occasionally these cracks were accompanied by leaching. Discoloration of the concrete deck was present under all open deck joints, but no major spalls were found, and no hollow areas were detected when the concrete was sounded with a hammer. The condition of the deck does not appear to require a reduction of the load ratings of the bridge.

Table 12.7 Rating Factors for Truss Stringers

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.73	1.30
Tri-Axle	0.54	0.97
Concrete Truck	0.59	1.04
3S3 (AL)	0.75	1.36
3S2 (AL)	0.84	1.44
School Bus	1.34	2.70

Table 12.8 Rating Factors for Truss Floorbeams

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.58	1.27
Tri-Axle	0.41	0.90
Concrete Truck	0.46	1.01
3S3 (AL)	0.58	0.95
3S2 (AL)	0.63	1.03
School Bus	1.27	2.08

CONCLUSIONS AND RECOMMENDATIONS

Rating calculations indicate that the load capacities of the riveted truss connections are very low when the allowable shear stress for Grade 1 rivets is used. The truss connections do not control the bridge ratings when the allowable shear stress for Grade 2 rivets is used. Because it is not known whether Grade 1 or Grade 2 rivets were used in the connections, it would be conservative to assume that Grade 1 rivets were used. It is suggested that such a conservative assumption is not necessary since no distress of the riveted connections was found during the bridge inspection. It is recommended that load restrictions not be placed on the bridge based on the connection capacities. If future inspections reveal distress of the connections, this recommendation should be reconsidered.

The suggested final bridge ratings are controlled by the flexural capacities of the floorbeams of the truss span. A summary of the Present–Condition ratings for the bridge based on the load capacity of the floorbeams is given in Table 12.9.

The field inspections also identified the following maintenance and repair that should be carried out:

1. River bank protection and river bed scour protection by the addition of stone riprap may be needed to prevent further erosion and slumping.

2. The cracked pier cap of pier 6 and the spall on pier 4 should be repaired. The void (4' long) beneath the north abutment breastwall should be filled with crushed stone.

3. All damaged truss members should be repaired by heat straightening the lesser damaged members (bent not torn) and heat straightening plus the addition of reinforcement plates to the torn and damaged members.

4. All welding performed on truss members in tension should be monitored for signs of distress during future inspections.

5. All damaged and deteriorated bridge railings should be repaired.

6. Approach guardrails should be installed at the ends of the structure to protect against impact of the end of the bridge rail.

Table 12.9 Summary of Present-Condition Ratings for Coffee County Structure No. 125-16-000.2

Truck Type	Standard Weight (tons)	Inventory Rating Factor	Inventory Rating (tons)	Operating Rating Factor	Operating Rating (tons)
Two Axle	29.5	0.58	17	1.27	>29.5
Tri-Axle	37.5	0.41	15	0.90	34
Concrete Truck	33	0.46	15	1.01	>33
3S3 (AL)	42	0.58	24	0.95	40
3S2 (AL)	40	0.63	25	1.03	>40
School Bus	12.5	1.27	>12.5	2.08	>12.5

7. New joint seals should be placed at all open joints on the approach stringer spans, above piers 1, 5, and 12, to prevent water and debris from falling on surfaces below the joint.

8. Install A325 high strength bolts in the 6 fabricator error holes of floorbeam 8 (FB8) of the truss span.

9. The field inspection crew noted that the approach roadway joints varied in width and 4 out of 8 joints were closed. AHD should monitor the joint openings at various temperatures to determine if the joints are working properly and that unusual movement patterns are not occurring. If joints are determined to remain in hard contact at low temperatures, then jacking and repositioning of selected spans may be necessary.

10. AHD may wish to consider replacing the frozen sliding bearing of the truss span with PTFE slider plate bearing. All deteriorated or missing anchor bolts and nuts should be replaced.

11. The flame cut copes at the ends of the stringers in the truss span should be carefully inspected for possible fatigue cracks during future bridge inspections.

CHAPTER THIRTEEN
TRUSS SPAN OF CHEROKEE COUNTY
STRUCTURE NO. 068–10–19.9

Cherokee County Structure No. 068–1–19.9 was constructed in 1930 and currently carries AL Route 68 traffic over the Chattooga River at Cedar Bluff, Alabama. The bridge consists of a 120 feet long Pratt thru–truss (span 3) with three steel stringer approach spans on the west approach and twelve steel stringer approach spans on the east approach. The roadway is 23 feet 7 inches curb–to–curb. An elevation of the bridge illustrating the arrangement of the spans and span numbering is shown in Figure 13.1. The bridge is currently posted for a 10 ton limit on all truck loads. A summary of the structural analyses and load rating results for the truss span are presented in this chapter. Ratings and condition inspection for steel stringer approach spans are not included in this project. Results from the field inspection and condition survey are summarized by A.G. Lichtenstein & Associates (Cherokee County 1992).

TRUSS RATINGS

The eight–panel Pratt truss has double diagonals in two center panels. An elevation showing the overall geometry, support conditions and joint numbering for the truss is given in Figure 13.2. A typical cross section of the truss is shown in Figure 13.3. Cross sectional properties for the truss members given in Table 13.1 were evaluated based on the Field Drawing sheet No. 2, Truss Member Sizes and Details, provided by A.G. Lichtenstein & Associates. Based on the time of construction the steel members are assumed to have a yield strength of 30 ksi as suggested by AASHTO (Manual 1983).

Structural analyses of the truss were performed using the in–house computer program BRIDGE to determine the dead load and live load stresses in each of the members. Because the truss span length (120 feet) is less than 200 feet only one standard truck was placed in each lane per span as per provision 5.2.2 of AASHTO (Manual 1983). A summary of the dead load and maximum live load stresses is given in Table 13.2. Top and bottom chords develop the maximum stress under a six axle vehicle (3S3). The tri–axle dump truck produces the maximum stresses in verticals and diagonals for which the influence lines are confined within the adjacent panels.

Summaries of the minimum As–Designed inventory and operating rating factors calculated for the truss members are given in Table 13.3. These ratings are based on the axial stresses created in the truss members by the standard truck loadings. The inventory ratings were controlled by tension member U2L3. The operating ratings were controlled by compression members U2L2 and U4L3.

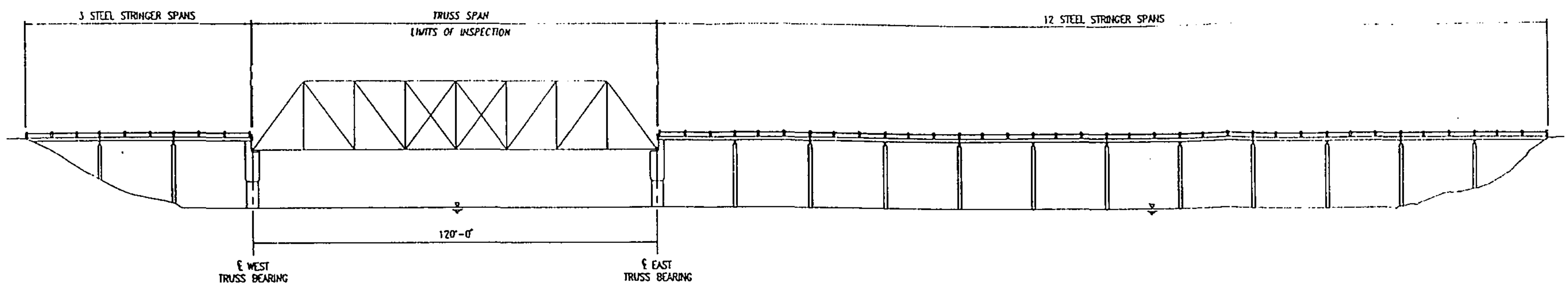


Figure 13.1. South Elevation of Cedar Bluff Bridge (068-10-19.9)

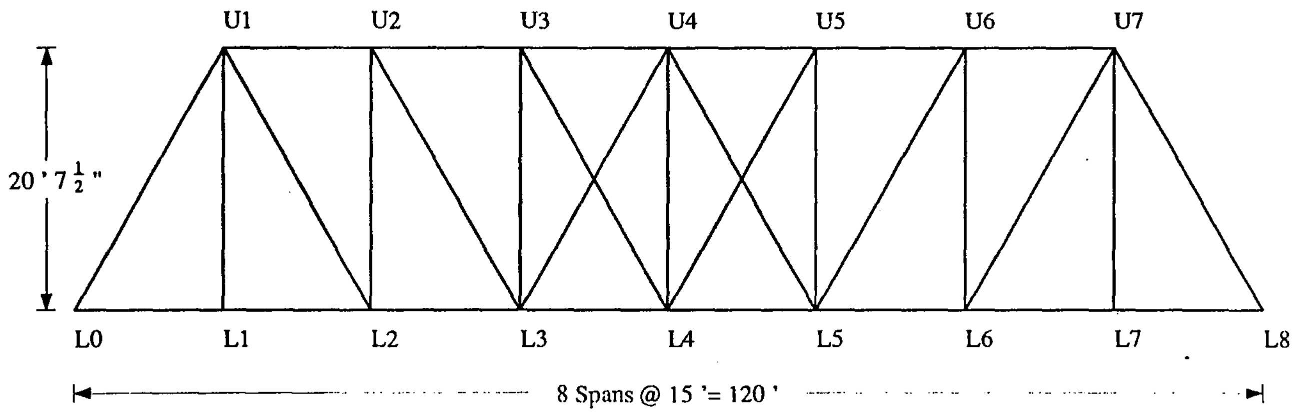


Figure 13.2. Elevation of Cedar Bluff Truss

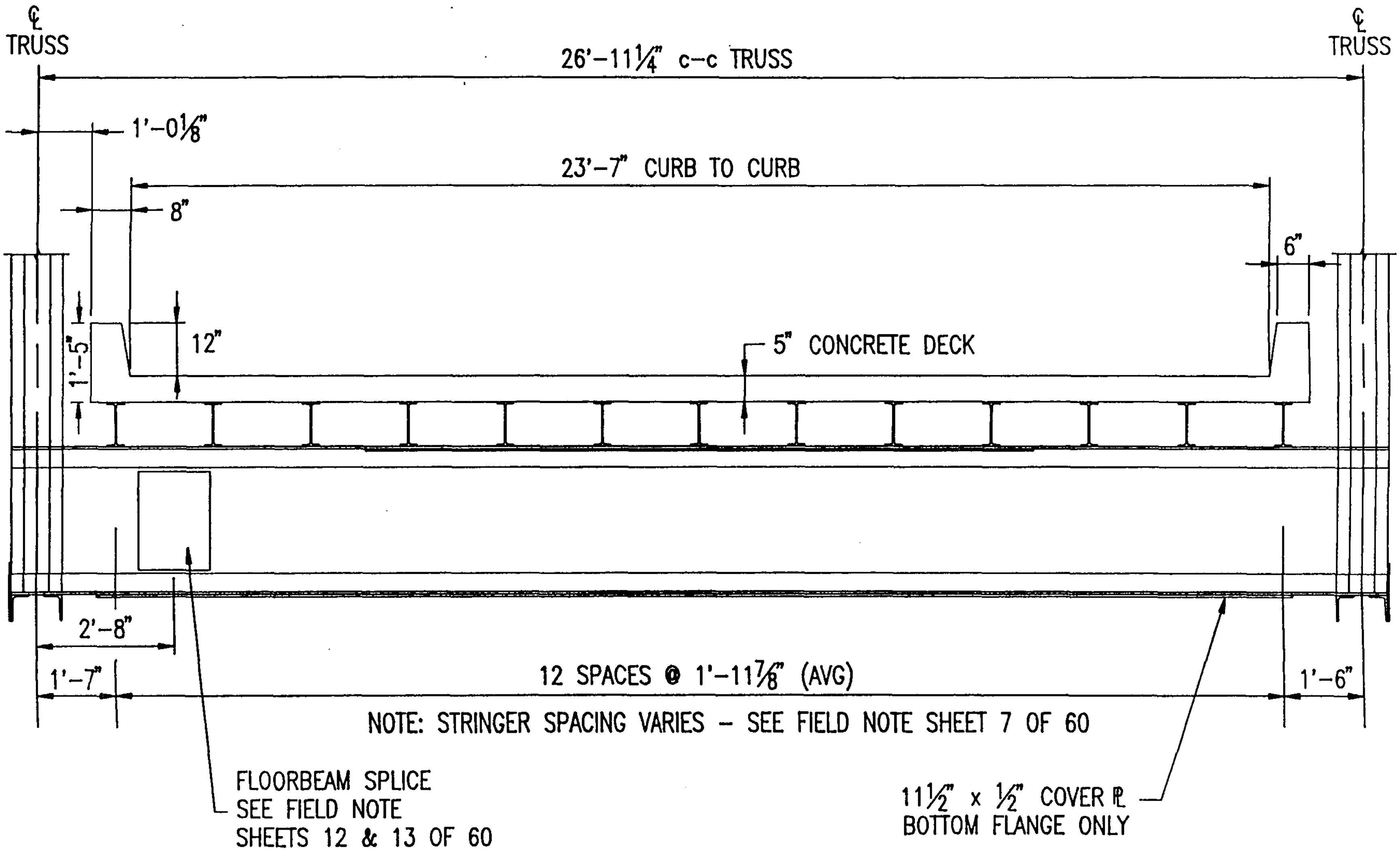


Figure 13.3. Cross Section at Cedar Bluff Truss Span

Table 13.1 Truss Member Cross Section Properties

Member	Area ($inch^2$)	I_x ($inch^4$)	I_y ($inch^4$)
L0U1	16.45	248.8	376.1
U1U2,U2U3 U3U4	17.40	259.9	397.7
L0L1,L1L2	7.94	29.5	88.4
L2L3	13.62	28.2	124.8
U1L1	6.50	70.2	10.1
L2U2,L3U3,L4U4	6.72	64.6	101.2
L3L4	17.72	57.7	150.4
U1L2	9.00	33.2	98.9
U2L3	6.10	15.6	66.5
U3L4	3.24	2.8	37.5
U4L3	3.56	4.4	42.1

Table 13.2 Dead Load Stresses and Maximum Live Load Stresses

Member	Dead Load (ksi)	Maximum Live Load Plus Impact (ksi)
L0L1	6.95	7.69
L1L2	6.95	7.69
L2L3	6.96	7.59
L3L4	6.93	7.65
L0U1	-5.75	-6.31
U1U2	-5.45	-5.95
U2U3	-6.82	-7.66
U3U4	-7.05	-7.55
L1U1	2.99	10.49
U1L2	7.53	9.77
L2U2	-5.20	-8.68
U2L3	6.70	11.82
L3U3	-1.10	-3.61
U3L4	2.14	9.27
L3U4	-1.98	-8.60
L4U4	1.42	5.00

Table 13.3 Rating Factors for Truss Members

Truck Type	As-Designed Inventory	As-Designed Operating	Present-Condition Inventory	Present-Condition Operating
Two Axle	0.99	1.57	0.64	1.29
Tri-Axle	0.79	1.24	0.50	1.01
Concrete Truck	0.88	1.40	0.56	1.14
3S3 (AL)	0.80	1.25	0.51	1.05
3S2 (AL)	0.83	1.31	0.53	1.07
School Bus	2.23	3.53	1.51	3.08

The corrosion losses found by the field inspection crew in the truss members were caused by localized rust, pitting, or impacted rust. The worst case noted was on the north truss in member L4U4 which displayed complete loss of the channel web section at the connection to the lower chord. Corrosion losses were also noted at the lower chord gusset plate at the same location. These corrosion losses were detailed in the Field Notes, pp. 43–44 of 60. There were many significant defects found on the truss mainly due to vehicular collision which had resulted in maintenance repair plates being added to the truss members throughout the truss span. Miscellaneous maintenance and repair welding was found on both tension and compression elements. Transverse cracks were noted in the diagonals L2U1 and L6U7 of the north truss. Member L6U7 has been replaced in–kind. Diagonal U2L1 should be monitored periodically by AHD personnel for any change in condition as noted in the Condition Report (Cherokee County 1992, pp. 5) and should be replaced in-kind as soon as possible. The 3 3/4 inch transverse crack in one of the two angles in the diagonal U2L1 severely limits the Present–Condition ratings given in Table 13.3. The rating results presented here should not be used until the member is replaced. Collision damage found on the upper lateral bracing, intermediate and end portals as detailed in the Field Notes (pp. 48 through 55/60). Except for the two end portals, L0U1 and L8U7, all of the collision damaged members were tension members. Therefore, the load carrying capacity of the tension members, including vertical member L7U7 of north truss which was twisted and kinked about 8 inches out of line was not significantly affected (see Photo 14 of the Condition Report (Cherokee County 1992) and in the Field Notes sheet 47). There were other numerous cracks found in secondary members (intermediate portals) as summarized in the Condition Report (Cherokee County 1992, pp.5). While the secondary members are not considered for load rating calculations, it is suggested that these members be repaired in the near future.

Capacities of the truss connections were also evaluated. The connection on every joint of the truss was examined based on the details given in the Field Notes (pp. 14 through 25) and field photographs. The head diameter of rivets was measured as 1 1/4 inches (pp. 14 through 23 of Field Notes). The shank diameter of the rivets was taken as 3/4 inch from page 4–135 of AISC (1980). It appears that the condition of the connections is generally fair with no significant corrosion damage that would adversely affect the ratings. The rivets were conservatively assumed to be ASTM A502 Grade 1 rivets. It was found that the connection at joint L2 controlled the connection ratings. Rating factors for the truss connections are summarized in Table 13.4. The ratings shown are based on the shear capacity of the rivets. Ratings based on bearing stresses were found to be much higher than those shown in Table 13.4.

Table 13.4 Rating Factors for Truss Connection Capacities

Truck Type	As-Designed/Present Condition Inventory	Operating
Two Axle	1.07	1.76
Tri-Axle	0.84	1.38
Concrete Truck	0.95	1.56
3S3 (AL)	0.87	1.43
3S2 (AL)	0.89	1.46
School Bus	2.56	4.19

TRUSS FLOOR SYSTEM RATINGS

A typical section through the truss span floor system is shown in Figure 13.3. Between floorbeams there are typically thirteen stringers of S9x20.5. Cross sectional properties for the stringers were taken from AISC (Iron and Steel Beams 1985). The spacing of these stringers varies from 1 foot 6 inches at the exterior to an interior maximum of 2 feet 1 13/16 inches center to center as shown in the Field Notes (pp. 7). As per provision 3.23.2.3.1.5 and Table 3.23.1 of AASHTO (1989), the wheel load distribution factor was determined by S/5.5 (S being 2 feet 1 3/16 inches). Structural analyses were performed on the stringers using the in–house computer program MOMENTS assuming each stringer to be a simple beam. The top flanges of the stringers were in contact with the underside of the deck. There were no signs of lateral movement of the top (compression) flange, so the top flanges were assumed to be continuously braced. Summaries of the minimum inventory and operating rating factors calculated for the stringers are given in Table 13.5. These factors were controlled by the absolute maximum positive moment carrying capacity of the stringer. Rating factors for the shear capacity were also evaluated. The resulting rating factors were substantially higher than those given in Table 13.5 for moment capacity. Hence they are not presented in this report. The stringers were found to be in generally fair condition with no significant losses as shown in photographs 11 and 21 of the Condition Summary Report (Cherokee County 1992). It was judged that the As–Designed rating factors given in Table 13.5 are sufficiently accurate for use as Present–Condition ratings.

There are nine floorbeams in the truss span designated as FB0 to FB8 from the west to the east. The floorbeams span between the lower chords of the trusses at the interior panel points. Each floorbeam is a built–up plate girder with two 5x3 1/2x1/2 angles each for the top and bottom flanges connected by a 26 1/2 x 7/16 web plate. Cover plates are added to the top and bottom flanges. Details for the floorbeam are shown in Figure 13.3, Field Drawing sheet No. 2, and Field Notes (pp. 7). Cross sectional properties were evaluated based on dimensions taken from these three details.

The bending moments and shearing forces were evaluated by assuming the riveted connections at the floorbeam ends to act as simple supports. The compression flanges were assumed to be braced by the stringers. Summaries of the minimum inventory and operating rating factors for the truss floorbeams are given in Table 13.6. These rating factors were based on the flexural capacities of the members. It was found that rating factors based on the shear capacities of the members and the stringer to floorbeam and floorbeam to truss connections are substantially higher than those presented in Table 13.6. Hence, those rating factors are not presented in this report. Examination of the Field Notes, the Condition Summary Report (Cherokee County 1992), and photographs reveals that there are no significant losses or damage noted for the floorbeams which would affect the live load rating. It was judged that the

Table 13.5 Rating Factors for Truss Floor Stringers

Truck Type	As-Designed/Present Condition Inventory	Operating
Two Axle	0.69	1.03
Tri-Axle	0.51	0.77
Concrete Truck	0.55	0.82
3S3 (AL)	0.72	1.08
3S2 (AL)	0.78	1.17
School Bus	1.21	1.81

Table 13.6 Rating Factors for Truss Floorbeams

Truck Type	As-Designed/Present Condition Inventory	Operating
Two Axle	1.03	1.55
Tri-Axle	0.71	1.08
Concrete Truck	0.83	1.24
3S3 (AL)	0.98	1.48
3S2 (AL)	1.08	1.63
School Bus	2.10	3.17

rating factors given in Table 13.6 are sufficiently accurate for use as Present–Condition rating factors.

CONCRETE DECK ON TRUSS SPAN

Reinforcement details for the truss span deck were not available. Therefore, a quantitative rating for the concrete deck was not possible. The field inspection results as summarized in the Condition Summary Report (Cherokee County 1992) indicate that the concrete deck appears to be in generally fair condition. The underside of the deck typically exhibited random transverse hairline cracks and minor areas of map cracking. The top of the concrete deck was found to be in fair condition with moderate to heavy wear and exposed aggregate throughout. Random areas of hairline transverse cracks and small ruts are also present. It was judged that the condition of the concrete deck will not adversely affect the live load carrying capacity of the bridge.

CONCLUSIONS AND RECOMMENDATIONS

Structural analyses and load ratings of the truss indicate that the final truss ratings are controlled by the flexural capacity of the stringers in the truss floor system. A summary of the Present–Condition ratings for the truss is given in Table 13.7. The field inspection also identified the following maintenance and repairs that should be carried out:

1. Continue monitoring of crack at the north truss diagonal L2U1 and schedule in–kind replacement of the same as soon as possible. The rating results presented in this report should not be used until member L2U1 is replaced.

2. The truss members have sustained significant vehicular impact damage throughout the years. All damaged members, end portals, and sway frames should be repaired by heat straightening the lesser damaged members and heat straightening with the addition of reinforcement plates to the more severely damaged members. The cracked angles connecting the intermediate portals to the truss members should be repaired.

3. New joint seal should be placed at all open joints on the truss span to prevent water and debris from falling on surfaces below the joint.

4. All damaged railings should be repaired.

5. Clean all rocker bearings of debris/rust and lubricate.

6. Secure all loose or missing nuts on the stringer/floorbeam connections.

Table 13.7 **Summary of Present-Condition Ratings for Truss Span (Cherokee County Structure No. 068-10-19.9)**

Truck Type	Standard Weight (tons)	Inventory Rating Factor	Inventory Rating (tons)	Operating Rating Factor	Operating Rating (tons)
Two Axle	29.5	0.69	20	1.03	>29.5
Tri-Axle	37.5	0.51	19	0.77	29
Concrete Truck	33	0.55	18	0.82	27
3S3 (AL)	42	0.72	30	1.08	>42
3S2 (AL)	40	0.78	31	1.17	>40
School Bus	12.5	1.21	>12.5	1.81	>12.5

7. Revise vertical clearance sign on the west approach and post the vertical clearance on the east approach.

8. Repair deck spall (westbound lane between panel point L5 and L6) to reduce the likelihood of additional deterioration of the deck.

9. Repair all spalled areas in the piers and epoxy inject all vertical and horizontal cracks. Continue monitoring piers for additional crack development.

CHAPTER FOURTEEN
TRUSS SPAN OF WILCOX COUNTY STRUCTURE NO. 28–66–12.0

Wilcox County Structure No. 28–66–12.0 was constructed in 1929 and currently carries AL Route 28 traffic over the Alabama River (William Donnelly Reservoir) in the town of Millers Ferry. The bridge consists of 89 steel stringer approach spans (1–40, 46–94), four Warren deck truss approach spans (41,42,44 and 45, truss spans 1,2,4 and 5), and one main span through truss (43, truss span 3). The four approach trusses have a span length of 160 feet, and the main span, which provides the greatest vertical clearance for navigational traffic (measured to be 48 feet 9 inches on January 24, 1992), has a span length of 160 feet 1 inch as shown in Figure 14.1. All of the truss spans are supported by concrete piers, and the steel stringer approach spans are supported by steel bents. The Millers Ferry lock and dam, located approximately 0.5 miles downstream (south) of the bridge, controls the water level of the reservoir and the passage of navigational traffic. The bridge is currently posted for a maximum HS20–44 load of 30 tons. A summary of the structural analyses and load rating results for the truss spans are presented in this chapter. Ratings for steel stringer approach spans are not included in this project. Results from the field inspection and condition survey are summarized by A.G. Lichtenstein & Associates (Wilcox County 1992).

TRUSS RATINGS

The five truss spans show three distinct geometries although all the span lengths are approximately 160 feet. Elevations showing the overall geometry, support conditions and joint numbering for these three trusses are given in Figures 14.2 through 14.4. Truss spans 1 and 5 are typical eight–panel Warren trusses. Truss spans 2 and 4 are modified six–panel Warren trusses. There are five panels of 26 feet, and the last panel length between L5 and L6 is 30 feet. The heights of the verticals also vary. Truss span 3 has variable panel lengths and heights. Truss spans 2, 3, and 4 constitute a three–span continuous configuration. Typical cross sections of the trusses are shown in Figures 14.5 through 14.7. Cross sectional properties for the truss members, given in Tables 14.1 through 14.3, were evaluated based on Field Drawing sheet Nos. 2, 3, 4 provided by A.G. Lichtenstein & Associates. Based on the time of construction the steel members are assumed to have a yield strength of 30 ksi as suggested by AASHTO (Manual 1983).

Structural analyses of the trusses were performed using the in–house computer program BRIDGE to determine the dead load and live load stresses in each of the members. Because the truss span lengths (160 feet) were less than 200 feet only one standard truck was placed in each lane per span as per provision 5.2.2 of AASHTO (Manual 1983). A summary of the dead load and

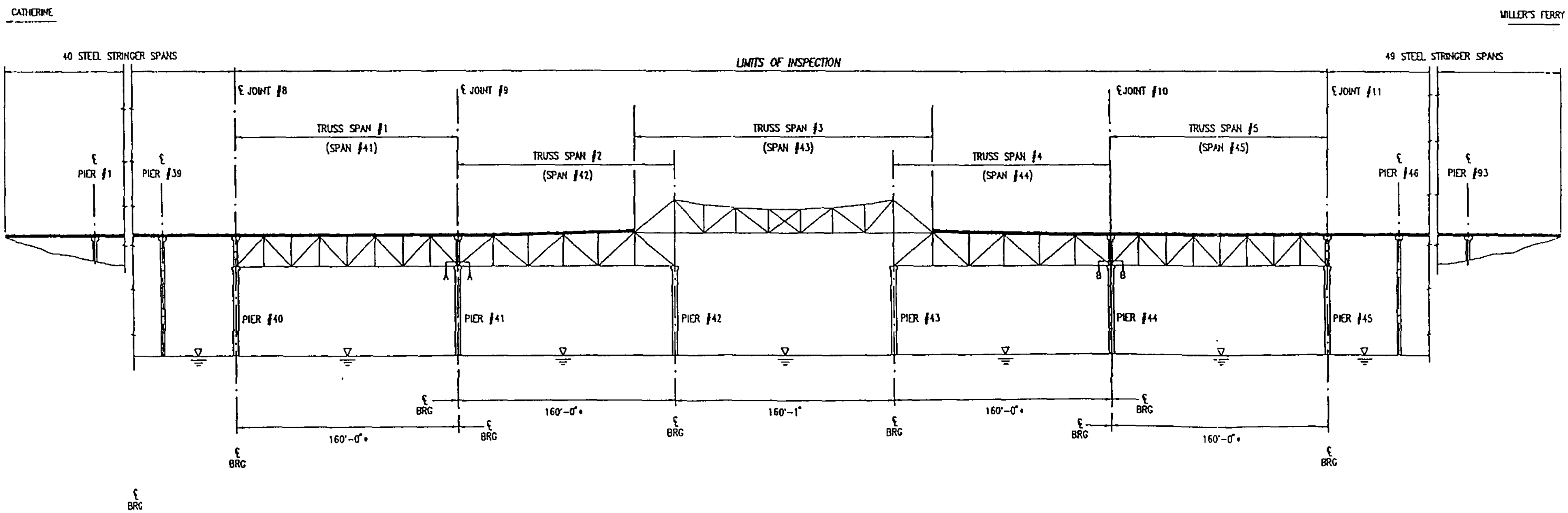


Figure 14.1. South Elevation of Millers Ferry Bridge (28-66-12.0)

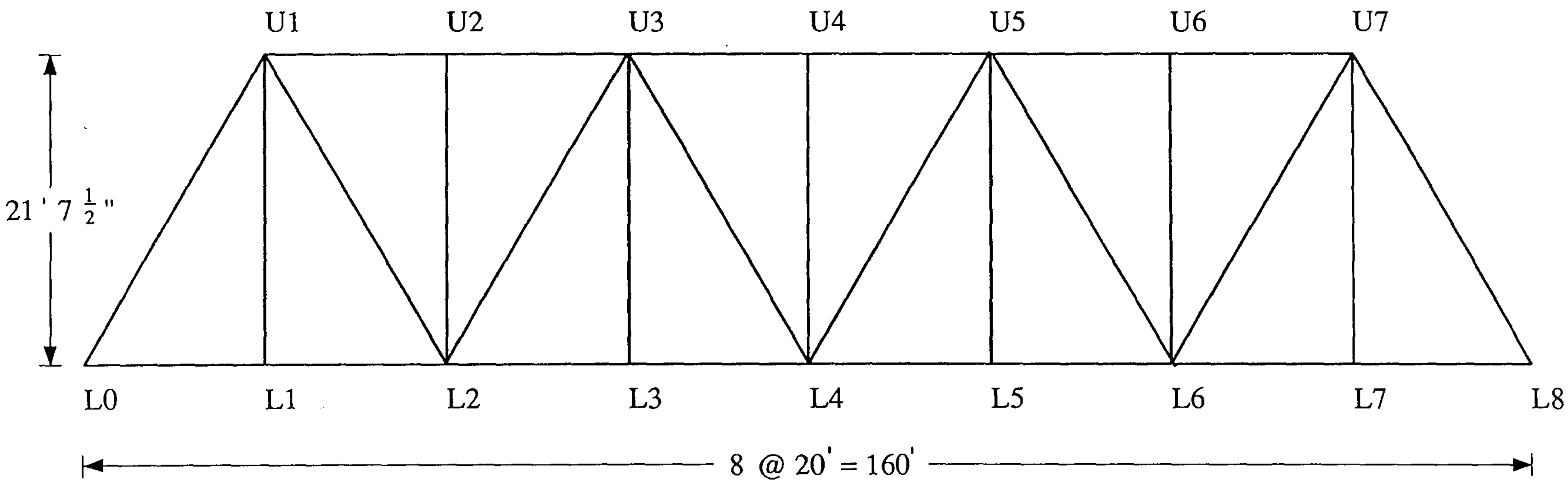


Figure 14.2. Elevation of Truss Spans #1 and #5

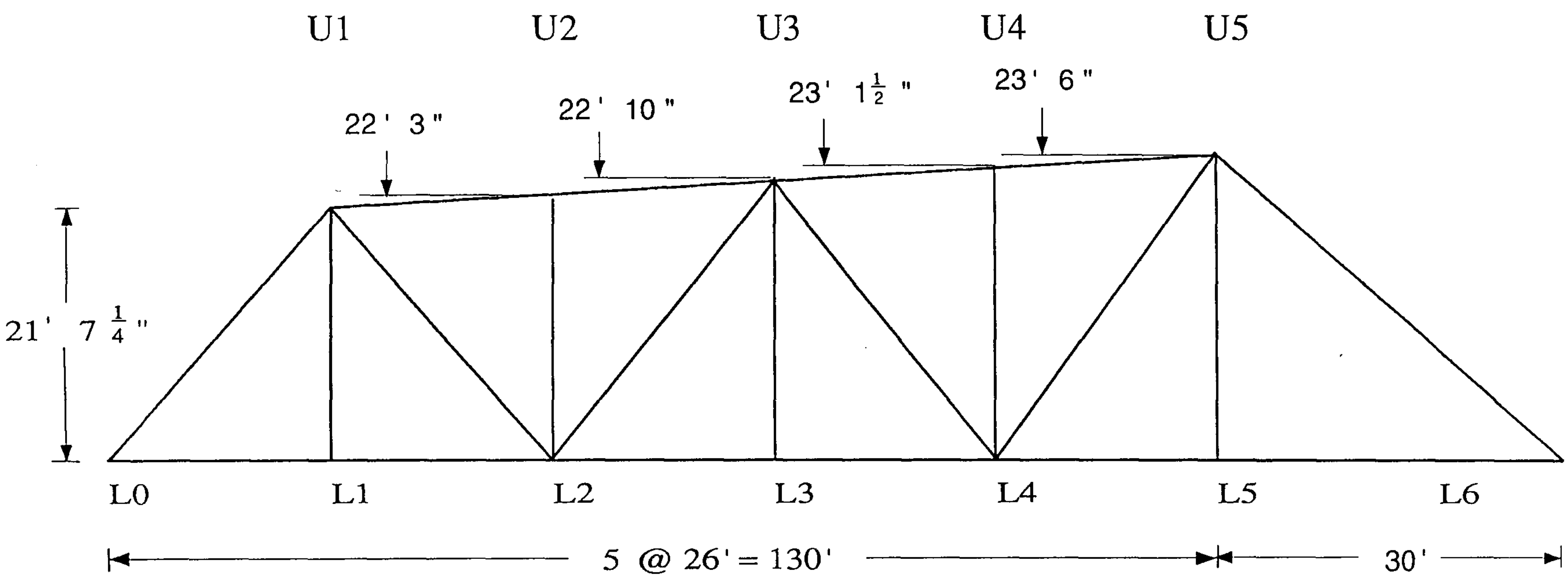


Figure 14.3. Elevation of Truss Spans #2 and #4

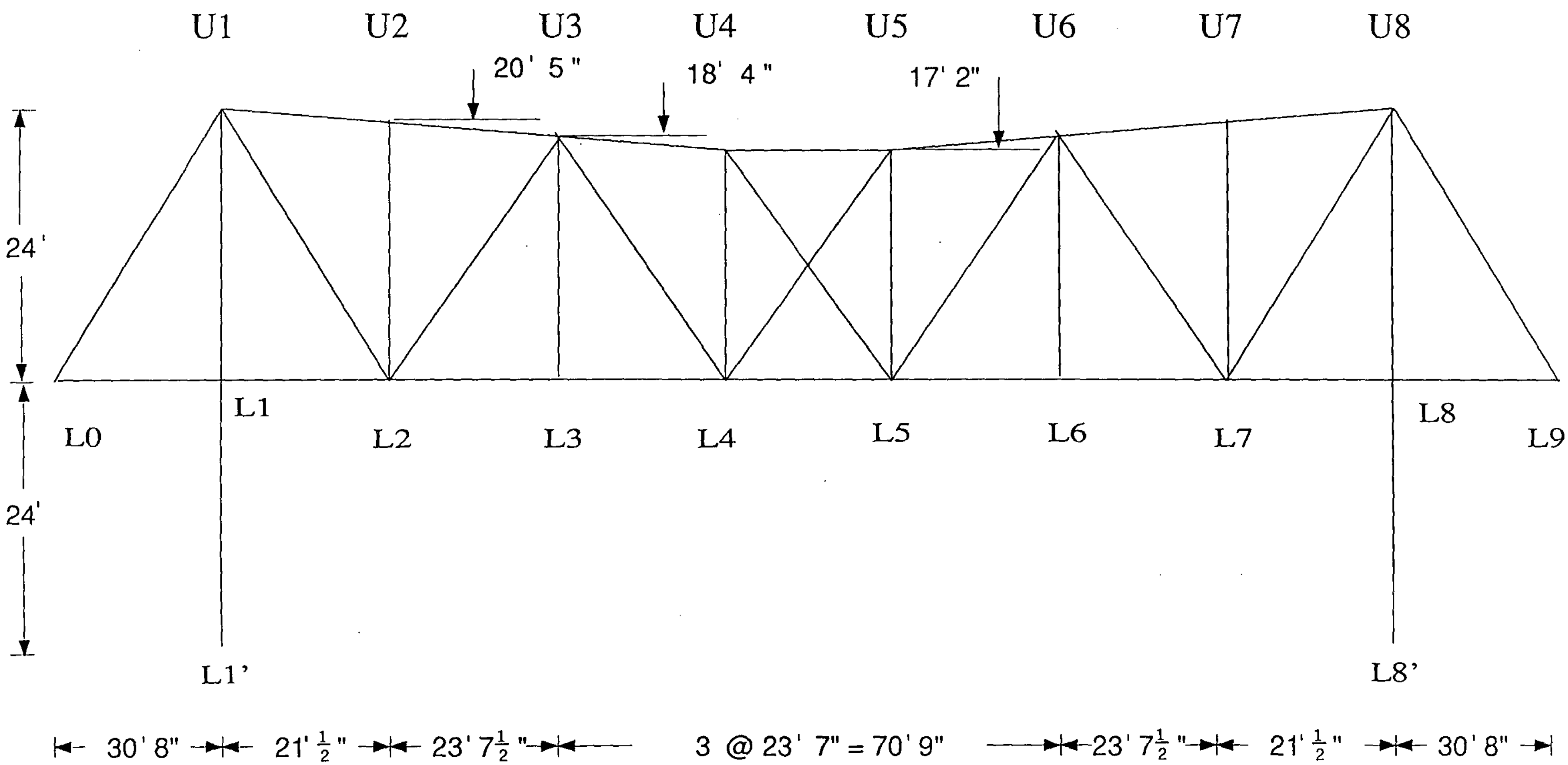


Figure 14.4. Elevation of Truss Span #3

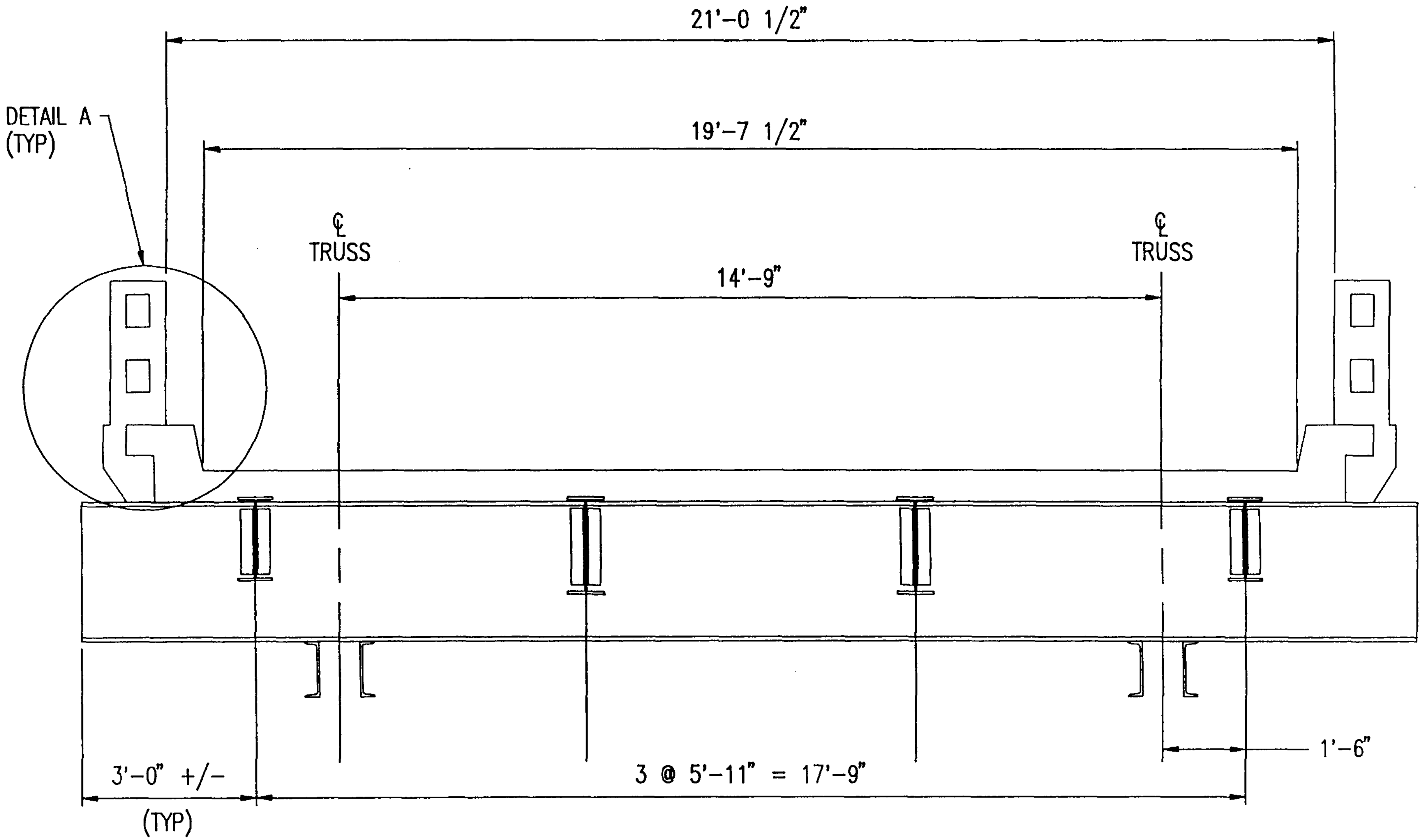


Figure 14.5. Cross Section at Truss Spans #1 and #5

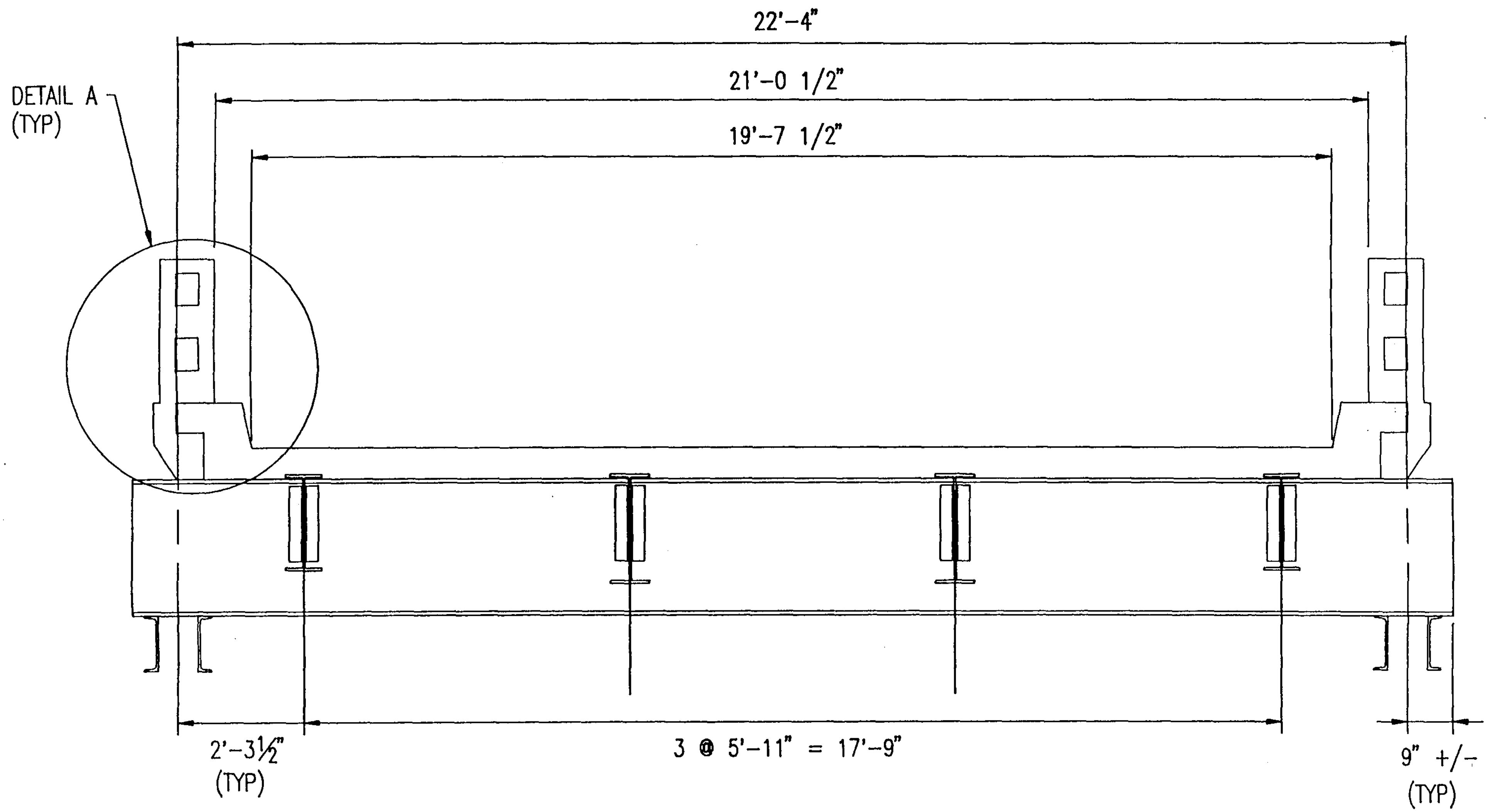


Figure 14.6. Cross Section at Truss Spans #2 and #4

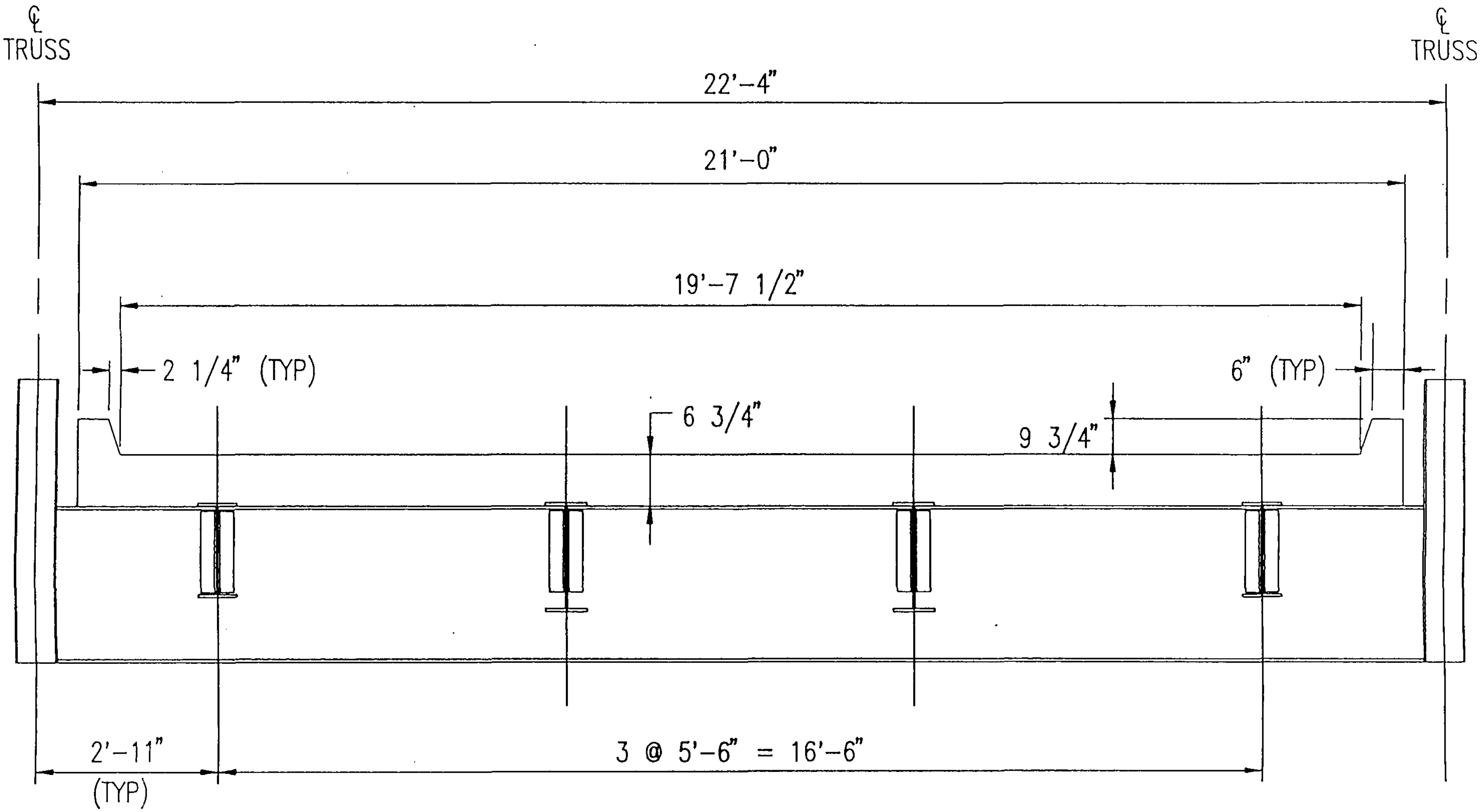


Figure 14.7. Cross Section at Truss Span #3

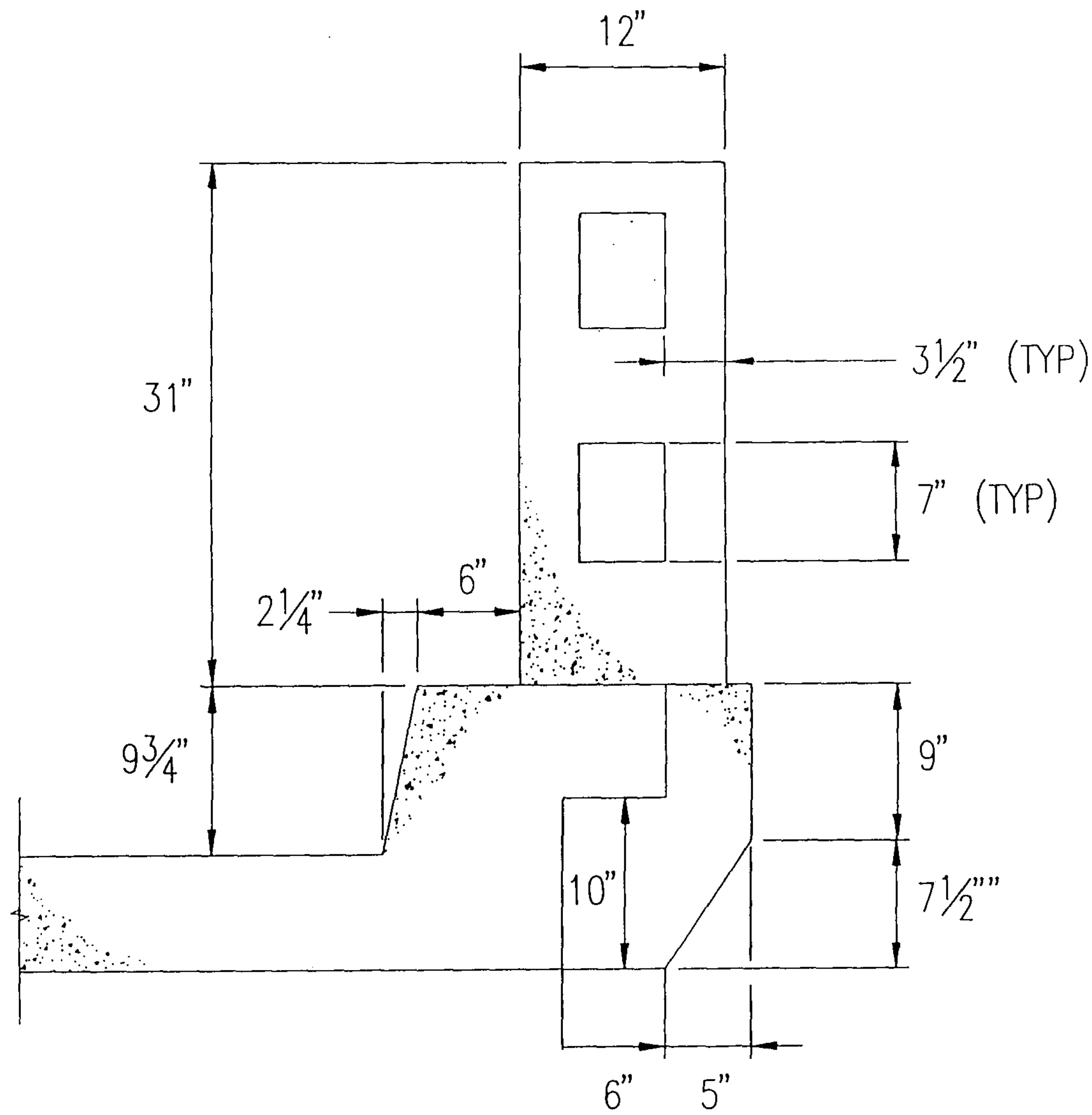


Figure 14.8. Typical Detail of Parapet

Table 14.1 Truss Member Cross Section Properties for Truss Span 1 and 5

Member	Area ($inch^2$)	I_x ($inch^4$)	I_y ($inch^4$)
L0U1, U1U2,U2U3 U5U6,U6U7,L8U7	26.46	911.7	690.6
U3U4,U4U5	29.40	974.9	768.4
L0L1,L1L2 L6L7,L7L8	20.46	637.4	562.6
L2L3,L3L4 L4L5,L5L6	26.34	747.8	723.3
U1L1,L3U3 L5U5,L7U7	7.06	84.3	18.3
L2U2,L4U4,L6U6	9.10	110.9	36.7
U1L2,L6U7	14.64	287.0	392.4
L2U3,U5L6	12.06	256.2	326.1
U3L4,L4U5	8.04	71.6	203.8

Table 14.2 Truss Member Cross Section Properties for Truss Span 2 and 4

Member	Area ($inch^2$)	I_x ($inch^4$)	I_y ($inch^4$)
L0U1	23.58	493.7	587.6
U1U2,U2U3	20.52	357.6	544.7
L5U5	12.06	256.2	318.4
U3U4,U4U5	17.58	322.4	459.6
L0L1,L1L2,U3L4	14.64	287.0	392.4
L4L5,L5L6	12.06	256.2	326.1
L1U1,L3U3,L5U5	7.06	84.3	18.3
L2L3,L3L4	17.58	322.4	470.8
L2U2,L2U3 L4U4,U1L2	11.36	157.0	296.8
L4U5	17.58	322.4	459.6

Table 14.3 Truss Member Cross Section Properties for Truss Span 3

Member	Area ($inch^2$)	I_x ($inch^4$)	I_y ($inch^4$)
L0U1,L0L1 U8L9,L8L9	23.40	692.6	640.4
U1U2,U2U3,U3U4 U4U5,U5U6,U6U7 U7U8	12.06	256.2	313.4
L1L2,L7L8	23.46	393.0	637.2
L2L3,L3L4 L4L5,L5L6,L6L7	12.06	256.2	313.4
L1'L1U1 L8'L8U8	33.96	1,341.4	974.8
L2U2,L3U3,L4U4 L5U5,L6U6,L7U7	7.06	84.3	18.3
U1L2,L7U8	17.58	322.4	459.6
L2U3,U6L7	14.64	287.0	383.1
U3L4,L5U6	11.36	157.0	254.7
L4U5,U4L5	8.04	71.6	198.8

maximum live load stresses plus impact is given in Tables 14.4 through 14.6. Top and bottom chords develop the maximum stress under a six axle vehicle (3S3). The tri–axle dump truck produces the maximum stresses in verticals and diagonals for which the influence lines are confined within the adjacent panels.

Summaries of the minimum As–Designed inventory and operating rating factors calculated for the truss members of truss spans 1 and 5 are given in Tables 14.7 through 14.9. These ratings are based on the axial stresses created in the truss members by the standard truck loadings.

The rating factors given in Table 14.7 are controlled by the compression capacity of members U3U4 and L4U4. Most of the inventory rating factors are controlled by member U3U4. The compression capacity of vertical L4U4 controls the operating rating factors for the two axle, tri–axle, and concrete trucks. Member U3U4 controls the operating rating factors for the 3S3 and 3S2 trucks. It should be noted from Table 14.1 that the cross sectional area for the verticals (L2U2, L4U4, and L6U6 which are subjected to compression due to the location of the roadway deck) is quite small as compared to that for top chords. Table 14.4 shows that the member stresses in these verticals are comparable to those in top chords.

Both the inventory and operating rating factors given in Table 14.8 for truss spans 2 and 4 are controlled by the compression capacity of top chords of the truss (U1U2,U2U3).

The compression capacity of member U3U4 of truss span 3 controls both the inventory and operating rating factors given in Table 14.9 for all vehicles.

The field inspection indicated that the truss members in truss spans 1, 2, 4, and 5 were in generally good condition with isolated areas of poor conditions. Because the trusses in these spans are deck trusses, there was obviously no vehicular impact damage. All members, however, exhibited paint loss on up to 15% of their surface area and minor surface rust. Several vertical gusset plates at lower chord joints were found to have losses due to pitting horizontally across the plate along the top flange of the bottom chord. The Condition Report (Wilcox County 1992, pp. 6–7) documents the losses at each of the severely corroded connections. Further details of corrosion damage are presented on the Field Notes sheets 122, 123, and 144. None of the corrosion damage on the gusset plates had a significant effect on the rating calculations. It was judged that the As–Designed rating factors given in Tables 14.7 and 14.8 are sufficiently accurate for use as Present–Condition ratings.

The truss members in truss span 3 were found to be in serious condition. All members displayed paint loss on more than 50% of their surface area and minor surface rust. The majority of verticals and diagonals suffer from vehicular impact damage and are out of their proper vertical alignments. Details of

Table 14.4 Dead Load Stresses and Maximum Live Load Stresses for Truss Spans 1 and 5

Member	Dead Load (ksi)	Maximum Live Load Plus Impact (ksi)
L0U1	-5.92	-3.88
L0L1	5.15	3.41
U1U2	-6.83	-4.46
L2U1	7.63	5.87
L1L2	5.15	3.41
L2U2	-3.18	-7.27
U2U3	-6.83	-4.46
L2U3	-5.56	-5.81
L2L3	8.58	5.50
U3U4	-8.20	-5.20
L4U3	2.81	6.85
L3L4	8.58	5.50
L4U4	-3.21	-7.27

Table 14.5 Dead Load Stresses and Maximum Live Load Stresses for Truss Spans 2 and 4

Member	Dead Load (ksi)	Maximum Live Load Plus Impact (ksi)
L0U1	-5.71	-4.54
L0L1	7.04	5.48
U1U2	-7.38	-5.82
U1L2	5.63	6.55
L1L2	7.04	5.48
L2U2	-3.29	-6.24
U2U3	-7.38	-5.82
L2U3	0.11	4.16
L2U3	0.11	-4.48
L2L3	8.61	7.25
U3U4	-6.03	-5.15
U3L4	-4.13	-4.81
L3L4	8.61	7.25
L4U4	-3.33	-6.29
U4U5	-6.03	-5.18
L4U5	6.96	5.39
L4L5	1.36	4.39
U5L6	-1.76	-5.52
L5L6	1.36	4.39

Table 14.6 **Dead Load Stresses and Maximum Live Load Stresses for Truss Span 3**

Member*	Dead Load (ksi)	Maximum Live Load Plus Impact (ksi)
L0U1	7.50	2.77
L0L1	-5.79	-2.77
L1'L1U1	-7.26	-3.20
U1U2	5.42	4.25
U1L2	6.32	6.16
L1L2	-5.78	-2.77
U2U3	5.36	4.21
L2U3	-6.37	-6.89
L2L3	0.71	6.42
L3U3	4.06	9.83
U3U4	-4.24	-10.22
U3L4	4.87	7.51
L3L4	0.71	6.42
L4U4	-0.59	-2.56
U4U5	-4.18	-8.93
U4L5	-0.21	-3.76
L4L5	4.37	8.53

*Truss Geometry and member stresses are symmetric about midspan

Table 14.7 Rating Factors for Truss Members of Truss Spans 1 and 5

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	1.36	1.92
Tri-Axle	1.03	1.39
Concrete Truck	1.16	1.57
3S3 (AL)	1.07	1.73
3S2 (AL)	1.11	1.80
School Bus	2.44	3.30

Table 14.8 Rating Factors for Truss Members of Truss Spans 2 and 4

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	1.24	1.96
Tri-Axle	0.96	1.52
Concrete Truck	1.10	1.72
3S3 (AL)	0.98	1.54
3S2 (AL)	1.02	1.61
School Bus	2.04	4.60

Table 14.9 Rating Factors for Truss Members of Truss Span 3

Truck Type	As-Designed/Present-Condition	
	Inventory	Operating
Two Axle	1.16	1.58
Tri-Axle	0.89	1.22
Concrete Truck	1.01	1.38
3S3 (AL)	0.94	1.27
3S2 (AL)	0.97	1.33
School Bus	2.78	3.79

damage condition are summarized in the Condition Report (Wilcox County 1992, pp. 7–8). Further details of damage are presented in the Field Notes on sheets 126, 127, 129, 132, 133, 134, and 135. Repair plates, knee brackets, and cable support brackets have been welded to several truss members (Field Note sheet 126 through 132 and 137). The quality of these welds is poor with pin holes and weld undercuts throughout. The presence of the welds on verticals and lower chord tension members creates a fatigue sensitive detail on these fracture critical members. The end portals, intermediate portals, and top lateral bracing were also found to be in serious condition. The majority of these members suffered from apparent vehicular impact damage.

The damage to the truss members of truss span 3 was considered in calculations of Present–Condition ratings. Members L8'L8U8 (in north truss) and L1'L1U1 (in both north and south trusses) were the most critically damaged members. The west flange of the north channel of member L1'L1U1 (of the south truss) had only 1 1/8 inches remaining (from the original length of 4.2 inches), and the east flange was bent 2 inches out of its original alignment as detailed in the Field Notes on sheet 126. Present-Condition rating factors calculated based on these damaged conditions of member L1'L1U1 were higher than those given in Table 14.9 which are based on member U3U4. Hence, it was judged that the As-Designed rating factors given in Table 14.9 are appropriate for use as Present-Condition ratings.

Capacities of the truss connections were also evaluated. The connection at every joint of each truss was examined using the details given in the Field Notes (pp. 6 through 14 for truss spans 1 and 5, pp. 24 through 33 for truss spans 2 and 4, and pp. 50 through 58 for truss span 3) and field photographs. The field measurements of the head diameter of rivets varied from 1 1/4 to 1 3/8 inches. The shank diameter of the rivets was conservatively taken as 3/4 inch from page 4–135 of AISC (1980). The condition of the connections was generally fair with no significant corrosion damage that would adversely affect the ratings. The rivets were conservatively assumed to be ASTM A502 Grade 1 rivets. It was found that the controlling joints are L4 of truss spans 2 and 4, U1 of truss spans 1 and 5, and L4 of the truss span 3. Rating factors for the truss connections are summarized in Tables 14.10 through 14.12. The ratings shown are based on the shear capacity of the rivets. Ratings based on bearing stresses were found to be much higher than those shown.

TRUSS FLOOR SYSTEM RATINGS

Typical sections through the truss span floor system are shown in Figures 14.5 through 14.7. Between floorbeams there are typically four stringers; one identical pair of interior stringers and one identical pair of exterior stringers. In truss spans 1 and 5 the exterior stringers are W18x50, and the interior stringers are W21x62. The exterior stringers of truss spans 2 and 4 are CB211 21x8x64, and the interior stringers are CB241 21x8 1/2x70. In truss span 3, four CB241

Table 14.10 **Rating Factors for Truss Connection Capacities for Truss Spans 1 and 5**

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	1.91	3.11
Tri-Axle	1.46	2.42
Concrete Truck	1.68	2.74
3S3 (AL)	1.48	2.40
3S2 (AL)	1.59	2.53
School Bus	4.51	7.33

Table 14.11 **Rating Factors for Truss Connection Capacities for Truss Spans 2 and 4**

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.77	1.59
Tri-Axle	0.59	1.22
Concrete Truck	0.66	1.38
3S3 (AL)	0.59	1.22
3S2 (AL)	0.62	1.29
School Bus	1.81	3.77

Table 14.12 Rating Factors for Truss Connection Capacities for Truss Span 3

Truck Type	As-Designed/Present-Condition Inventory	As-Designed/Present-Condition Operating
Two Axle	1.05	1.67
Tri-Axle	0.81	1.30
Concrete Truck	0.92	1.46
3S3 (AL)	0.85	1.36
3S2 (AL)	0.91	1.45
School Bus	2.47	3.95

24x8 1/2x70 are used for the exterior and interior stringers in the end panels where the panel length is the longest (30 feet 8 inches). In the first interior panel (21 feet 1/2 inch), the exterior stringers are CB181 18x7 1/2x51, and the interior stringers are CB211 21x8x55. In the panel L2L3, four CB211 21x8x58 are used for both the exterior and interior stringers. In the middle three panels (panel length of 23 feet 7 inches), the exterior stringers are CB211 21x8x64, and the interior stringers are CB211 21x8x58. Cross sectional properties of the stringers for use in the rating calculations were taken from AISC (Iron and Steel Beams 1985). Due to the varying panel length of the trusses, the stringer span lengths vary. The top flanges of the stringers in all truss spans were in contact with the underside of the deck. There were no signs of lateral movement of the top (compression) flange, so the top flanges were assumed to be continuously braced. Structural analyses were performed on the stringers assuming each stringer to be a simple beam. The truck loadings were distributed to the stringers using the wheel load distribution factor (S/5.5) from Table 3.23.1 of AASHTO (1989). Summaries of the minimum inventory and operating rating factors calculated for the stringers based on flexural capacity are given in Tables 14.13 through 14.15.

The rating factors given in Table 14.13 for truss spans 1 and 5 (constant panel length of 20 feet) and Table 14.14 for truss spans 2 and 4 (constant panel length of 26 feet) were controlled by the lighter exterior stringers. For truss span 3, the rating factors given in Table 14.15 were controlled by the stringers in the end panels (L0L1 and L8L9 where the panel length of 30 feet 8 inches is the longest).

Rating factors for the shear capacities and stringer to floorbeam connection capacities were also evaluated. The resulting rating factors were substantially higher than those given in Tables 14.13 through 14.15 for moment capacities. Hence they are not presented in this report.

The field inspection revealed that the stringers were in satisfactory condition with no significant losses (Wilcox County 1992, pp. 9). Some broken and loose rivets were found in the stringer to floorbeam connections and are noted in the Condition Report (Wilcox County 1992, pp. 9–10). These rivets should be replaced with A325 bolts and hardened washers. Near right angle flame cut copes were also found in the stringer web at the stringer to floorbeam connections. These copes should be carefully inspected for fatigue cracks in the future inspections. Overall, it was judged that the As–Designed rating factors given in Tables 14.13 through 14.15 are sufficiently accurate for use as Present–Condition ratings assuming that the missing rivets in the stringer to floorbeam connections are replaced with high strength bolts.

There are seven floorbeams (W21x62) in each of the truss spans 1 and 5 designated as FB1 to FB7. The floorbeams span between the top truss joints at U1 through U7 and are numbered from the west to the east. The floorbeams are

Table 14.13 Rating Factors for Truss Floor Stringers of Truss Spans 1 and 5

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.73	1.14
Tri-Axle	0.52	0.82
Concrete Truck	0.58	0.91
3S3 (AL)	0.73	1.13
3S2 (AL)	0.83	1.30
School Bus	1.39	2.17

Table 14.14 Rating Factors for Truss Floor Stringers of Truss Spans 2 and 4

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.80	1.26
Tri-Axle	0.56	0.89
Concrete Truck	0.64	1.01
3S3 (AL)	0.77	1.22
3S2 (AL)	0.87	1.38
School Bus	1.60	2.53

Table 14.15 Rating Factors for Truss Floor Stringers of Truss Span 3

Truck Type	As-Designed/Present-Condition	
	Inventory	Operating
Two Axle	0.75	1.17
Tri-Axle	0.54	0.84
Concrete Truck	0.60	0.93
3S3 (AL)	0.75	1.15
3S2 (AL)	0.86	1.33
School Bus	1.45	2.25

supported by the top chords of the deck trusses and have an overhang of 4 feet 6 inches on both ends as show in Figure 14.5.

The bending moments and shearing forces were evaluated based on a simple beam of 14 feet 9 inches span with 4 feet 6 inches overhang on both ends. The compression flanges were assumed to be braced by the stringers at the interior portion, and unbraced lengths of 4 feet 6 inches were used at the overhangs. Summaries of the minimum inventory and operating rating factors for the floorbeams of the truss spans 1 and 5 are given in Table 14.16. These rating factors were based on the flexural capacities of the members. It was found that rating factors based on the shear capacities of the floorbeams are substantially higher than those presented in Table 14.16. Hence, those rating factors are not presented in this report.

CB301 30x115 shapes are used for all floorbeams in the truss spans 2, 3, and 4 despite different panel spacings. The floorbeam between truss span 2 and truss span 3 is the most heavily loaded floorbeam since it supports the greatest length of roadway. Hence, the floorbeam ratings for truss spans 2,3, and 4 were based on the capacity of the floorbeam at joint U5 of truss span 2. The bending moment and shearing forces were evaluated assuming the floorbeam to be a simple beam of 22 feet 4 inches span length. Summaries of the minimum inventory and operating rating factors for the floorbeam are given in Table 14.17. These rating factors were based on the flexural capacity of the member. It was found that rating factors based on the shear capacity are higher than those presented in Table 14.17. Hence, those rating factors are not given in this report.

The capacities of the floorbeam to truss connections were also evaluated. Since the floorbeams in truss spans 1, 2, 4, and 5 are placed directly on top of top chords as shown in Figures 14.5 and 14.6, there is no critical joint to be rated. However, the floorbeams in the truss span 3 are connected to verticals of the truss. Rating factors based on these connection capacities were evaluated assuming the rivets to be ASTM A502 Grade 1. The field measurements of the head diameter of rivets varied from 1 5/16 to 1 3/8 inches (pp. 48 through 51, Field Notes, Wilcox County 1992). The shank diameter of the rivets was conservatively taken as 3/4 inch (pp. 4–135, AISC 1980). The resulting rating factors are summarized in Table 14.18. The ratings shown are based on the shear capacity of the rivets.

Examination of the Field Notes, the Condition Summary Report (Wilcox County 1992), and photographs reveals that there are no significant losses or damage noted for the floorbeams which would affect the live load ratings. The locations of a few missing rivets from the floorbeam to truss connections are summarized on pages 9 through 10 of the Condition Report (Wilcox County 1992). The missing rivets need immediate attention and should be replaced with A325 bolts and hardened washers. Based on the Condition Report and

Table 14.16 Rating Factors for Truss Floorbeams of Truss Spans 1 and 5

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.55	1.12
Tri-Axle	0.40	0.81
Concrete Truck	0.45	0.92
3S3 (AL)	0.59	1.19
3S2 (AL)	0.63	1.27
School Bus	1.33	2.70

Table 14.17 Rating Factors for Truss Floorbeams of Truss Spans 2, 3 and 4

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	0.71	1.19
Tri-Axle	0.53	0.89
Concrete Truck	0.60	1.01
3S3 (AL)	0.72	1.21
3S2 (AL)	0.75	1.26
School Bus	1.74	2.92

Table 14.18 Rating Factors for Truss Floorbeam to Truss Connections of Truss Span 3

Truck Type	As-Designed/Present-Condition Inventory	Operating
Two Axle	1.04	1.56
Tri-Axle	0.78	1.17
Concrete Truck	0.88	1.33
3S3 (AL)	1.05	1.59
3S2 (AL)	1.09	1.65
School Bus	2.54	3.82

calculations it was judged that the rating factors given in Tables 14.16 through 14.18 are sufficiently accurate for use as Present–Condition rating factors.

CONCRETE DECK ON TRUSS SPANS

Reinforcement details for the truss span decks were not available. Therefore, a quantitative rating for the concrete deck was not possible.
The field inspection results as summarized in the Condition Summary Report (Wilcox County 1992) indicate that the concrete deck appears to be in fair condition. The underside of the deck typically exhibited random transverse hairline cracks in all spans. Occasionally these cracks were accompanied by leaching. Except for a few popouts and minor spalls (<2 inches deep), no major spalls were found, and no hollow areas were detected when the concrete was sounded by a hammer. The top of the concrete deck was found to be in fair condition with light to moderate scaling. Random areas of hairline map cracking and transverse cracks are also present. There typically is a pattern of transverse cracking over the floorbeams. It was judged that the condition of the concrete deck does not adversely affect the live load carrying capacity of the bridge.

CONCLUSIONS AND RECOMMENDATIONS

Structural analyses and load ratings of the five truss spans indicate that the final bridge ratings are controlled by the flexural capacity of the stringers and the floorbeams in the truss spans 1 and 5. A summary of the Present–Condition ratings for the truss spans is given in Table 14.19. The field inspection (Condition Report, Wilcox County 1992, pp. 11–12) also identified the following maintenance and repairs that should be carried out:

1. All damaged truss members, end portals, and sway frames should be repaired by heat straightening the lesser damaged members and heat straightening with the addition of reinforcement plates to the more severely damaged members. The cracked angles connecting the intermediate portals to the truss members should be repaired.

2. Field measurements taken at the truss rocker bearings at locations of apparent overstress show the pin hole is oversized by up to 1/2 inch for the pin diameter. All misaligned pin nuts should be realigned and tightened. Although load ratings on bearing assemblies and substructures are beyond the scope of the contract, A.G. Lichtenstein and Associates and Auburn University strongly suggest AHD arrange UT testing of all pins to ensure that there are no hidden internal cracks in the pins. It is recommended that the pin assemblies be monitored by AHD periodically to provide timely detection of any further distress.

Table 14.19 Summary of Present-Condition Ratings for Truss Spans (Wilcox County Structure No. 28-66-12.0)

Truck Type	Standard Weight (tons)	Inventory Rating Factor	Inventory Rating (tons)	Operating Rating Factor	Operating Rating (tons)
Two Axle	29.5	0.55	16	1.12	>29.5
Tri-Axle	37.5	0.40	15	0.81	30
Concrete Truck	33	0.45	15	0.91	30
3S3 (AL)	42	0.59	25	1.19	>42
3S2 (AL)	40	0.63	25	1.27	>40
School Bus	12.5	1.33	>12.5	2.17	>12.5

3. All welding performed on truss members in tension should be monitored for signs of distress during future inspections. Although evaluation of remaining fatigue life is beyond the scope of the contract, the addition of the poor quality welds on repair plates to the lower chords of the trusses creates a fatigue sensitive detail in the fracture critical truss lower chord members.

4. All cracks and spalls in the truss bearing pads should be raked and cleaned to remove all loose material, regrouted, and sealed.

5. The flame cut copes at the ends of the stringers should be carefully inspected for possible fatigue cracks during future bridge inspections.

6. The stringer to floorbeam and floorbeam to truss connections which have broken rivets should be retrofitted with A325 bolts and hardened washers.

7. The vertical clearance signs on the approaches should be revised.

8. All damaged railings should be repaired to current AASHTO Specifications.

9. New joint seals should be placed at all open joints on the truss spans to prevent water and debris from falling on surface below the joints.

10. The top of deck over floorbeams should be sawcut and then filled with poured joint material to seal and control the transverse cracking in the deck. Existing cracks which do not coincide with the sawcut line should be epoxy injected.

11. All spalls in the piers should be repaired and all pier cracks epoxy injected.

12. The installation of speed reduction signs should be considered at both approaches, and AHD should examine the need to request assistance from local and state police in the enforcement of speed and load capacity posting.

13. The stability of truss spans 1 and 5 should be investigated. The fact that the trusses are taller than they are wide coupled with the higher center of gravity could cause instability of these trusses in severely high winds. Sound bearing assemblies and proper anchorage of these truss spans appears very critical.

CHAPTER FIFTEEN
CONCLUDING REMARKS

Inspections and load ratings were performed for twelve bridges on this project. The main spans of most of the bridges were truss structures. Load ratings for the twelve bridges had never been established because sufficient information about the bridge structures was not available. The specific project objectives were: (1) to perform a thorough field inspection of each bridge, (2) to collect all pertinent field data on the geometry and details of each bridge, and (3) to perform a structural analysis and load rating of each bridge based on the superstructure's load carrying capacity. Each of the project objectives was achieved. The results for each bridge were documented with interim reports of sufficient detail, and in the necessary format, for use by the Maintenance Bureau in updating the bridge inventory records.

An examination of the field inspection and load rating results for all the bridges leads to the following concluding remarks.

1. The load capacity of the truss spans was typically limited by the capacity of the stringers or floorbeams instead of the capacities of the truss members or truss connections.

2. All twelve bridges exhibit signs of deterioration to varying degrees. The major causes of deterioration are: corrosion (loss of member cross sectional area), vehicle impacts due to overheight vehicles and narrow lanes, and fatigue fracture of members and rivets. Most of the bridges exhibited deterioration from all of these.

3. Corrosion of a number of stringers, floorbeams, and bearing assemblies was found to result from leakage of water through deck joints or from drainage through deck drains onto structural members below.

4. Fatigue sensitive details were created in a number of fracture critical members by field welding performed to repair corrosion or impact damage.

REFERENCES

A.G. Lichtenstein & Associates (1991). Condition Report for St. Clair County Structure No. 004-58-021.9, Framingham, MA.

A.G. Lichtenstein & Associates (1991). Condition Report for Keller Memorial Bridge Structure No. 003-52-021.9A, Framingham, MA.

A.G. Lichtenstein & Associates (1991). Condition Summary Report for Morgan/Madison County, Structure No. 053-52-008.7B, Framington, MA.

A.G. Lichtenstein & Associates (1991). Condition Summary Report for Jackson County, Structure No. 035-36-009.8A, Framington, MA.

A.G. Lichtenstein & Associates (1991). Condition Summary Report for Mobile County, Structure No. 013-49-003.2, Framington, MA.

A.G. Lichtenstein & Associates (1991). Condition Summary Report for Montgomery County, Structure No. 009-51-040.7B, Framington, MA.

A.G. Lichtenstein & Associates (1991). Condition Summary Report for Wilcox County, Structure No. 021-66-023.2, Framington, MA.

A.G. Lichtenstein & Associates (1991). Condition Summary Report for Mobile/Baldwin County, Structure No. 016-49-037.6B, Framington, MA.

A.G. Lichtenstein & Associates (1991). Condition Summary Report for Lauderdale County, Structure No. 002-39-008.7A, Framington, MA.

A.G. Lichtenstein & Associates (1991). Condition Summary Report for Coffee County, Structure No. 125-16-000.2, Framington, MA.

A.G. Lichtenstein & Associates (1992). Condition Summary Report for Cherokee County, Structure No. 068-10-19.9, Framington, MA.

A.G. Lichtenstein & Associates (1992). Condition Summary Report for Wilcox County, Structure No. 28-66-12.0, Framington, MA.

American Association of State Highway and Transportation Officials (1983). Manual for Maintenance Inspection of Bridges, Washington, D.C.

American Association of State and Transportation Officials (1989). Standard Specifications for Highway Bridges, 14th Edition, Washington, D.C.

American Concrete Institute (1977). Building Code Requirements for Reinforced Concrete, Detroit, MI.

American Institute of Steel Construction (1927). Steel Construction, 1st Edition, New York, NY.

American Institute of Steel Construction (1963). Manual of Steel Construction, 6th Edition, Chicago, IL.

American Institute of Steel Construction (1980). Manual of Steel Construction, 8th Edition, Chicago, IL.

American Institute of Steel Construction (1985). Iron and Steel Beams 1873 to 1952, 10th Printing, Chicago, IL.

American Iron and Steel Institute (1971). Bulletin No. 19, Criteria for the Deflection of Steel Bridges, by Wright and Walker, New York, NY.

Carnegie Steel Company (1971). Pocket Companion, Pittsburg, PA.

Hibbitt, Karlsson & Sorensen, Inc. (1989). ABAQUS User's Manual, Version 4.8, Newark, CA.

McCormac, Jack C. (1971). Structural Steel Design, 2nd Edition, Intext Educational Publishers, Scranton, OH.

Portland Cement Association (1959). Continuity in Concrete Building Frames, 4th Edition, Skokie, IL.

White, K.R., Minor, John, Derucher, K.N. and Heins, C.P., Jr. (1981). Bridge Inspection and Evaluation, M. Dekker, New York, NY.

Yoo, C.H. and Siegel, C.R. (1986). "Column Loadings on Telescopic Power Cylinders," Computers & Structures, 22(3), 245-251.