
T.J. Wipf, F.W. Klaiber, J. Witt, T.L. Threadgold

Use of Railroad Flat Cars for Low-Volume Road Bridges

August 1999

Sponsored by the
Iowa Department of Transportation
Project Development Division and the
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Iowa DOT Project TR-421



Iowa Department
of Transportation

Final

REPORT

IOWA STATE UNIVERSITY
OF SCIENCE AND TECHNOLOGY

Department of Civil and Construction Engineering

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Highway Division of the Iowa Department of Transportation.

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ABSTRACT

In an attempt to solve the bridge problem faced by many county engineers, this investigation focused on a low cost bridge alternative that consists of using railroad flatcars (RRFC) as the bridge superstructure. The intent of this study was to determine whether these types of bridges are structurally adequate and potentially feasible for use on low volume roads.

A questionnaire was sent to the Bridge Committee members of the American Association of State Highway and Transportation Officials (AASHTO) to determine their use of RRFC bridges and to assess the pros and cons of these bridges based on others' experiences. It was found that these types of bridges are widely used in many states with large rural populations and they are reported to be a viable bridge alternative due to their low cost, quick and easy installation, and low maintenance.

A main focus of this investigation was to study an existing RRFC bridge that is located in Tama County, IA. This bridge was analyzed using computer modeling and field load testing. The dimensions of the major structural members of the flatcars in this bridge were measured and their properties calculated and used in an analytical grillage model. The analytical results were compared with those obtained in the field tests, which involved instrumenting the bridge and loading it with a fully loaded rear tandem-axle truck. Both sets of data (experimental and theoretical) show that the Tama County Bridge (TCB) experienced very low strains and deflections when loaded and the RRFCs appeared to be structurally adequate to serve as a bridge superstructure. A calculated load rating of the TCB agrees with this conclusion.

Because many different types of flatcars exist, other flatcars were modeled and analyzed. It was very difficult to obtain the structural plans of RRFCs; thus, only two additional flatcars were analyzed. The results of these analyses also yielded very low strains and displacements.

Taking into account the experiences of other states, the inspection of several RRFC bridges in Oklahoma, the field test and computer analysis of the TCB, and the computer analysis of two additional flatcars, RRFC bridges appear to provide a safe and feasible bridge alternative for low volume roads.

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1. INTRODUCTION

1.1 Background

In the United States, the bridge problem has been well documented. More existing bridges are becoming structurally inadequate while funds to repair or replace these bridges are limited. The bridge problem is significant in Iowa because many of the state's 25,000 bridges (approximately 81%) are on secondary roads and thus the responsibility of the counties. The number of bridges in Iowa ranks it 5th in the nation (behind Texas, Ohio, Kansas, and Illinois) while Iowa's population ranks 30th, which limits the state's tax base. Based on these two facts (large number of bridges and limited tax base), few states have more severe bridge problems than Iowa.

Based on surveys of the Iowa counties, the Bridge Engineering Center (BEC) at Iowa State University found that a large number of Iowa counties (69%) have the ability and interest to use their own forces to design and construct short span bridges provided the construction procedures are relatively simple. At the request of Cerro Gordo County, the BEC proposed investigating the feasibility of using railroad flatcars (RRFC), a low cost bridge alternative, for low volume road bridges. To determine the interest of other counties in using RRFC as a bridge alternative, the BEC surveyed Iowa counties. Fifty-seven (58%) counties returned the questionnaire; of those returning the questionnaire, over 47% indicated that they were interested in the concept.

The RRFC bridge concept involves using salvaged (i.e., no longer used by the railroad industry) flatcars as the superstructure of low volume road bridges.

Either existing or new bridge abutments can support these flatcars, which may be coupled to increase the bridge width. Based on knowledge to date, RRFC bridges initially appeared to offer several advantages: low cost, easy and quick installation, variable span length availability, and low maintenance. It is known that the girders in the railcars were originally produced with high quality fabrication and welding. They exhibit very high torsional strength and stiffness in addition to the required flexural strength and stiffness required in bridge replacement alternatives. Preliminary investigations revealed that several states have used railcars in different bridge applications with span lengths ranging from 20 to 80 ft.

1.2 Objective and Scope

The overall objective of this project was to determine whether RRFC bridges could provide a viable bridge replacement alternative for low volume roads in Iowa. Many variables were considered when determining the feasibility of this bridge concept, the most significant being the bridge's load carrying capacity. This study consisted of an analytical investigation of various RRFC structural systems (plans of which were obtained from various flatcar manufacturers) and a field load test of an existing RRFC bridge in Tama County, Iowa. The cost, construction, and maintenance issues associated with RRFC bridges were also addressed using information from states that have constructed and maintained a significant number of RRFC bridges.

The purpose of the first task was to collect existing information on RRFC bridges. Comprehensive computer and library searches were performed and

personnel from the railroad industry were contacted. In addition, a questionnaire was sent to several state bridge engineers and other members of the AASHTO Bridge Committee to identify other states' use of the RRFC bridge concept. Through railroad industry contacts, structural drawings of various flatcars were obtained.

In the analytical portion of the project, a computer model of the RRFC bridge located in Tama County was developed using the grillage method of analysis. Using the same modeling procedures, computer models were developed to simulate the Tama County Bridge geometry using the structural plans of other flatcars. Loads were applied at critical locations and results obtained in the analyses of the various flatcars were compared.

Before the field load test was performed, the RRFC bridge in Tama County was carefully inspected. The bridge was then instrumented to measure strains and deflections at critical locations. During the load test, the bridge was first loaded with an empty single-axle dump truck and then with a fully loaded rear tandem-axle dump truck. The measured strain and deflection data from the field load test were compared to the analytical results obtained from the computer model subjected to the same loading. Together, these results were used to determine whether RRFC bridges have an adequate strength capacity for Iowa legal loads. A load rating was also calculated for the RRFC bridge in Tama County.

Bridge and county engineers and individuals with significant experience in RRFC bridge maintenance and construction were contacted for information on

other aspects of the feasibility issue, including cost, construction, and maintenance. After considering all aspects of this investigation, the overall feasibility of using RRFC for low volume road bridges was determined. The procedures and results of the analytical and experimental portions of this investigation, as well as information obtained regarding the use of RRFC bridges in other states, are summarized in this report.

2. LITERATURE REVIEW

The initial task of the RRFC bridge project was to conduct a literature search to collect any existing information on the subject. Comprehensive computer and library searches were performed as part of this task. Various personnel from the railroad industry were contacted to answer specific questions. In addition, a questionnaire was sent to members of the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Bridges and Structures to obtain their input on the feasibility of using RRFC for low volume bridges.

The information found in reports and articles obtained through the questionnaire is summarized in the following sections. These sections focus on the experiences of the states of Arkansas, California, Wyoming, and Montana, which provided detailed accounts of their use of RRFC bridges. The attitudes and concerns of other states are also included. A private company who specializes in RRFC bridges, the Skip Gibbs Company from California, was also contacted for its experiences with RRFC bridges. Very little information was found through the computer and library searches; thus, essentially all information presented in this chapter was obtained through the questionnaire or personal contacts. Research reports from Arkansas and Wyoming were discovered through the questionnaire.

2.1 State Department of Transportation Questionnaire

A questionnaire was developed and sent to 59 bridge engineers from across the United States and Canada (see Appendix A for questionnaire). The

goal of the survey was to obtain information on the use of RRFC bridges in other states and to identify any problems with these bridges. The questionnaire also provided current information on completed research and on the opinions of bridge engineers related to RRFC bridges.

2.1.1 Questionnaire Results

Of the 59 questionnaires sent, 49 responses were returned, giving an excellent return rate of 83%. Nearly one half of the respondents (24) noted that railroad flatcars were used as bridges in their state. Presented in Table 2.1 is the number of RRFC bridges in the states that responded to the questionnaire.

States with large rural populations such as Oklahoma, Texas, Arkansas, and Montana reported the highest use of RRFC bridges. States located on the East Coast noted that these types of bridges would not be practical in urban areas.

According to the survey results, Arkansas is the only state that had conducted research on RRFC bridges while Wyoming had three RRFC bridges load tested by a bridge consulting firm. No state reported any permanent RRFC bridges on

Table 2.1. RRFC bridges in states responding to questionnaire.

Number of RRFC bridges	States	Percent
0	25	51.0
1 - 10	6	12.3
11 - 25	8	16.3
26 - 50	2	4.1
51 - 100	3	6.1
100 +	3	6.1
unknown	2	4.1
	<u>49</u>	<u>100.0</u>

their state highway system; all flatcar bridges were reported to be located on county or private roadways. The California Department of Transportation (Caltrans) permits RRFC bridges to be used as temporary bridges in emergency situations.

2.1.2 Questionnaire Comments and Concerns

The responses from State Bridge Engineers regarding the use of railroad flatcars on low volume road bridges are listed below. Many of these comments are shared by the Bridge Design Standard Practices Committee of Connecticut (a division of the Connecticut Department of Transportation) which had held a meeting to determine if railroad flatcars should be used by the state.

- Contractors in our state have used them for temporary detours during construction. They seem to work well.
- Two cars side by side with a unified reinforced concrete deck and supported at the original wheel locations make very good bridges.
- If properly retrofitted, installed, and maintained, RRFC bridges appear to serve low volume roads adequately.
- They are low cost and easy to install. Good for low volume roads with few trucks.
- RRFC bridges have proven to be a low cost alternative for replacing old timber structures.
- There may be some very limited use if they can be transported and lifted.
- The idea has merit if there is a readily available supply of inexpensive railroad flatcars.
- What would the deck system be?
- These bridges may be temporary in nature.

- Concerned with profile depth compared to other bridge types and the impact on hydraulic opening under bridge.
- We have concerns about installing adequate railings.
- They are very difficult to structurally evaluate. Recommended for pioneer type roadways only.
- Since the load history of flatcars is not known, estimation for the remaining fatigue life would be difficult, if not impossible.
- Flatcars are not appropriate for skewed bridges without extensive modifications.
- It is doubtful that flatcars would be considered aesthetically acceptable.
- Don't do it!

Taking into account many of these reasons, the committee in Connecticut decided not to support the use of railroad flatcars for permanent bridges, but would consider their use of as components of temporary bridges [1].

In response to the questionnaire, the states of Arkansas, California, Wyoming, and Montana provided detailed accounts of their involvement with RRFC bridges. Caltrans developed an emergency bridge composed entirely of railroad flatcars. To insure public safety, the state of Montana has developed a posting policy for railroad cars used as bridges. The experiences of these four states are described in more detail in the following sections.

2.2 Arkansas Research and Experience

According to its response to the questionnaire, Arkansas has been involved in the use of more than 340 railroad car bridges composed of various types of railroad cars including flatcars, gondola cars, and boxcars. In 1991, Arkansas State University studied bridges constructed from railroad cars for the Arkansas

State Highway and Transportation Department (AHDT) [2]. The US Department of Transportation and the Federal Highway Administration also supported this project. The wide range of objectives of the study included determining present and future use of railroad car bridges, development of a railroad car data archive, and development of load ratings software for railroad cars. Of particular interest to this investigation were Arkansas' findings on the use of railroad car bridges in their state and results from field load tests of two RRFC bridges.

2.2.1 Use of Railroad Car Bridges in Arkansas

In the Arkansas State University study, a survey was sent to the county judges and city governments to determine the interest and use of railroad car bridges in the state. No city government reported the use of railroad car bridges. Approximately one half of the counties reported using railroad bridges for a total of 167 railroad bridges in the state. Of these bridges, 128 were single span bridges while two bridges consisted of three or more spans. Three bridges had a span length less than 20 ft while 57 bridges had a span length greater than 56 ft. The most common railroad car bridge consisted of two cars side-by-side but several bridges were only one car wide. Due to the manner of construction, a gap was left in the bridge deck in a few bridges that were two flatcars wide.

The investigators in this study visited 27 railroad car bridges in six counties to document differences in construction and types of railroad cars. Several methods of attaching the railroad car to the abutment were observed during the field visits. The most common method was placing the cars on top of concrete or steel abutments. Other methods included casting the RRFC into

concrete abutments and coping the ends of main beams to maintain the road grade. The 27 bridges visited were made up of 52 railroad cars. Of these, only eight bridges used flatcars with the majority being boxcars with their sides and tops removed. In a few cases, the date of manufacture, which ranged from 1957 to 1979, was printed on the cars. A significant number of railroad cars had structural damage, mainly deformations of the exterior longitudinal members. The investigators did not note these damages as a problem.

2.2.2 Load Rating Program and Field Testing

A part of the Arkansas study included the field testing of four railroad car bridges, two of which were constructed from RRFC. The purpose of the testing was to check the accuracy of a finite element computer program that was written to calculate the load rating of the individual railroad cars. For the load tests, the bridges were instrumented with 30 to 54 strain gages on the structural frame of the cars. With the use of AHDT trucks, static load tests were performed with the trucks positioned to maximize the forces (and thus the strains) in the various structural members. The dynamic response of the bridges was also obtained by driving the trucks over the bridge at normal operational speeds.

As part of the Arkansas project, a load rating program was developed which predicted the load rating of individual railroad cars. After the properties and spacing of the railroad car members were input, a finite element analysis determined the moment capacity of each structural member. Dead load and vehicle loads were used in the computer model to determine factored dead and live load moments in each member. Using this information (factored moments

and member capacities), a bridge rating was calculated. The results of this program were compared with results from field tests. Only the load tests performed on bridges composed of flatcars are discussed in the following paragraphs.

The first RRFC bridge load tested, which had a clear span of 81.6 ft, was constructed by placing two flatcars (with tapered floor beams) side-by-side. The two flatcars were connected by welding 4 in. channels between the adjacent outside girders on approximately 4 ft centers. A thin layer of asphalt was placed over the existing steel deck to provide a road surface. The center girder of the flatcars had a depth of 11.50 in. at the ends and 25.75 in. at the center. Small channel sections provided the exterior girders of the flatcars. The truck used for this test was an empty single-axle dump truck weighing 11 kips. For the static load test, a maximum strain of 158 microstrains (4.6 ksi) was measured in the center girder at the quarter span of the bridge with the truck positioned at the center of one flatcar. Because of the tapered center member and its large moment of inertia in the region near midspan, the centerline strains were only 82 microstrains (2.4 ksi) during both the static and dynamic load tests. There was major buckling in the center girder at the one-third point in the span of one of the flatcars that experienced a maximum strain of 121 microstrains (3.5 ksi), approximately 45% higher than estimated. The estimated strain was determined by averaging strains recorded at both ends of the buckle.

The second RRFC bridge load tested, which also consisted of two flatcars adjacent to each other, had a clear span of 73.7 ft. The flatcars were connected

together with 7 in. channels welded to the side girders. An asphalt deck over the flatcar steel deck provided the road surface. The center girder of the flatcars had a depth of 13.38 in. at the ends and 30.77 in. at the center. Small angle-shaped members made up the exterior girders of the flatcars. For loading, the same truck was used as in first bridge. In addition, a single-axle dump truck loaded with gravel (weighing 29.3 kips) was also used in the testing of the second bridge. From the strain data, it was observed that a maximum strain of 180 microstrains (5.2 ksi) occurred in the center girder when the loaded truck was centered with respect to the width of the car. When the truck was positioned on the edge of the flatcar, the exterior girder experienced a maximum strain of 413 microstrains (12 ksi). For these data, the rear-axle of the truck was located at the longitudinal center of the bridge.

Results obtained from the finite element program that was developed were compared with the field test results. It was concluded that the load rating program predicted the behavior of the flatcars with reasonable accuracy, but the Arkansas research report did not include a specific load rating for either RRFC bridge. Even though specific load carrying capacities were not reported for these bridges, the load test results and bridge descriptions provide information that can be compared with field testing done by other states (including Iowa) to help understand the behavior of RRFC bridges. The first bridge, with the main girder depth of 25.75 in., had a maximum stress of 4.6 ksi due to a gross load of 11 kips. The second bridge had a main girder depth of 30.77 in. and a maximum stress of 12 ksi due to a gross load of 29.3 kips. It should also be noted that the

exterior members of both flatcar bridges are relatively shallow in comparison to the main center girders.

2.3 California Emergency Bridge System

Because of California's vulnerability to natural disasters, a quick, reliable, and inexpensive method for reopening interstates and roads is needed. With over 80 RRFC bridges privately used in the state of California, the RRFC bridge concept was used to develop an emergency bridge kit. A modular steel, multilane freeway bridge was constructed and evaluated in March 1994 by Caltrans as a temporary bridge for emergency freeway repair. The structure can be erected on-site within a few days without extensive site preparation and provides an inexpensive and reliable way to restore traffic.

The bridge system, shown in Figure 2.1, incorporates salvaged 53.5 ft railroad flatcars. A single car is placed on the ground upside down to act as a footing. Another car is cut transversely in half, each half then being placed vertically at the ends of the footing flatcar to make up the two columns. These footing and column elements are then topped with a single flatcar, placed right side up, on top of the column elements. This method, using a total of three railroad flatcars and miscellaneous attachment clips and cross bracing, forms a bent system. These bent systems serve as abutments or piers that support the superstructure of the bridge, which is made of four flatcars placed side-by-side. Together, the bents and the superstructure complete a 42 ft wide by 53.5 ft long single span of the temporary bridge system. A modular open-grid steel deck is specified for speed of erection, but heavy steel plate or a less expensive

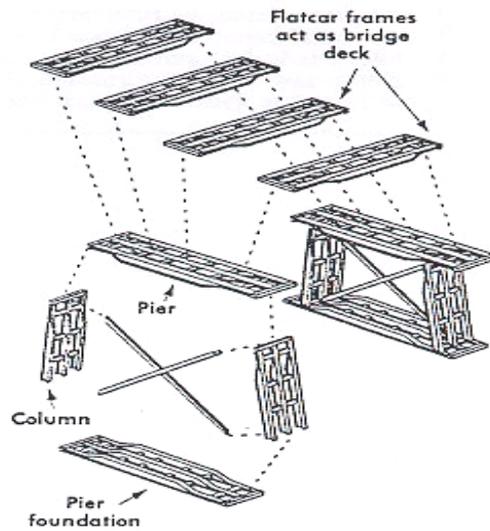


Figure 2.1. Single span of Caltran's emergency bridge system [3].

concrete deck is also acceptable. Caltrans' temporary bridge kit has enough flatcars to make three such spans to form a bridge structure 42 ft wide and 160.5 ft long.

Caltrans tested the as-built modular bridge with a static load of 110 metric tons (242 kips) at the center of the span [4]. The observed maximum vertical deflection was only 0.3 cm (0.12 in.). W. H. Wattenburg, a scientist with Lawrence Livermore National Laboratory who proposed the bridge idea to Caltrans, constructed a finite element model of the modular bridge system to understand its behavior due to earthquake aftershocks using ground motion data from the Petrolia-Cape Mendocino earthquake. The analytical behavior of the modular bridge showed little amplification of ground motion in the transverse direction. In the longitudinal direction, the behavior was close to that measured in permanent conventional bridges during the Petrolia-Cape Mendocino

earthquake. Based on the static load test and the analytical dynamic analysis, the modular flatcar bridge exhibited the ability to withstand significant earthquake ground motion. The static and dynamic test results also indicated that this inexpensive modular bridge could be used for permanent bridges in many areas where funds are not available for bridges of current designs.

When Interstate 5 collapsed over Arroyo Pasajero Creek in the spring of 1995, Caltrans successfully employed its emergency bridge kit. At this site, Caltrans needed a three-span bridge with a deck that was four flatcars wide for one-lane of traffic in each direction. With a few on-site modifications, such as replacing the substructure with steel H-piles and angle cross bracing and using steel shims to acquire a uniform roadway surface, the flatcar bridge system allowed traffic to be restored only eight days after the collapse. The cost to install the flatcar bridge system was estimated at approximately \$19,000 per flatcar with decking and modification costs included. Considering the benefit of having the temporary bridge in place to avoid the cost of detouring freeway traffic, Caltrans estimated a net savings of about \$500,000 at the Arroyo Pasajero site [7].

2.4 Wyoming's Bridge Tests

From the questionnaire, it was found that the state of Wyoming has approximately 25 RRFC bridges on their county road system and is responsible for load rating them. The Wyoming DOT chose to have load testing performed on three RRFC bridges by a private firm. Bridge Diagnostics, Inc. (BDI) performed the load testing, developed analytical models, and recommended load

ratings for the three bridges. Presented in the following sections are brief descriptions of the three bridges, the testing procedures used, and the suggested rating for each of the RRFC bridges [8].

2.4.1 Bridge CN13-41

This bridge was a single span composed of a single railroad flatcar located on a county road with minimal traffic. Transverse timber planks with longitudinal timber runners were used for the bridge deck. The main span of the bridge was 50 ft with an overall width of 10.6 ft and roadway width of 8 ft.

A tapered main girder centered down the length of the car and two 13 in. deep channels for the exterior girders are the primary structural members in the railroad flatcar. The main girder is riveted and is composed of two 35 in. deep sections connected with a 1/2 in. top plate. Four large tapered floor beams tie the exterior beams to the main girder and several smaller channel diaphragms carry loads from small Z-shaped stringers to the main girder and exterior beams. This bridge also had unusual support braces at the abutments. Several 4 in. diameter pipe sections were attached at approximately a 45-degree angle between the abutment piles and the interior girder and exterior beams. It was believed that these support braces were added to provide additional torsional stability to the one-flatcar bridge.

During the load test, the majority of the instrumentation was placed on the main girder and exterior beams. However, several diaphragms, Z-shaped stringers, and the pipe braces were also instrumented. Attention was given to

the effect that the pipe bracing had on the support conditions of the bridge. A tandem-axle truck with a weight of 34.66 kips was used for the load test.

After the load test, an initial review of the load data indicated that all responses were linear-elastic and there were no signs of distress in any of the members. It was also determined that the pipe supports had a significant impact on the support conditions and produced a negative moment region in the exterior beams. A maximum compressive stress of 3.2 ksi occurred in the negative moment region of the exterior beams. The main girder experienced a tensile stress of 2.7 ksi while the Z-shaped stringers experienced a tensile stress of 1.6 ksi.

Using the load test data, BDI refined their analytical model and determined a load rating for the bridge. Based on the load test and analysis results, it was determined that the bridge superstructure could safely carry unrestricted design loads. Load ratings of 48, 85, and 79 tons were calculated for rating vehicles: Types 3, 3S2, and 3-3, respectively. The load rating values were for the main components of the flatcar and did not consider the condition of the timber deck or abutments. It was also noted that because the bridge consisted of a single flatcar, the pipe bracing significantly improved the torsional stability of the bridge.

2.4.2 Bridge CN18-200

This bridge, which is also on a low volume county road, is made up of two railroad flatcars placed side-by-side supported on stone abutments. Transverse timber decking and longitudinal runners provide a roadway width of 12 ft. The main span of the bridge is 50 ft with an overall width of 21.5 ft. Each flatcar

contains two riveted C-sections for the exterior girders and a W30x190 beam down the center of each car. Large riveted diaphragms are placed at approximately the one-third points and smaller channel diaphragms are spaced at 4 to 5 ft intervals. Small I-beams and timber stringers span over the small channel diaphragms and butt into the large diaphragms.

In the load test, the majority of the instrumentation was placed on the main girder and exterior beams. Several diaphragms were instrumented along with a timber stringer and one small I-beam. A tandem-axle truck with a weight of 35.58 kips was used for the load test.

After the load test, an initial investigation of the load data indicated that all responses were linear-elastic and there were no signs of distress in any of the members. A maximum tensile stress of 4.4 ksi occurred at an interior channel. The main girder experienced a maximum tensile stress of 3.7 ksi while the exterior channels experienced a maximum tensile stress of 2.9 ksi.

Based on a refined analysis and the load test, BDI determined that the superstructure of the bridge could safely carry unrestricted design loads. Load ratings of 33, 64, and 69 tons were calculated for rating vehicles: Types 3, 3S2, and 3-3, respectively. It was again noted that for this load rating, the condition of the timber deck and substructure was not considered. During the field test, it was noted that the abutment showed signs of severe cracking and separation from the wing walls, a condition that existed prior to the load test. Obviously, the condition of the abutment should also have been considered when one determines the overall load rating of this bridge.

2.4.3 Bridge CN18-151

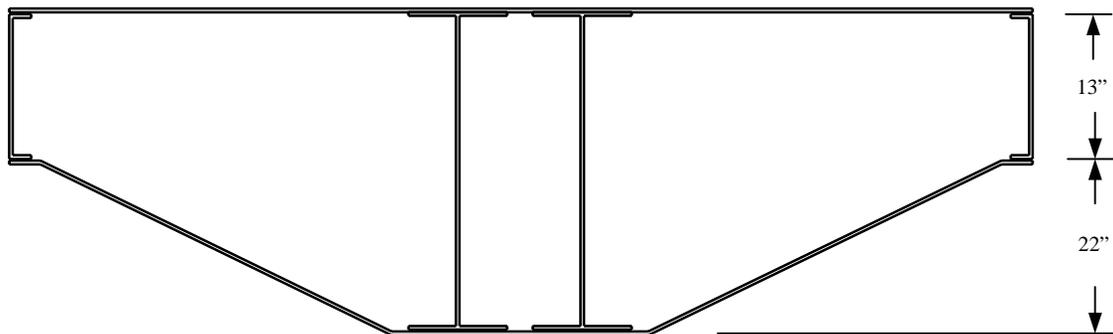
This bridge, on a county road with minimal traffic, consisted of three-spans with the main span having two railroad flatcars placed side-by-side. Timber bridges make up the two approach spans. Since the flatcar portion of the bridge (spanning 36 ft with an overall width of 18.2 ft) controlled the current load ratings, it was the focus of the test. The bridge has a timber deck with a roadway provided by timber runners. Each flatcar frame is composed of a single relatively shallow main girder, located along the car centerline. Several main transverse members are essentially cantilevered from the main girder (minimal support provided by the exterior girders) and carry secondary timber stringers. The main girder is made from two 15 in. deep channels placed back to back with a 1/4 in. top plate attached to the top flanges. Since the exterior girders are relatively small angles (6 in. x 3 1/2 in. x 3/8 in.), the main girder acted as the only significant longitudinal member.

In the load test, the majority of the instrumentation was placed on the main girder and small exterior angles. A tandem-axle truck with a weight of 35.58 kips was used for the load test. The initial findings of the load test revealed that the main girders in both flatcars have a high probability of failure when subjected to heavy loads. Stresses on the main girder of one car reached 27.8 ksi, which is significantly higher than the stresses found in the first two flatcar bridges tested. The exterior stringers were relatively flexible and any load carried by these members was transferred back to the main girder by the transverse floor beams.

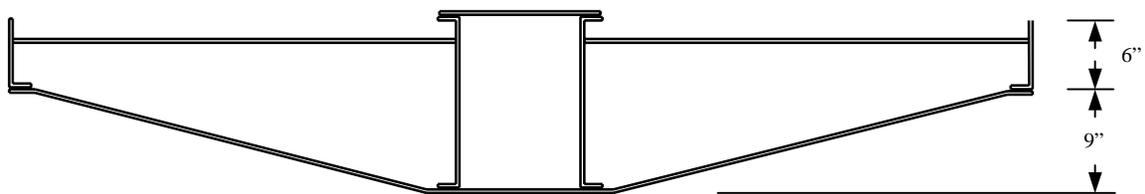
Because of the high stresses in the main girders, BDI recommended a load limit be posted on this bridge of 5.7, 10, and 10 tons for rating vehicles: Types 3, 3S2, and 3-3, respectively. Because of the non-redundant geometry of these particular flatcars, the main girder becomes the sole carrier of the truck loads to the abutments. Any cracks or deterioration of the main member could cause the bridge to fail. The small exterior members provide minimal lateral support which subjects the main girder to significant torsion and lateral bending when loads are not applied symmetrically over the main girders. For these reasons, BDI does not recommend using this type of flatcar in highway bridges.

2.4.4 Overall Conclusions

After the field load testing of three RRFC bridges, BDI concluded that railroad flatcars appear to be an adequate solution for a bridge superstructure on low volume roads if the railroad cars with appropriate geometry are selected and maintained. From the three load tests, it can be concluded that the effectiveness of railroad flatcar bridges depends greatly on the particular design and style of car. Flatcars with relatively deep main girders and significant exterior stringers appear to function very well under legal loads, as determined in the first two load tests. Flatcars with shallower main girders and non-redundant designs should be avoided, as seen in the third bridge test. Shown in Figure 2.2 are general cross-sections of the two types of flatcars discussed. Presented in Figure 2.2a is the redundant cross-section found in the first two bridges tested with its deep interior and exterior girders. In this particular situation, redundancy refers to a RRFC design in which more than one longitudinal load path is present (i.e., exterior



a. Redundant cross-section



b. Non-redundant cross-section

Figure 2.2. Comparison between two types of flatcar cross-sections.

girders in addition to the main interior girder provide load paths to the abutments). Illustrated in Figure 2.2b is the non-redundant (i.e., interior girder provides the only load path to abutments) cross-section in the third bridge tested. Because of the shallow interior girder and small exterior girders, these types of flatcars do not provide for an effective bridge superstructure. From the results of the first two load tests, it can also be seen that the redundant flatcars produce an effective bridge superstructure in both single and double wide configurations.

From the field inspections, BDI noticed that each bridge structure is unique and retrofits (such as adding tension rods, support pipes, or secondary steel stringers) during field installation could affect the load path. With these modifications, it may be difficult to evaluate a structure without some basic field testing. As noted in the second field test, the decking, abutments and support conditions were not studied by BDI; these elements, however, may be a controlling factor in determining a load limit for railroad flatcar bridge structures. A final note was made concerning the fatigue life of the flatcars. Even though the amount of remaining fatigue life for these structures would be virtually impossible to quantify, BDI believes that fatigue should not be a major issue in low volume traffic environments.

2.5 Montana Load Rating Procedure

Montana reported that as many as 100 RRFC bridges exist on their private and county road system. To insure the safety of the traveling public, the Montana Department of Transportation (MDT) is required to inspect and load rate all structures with a span greater than 20 ft. The MDT does not have the resources available to accurately rate RRFC bridges, and because of the uncertainty of the size, condition, and material strength of the flatcar members, the MDT has established the following load rating procedure for railroad car bridges [9]:

Option 1) Assign a five ton limit unless Option 2) or 3) is used.

Option 2) The county may load test the bridge by placing a vehicle of known weight on the superstructure. The axle configuration, front to back, and the weight applied to each axle along with a picture of the test must be submitted

to the Bridge Bureau. The Bridge Bureau will then convert the test truck position into an equivalent weight given for a type 3 truck configuration. The posting limits will be 40% of this weight. The reduction is necessary to account for the effects of impact and the factor of safety involved in an inventory rating.

Option 3) Hire a consultant engineer registered as a professional engineer in Montana to gather information and to accomplish the work outlined below. Develop a cross sectional diagram showing the size and spacing of structural members to determine how the live load is distributed. Establish the condition of the railroad car, which includes providing all information supporting that determination. This will likely require some sort of nondestructive test of steel elements. Perform a structural analysis to determine the load carrying capacity of the car as a bridge. After the above work is completed and received by the Bridge Bureau, a calculated load rating will be assigned to the bridge (p. 5).

2.6 The Skip Gibbs Company

The Skip Gibbs Company, which specializes in completed ready-to-install bridge superstructures in the Western United States, was contacted for information regarding their experience with RRFC bridges (contact information is provided in Appendix D). A sampling of their projects shows that RRFC bridges have been successfully used in a variety of bridge situations. They have installed several hundred RRFC bridges as temporary bridges on a variety of roads and as permanent bridges on low volume rural roads [10]. Many of Skip Gibbs' clients include county and state governments, logging and timber companies, heavy construction contractors, and mining and gravel companies. They provide bridges that are capable of carrying AASHTO highway loads, extra heavy off-highway loads and even light recreational loads. The following agencies and companies are only a few who have approved the use of Skip Gibbs flatcars and have placed them in service [11,12]:

- The County of Davis, UT installed a heavy haul road bridge to a state park. The bridge carried 6,700 vehicles weighing 125 tons each.
- The U. S. Bureau of Reclamation used railroad flatcars for a dam project bridge in Shasta County, CA that carried 16,000 vehicles weighing 88 tons each. This agency also used two flatcar bridges for a temporary detour of U. S. Hwy 93 (12,000 vehicles per day).
- The County of Humbolt, CA has permanently installed several single and double flatcar bridges including one bridge within range of ocean salt spray.
- The Pacific Lumber Company installed three permanent haul road bridges in Fortuna, CA.
- The Kasler Corporation used railroad flatcar bridges for freeway construction bridges that carried 19,000 vehicles weighing 93 tons each and 2,800 vehicles weighing 50 tons each.

These projects provide evidence that RRFC provide effective bridge superstructures. According to Skip Gibbs, savings can typically range from 30 to 70% when compared to traditional bridge designs for several reasons [13]: installation speed, length of span, ease of design, long life, low maintenance, and low initial cost. The weight of a flatcar superstructure is usually much less than the weight of traditional bridges, thus lowering the dead load of the bridge. Simple supports to a properly designed substructure allow for practical bridge loadings. When used for temporary purposes, a flatcar can be easily moved and re-used several times, thereby increasing its economy.

2.7. Field Inspection by Research Team

In Grant County, Oklahoma there are over 40 RRFC bridges in service. The majority of these are located on unimproved roads, have no guardrail system, and no wearing surface. In a few instances, RRFC bridges have been

used on asphalt and concrete roads. In these cases some type of guardrail system is used. Although some of the bridges inspected are posted, the majority of them are not. All indications are that these RRFC are working well and have provided an economic alternative. See Appendix E for additional details of some of the bridges that were inspected during a field trip by the research team in July 1999.

3. TAMA COUNTY BRIDGE

One of the main focuses of this investigation was to study a RRFC bridge located in Tama County. This chapter provides a detailed description of the Tama County Bridge (TCB) that was both field tested and modeled analytically to help understand the behavior of RRFC bridges. The modeling and testing procedures used are discussed in the following chapters.

3.1 Description of the Tama County Bridge

This bridge consists of two RRFC set side-by-side on timber abutments that span 42 ft over a small creek on a Class B, rural gravel road. Located approximately two miles east of Chelsea, Iowa, the bridge provides access to farming fields and a state wildlife recreation area. A map showing the location of the TCB is provided in Figure 3.1. The actual flatcars that make up the TCB are 51 ft in length but are supported at the bolster locations thus creating a span of 42 ft center to center of abutments. At each end of the flatcars, the remaining 4 ft – 6 in. sections bear directly on the ground. Each RRFC has a width of 9 ft with an 8 in. space between the flatcars. The bridge deck consists of metal grating over the entire bridge surface that is topped by 4 in. x 12 in. timbers across the center 12 ft width of the bridge to create a driving lane. An overview of the bridge is shown in Figures 3.2 and 3.3. As seen in the photographs, only one lane of traffic is possible, no guardrails exist on the bridge, and the bridge is located in a rural setting. A view from underneath the bridge showing the flatcar members and timber abutments is provided in Figure 3.4.

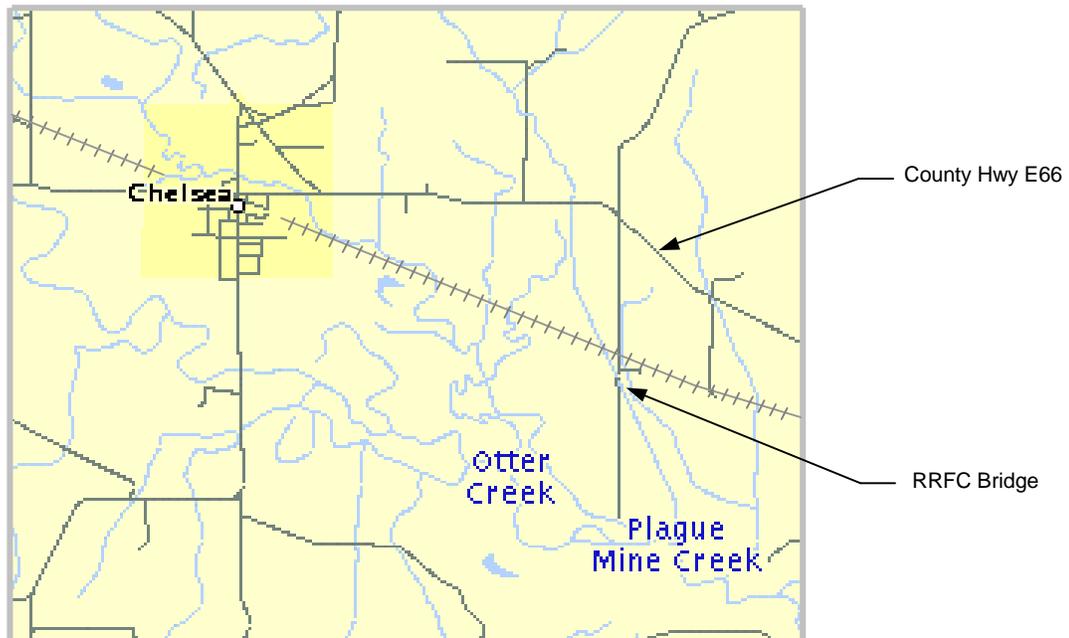


Figure 3.1. Map showing location of Tama County RRFC bridge [14].



Figure 3.2. Elevation view of TCB and south abutment (Note that the girder extends past the abutment cap beam).



Figure 3.3. Top view of TCB looking north (Note the metal grating and transverse timber planks over center width of bridge).



Figure 3.4. South abutment and underside of TCB superstructure.

A plan view of the flatcars that identifies the transverse and longitudinal members is shown in Figure 3.5; each RRFC contains four major longitudinal members and six major transverse members. The exterior longitudinal members of each RRFC are built up C-sections composed of a plate for the web and angles riveted to the top and bottom of the web for the flanges. Each exterior member has a depth of 24 in. at the center of the bridge and tapers to 12 in. at the abutments. Two main interior longitudinal members of each RRFC consist of built up I-sections; each I-section is composed of a plate for the web and four angles riveted to the web for the top and bottom flanges. Each interior member has a depth of 30 in. at the center of the bridge and also tapers to 12 in. at the abutments. A detailed drawing showing the tapering of the longitudinal members is provided in Figure B.1 of Appendix B. A cross-section view (cross-section A-A in Figure 3.5) of one flatcar is shown in Figure 3.6, which includes member sizes and flatcar dimensions.

Each RRFC has six major transverse members that are built up I-shapes consisting of a 1/4 in. web plate attached to a top and bottom flange. The web of the transverse members is also connected to the webs of the major longitudinal members. A 3/8 in. x 9 in. plate that runs the entire width of the flatcar makes up the top flange of the transverse members. This plate also connects the top flange of the four major longitudinal members. The bottom flange is a 3/8 in. plate which also runs the entire width of the flat car but varies in width, 4 in. at the exterior longitudinal members to 9 in. at the interior longitudinal members. At the exterior longitudinal members, the transverse web has a depth of 24 in. and

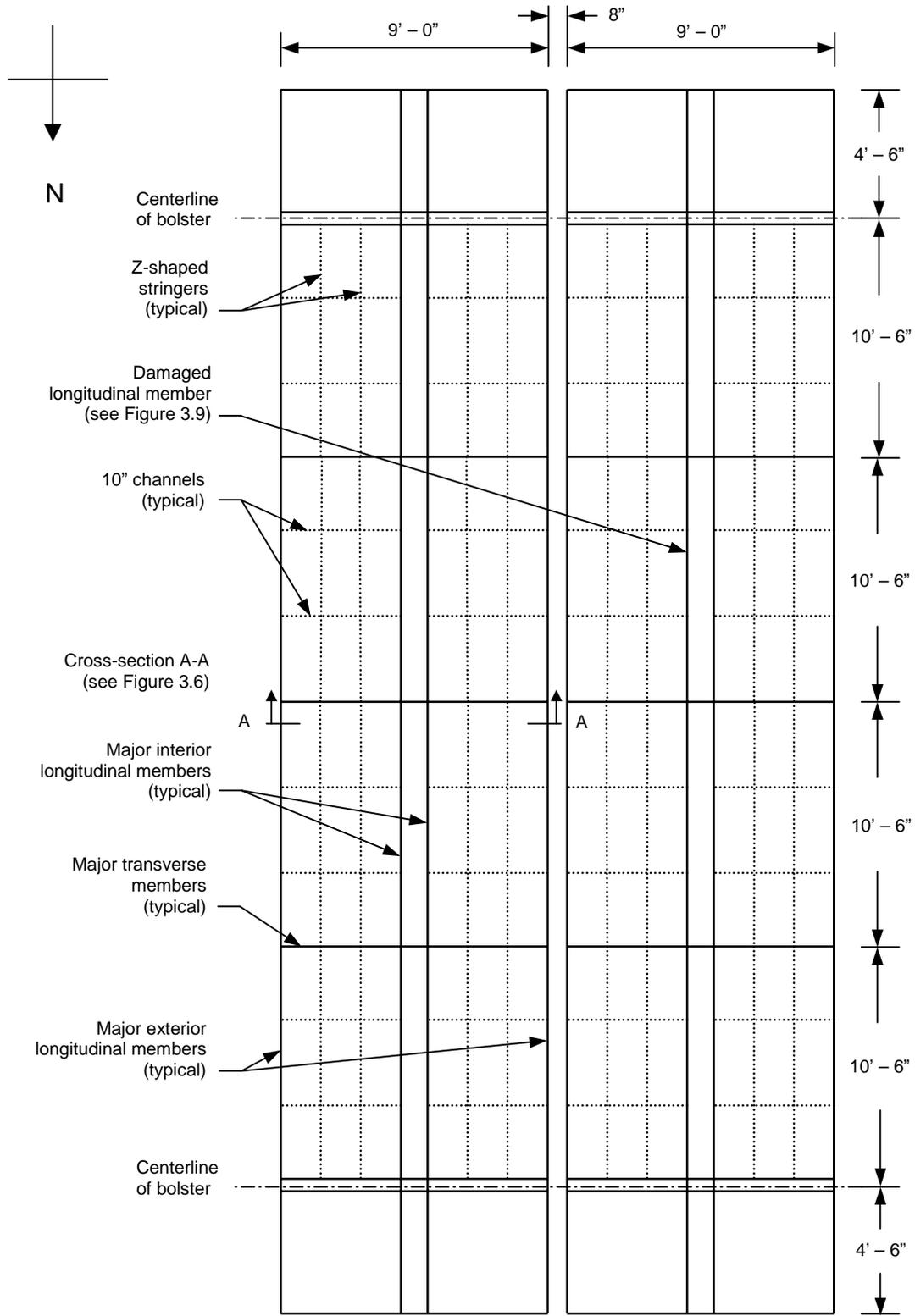


Figure 3.5. Plan view of TCB (Note that timber deck is not shown).

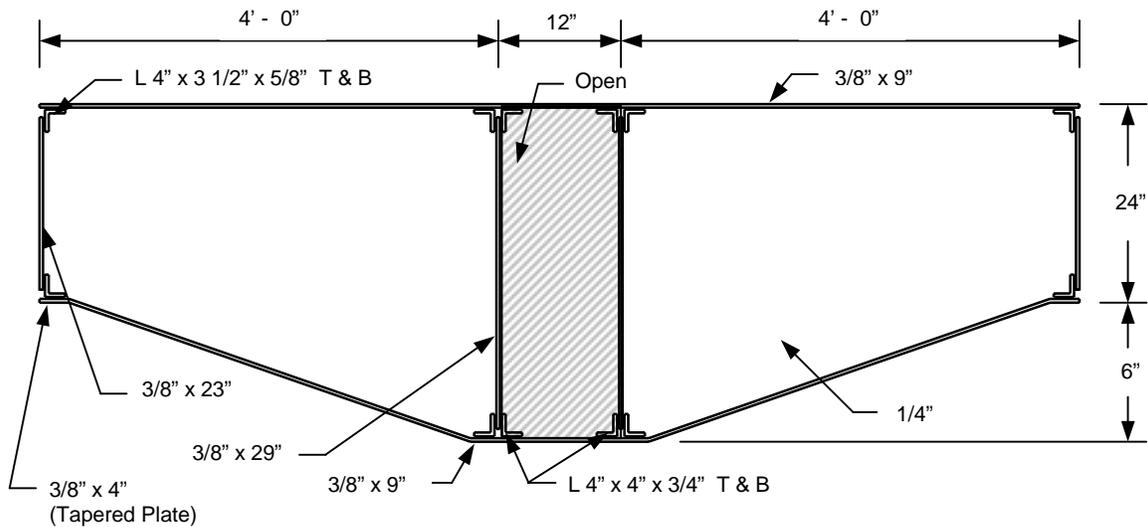


Figure 3.6. Cross-section view A-A (see Figure 3.5).

tapers to a depth of 30 in. at the interior. Unlike the flanges, the web does not extend between the two interior longitudinal members.

As shown in Figure 3.5, each RRFC also contains smaller channels and Z-shaped members that span between the major transverse and longitudinal members, respectively. Between major transverse members, two 10 in. channels span 4 ft between exterior and interior longitudinal members. The Z-shape stringers span 10 ft – 6 in. longitudinally between the major transverse members and are supported by the channels. The primary purpose of these secondary members is to provide support to the deck members and transfer loads from the bridge deck to the major structural members of the flatcar.

3.2 Condition Assessment

The overall condition of the TCB superstructure is not good. Tama County officials noted that these flatcars were damaged before put in service and the

damage did not result from being used as a bridge. Several major transverse members of each RRFC have severe out-of-plane deformations. A damaged transverse member in the west flatcar is illustrated in Figure 3.7. Of the 12 major transverse members in the two flatcars, six are damaged. Three of the six damaged transverse members have damage similar in magnitude to that shown in Figure 3.7. The out-of-plane damage in the other three members is not as severe. In the west flatcar, the exterior longitudinal member located along the longitudinal center of the bridge has significant out-of-plane bending along the entire length of the bridge. This damaged member can be seen in Figure 3.8 along with its undamaged counterpart in the east flatcar. An interior longitudinal member of this same flatcar also has major damage as is shown in Figure 3.9 (See Figure 3.5 for location of this member). The member in Figure 3.9 has



Figure 3.7. Damaged transverse member in west flatcar.



Figure 3.8. Damaged exterior member of west flatcar at center of bridge looking north.



Figure 3.9. Damaged interior longitudinal member in the west flatcar looking south.

out-of-plane bending in its web and bottom flange for the 10 ft – 6 in. between the transverse members. Not only are the structural members in poor condition, many rivets in critical connections are either loose or missing. From the photographs of the damaged members (Figures 3.7, 3.8, and 3.9), it can be seen that the members have undergone horizontal and upward deformations. These types of deflections would not be caused from gravity bridge loads and support the assumption that the flatcars were damaged before installation.

The transverse abutment support locations are different for each flatcar. At the supports of both flatcars, several steel plates (12 in. x 2 1/2 in. x 3/4 in.) are placed between the bolster of the flatcar and the timber abutment at three locations as shown in Figure 3.10. Both the north and south ends of the TCB are supported as shown in Figure 3.10. Steel plates supporting the east flatcar are located at the center and edges of the flatcar; the west flatcar is supported at the center and 27 in. from the both edges. The support conditions for the exterior longitudinal members of the east and west flatcars at the center of the bridge are shown in Figure 3.11. Notice that one longitudinal member is supported by steel plates while the other is not. Because of these support differences, the west car likely has less torsional stability than the east car. Both flatcars are bolted to the timber abutments with two 3/4 in. diameter bolts at each abutment. The location of these bolts is also shown in Figure 3.10. It should also be noted that some of the gravel fill behind the south abutment had been washed out and caused rutting just beyond south abutment from truck loads.

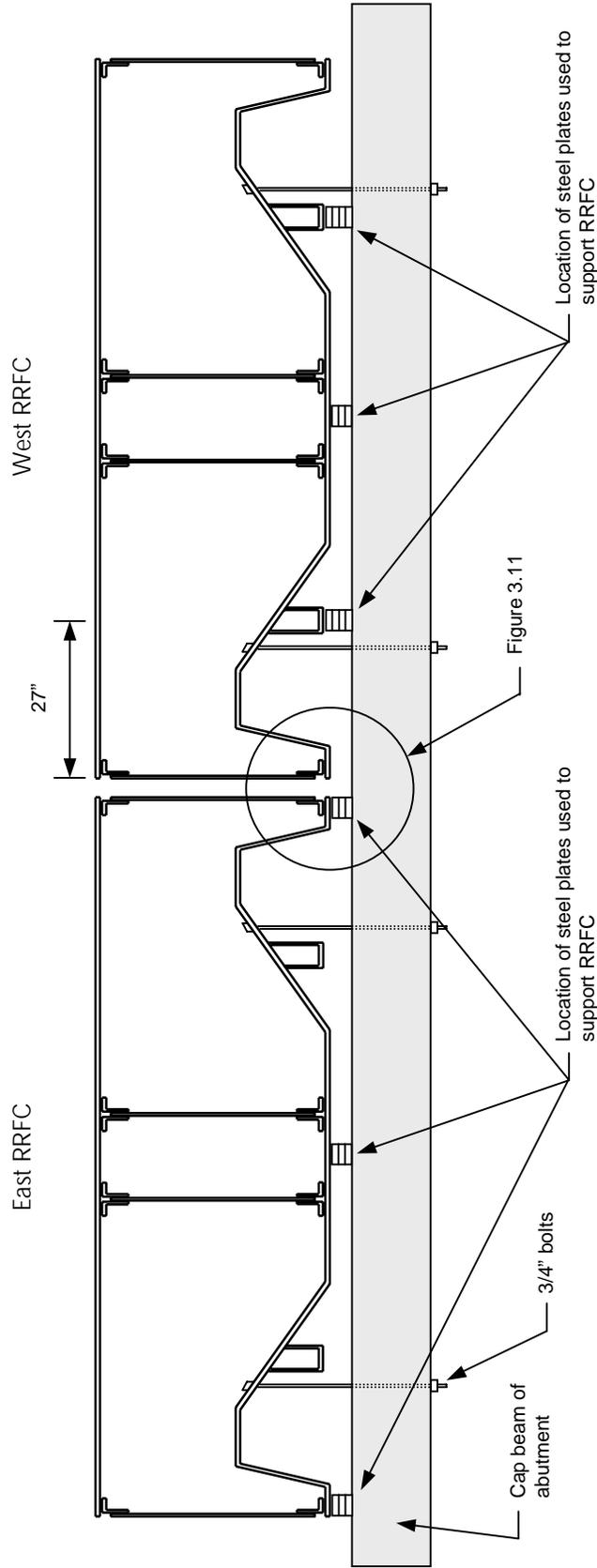


Figure 3.10. Location of TCB abutment bearings.



Figure 3.11. Transverse abutment support locations differed between the east and west flatcars (see Figure 3.10).

With different support locations, the load paths to the abutments are different for each flatcar. For the east RRFC, each longitudinal member provides a direct load path to the abutments since the longitudinal members are directly supported. In the west flatcar, the transverse support members provide the critical load path for loads carried by the exterior longitudinal members since the exterior members are not directly supported. Because of this, the redundancy (multiple load paths to the abutments) of the west RRFC may be questioned. Originally, the deck of the TCB consisted solely of the metal decking. After the bridge had been in service, deformations were noticed in the metal decking and timber deck planks were added. With the use of bolts, the timber planks were attached to RRFC frame. Three bolts were used per timber, two at each end of

the timber and one at the center. All timber deck planks appeared to be in good condition as no signs of splitting or rotting were noticed. The bolts that connected the timbers to the RRFC also appeared to in good condition with no loose or missing bolts.

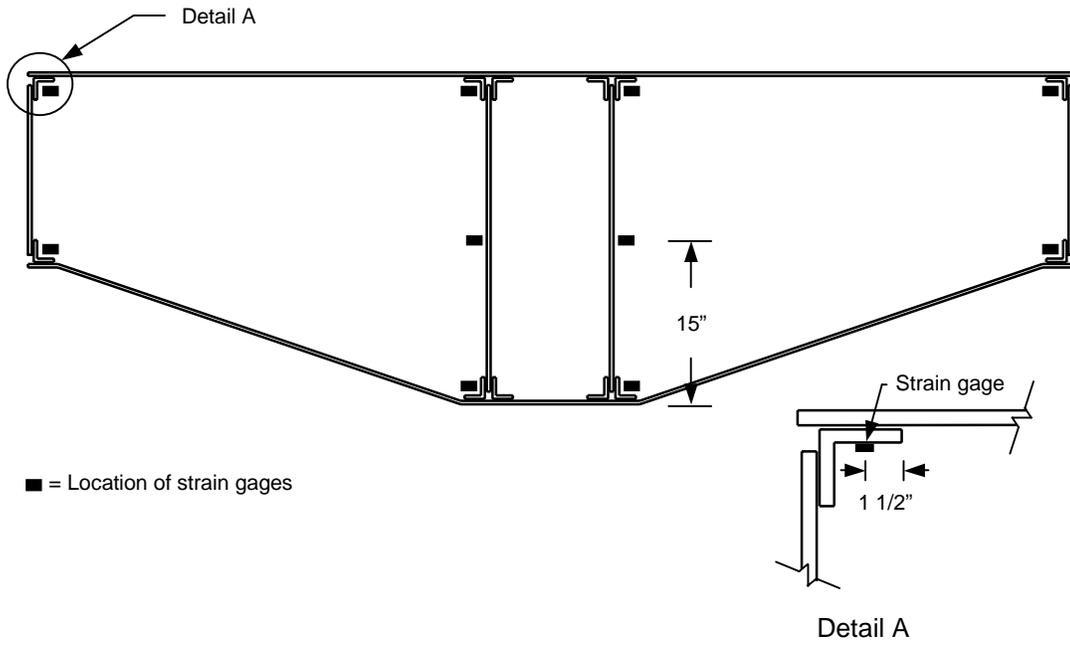
4. FIELD LOAD TEST

In this part of the investigation, a field load test was performed on the TCB. In the following sections, a description of the field load test instrumentation and procedures is presented. The results of the field load test are presented with analytical results in Chapter 6.

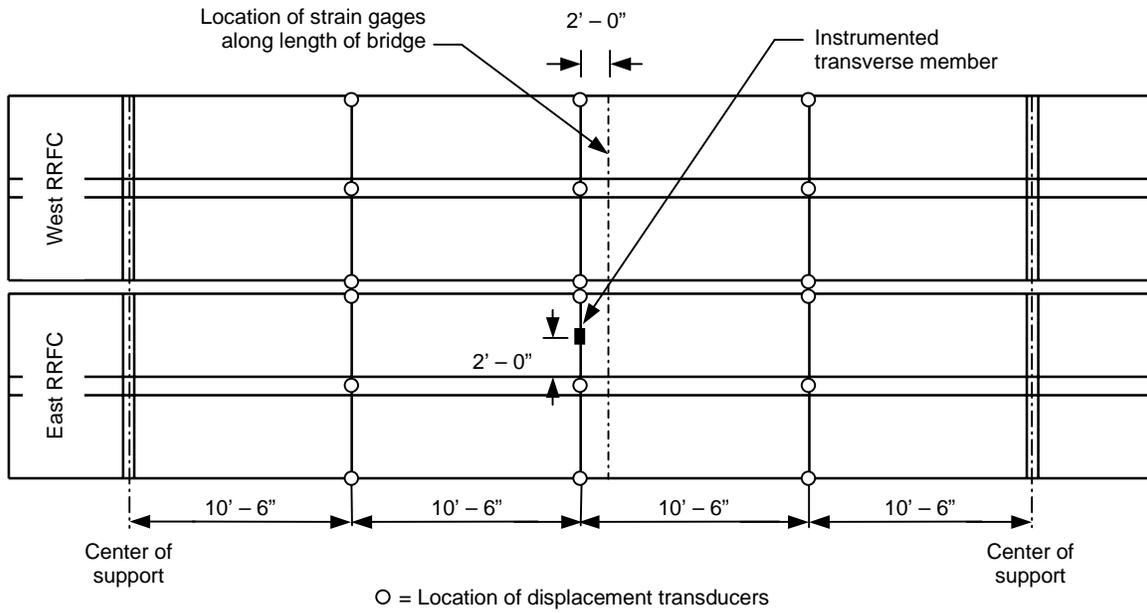
4.1 General Field Test Instrumentation and Procedures

The initial task of the field tests was to evaluate the TCB and to determine the number and location of displacement transducers (string potentiometers mounted on tripods) and strain gages needed to understand the behavior of the bridge. The main longitudinal members of each flatcar were the main focus of the instrumentation.

With simple supports, the maximum stresses were expected to occur at the midspan of the bridge. For this reason, 20 strain gages were placed on the main longitudinal members of both flatcars at the bridge midspan. The location of the strain gages in a cross-section view of the flatcar is shown in Figure 4.1a. Both flatcars have this same instrument placement. On each longitudinal member, strain gages were placed on the top and bottom flanges as well as the web of the interior members. To avoid potential localized strain effects of the connections between the longitudinal members and the transverse members, the strain gages were placed 2 ft from the midspan of the bridge (see Figure 4.1b). In addition to the longitudinal members, two strain gages were also placed on the top and bottom flange of a major transverse member of the east flatcar. This member was the only undamaged transverse member at the midspan of the



a. Strain gage location in cross-section of flatcar 2 ft from midspan



b. Instrumentation location along length of bridge

Figure 4.1. Instrumentation location for TCB load test.



Figure 4.2. Displacement transducers used during field load test of TCB.

bridge and was instrumented 2 ft from the interior longitudinal member. The transverse member and location of strain gages are identified in Figure 4.1b.

Because of the many damaged RRFC members, symmetry was not considered and the bridge was instrumented with displacement transducers along its entire length and width. In the cross-section of each flatcar, a displacement transducer was positioned at each exterior girder and between the two interior girders. Along the length of the bridge, the displacement transducers were positioned at the $1/4$, $1/2$, and $3/4$ spans. The location of the 18 displacement transducers is shown in Figure 4.1b; a photograph of the displacement transducers positioned under the TCB is presented in Figure 4.2.

Because the behavior of RRFC bridges was uncertain at this point, load testing began with an empty truck. After the initial load test, the strain and

displacement results were examined; it was determined that it was safe to increase the truck loading used in the bridge test.

For the first load test, a single-axle Tama County dump truck with an empty weight of 17.30 kips was used. Dimensions and axle weights of this truck are shown in Figure 4.3; a photograph of the truck is provided in Figure 4.4. After this initial load test, as previously noted, the experimental results indicated that the bridge had sufficient strength to support the fully loaded rear tandem-axle truck. Dimensions of the tandem-axle truck and its axle weights are presented in Figure 4.5; a photograph of the rear tandem-axle truck is shown in Figure 4.6. The total load of the second truck was 52.14 kips. For each load test, strain and deflections were recorded with the truck in nine positions on the bridge. These positions (A1, A2, A3, B1, B2, B3, C1, C2, and C3) are shown in Figure 4.7. A symbol (see Figures 5.3 and 5.5) was used to show the location of the rear axle and the direction the truck is heading. The nine positions correspond to the position of the rear axle both across the width and length of the bridge. At the 1/4, 1/2, and 3/4 spans (identified in Figure 4.7), the truck's rear axle is positioned at the center and at the two edges of the timber road surface as shown in Figure 4.8. Truck weights and positions from the field test were used in the computer model so that analytical and experimental data for each truck position could be compared. These comparisons and the results of the field testing of the TCB, as previously noted, are presented and discussed in the Chapter 6.

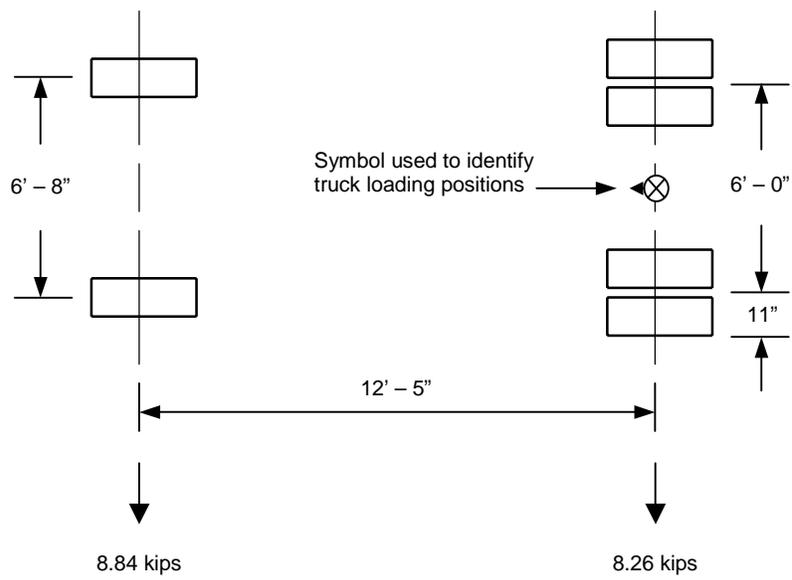


Figure 4.3. Single-axle truck dimensions and axle weights.



Figure 4.4. Photograph of single-axle test truck.

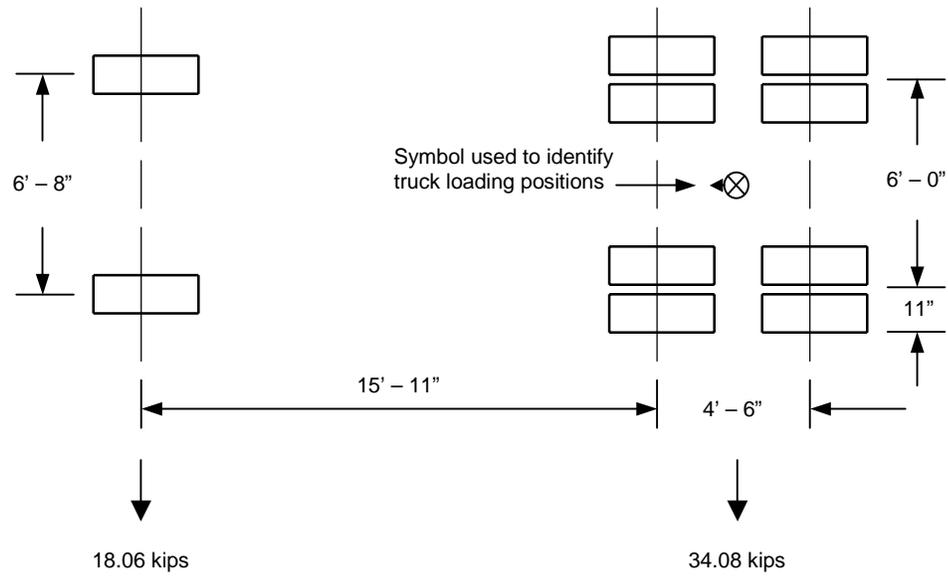


Figure 4.5. Rear tandem-axle truck dimensions and axle weights.



Figure 4.6. Photograph of rear tandem-axle test truck in position C2.

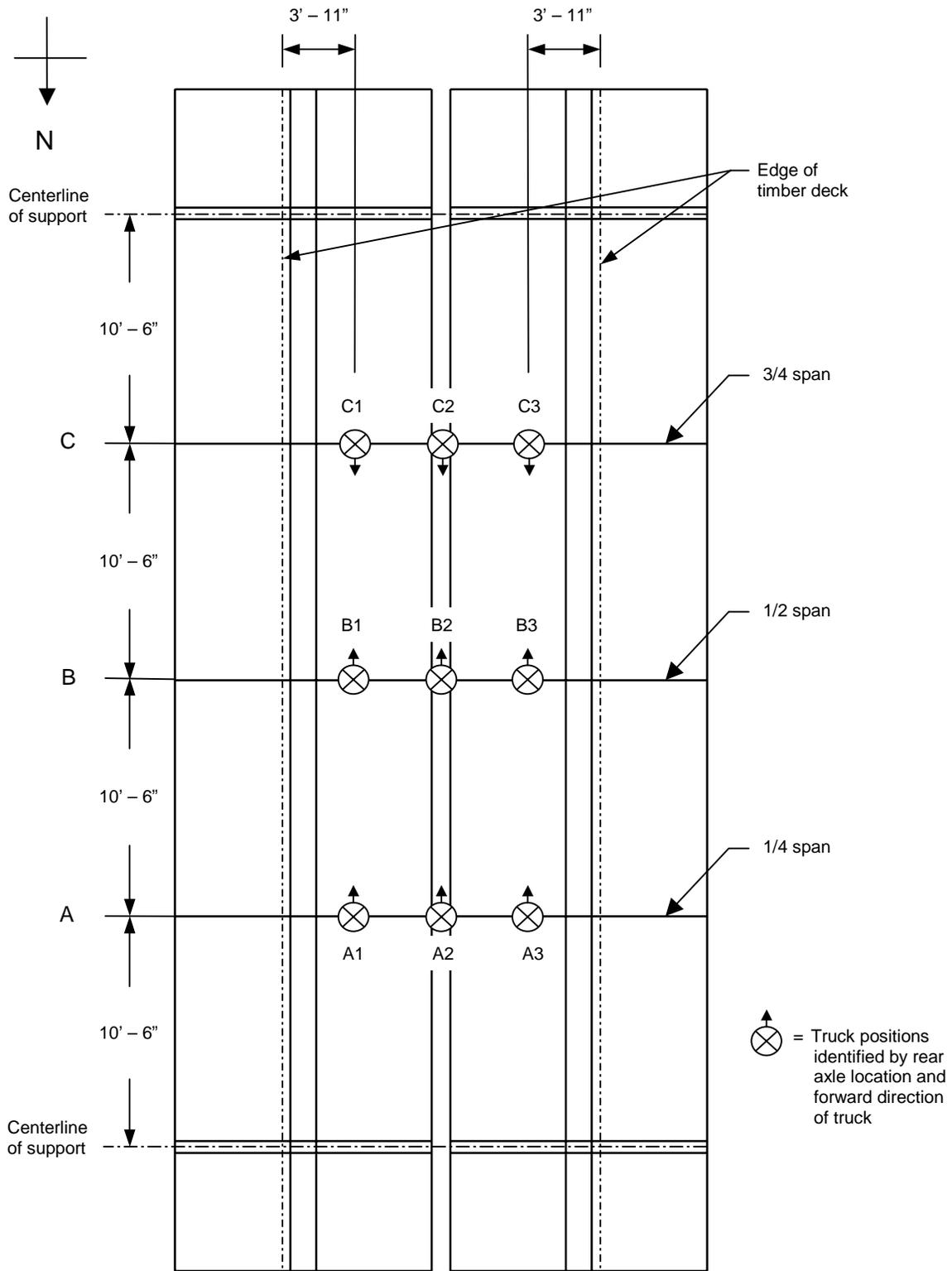
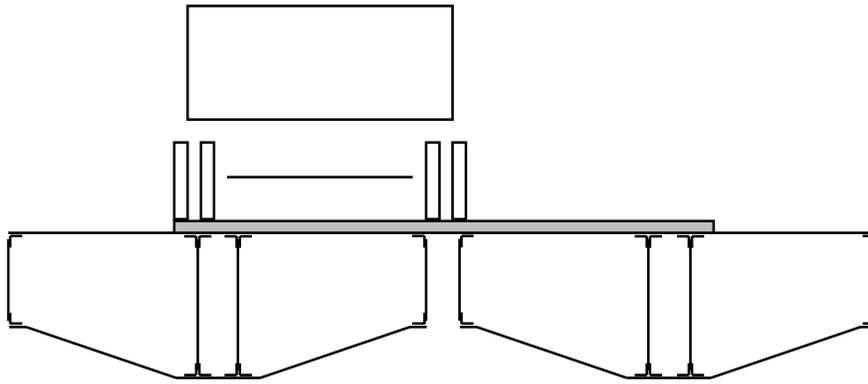
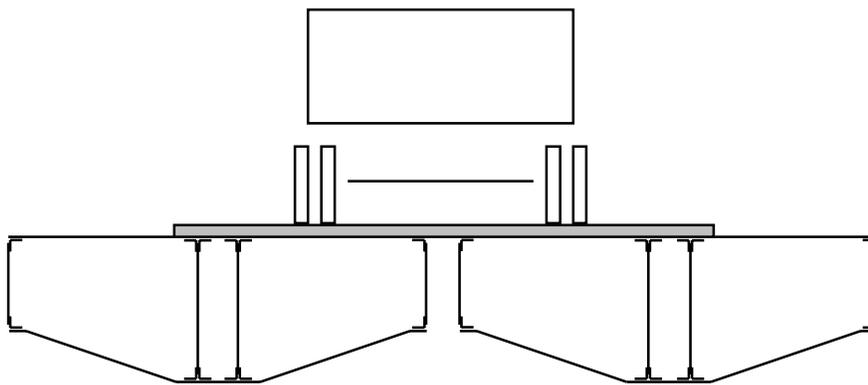


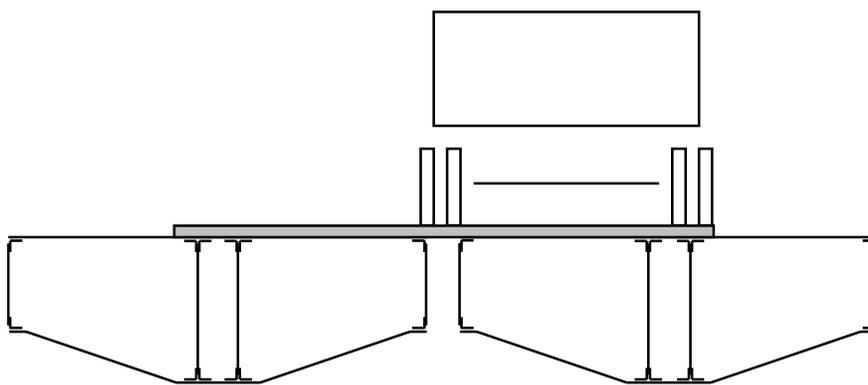
Figure 4.7. Plan view of TCB showing truck load positions.



a. Truck positioned at east edge of timber road surface.



b. Truck positioned at center of timber road surface.



c. Truck positioned at west edge of timber road surface.

Figure 4.8. Transverse loading positions.

4.2 Flatcar Connection Tests

As described in Chapter 3, the timber planks of the bridge deck provide the only connection between the two RRFC in the TCB. While the testing instrumentation was in place, angle-plate connections were made between the two RRFC. The bridge was re-tested using the same truck and same truck positions. A detail of the connections is shown in Figure 4.9. Holes were drilled into the web of an exterior longitudinal member of each flatcar and angles were attached to each flatcar using 3/4 in. bolts. A 3/8 in. plate was welded to each angle to complete the connection. These connections were added at the 1/4, 1/2, and 3/4 span locations along the bridge (see Figure 4.7). A photograph of

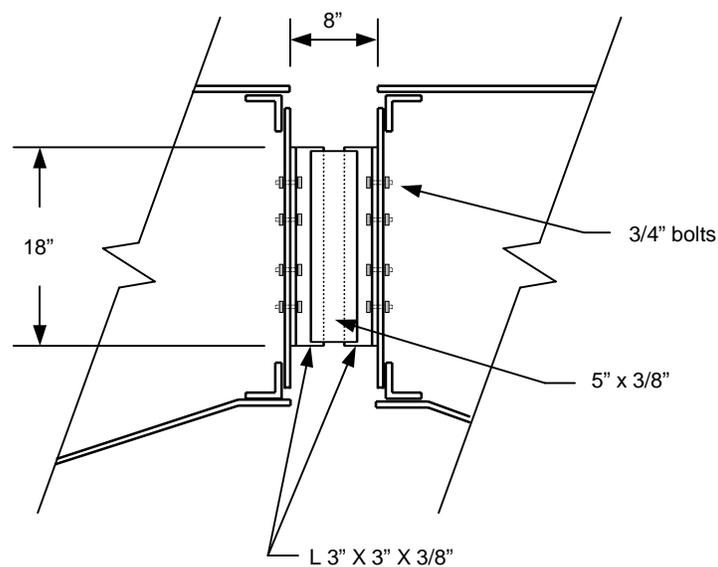


Figure 4.9. Angle-plate connection added between flatcars.

the angle-plate connections added to the TCB is presented in Figure 4.10. Initially, only the midspan connection was added and the bridge was re-tested. After this test, the two remaining connections (at the 1/4 and 3/4 spans) were added and the bridge was tested again. The data obtained from these tests are also presented in Chapter 6.

4.3 Second Field Load Test

As discussed in Chapter 6, questionable strain readings from the field load test were noticed at two locations in the bridge cross-section. For this reason, strain gages at these locations were replaced the bridge was re-tested to verify the results of the first load test. The rear tandem-axle truck used in the first load test was used for the second load test with a weight of 54.28 kips. Strain and

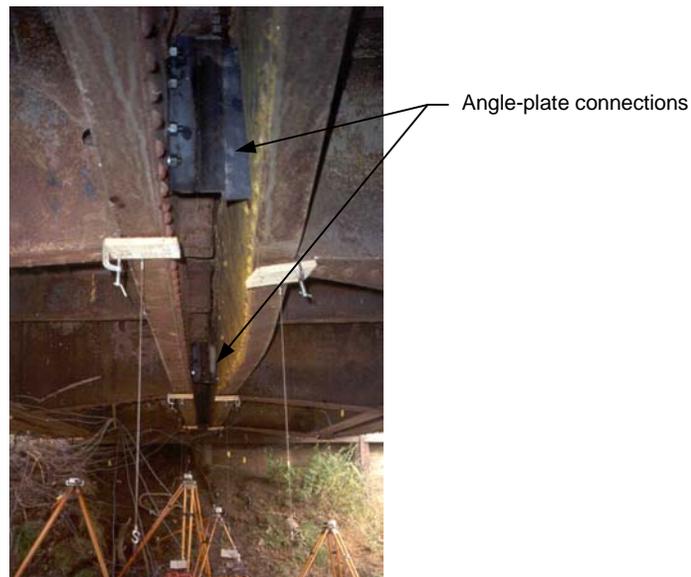


Figure 4.10. Photograph showing angle-plate connection added between flatcars.

deflection data were recorded for the nine truck positions described earlier. Chapter 6 provides the location of the questionable strain readings and the results of the second load test.

5. STRUCTURAL ANALYSIS

To better understand the behavior of RRFC bridges, a grillage analysis was performed on the TCB described in Chapter 3. In addition, structural drawings of two other flatcars were obtained from the railroad industry and the flatcars were modeled for application in RRFC bridges using the TCB geometry. Because so many different flatcar styles exist, these additional models help to better understand the general behavior of RRFC when used in bridges. The following sections provide a detailed description of the structural analysis procedures used to model and analyze the three different railroad flatcars. The results of the analyses are presented in Chapter 6 where they are compared with the results of the field load test performed on the TCB.

5.1 Grillage Modeling of the Tama County Bridge

An analytical model of the TCB was created using the grillage method of analysis. Beam elements were assigned properties of the main longitudinal and transverse members. Even though a grillage analysis is not the most accurate analysis method, this type of analysis was justified instead of a more complex finite element analysis due to the poor condition of the bridge. The following assumptions were made in the development of the analytical model:

- All members of the bridge are considered to be straight and undamaged. For ease of modeling, this assumption was made despite the previously discussed condition of many flatcar members.
- The secondary members and steel grid deck of the RRFC do not contribute to the transverse or longitudinal rigidity of the bridge.
- Because the two interior longitudinal members are connected with plates at five locations, they are considered to act as one member.

- Because of the bridge bearing condition, simple supports are assumed.
- The 4 ft – 6 in. of the flatcars that extend beyond the abutments was neglected.
- The bridge deck transfers the truck loading directly to the longitudinal members without consideration of the load distribution of the deck. Even though the timbers are bolted to these members, it is assumed that only vertical reactions are transferred (i.e., no moment/torsion transfer).
- The connections between the major longitudinal and transverse members are adequate to allow full moment transfer.
- The two RRFC were assumed to be 8 in. apart. Because one of the longitudinal members is severely bent, the actual spacing varies along the entire length of the bridge.

The analytical model was developed using ANSYS [15], a finite element program that has both tapered and prismatic elements. Tapered elements were used to model all major members (both longitudinal and transverse) of the flatcar while the prismatic element was used for the timber members of the deck. The tapered elements allow a different unsymmetrical geometry at each end and allow for translation and rotation in all three directions. At each end of the tapered elements, the area and moment of inertia about the strong and weak axis (A , I_x and I_y , respectively) were assigned. Because flatcar member properties vary along the member, a spreadsheet was developed to calculate the required section properties of any member at any location in the bridge based on the measured dimensions. The section properties of the various members in the Tama County flatcars are presented in Table 5.1. At the bridge midspan, the

Table 5.1. Calculated member properties of Tama County RRFC.

	Area (in ²)	I _x (in ⁴)	I _y (in ⁴)
Exterior Girders			
at supports	12.7	263	19
at midspan	17.2	1,421	22
Main Girder			
at supports	50.2	1,109	2,300
at midspan	63.7	9,597	3,003
Major Transverse Members			
at exterior	10.88	963.9	25
at interior	14.2	2,119	45

strong axis moment of inertia (I_x) of the exterior and interior girders increases significantly as may be seen in Table 5.1; also, the moment of inertia (both I_x and I_y) of the interior girders is much larger than that of the exterior girders. The moment of inertia of the transverse members more than doubles as the members span from the exterior to the interior of the bridge.

The ANSYS model was developed by creating three longitudinal members (as previously noted, the two interior longitudinal members of each RRFC were modeled as one member) and six transverse members for each RRFC. Each longitudinal member was composed of a fine mesh of 48 elements; this allowed input of the properties in the tapered regions of the member. At the abutment locations of each flatcar, elements were added to represent the transverse support beams. As previously noted, there was an 8 in. space between the two flatcars. The analytical model of the TCB is presented in Figure 5.1. Note that the bridge span was modeled based on its span length of 42 ft (center-to-center

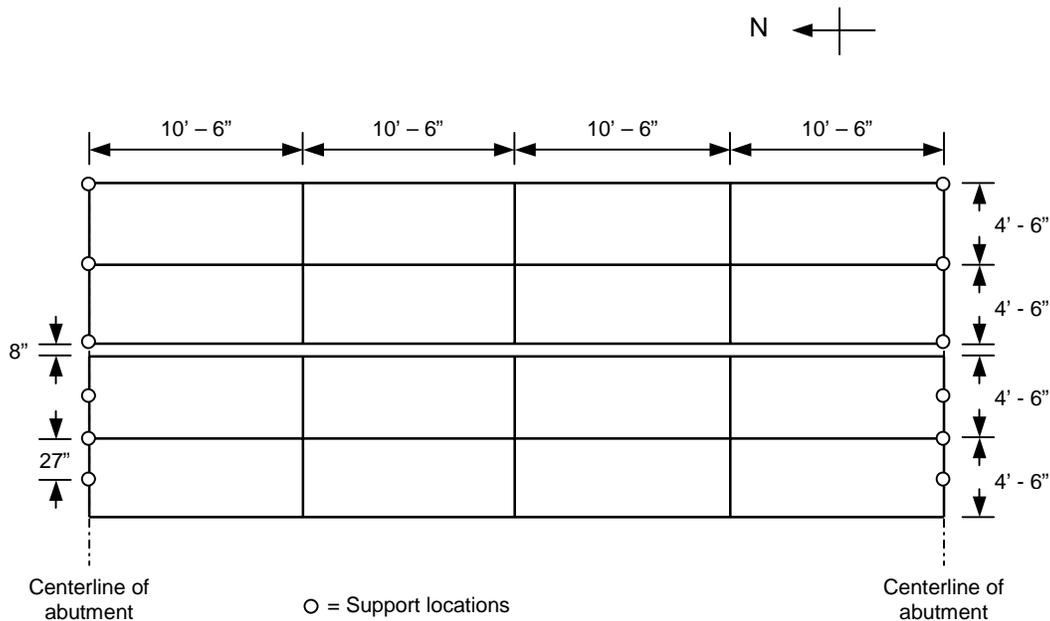


Figure 5.1. ANSYS grillage model of TCB (Note that the support locations for each flatcar are different).

of abutments) rather than its overall length of 51 ft. Because concentrated loads may only be applied at nodes in the model, nodes were closely spaced (approximately 1 ft apart) throughout the bridge so that one model could be used to analyze all load cases. Section properties were assigned to the elements at every node location. Vertical and horizontal restraints were applied at the ends of the bridge to simulate a simple span. These support restraints were assigned to the model at locations (shown in Figure 5.1) that correspond to the actual steel plate support locations of the TCB previously described.

To account for the structural rigidity of the timber deck, two different approaches were considered in the modeling procedure. Initially, the displacements between the two flatcars were set equal to each other at the

centerline of the bridge and the timber planks were ignored. This seemed reasonable because the timbers (with a thickness of 4 in.) spanned only 8 in. between the flatcars and would most likely cause displacements to be the same. The second approach was to incorporate each timber plank into the analytical model. This was done by creating prismatic elements 2 in. (half the timber deck thickness) above the flatcar and linking these elements to the rest of the model. Rigid links were applied between the timber elements and the center four longitudinal members in the bridge model. As stated earlier, it was assumed that no moment/torsion transfer occurred between the timbers and the RRFC. Therefore, the links created between the timber and RRFC elements consisted only of displacements in three directions (i.e., no rotation constraints). The TCB ANSYS model with the 41 timber elements included is shown in Figure 5.2.

Another objective of the structural analysis was to determine the lateral load distribution effects the timbers had on the bridge superstructure. To

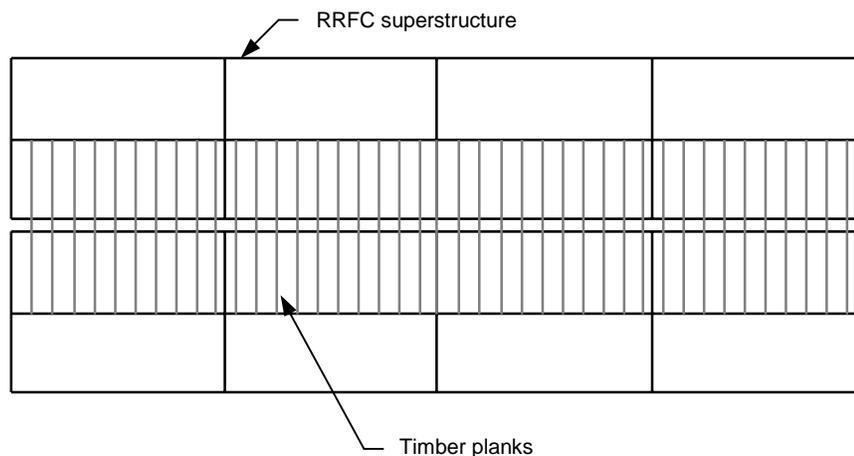


Figure 5.2. ANSYS model of TCB with a fine mesh grillage of the timber deck.

accomplish this, the TCB was modeled with coarser meshes than in the original model representing various numbers of equally spaced timbers (21, 11, 5, and 3). Results from these models were compared with the finer mesh model in which all timbers were included. The TCB modeled with 21 timbers spaced 2 ft apart is presented in Figure 5.3.

Based on an assumption stated earlier, the timber bridge deck is assumed to transfer the truck wheel loads to the main longitudinal members of the flatcar frame. Therefore, loads were applied directly to the main longitudinal members in the ANSYS model. This was done so that the same loads could be applied to both types of TCB models (with and without the timber deck). To determine how much of the truck load should be applied to each member, the timber deck was modeled as a continuous span beam with supports at the locations of the main longitudinal members of the flatcar. Truck wheel loads were applied to the continuous beam and reactions calculated. These reactions were then applied to

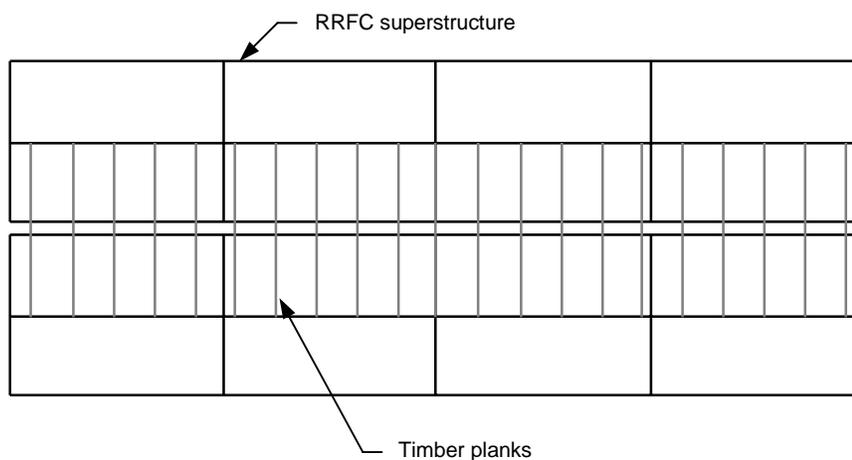


Figure 5.3. ANSYS model of TCB with a coarse mesh grillage of the timber deck.

the main longitudinal members of the RRFC ANSYS model. The configurations used to determine the loads applied to the ANSYS model is shown in Figure 5.4. When modeling a truck supported by only one RRFC, a two-span continuous system was used to determine the loads that would be applied to the RRFC grillage model as shown in Figure 5.4a. When modeling a truck supported by two RRFC, as is the case with the TCB, a three-span continuous system was used as shown in Figure 5.4b. When two RRFC carry the truck loads, the outer members were assumed to carry no truck loads directly since the timber deck does not extend to the exterior longitudinal members. Therefore, only reactions for the center four longitudinal members, which support the timber deck, were determined. Reactions were found for both the front and rear axle loads of a truck. In the ANSYS model, nodes were placed at the exact wheel locations along the length of the bridge so that no longitudinal distribution of wheel loads was required.

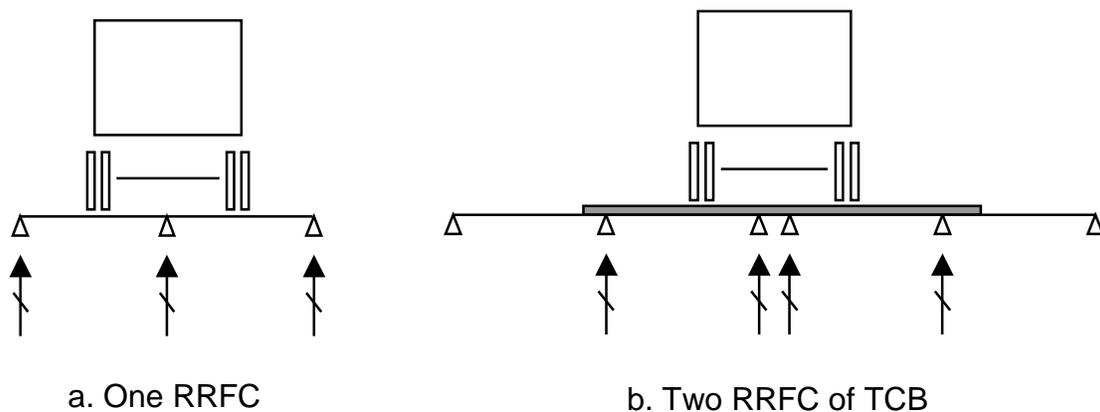


Figure 5.4. Reactions from truck loads that were applied to the ANSYS model.

The weight and dimensions of the truck used in the field test (as described in Chapter 4) were used to determine the appropriate loads to apply to the model. Various truck positions (as used in the field load test) were analyzed to determine the maximum strains and forces that would occur in members of the RRFC bridge.

5.2 Other Flatcars

Because a wide variety of flatcar designs exist, an analytical comparison of several flatcars was performed to determine the structural adequacy of RRFC bridges. Unfortunately, the structural plans of only two additional flatcars could be obtained. The same modeling procedures previously described for the TCB were used to develop the analytical models described in the following sections.

5.2.1 Thrall Manufacturing Flatcar

The Thrall Manufacturing flatcar for which structural plans were obtained was constructed in the early 1970s for the Northern Burlington Railroad. This type of flatcar is not uncommon and is still in use. The flatcar has structural features similar to those in the flatcars used in the TCB, but has a length of 61 ft and width of 9 ft - 4 in. If the flatcar's bolster were set on abutments, as done with the TCB, the flatcar would span 51 ft. The flatcar is composed of four main longitudinal members (two exterior and two interior) and eight major transverse members. Figure 5.5 shows a centerline cross-section view of the flatcar, and the member properties are presented in Table 5.2. As with the TCB, the main interior girder has a large moment of inertia about its strong axis (I_x) when compared to the rest of the flatcar members. The longitudinal members are

the ends of the flatcar. An elevation drawing of the Thrall flatcar showing location of the tapered regions of the longitudinal members is provided in Figure B.2 of Appendix B. Between the two interior longitudinal members, there is a 5/16 in. plate that connects the webs of the two interior members. These plates are located along the length of the flatcar at locations of the main transverse members. Steel plates (3/4 in. x 17 in.) connect the top and bottom flanges of the interior members. As in the TCB flatcars, there are tapered transverse members. At the interior of the flatcar, these members have a height of 23 in. and decrease to a height of 10 in. at the exterior. The built up transverse members are composed of 5/16 in. web plates with 6 in. x 3/8 in. flanges.

An analytical bridge model was developed using the properties obtained from the Thrall flatcar. As with the TCB model, only the major structural members were considered. The ANSYS bridge model using the Thrall flatcars is shown in Figure 5.6. Vertical and horizontal support conditions were placed at the ends of each longitudinal member as shown in Figure 5.6 (i.e., span length equals 51 ft). The procedure used to model the TCB was used to develop this model and determine the appropriate loading. The results of the Thrall flatcar analysis are presented in Chapter 6.

5.2.2 Canadian National Flatcar

A second set of RRFC structural plans was obtained from Canadian National Railways. This 100-ton flatcar was designed in the early 1970s and has an overall length of 63 ft (51 ft - 6 in. between bolsters) and width of 9 ft – 2 in. The structural framing of this flatcar is very similar to the one designed by Thrall

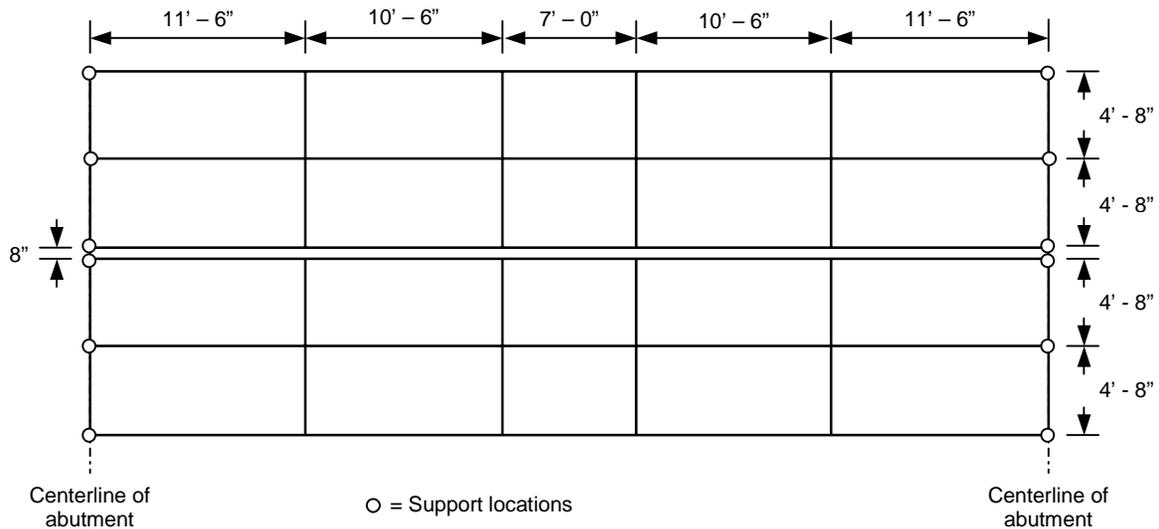


Figure 5.6. ANSYS bridge model of the Thrall flatcar.

Manufacturing. The flatcar is composed of three main longitudinal members (two exterior and one interior) and eight major transverse members. A centerline cross-sectional view of the flatcar is shown in Figure 5.7; the member properties are presented in Table 5.3. Again, the main girder has a much larger moment of inertia than any other member does in the flatcar. From Table 5.3, it can be noticed that the interior longitudinal girder has the largest moment of inertia of any flatcar discussed. Like the Thrall flatcar, the exterior longitudinal members of the Canadian National flatcar are MC18 x 42.7 channels and remain a constant depth for the entire length of the flatcar. The interior longitudinal member is a built up I-section with two 1/2 in. plates for the web and 1 1/8 in. and 1 1/4 in. plates for the top and bottom flanges, respectively. As in the previous two types of flatcars, there are tapered transverse members with a depth of 18 in. at the exterior of the flatcar and increases to 32 in. at the interior. An elevation drawing

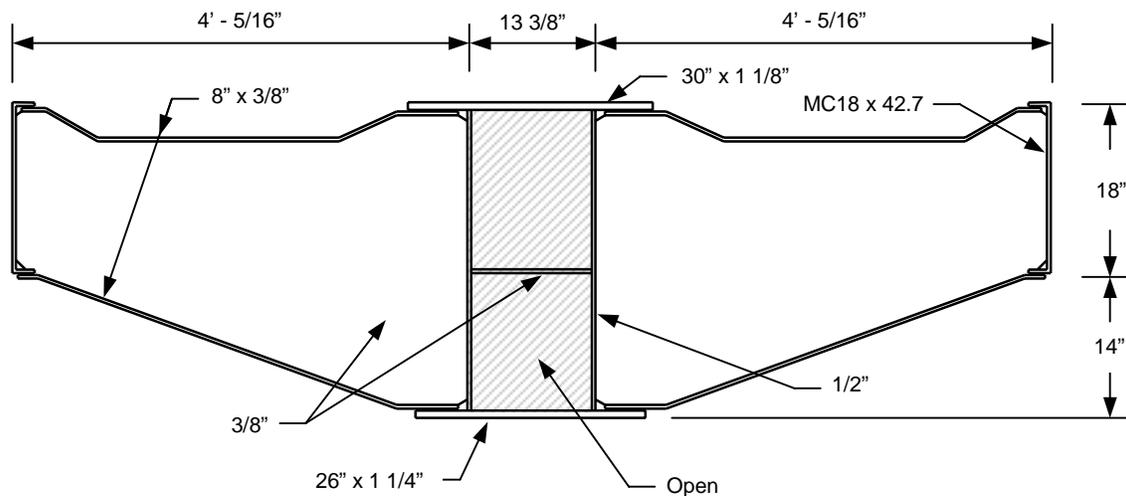


Figure 5.7. Midspan cross-sectional view of the Canadian National flatcar.

Table 5.3. Calculated member properties of the Canadian National RRFC.

	Area (in ²)	I _x (in ⁴)	I _y (in ⁴)
Exterior Girders			
at supports	12.6	554	14
at midspan	12.6	554	14
Main Girder			
at supports	78.7	2,757	4,868
at midspan	95.9	17,895	5,687
Major Transverse Members			
at exterior	12.5	655	32
at interior	16.8	2036	32

of the Thrall flatcar showing location of the tapered regions of the longitudinal members is provided in Figure B.3 of Appendix B. An analytical model of a bridge superstructure was created using the Canadian National flatcar; this ANSYS model is presented in Figure 5.8. Note that the support conditions were again applied at the ends of the longitudinal members (i.e., span length equals

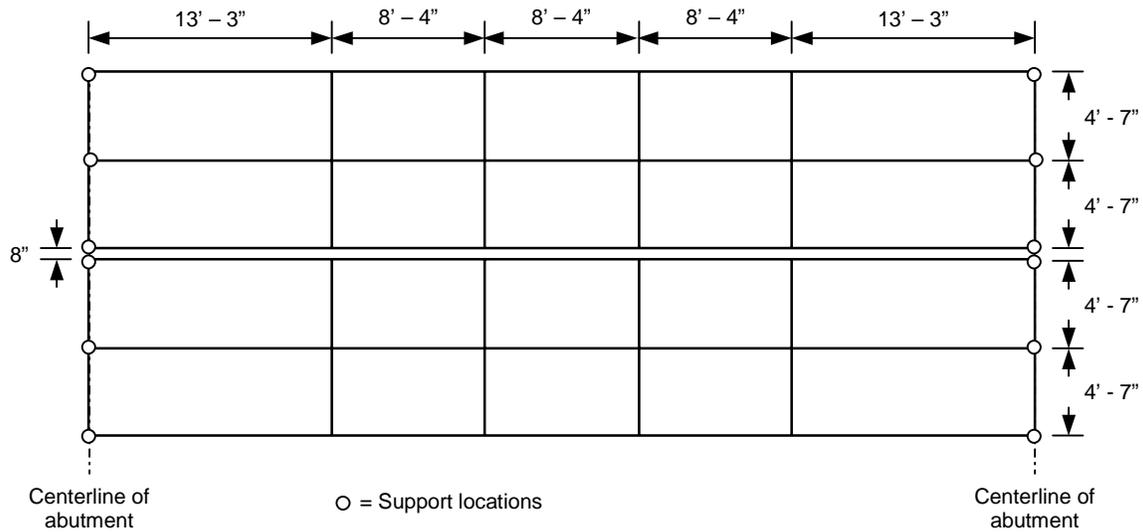


Figure 5.8. ANSYS bridge model of the Canadian National flatcar.

51 ft – 6 in.). Notice that only slight differences in element spacing exist between the Thrall flatcar the Canadian National flatcar models. Each of these two models has eight major transverse members per flatcar whereas the TCB model has only six. Difference in span lengths also exist between the three flatcar models; the models have spans of 42 ft, 51 ft, and 51 ft – 6 in for the TCB, Thrall Manufacturing, and Canadian National flatcars, respectively. A table comparing the member properties of the three flatcars is presented in Table B.1 of Appendix B.

6. EXPERIMENTAL AND ANALYTICAL RESULTS

This chapter presents the results obtained from the analytical and field load testing portions of this investigation. The results from the field load testing of the TCB are compared with those obtained from the grillage analysis. The analytical results using the Thrall and Canadian National flatcars with TCB geometry are compared with the analytical results of the TCB. Finally, a load rating for the TCB was calculated based on the theoretical and experimental results.

6.1 Field Load Test Results

Before obtaining data from load tests, visual field observations were made with regard to the bridge's response to the truck loadings by having the unloaded truck slowly cross the bridge. During loading, the supports, damaged members, and midspan region were closely observed. Both flatcars were visually stable; no excessive vertical deflection or rotation (in or out of plane) of any part of the flatcars was noticed.

6.1.1 Test Results without Flatcar Connections

The midspan deflections and bottom flange strains are presented in Figures 6.1, 6.2, and 6.3 for truck positions B1, B2, and B3 (see Figure 4.7), respectively due to the fully loaded rear tandem-axle truck (52.14 kips). These three truck positions produced the maximum strain and deflection results. From Figures 6.1, 6.2, and 6.3, it can be seen that the strains and deflections were low for the three truck positions. The maximum deflections were 0.32 in., 0.31 in., and 0.31 in. for truck positions B1, B2, and B3, respectively, compared to the

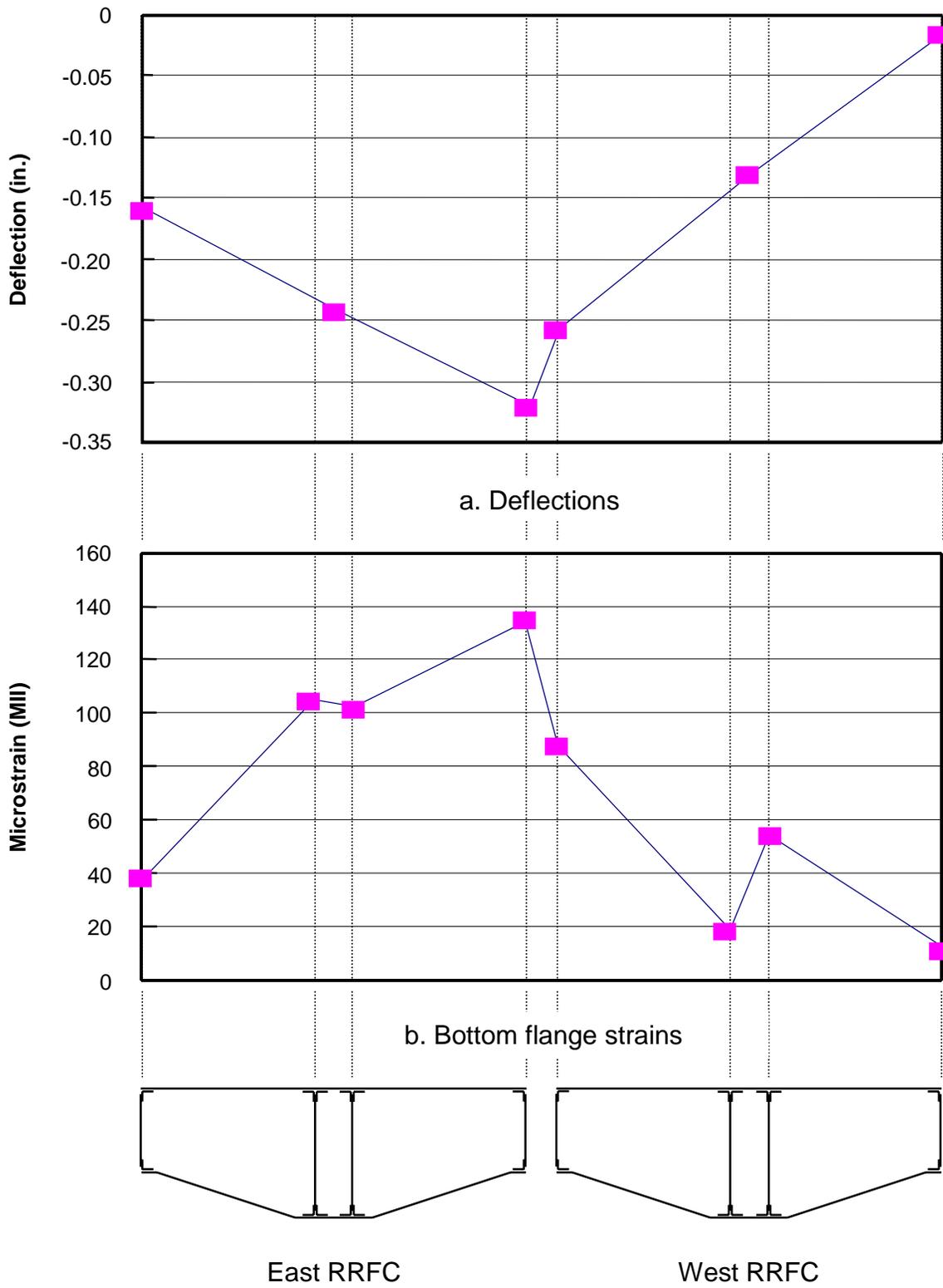


Figure 6.1. Midspan deflections and strains from field load test for truck position B1.

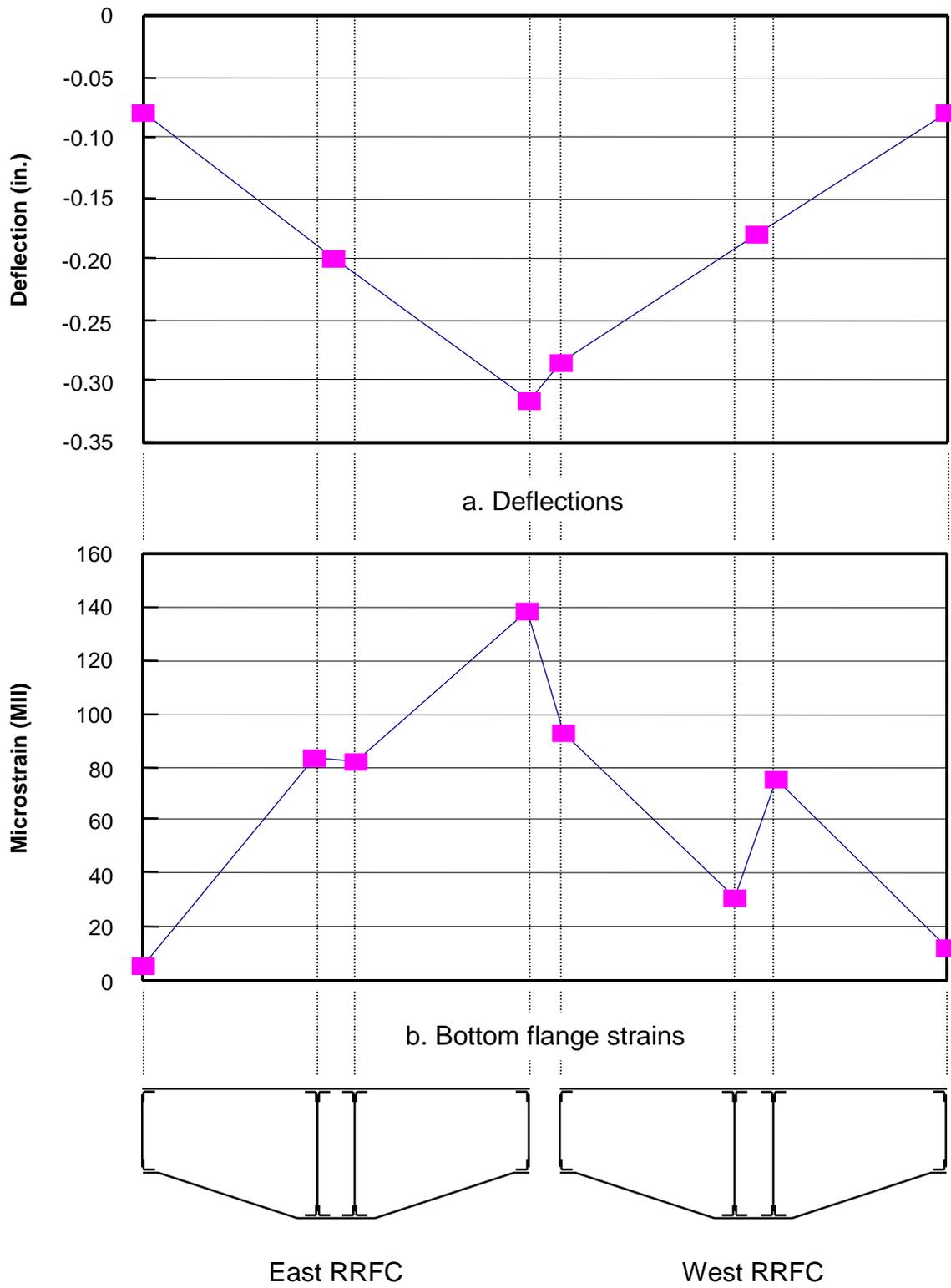


Figure 6.2. Midspan deflections and strains from field load test for truck position B2.

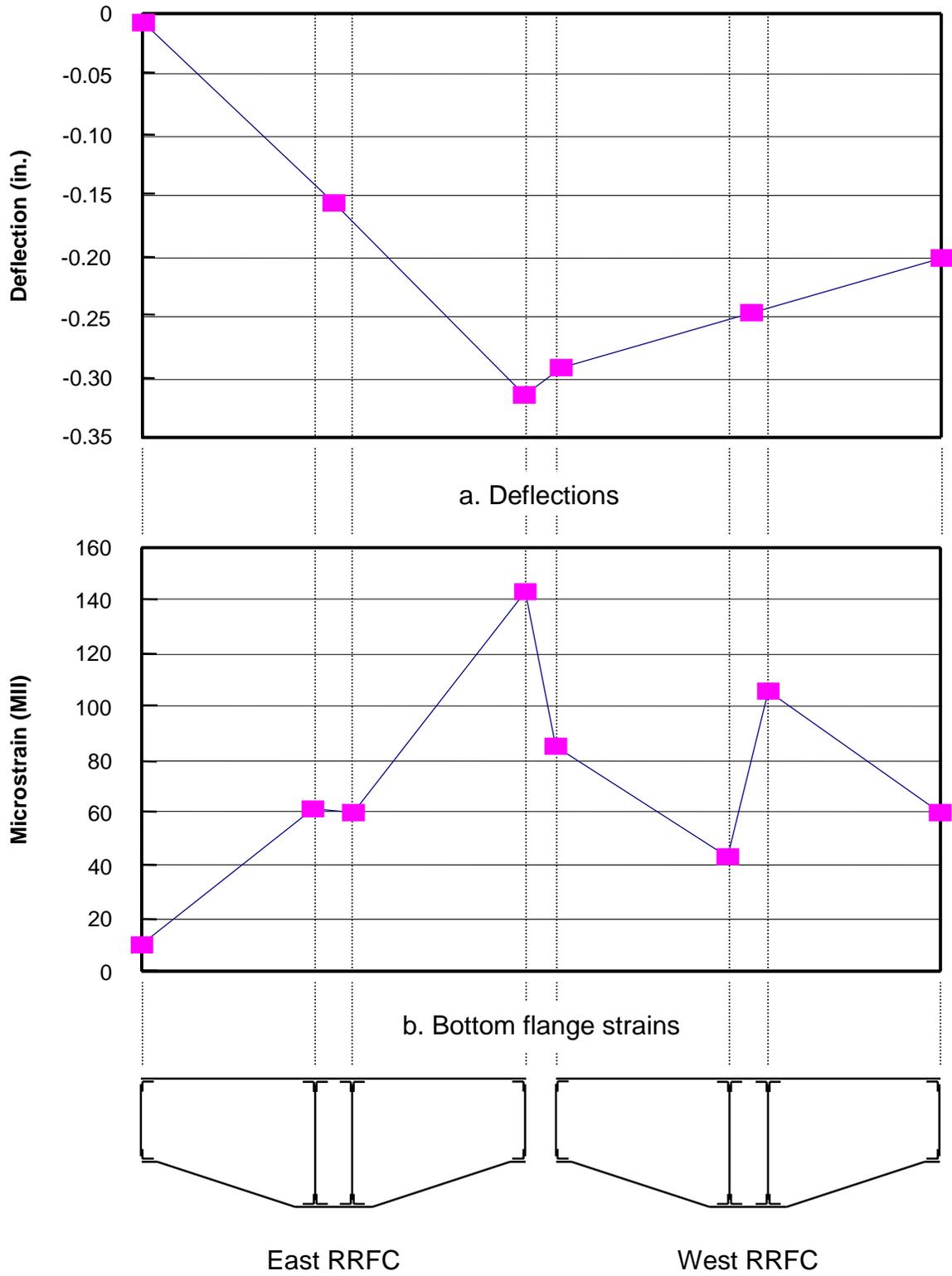


Figure 6.3. Midspan deflections and strains from field load test for truck position B3.

AASHTO live load deflection criteria of $L/800$ (0.63 in. for the TCB). Note that the deflections of the center two longitudinal members of the bridge differed, with the maximum difference produced by truck position B1. Recall that these two members had different support conditions previously described in Chapter 3.

By comparing the deflection results from truck positions B1 and B3 as shown in Figure 6.4a, the bridge's global behavior is somewhat symmetrical in nature. Symmetry is also noticed in the B2 truck position data (except for the dissimilar deflections at the center of the bridge) even though the east RRFC experiences slightly more rotation about its longitudinal axis. Also note that the outside longitudinal member deflections are relatively small, especially when the truck was positioned toward the opposite side of the bridge (i.e., deflection of the extreme east bridge member due to truck position B3).

The maximum strains are low for the three truck positions and occurred in the members with the maximum deflections. For truck positions B1, B2, and B3, the maximum strains were 135, 139, and 143 microstrains, respectively. These strains are equivalent to a tensile stress of approximately 4 ksi. Again, the test results show that the two exterior members of the bridge had very small strains when the truck loads were placed at the center of the bridge; in fact, the strains were nearly zero for truck position B2.

In all figures, the strains of the east RRFC have a similar pattern as the corresponding deflection data; the strains increase as the deflections increase. However, this is not evident in the west flatcar where two irregularities in the strain data are noticed. Different strains were recorded for the two interior

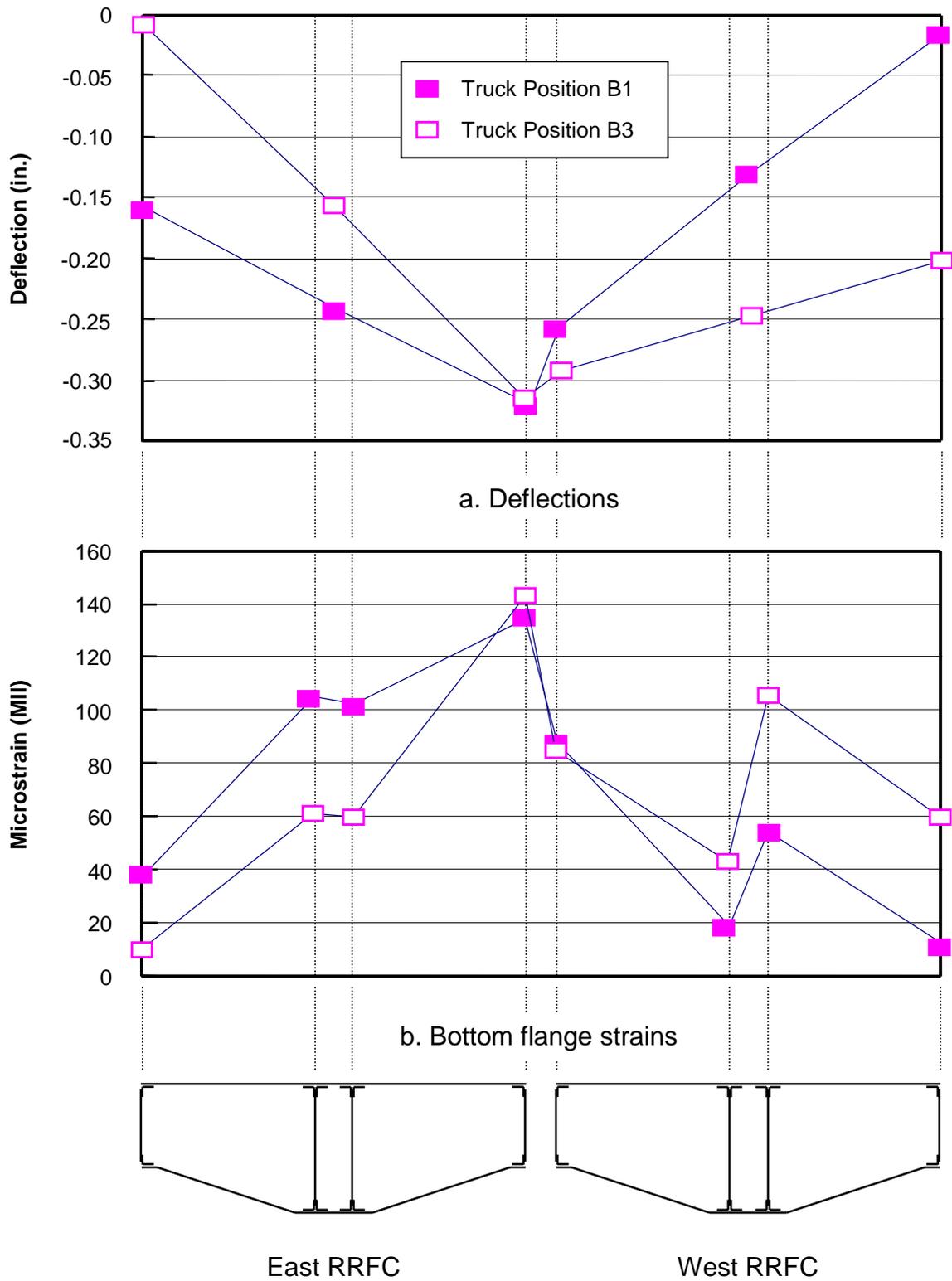


Figure 6.4. Midspan deflection and strain comparisons for truck positions B1 and B3.

longitudinal members of the west RRFC while the two interior members of the east RRFC had similar strains. In the interior of the west RRFC, one strain reading was the same magnitude as experienced in the interior longitudinal members of the east RRFC while the other strain was much lower. Also, the strains at the center two longitudinal members of the bridge were significantly different while their displacement differences were small. Even though each RRFC had different support conditions, the magnitude of these irregularities was not expected. Damaged members of the flatcar may be responsible for the unexplained differences; this is discussed later in the report where the field test data are compared with the analytical results.

As discussed in Chapter 4, strain gages were placed at the top and bottom flanges of a major transverse member of the east RRFC. The strains from this member experienced a maximum of 17 microstrains (equivalent to 0.49 ksi) during the load test. Because of the small strains in the transverse member, graphical data was not presented. Also, strain gages were placed at mid-height of the four longitudinal members. For linear behavior across the depth of the member to occur, the mid-height strains were expected to be zero. From this data, three of the four longitudinal members displayed linear behavior. The east interior longitudinal member (one of the members with questionable bottom flange strains) of the west flatcar did not display linear behavior.

6.1.2 Test Results with Flatcar Connections

A second aspect of the load test consisted of making angle-plate connections between the flatcars and re-testing the bridge. Figure 6.5 provides

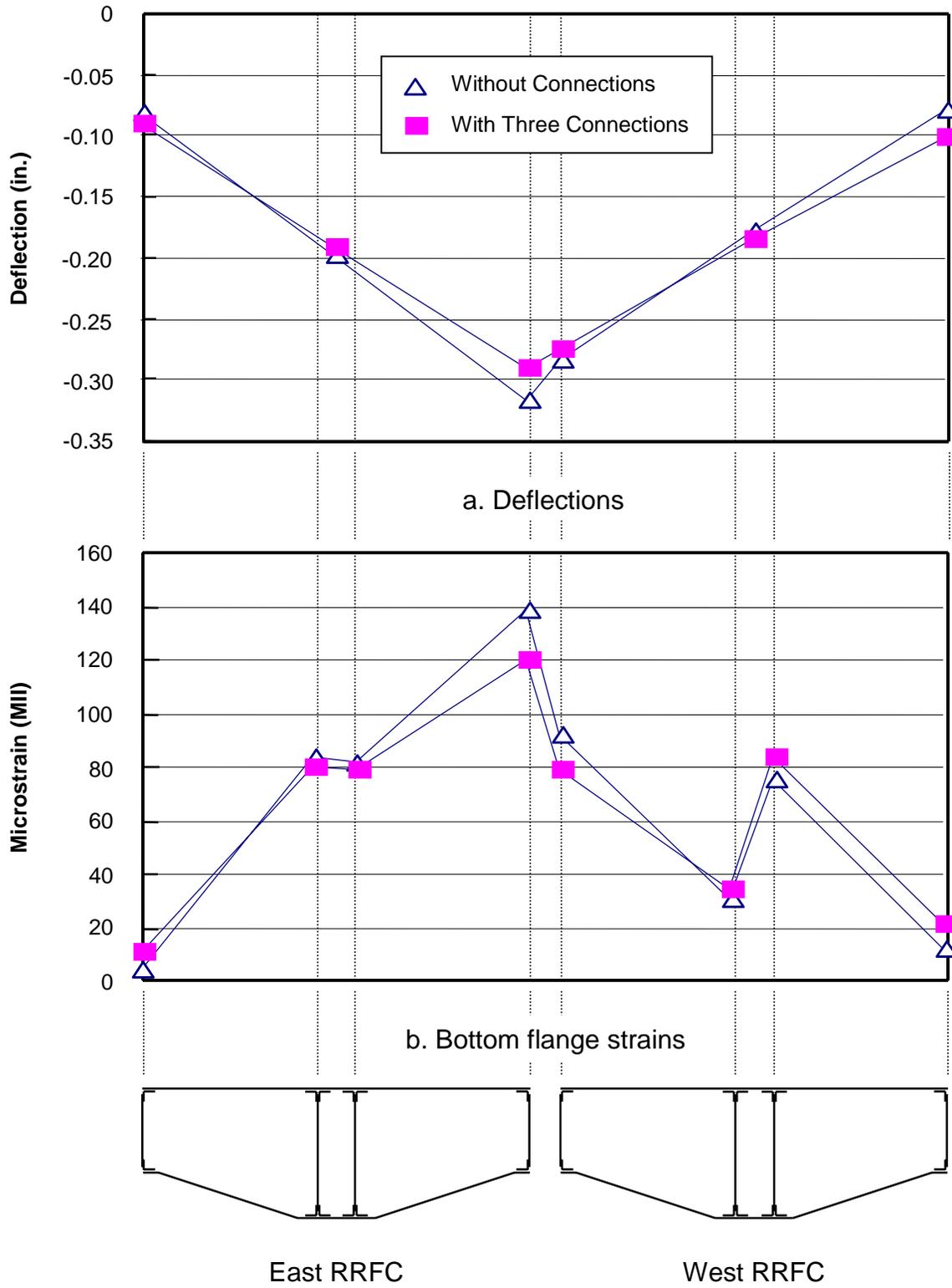


Figure 6.5. Midspan deflections and strains for truck position B2 with three connections made between flatcars.

the results of this test compared with the results when the bridge was unconnected. The data presented are for truck position B2 with the bridge having all three connections (at the 1/4, 1/2, and 3/4 span of the bridge) in place. These data are representative of all truck positions. Generally, it can be seen that the connections did not change the behavior patterns in either the strain or deflection data. At the adjacent exterior members of the flatcars, the strains were reduced approximately 14%, thus indicating that some of the load was distributed to other members. From Figure 6.5, it is seen that all other members in the bridge cross-section had relatively small changes in strain. From the deflection data, it is seen that both flatcars experience slightly less rotation about their longitudinal axis with the connections in place. Deflections at the center of the bridge are approximately 9 % lower with the connections in place. Despite reductions to the already low strain and deflection data at the center of the bridge, it can be concluded from Figure 6.5 that the angle-plate connections are not needed for sufficient lateral load distribution between the two flatcars. With only one angle-plate connection added to the TCB, reductions in strain and deflection were less than those recorded with three angle-plate connections added to the TCB. For this reason, data from the field load test with one connection added to the TCB are not presented.

6.1.3 Second Field Load Test

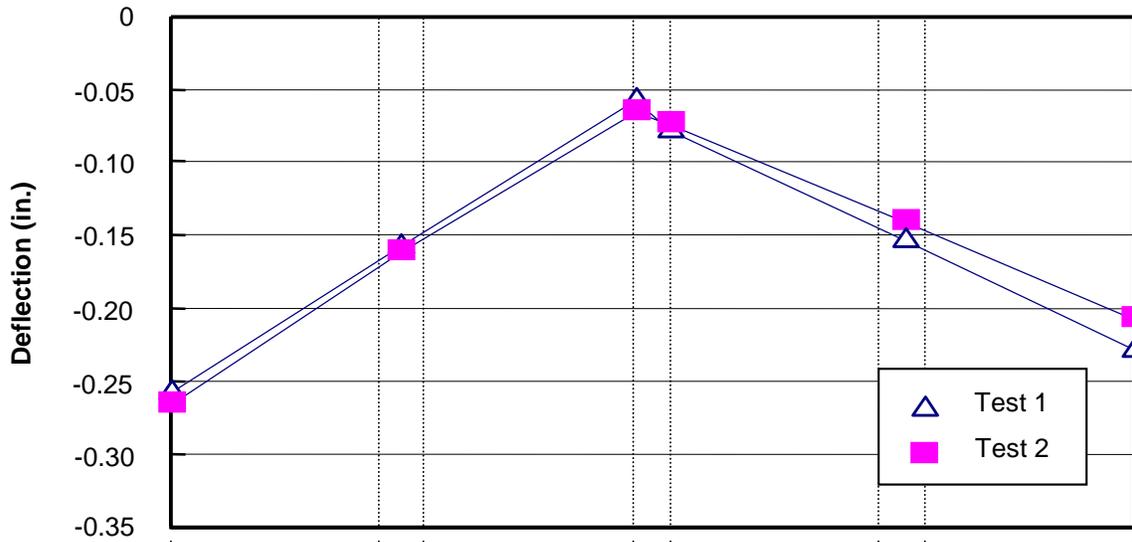
As discussed in Section 6.1.1, two questionable strains were recorded during the first field test. To verify these strains, checks were performed to insure that the correct data were plotted at the correct locations; the possibility of

some type of instrument malfunction was also investigated. After these checks revealed no apparent errors, it was decided to perform a second field test paying particular attention to the regions where questionable strain data were obtained. The bridge was instrumented with new strain gages where questionable data were obtained and loaded with a rear tandem-axle truck weighing 54.28 kips (compared with 52.14 kips from the first load test). The truck that was used in the first load test was also used in this load test along with the same truck positions.

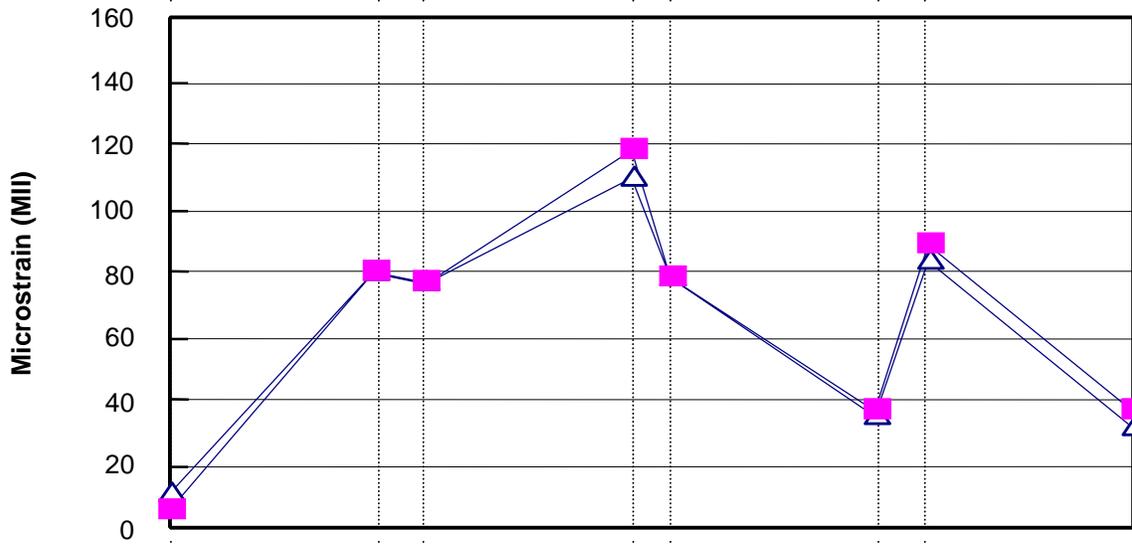
In Figure 6.6, the results of the second load test are compared with the results of the first load test with the truck in position B2. This figure is representative of all data recorded during the second load test and shows that the results from the first load test were accurate. The small differences in the strains and deflections between the two load tests can be attributed to the slight difference in the truck weights.

6.1.4. Field Load Test Conclusions

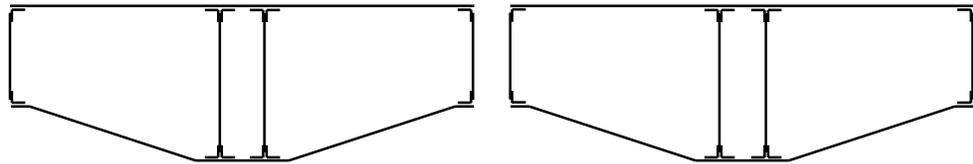
Based on the test data presented, the TCB experienced very small midspan deflections and strains when loaded with a rear tandem-axle truck weighing 52.14 kips. Regardless of truck position, the maximum strain recorded was only 143 microstrains (4.1 ksi) with the maximum deflection of the bridge being only 0.32 in (compared to AASHTO live load limit of 0.63 in.). These results provide evidence that the TCB is very capable of supporting legal Iowa highway loads. Incorporating both the experimental and analytical data, a load rating, which is discussed later in this chapter, was calculated for the TCB.



a. Deflections



b. Bottom flange strains



East RRFC

West RRFC

Figure 6.6. Midspan deflection and strain comparisons between first and second load test for truck position B2.

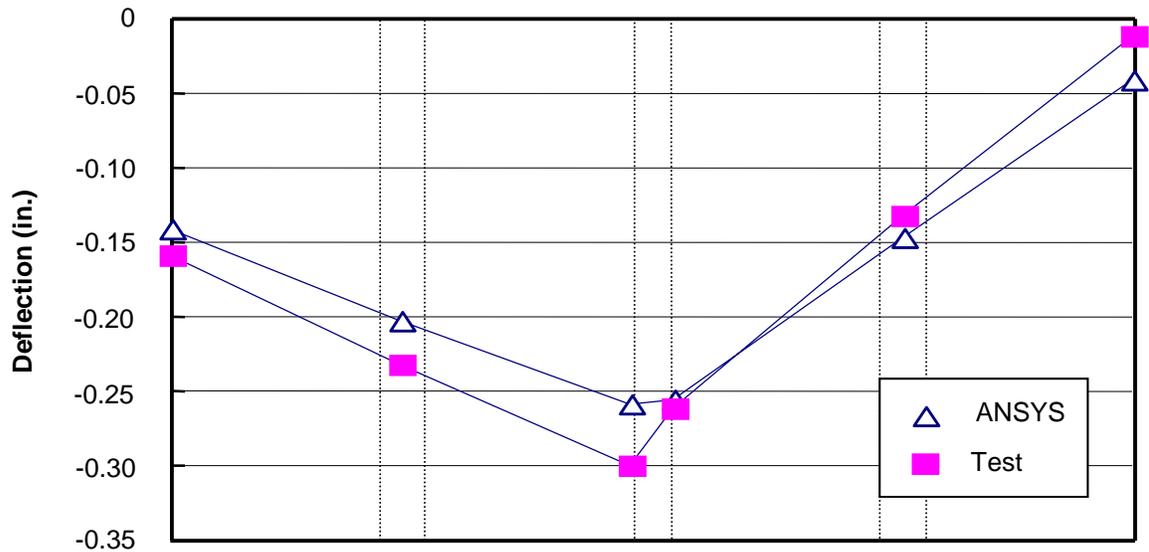
6.2 Analytical Results

Using the truck weights, dimensions, and positions from the field load tests, equivalent loads were applied to the TCB analytical model for comparison. Bridge models developed using the Thrall and Canadian National flatcars were also loaded and these results were compared with the TCB analytical data. Results from the TCB analytical models with coarse timber deck meshes are also presented in this section to determine the effect of the timber deck on the lateral load distribution characteristics of the TCB.

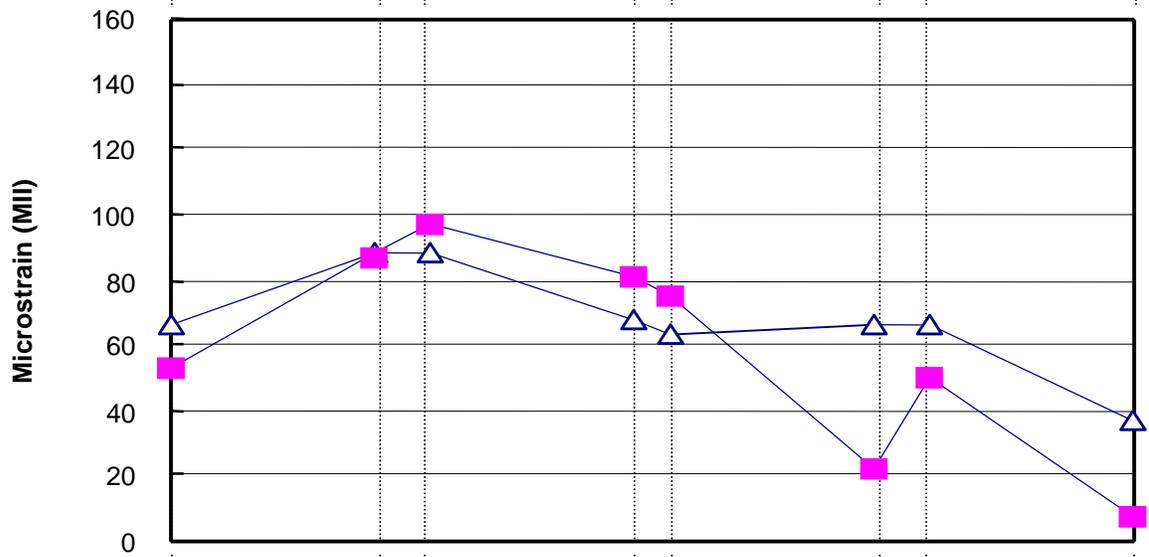
6.2.1 Comparison of Field Test and Analytical Results for TCB

The field deflections and strains and the corresponding analytical values are presented in Figures 6.7 through 6.13 for truck positions A1, A3, B1, B2, B3, C1, and C3 respectively. The data from truck positions A2 and C2 were not included because the results obtained from these load cases produce “symmetrical” results which are smaller than the results found for truck position B2.

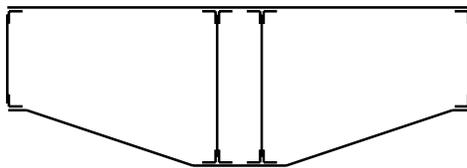
From the analytical data, it can be seen that the deflection and strain results are of the same low magnitude as the experimental results. The maximum midspan deflection was 0.27 in. due to loading at position B2; the maximum strain was 108 microstrains (3.1 ksi) in the interior members of the west RRFC with loading at position B3. Even with different support conditions between each RRFC, the two analytical deflections and strains at the center of the bridge were essentially the same for all truck positions. Also note that the deflection data trends are similar for both the computer model results and field



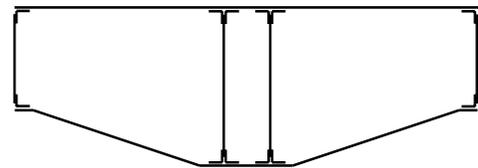
a. Deflections



b. Bottom flange strains



East RRFC



West RRFC

Figure 6.7. Midspan deflection and strain results for truck position A1.

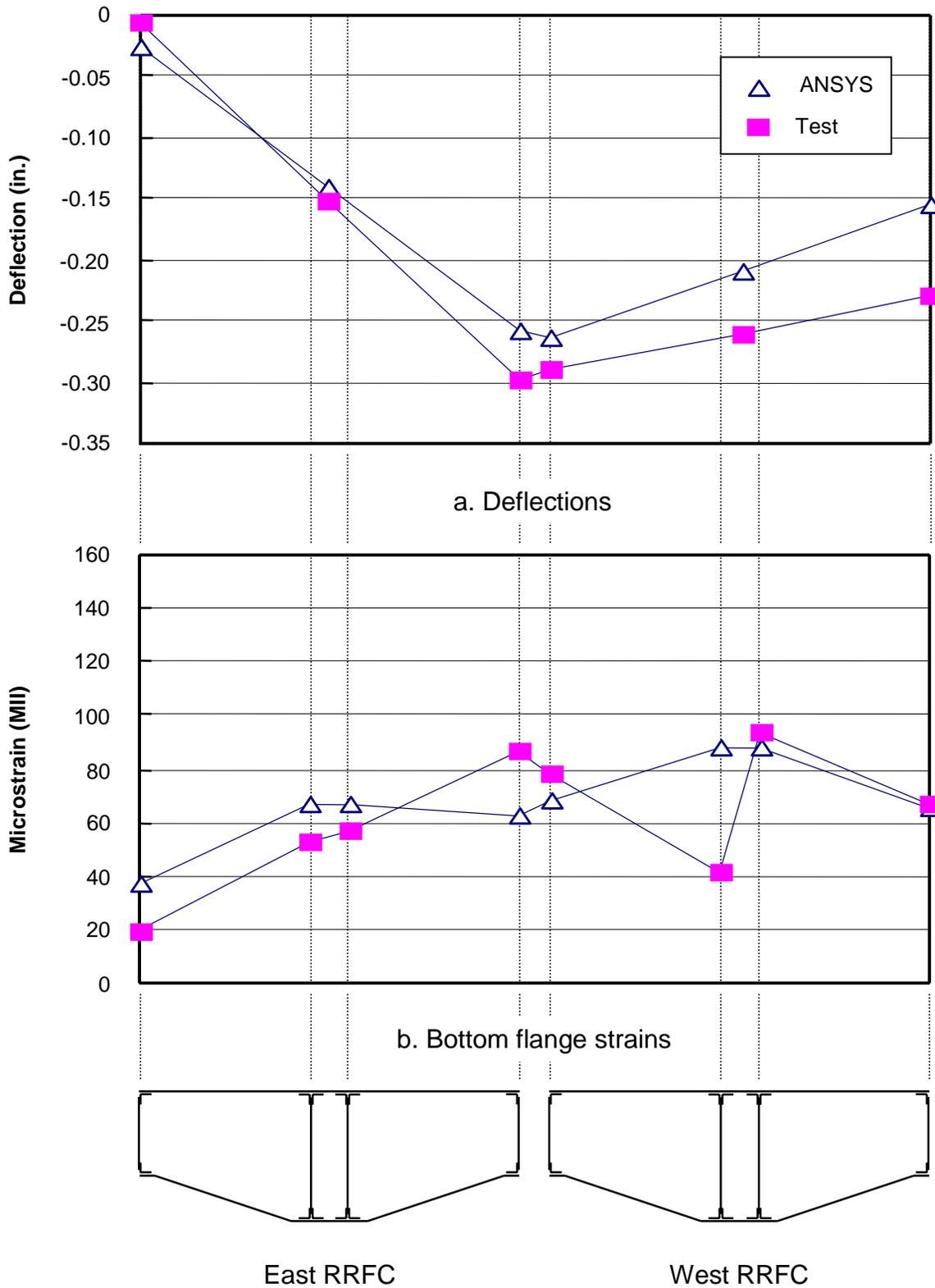


Figure 6.8. Midspan deflection and strain results for truck position A3.

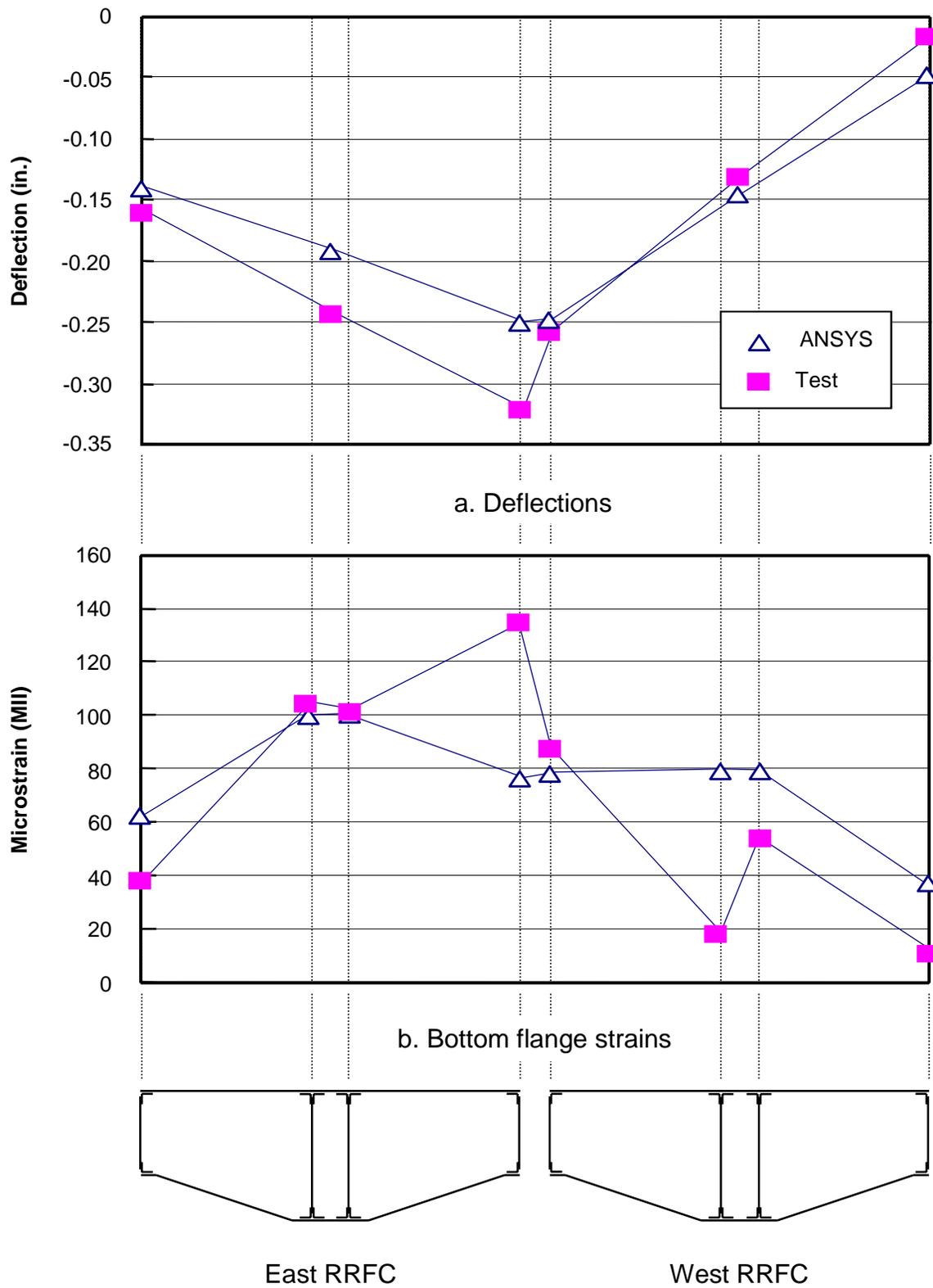
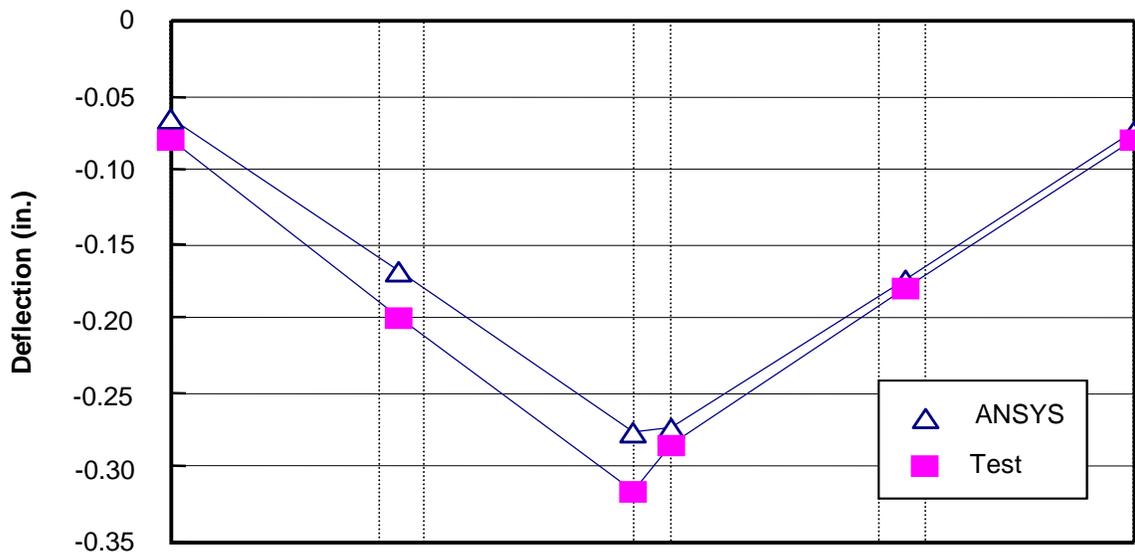
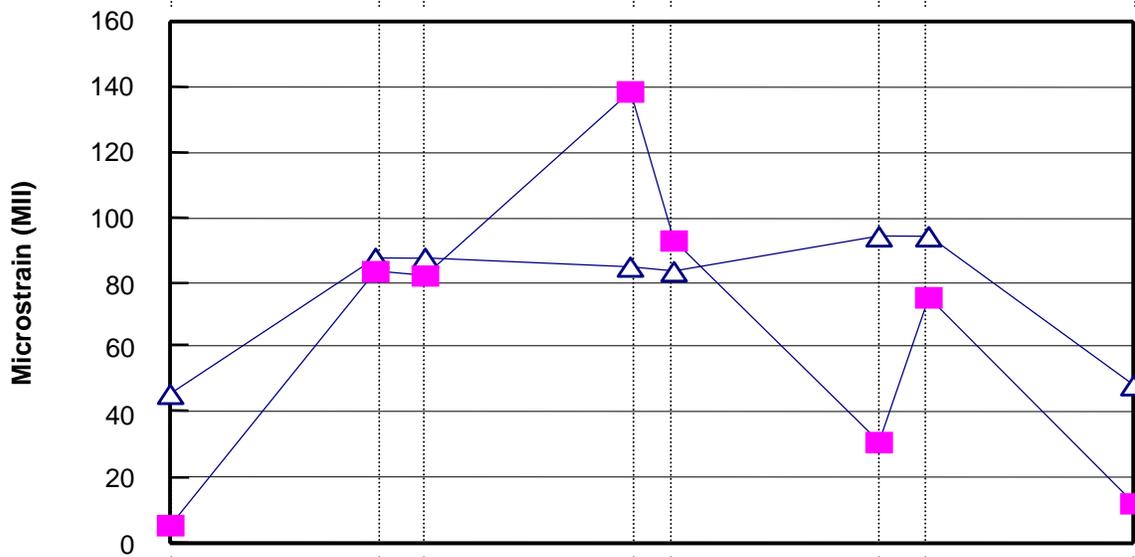


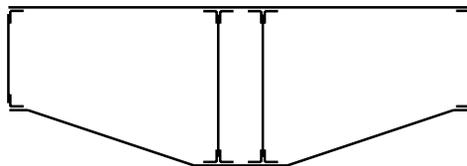
Figure 6.9. Midspan deflection and strain results for truck position B1.



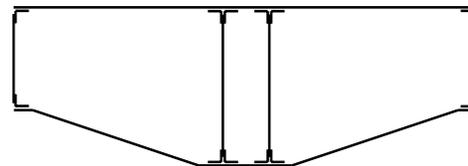
a. Deflections



b. Bottom flange strains



East RRFC



West RRFC

Figure 6.10. Midspan deflection and strain results for truck position B2.

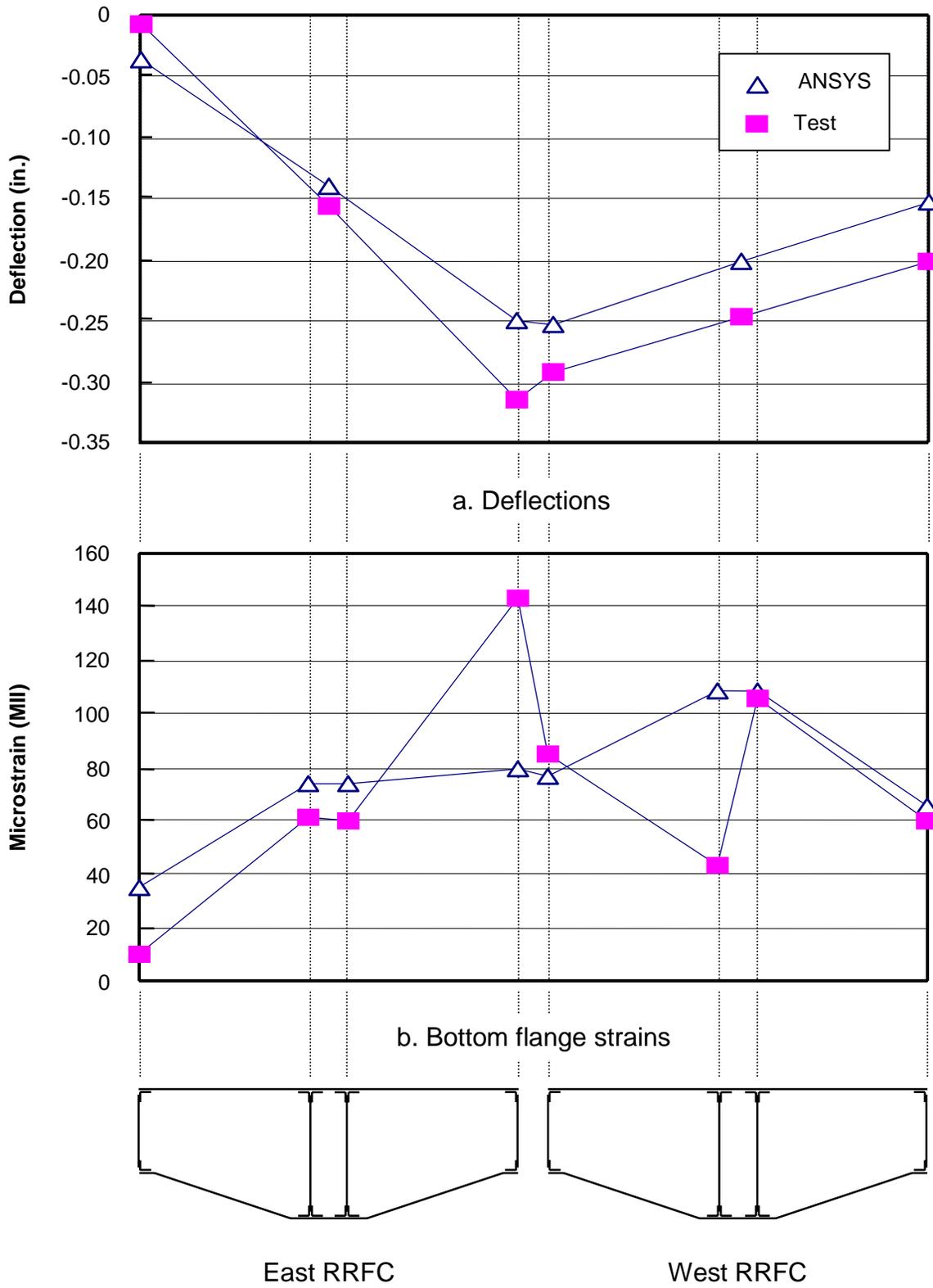


Figure 6.11. Midspan deflection and strain results for truck position B3.

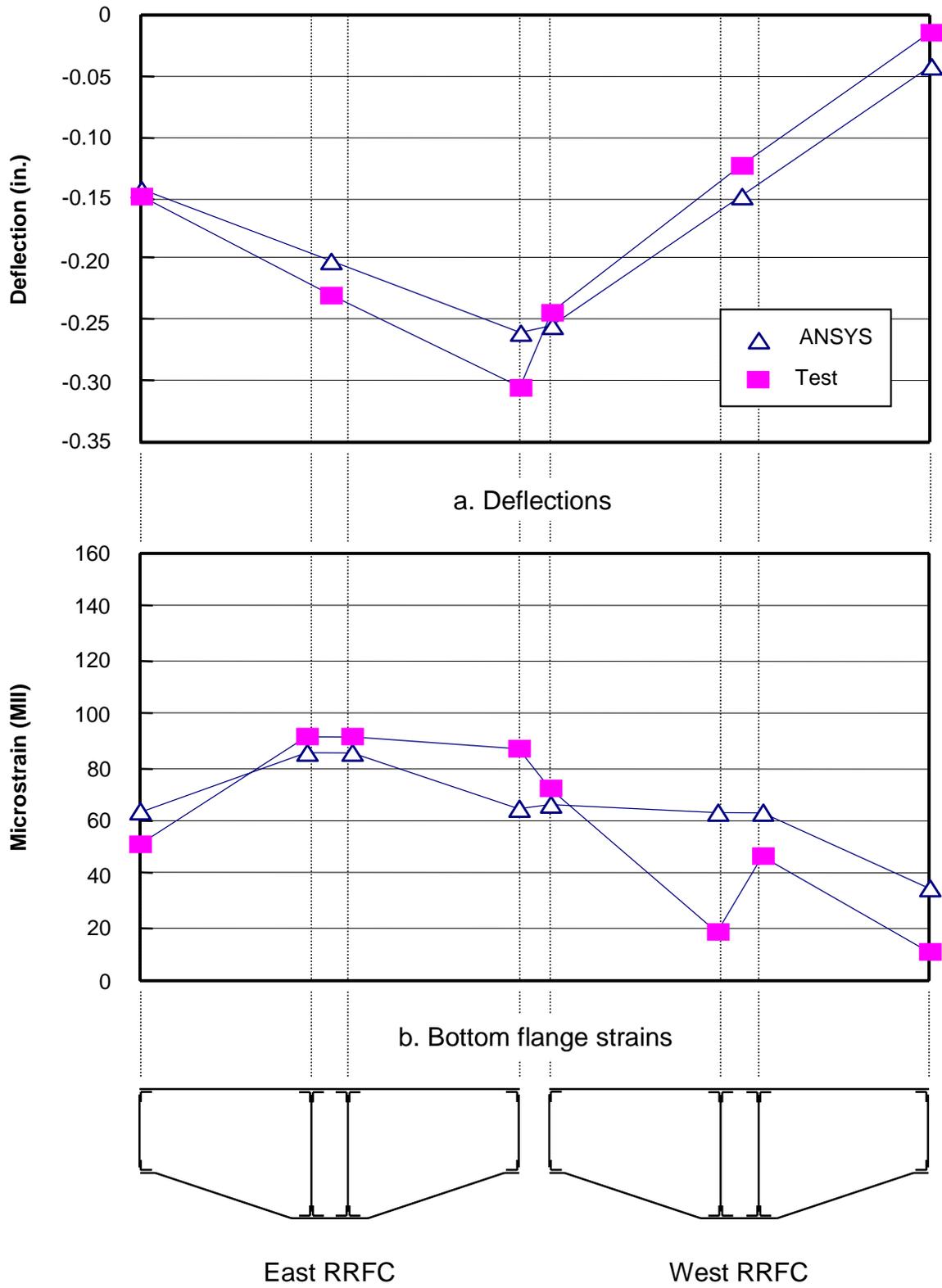
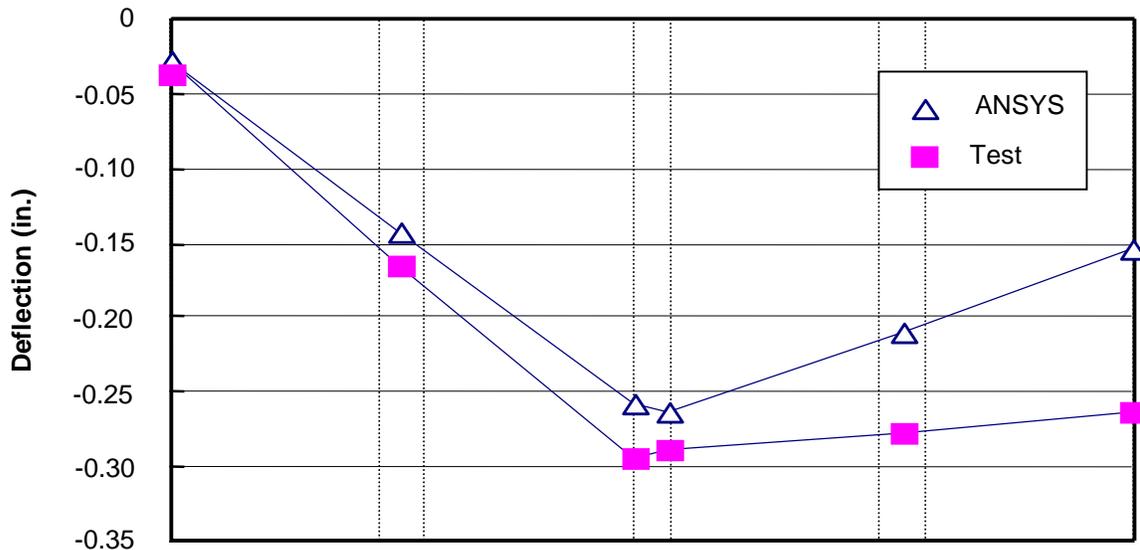
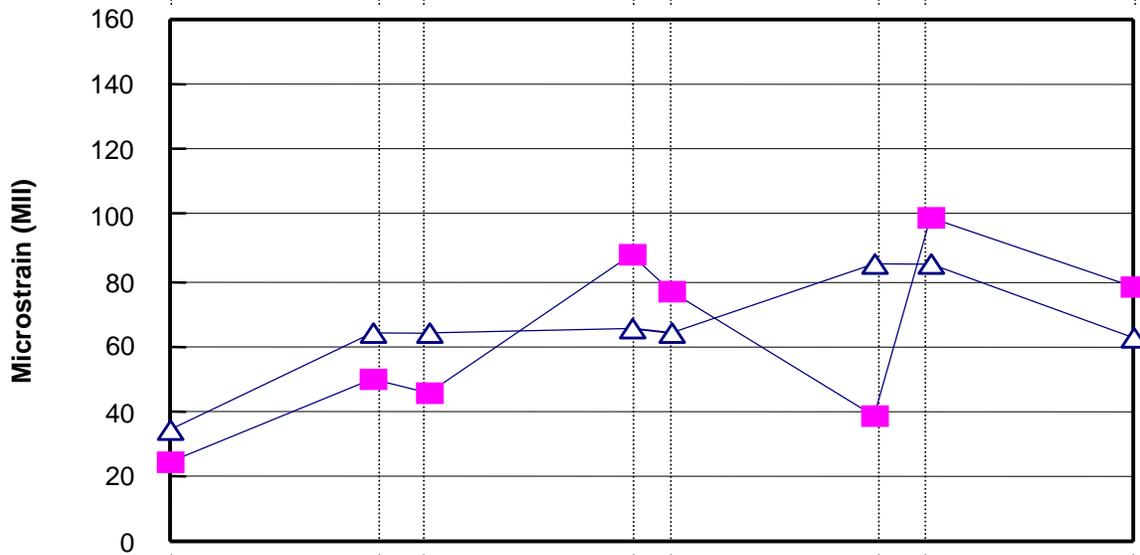


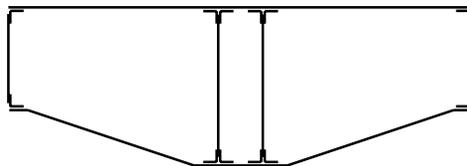
Figure 6.12. Midspan deflection and strain results for truck position C1.



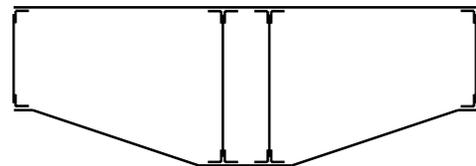
a. Deflections



b. Bottom flange strains



East RRFC



West RRFC

Figure 6.13. Midspan deflection and strain results for truck position C3.

test results. The main differences between the experimental and analytical deflection results are at the center of the bridge.

Similar trends also occur between the analytical and experimental strain results with the exception of two locations in the cross-section of the bridge, which are discussed in the next paragraph. The analytical and experimental strains for the interior members of the east RRFC were very close for all truck positions. At the two exterior members of the bridge, the analytical and experimental strains generally agreed, but the analytical data were slightly higher. At the center of the bridge, the experimental and analytical strains agree for the west member. This is not true for the east longitudinal member; large differences in the strain results are apparent. Also, large differences exist between the analytical and experimental strains in an interior longitudinal member of the west RRFC.

The portion of the TCB cross-section where the two questionable strain readings noted above occurred is shown in Figure 6.14. For truck positions B1, B2, and B3 (Figures 6.9, 6.10, and 6.11), the strains from the field test are significantly higher than the analytical results at the west exterior longitudinal member of the east flatcar. The strains from the field test are significantly lower than the analytical results at the east interior longitudinal member of the west flatcar for all truck positions. These two locations have been labeled S10 and S15 in Figure 6.14 for ease of discussion; the labels correspond to the strain gage number used in the field test. The east exterior longitudinal member of the west car (S12) and the west interior longitudinal member of the west flatcar (S18)

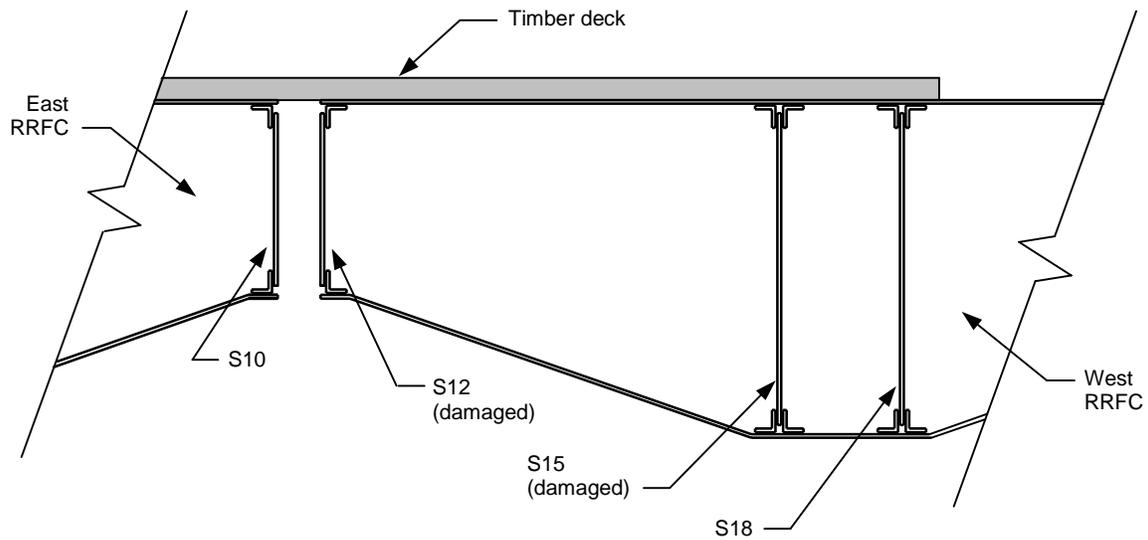


Figure 6.14. Member labels in region of questionable strains.

are also identified in the figure for discussion. The cross-section shown in Figure 6.14 is located longitudinally near the center of the bridge where the strain gages were positioned.

Because of the extreme damage to many of the flatcar members, an exact explanation for the questionable strains would be nearly impossible; a possible explanation follows. As described in Chapter 3, the S12 and S15 members were severely damaged and the S10 and S18 members were in good condition. When the bridge is loaded, the two straight (undamaged) members may “attract” loading from the two damaged members. If this were the case, the S12 and S15 strains would be less than their S10 and S18 counterparts. This type of load distribution would also cause the west flatcar to have less rotation about its longitudinal axis when compared with the east flatcar. From Figure 6.10 (truck position B2), the deflections from the field test indicate that the west flatcar does

experience less rotation and the S12 and S15 strains are much less than the strains of S10 and S18. This global explanation coupled with local behavior due to the member damage may be the reason for differences in the analytical and experimental strains.

The effect of the damaged members can also be noticed by comparing truck positions that apply symmetric loading. If the TCB superstructure was symmetric, truck positions A1 and C3 would produce similar results as well as truck positions A3 and C1. When studying Figures 6.7 and 6.13 (truck positions A1 and C3), the global behavior that was just described is apparent. For truck position A1, the truck is positioned at the east edge of the timber roadway and thus the east flatcar carries a majority of the load. The experimental and analytical deflection results show similar patterns of behavior. When the condition of the bridge was assessed, the east flatcar had far less damage than the west flatcar, especially in the longitudinal members. With truck position C3, the majority of the truck load is carried by the heavily damaged west flatcar. From the deflection results of this truck position (Figure 6.13), it can be seen that there are significant differences in the east RRFC analytical and experimental deflections. This indicates that the damaged west flatcar does have significantly less rotation than the east flatcar under the same loading, which is in agreement with the global explanation that was previously provided. A similar conclusion can be made when comparing truck positions A3 and C1, which also provides a symmetric loading situation.

Comparison of the field test results with the analytical results indicated similar patterns of behavior with the exception of the two strain locations noted. Even though the computer models predicted deflections and strains that were generally lower than those measured in the field tests, the models were seen to provide a close approximation to the bridge's behavior, especially considering the condition of the TCB.

6.2.2 Analytical Results for the Three Different Flatcars.

As discussed in Chapter 5, two approaches were taken to account for the timber deck of the TCB in the computer modeling; model each timber and set displacements between flatcars equal to each other. The results with truck position B2 applied to both modeling approaches are representative of all load positions and are presented in Figure 6.15. The results indicate that minimal differences occurred between the two models and by adding timbers to the TCB model, the bridge's lateral stiffness increases only slightly. Because both models are practically equivalent, the quicker approach (setting displacements between the two flatcars equal) was used in the analysis of the Thrall and Canadian National flatcars.

In addition to the nine truck positions that were used during the field test, analytical results were also determined for the case when only one RRFC supports the entire truck weight. From the questionnaire that was discussed in Chapter 2, many states reported the existence of RRFC bridges that consist of only one flatcar. For this analytical case, truck loads were applied to one flatcar and all connections between the flatcars were removed. Loads were applied that

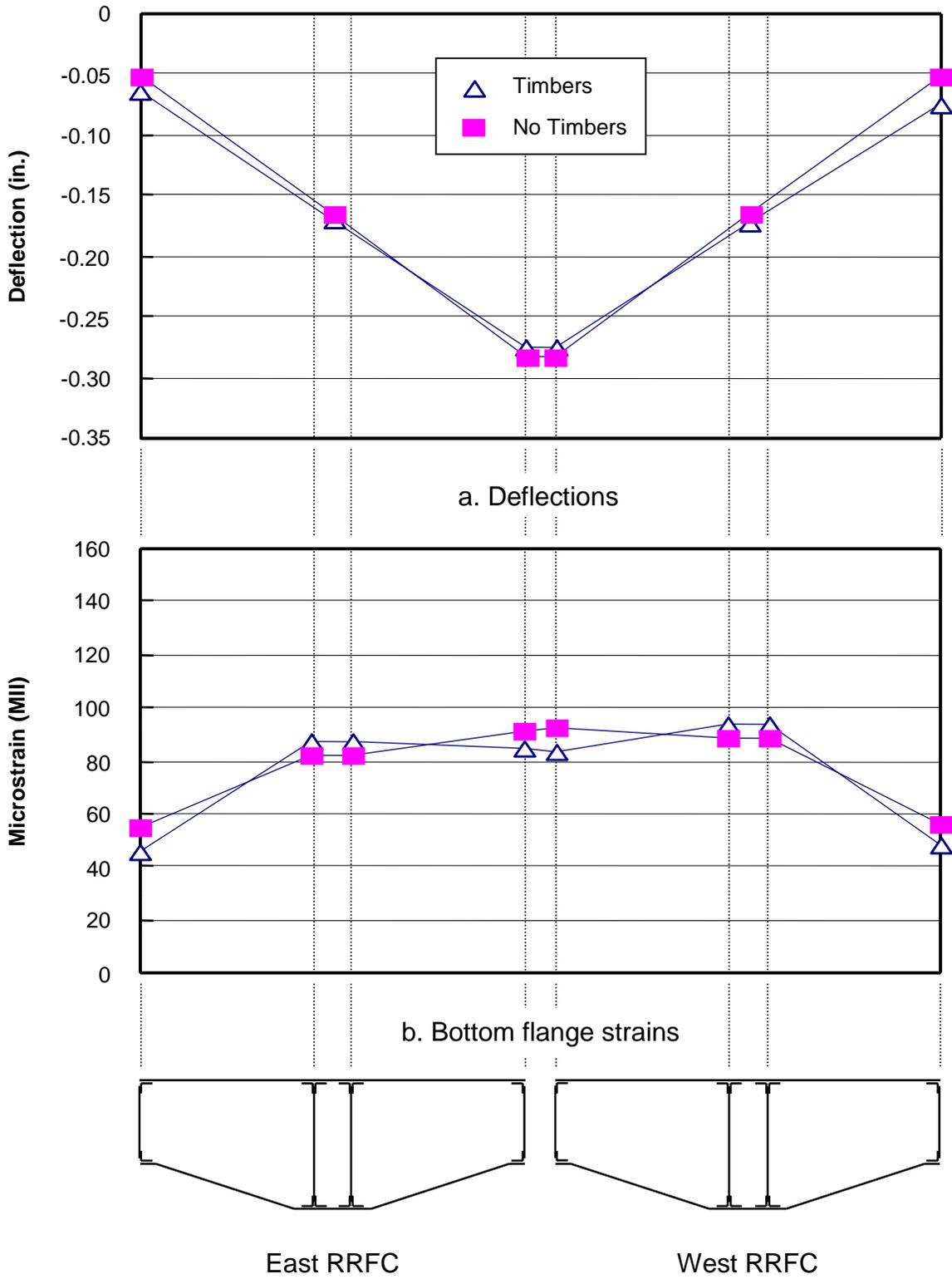


Figure 6.15. Analytical midspan deflections and strains for truck position B2 using two approaches to model the effect of the timber deck.

would be equivalent to a rear tandem-axle truck centered across one flatcar with its rear axle at the flatcar's midspan. Even though this position does not produce the maximum moment, it is consistent with the truck locations used in the field tests and the loading of the analytical models of two flatcars.

Analytical results for three truck positions are presented in Table 6.1 for the three different flatcars that were modeled. As discussed in Chapter 5, the span lengths are different for all three models: 42 ft for the Tama County flatcar, 51 ft for the Thrall flatcar, and 51 ft - 6 in. for the Canadian National flatcar. For all three flatcars, small strains and deflections were determined. When two flatcars act in combination to form a bridge superstructure, the midspan tensile stresses reach a high of 3.37 ksi for the Tama County flatcar, 4.56 ksi for the Thrall flatcar, and 4.66 ksi for the Canadian National flatcar. The maximum deflections are only 0.27 in., 0.46 in., and 0.36 in. for the three flatcars, respectively. When the truck load is applied to only one flatcar, the midspan tensile stress in all three flatcars reaches a maximum of only 6.75 ksi (for the Thrall flatcar). The analytical results of three different flatcars provide strong evidence that RRFC are structurally adequate to act as bridge superstructures both individually and in combination.

6.2.3 Lateral Load Distribution Effects of the Timber Deck.

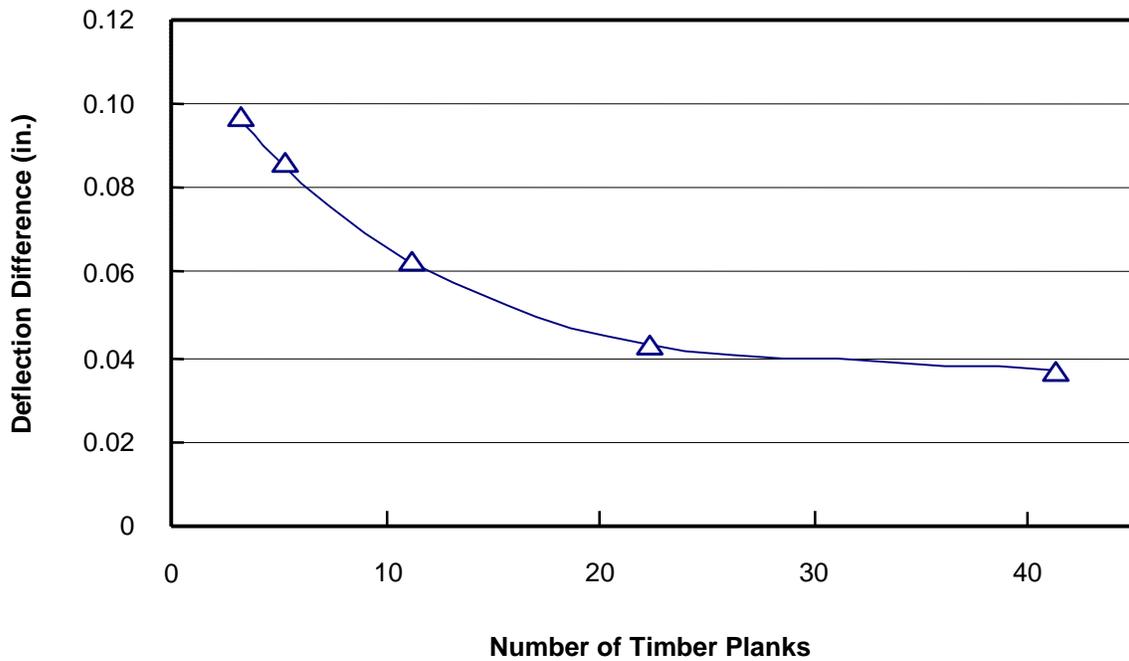
Through the questionnaire described in Chapter 2, many RRFC bridges were reported to have a bridge deck composed of transverse timber planks with timber runners providing the driving surface. In this type of bridge deck, the transverse timbers are spaced along the length of the bridge. Through computer

Table 6.1. Analysis results from three different RRFCs.

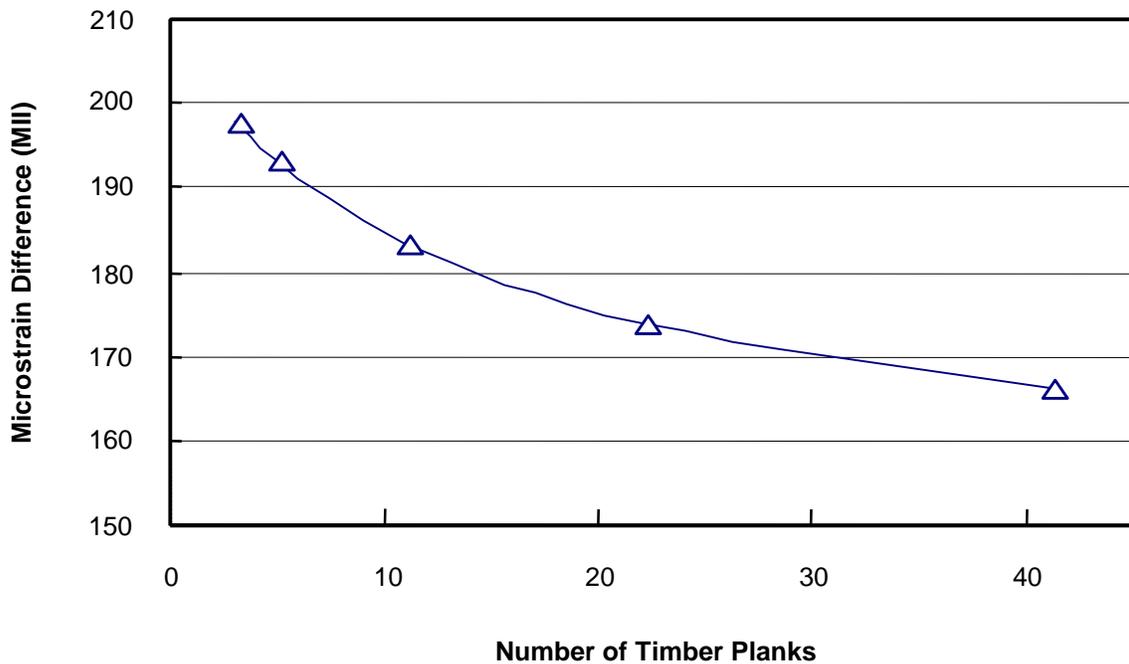
	Truck Position for Models with 2 RRFC		Truck Load Centered on One RRFC
	B2	B1	
<u>Tama County RRFC (42 ft span)</u>			
Midspan Tensile Stresses (ksi)			
Interior girder	3.03	3.37	5.14
main girder	2.80	3.31	5.29
Exterior girder	2.01	2.57	5.14
Transverse member	1.91	1.12	1.64
Max. Midspan Deflection (in.)	0.27	0.27	0.33
<u>Thrall Manufacturing RRFC (51 ft span)</u>			
Midspan Tensile Stresses (ksi)			
Interior girder	4.56	2.88	6.62
main girder	3.40	4.10	6.75
Exterior girder	1.82	2.16	6.62
Transverse member	3.15	1.62	2.96
Max. Midspan Deflection (in.)	0.46	0.41	0.59
<u>Canadian National RRFC (51 ft – 6 in. span)</u>			
Midspan Tensile Stresses (ksi)			
Interior girder	3.92	4.66	5.42
main girder	2.48	2.93	4.92
Exterior girder	3.92	1.45	5.42
Transverse member	2.04	0.80	2.07
Max. Midspan Deflection (in.)	0.36	0.33	0.42

modeling of the timbers, the load distribution behavior of a timber deck was studied as the number of timbers varied along the length of the bridge. In Chapter 5, it was noted that several TCB computer models were developed with a different number of timbers in each model (3, 5, 11, 21, and 41 evenly spaced timbers). A concentrated load of 50 kips (the magnitude of a typical gross truck load) was applied at the center of one flatcar at several locations along its length. Note that a concentrated load applied at the center will produce a greater moment than that caused by the actual truck wheel loads. The differences in deflection and strain between the adjacent exterior longitudinal members were determined as the number of timbers varied. Because only one flatcar was loaded, the differences in strain and deflections between the flatcars indicate the effectiveness of the timbers to laterally distribute load. The maximum differences in deflection and strain occurred when the concentrated load was applied at the midspan of the flatcar; these results are presented in Figure 6.16. The results show that the strain and deflection differences between the adjacent exterior longitudinal members vary little between the model with 41 timbers (no spacing between timbers) and the model with 21 timbers (1 ft spacing between timbers). When the timbers are spaced more than 1 ft apart (the models with 11, 5, and 3 timbers), the deck loses its ability to effectively distribute the load.

From the connection field test (as discussed in Section 6.1.2) and the computer modeling study, it was determined that a continuous transverse timber deck (i.e., no transverse space between timbers) provides adequate lateral load distribution for the TCB. Additional connections between the flatcars do not



a. Deflections



b. Bottom flange strains

Figure 6.16. Deflection and strain differences between exterior adjacent longitudinal members as a function of the number of timber planks used.

significantly improve the bridge's behavior. If a deck system were to be used in which timbers are spaced along the bridge length with timber runners providing the driving surface, it was shown that a spacing of 1 ft between timbers provides similar load distribution to that of a continuous transverse timber deck. For timber spacing greater than 1 ft between planks, additional connections would be required to accomplish the same distribution characteristics.

6.3. Load Rating of the Tama County Bridge

A load rating was performed on the TCB using the load and resistance factor rating (LRFR) method as described in a report by Streeter [16]. This method is expressed by the following equation:

$$RF = \frac{\phi R_n - \gamma_d D}{\gamma_L L (1+I)}$$

where:

- RF = Rating factor
- ϕ = Resistance factor
- R_n = Nominal resistance
- γ_d = Dead load factor
- D = Nominal dead load
- γ_L = Live load factor
- L = Nominal live load
- I = Live load impact factor

After a field load test has been conducted, the following equation is used to modify the theoretical rating:

$$RF_T = RF_c(1+K_a K_b)$$

where:

RF_T = The load rating factor after the results from the load test have been applied.

RF_c = The theoretical load rating factor.

K_a = A factor obtained from the comparison of the results obtained from the theoretical model with those obtained from the load test.

K_b = A factor which takes into account the frequency of inspections, the presence of special structural features such as redundancy, and the ability of the test team to explain the results obtained from the load test.

After the modified load rating factor has been calculated, a load rating can be determined by multiplying the load rating factor by the weight of the vehicle used to obtain the experimental and analytical results.

For the TCB, two members were the focus of the load rating procedure; the exterior built-up channels and the interior built-up I-shaped members. The nominal resistance was calculated for both members and maximum dead and live loads were obtained from the analytical model to develop a theoretical rating factor. By comparing maximum strains obtained in the load test with those from the analytical model, the theoretical factors were adjusted and final load ratings were calculated for the TCB. Table 6.2 provides a summary of the final load ratings developed for each of the two different longitudinal members in the Tama County flatcars. The load ratings suggest that the TCB has sufficient strength capacity to support Iowa legal loads. Note that this load rating is only for a Type 3 load rating vehicle. As discussed earlier, the analytical and experimental strains differed greatly for member S10. Because of this, the theoretical rating

Table 6.2. Load rating summary for the TCB.

Load rating parameters	Longitudinal member of RRFC	
	C-shape	I-shape
D (in-k)	114.17	358.56
L (in-k)	311.00	1031.10
M_n (in-k)	3,785.15	14,778.23
RF_C	5.18	6.16
K_a	-0.39	-0.05
K_b	0.80	0.80
RF_T	3.57	5.94
Final Load Rating (kips)	178.50	297.00

factor was decreased by 31% for the C-shaped members. Load rating calculations for the C-shaped members are presented in Appendix C. These rating values assume linear behavior in the flatcar members of the TCB. With the severe damage in the bridge, this is obviously not the case and the values presented in Table 6.2 should not be used in practice. The field load tests performed on the bridge should be considered a “proof” test where the TCB proved that it could support a Type 3 truck with a weight of 52.14 kips (which is slightly higher than the legal load for this type of truck). Because of the existing damage, the amount of additional load that would cause buckling or some other failure to occur is uncertain.

7. OTHER CONSIDERATIONS

Thus far, the investigation into RRFC bridges has focused on determining the strength capacity of this type of bridge through computer modeling and field testing. Other considerations such as availability of flatcars, overall costs, construction difficulty, and maintenance also need to be addressed. A major concern that was expressed by state bridge engineers in the questionnaire was the condition of flatcars and their remaining fatigue life. Flatcar manufacturers, bridge and county engineers, railroad salvage yards, and individuals with significant experience in RRFC bridges were contacted to gather information on these subjects. This chapter addresses these considerations that need to be addressed when determining the feasibility of using RRFC for bridges.

7.1 Availability of Railroad Flatcars

Through contact with local scrap metal dealers and regional railroad salvage yards, the issue of RRFC availability was addressed. The local scrap dealers were found to have limited involvement with used railroad cars. Even though most local scrap dealers have dealt with flatcars from time to time, their involvement is infrequent. Thus, they should be considered a secondary source of RRFC. Two companies that purchase used railroad cars directly from the railroad industry were also contacted; the Ermann Corporation based in Kansas City, Kansas and the Chambliss Bridge Company based in Hampton, Arkansas (contact information is provided in Appendix D). Both companies have a history of providing private buyers and counties with RRFC for use in bridges. These two companies have supplied several hundred RRFC bridges to many

Midwestern states and indicated that very few companies like themselves (with direct access to used railroad cars) exist in the United States.

Three main reasons for the retirement of flatcars were given by the railroad salvage yards: age, derailments, and economics. By law, railroad cars are salvaged after 50 years of service even though most are retired before this because of derailments or economic reasons. Derailments account for many flatcars being retired and it was noted that in minor railroad accidents, usually only the ends of the flatcars are damaged. In minor accidents, the structural portion of the flatcars that spans between the bolsters is usually not damaged and thus the structural integrity of the RRFC remains intact. Economics is the primary reason railroad cars are taken out of service. Either repairs are too costly or more efficient, cost-effective flatcar designs become available. Once the repair costs exceed the depreciated value of the car, it is removed from service. Repair costs include replacement of decking, repair of running gear, car seals, and other mechanical components of the cars. The maximum gross weight of a railroad car (which includes self-weight and maximum load) is limited by the railroad and railroad bridge limits. By reducing the weight of the flatcar, the load placed on the flatcar (i.e., live load) may be increased. As more efficient designs become available, older and heavier flatcars are replaced. Market and economy conditions also affect the removal of railroad cars. If a railroad company has more flatcars than they need, some will be sold to private industries or railroad salvages businesses.

Taking into account these reasons for removal, it can be seen that the railroad industry continuously removes railroad cars from service. Both the Ermann Corporation and the Chambliss Bridge Company admit that their flatcar supply is not at the stockpile capacity it once was, but they both continually receive flatcars from the railroad industry. Because so many railroad flatcars are currently used by the railroads, they expect the supply to continue for quite some time and actually predict an increase in supply within the next 12 to 18 months.

7.2 Condition of Flatcars and Fatigue Issues

One of the major concerns voiced by state bridge engineers through the questionnaire was the uncertain condition of RRFC when retired by the railroad companies. Many believe that the flatcars cannot be structurally sound if the railroads dispose of them. This idea has merit for flatcars removed due to derailment or age. But as stated earlier, many flatcars are retired for non-structural and economic reasons. The members in these flatcars are considered to be structurally sound.

A problem obviously arises when selecting flatcars to be used in bridges. When the flatcars are purchased through local scrap dealers who have little or no experience in providing RRFC for bridge superstructures, an accurate assessment of the flatcar condition is difficult. On the other hand, the railroad salvage yards that deal with the railroads on a regular basis have access to the reasons for the removal of the flatcars. If the flatcars are in sound condition (based on experience), the flatcars may be sold for use in bridges. If they are in poor condition, the flatcars are simply cut down and sold as scrap metal. Having

supplied several hundred RRFC bridges, the railroad salvage yards have the experience to initially determine which flatcars are structurally adequate and thus can be used in bridges.

Evaluation of the remaining fatigue life of individual flatcars may be both impossible and impractical. Fatigue damage primarily depends on the stress range level, number of cycles of the applied load, and the type of structural details. Because the load history of individual flatcars is unknown and may vary greatly between flatcars, it would be difficult to predict an accurate fatigue life for individual RRFCs.

To address the issue of fatigue, bridge and county engineers, railroad salvage yards, the Skip Gibbs Company, and individuals with significant experience in RRFC bridges were contacted. Collectively, these contacts do not consider fatigue a problem when RRFC are used on low volume roads. Flatcars that are acceptable to be used as bridge superstructures (i.e., flatcars not removed because of age or derailment) in theory have not reached their design life. The stress ranges magnitude and number of cycles of applied loading should be much greater for a flatcar when used by the railroad industry than one used as a bridge on a low volume road, since the flatcars are designed to support between 50 and 100 ton loads.

Another reason fatigue is not thought to be a problem is that only the major structural members of the flatcar are used as the bridge superstructure. Smaller mechanical components that are prone to fatigue damage are not critical in this particular application of flatcars. As determined during the field test

performed on the Tama County RRFC bridge, the maximum stresses were relatively very small. Finally, a number of users of RRFC bridges (Arkansas, Oklahoma, Skip Gibbs, Ermann Corporation, and Chambliss Bridge Company) provided their experience to verify that fatigue has not been a problem in flatcars used on low volume bridges. As noted in the report to the Wyoming Department of Transportation, BDI also believes that in low traffic environments, fatigue should not be a major issue on RRFC bridges [8].

7.3 Cost, Construction, and Maintenance

One reason to consider using RRFC for low volume bridges is that they present an inexpensive bridge alternative for rural areas with limited resources. Organizations that have used RRFC bridges cite a number of reasons for this, including low initial first cost and construction cost. As an example, the Ermann Corporation estimates the cost of a RRFC from \$7,000 to \$9,000 delivered.

Oklahoma and Arkansas county personnel, two states with significant experience in constructing and maintaining RRFC bridges, were contacted to gather information about the cost, construction and maintenance of RRFC bridges in comparison to traditional steel or reinforced concrete bridges. In this section, the costs presented include everything for complete installation (materials, construction, engineering, etc.). Construction time includes the time it takes to install both the substructure and superstructure of the bridge.

Grant County, Oklahoma, which has between 50 and 60 RRFC bridges, a population of 5,500, and over 2,000 miles of roads, believes that RRFC bridges have saved them millions of dollars. Using only a county crew, a RRFC bridge

can be constructed (substructure and superstructure) in 30 to 45 days at an approximate total cost of \$25,000. In their experience, this compares with a cost of \$75,000 to \$90,000 to build an equivalent structural steel or reinforced concrete bridge. Due to increased regulations, the cost of the conventional bridges can increase even more and the life of the project may extend to 3 years if federal funds are involved in the process. From their experiences, very minimal maintenance has been required for the RRFC bridges.

The Chambliss Bridge Company, based in Arkansas, sells railroad cars and constructs RRFC bridges for counties and private individuals. For many jobs, they bid to install RRFC bridges with savings ranging from 1/3 to 1/2 of the total cost of the bridge when compared to conventional bridge designs. Because of their construction experience, they can completely install a RRFC bridge (both substructure and superstructure) in less than a week. A county crew with no previous experience would normally require up to 3 weeks for the same project. Bulldozers, front-end loaders, backhoes, and cranes have all been used successfully to set the flatcars in place. Once in place, minimal maintenance requirements were required.

Arkansas Highway Department (AHD) officials indicated a few problems associated with RRFC bridges. By law, Arkansas counties do not require county engineers and thus many counties do not have them. County judges are primarily responsible for the bridges and roads in the county. When counties install their own RRFC bridges, many times the main structural members have been altered with unknown consequences. One example given was when

bottom flanges of the main girders were removed to set the RRFC at the desired grade. These alterations make inspection and load rating, which are the responsibility of the AHD, very difficult. Due to the RRFC unknowns (condition of flatcars, type of steel, altered members, etc.), conservative assumptions are made by the AHD and load ratings can sometimes be low and undesirable. It was noted that installation guidelines and accurate load rating techniques could eliminate some of these problems.

8. SUMMARY AND CONCLUSIONS

8.1 Summary

In this investigation, the feasibility of using RRFC for bridge superstructures on low volume roads was studied. The first portion of the project focused on determining the use of RRFC bridges by other states. Through a questionnaire sent to state bridge engineers, it was found that many states with large rural populations use a significant number of RRFC bridges, almost exclusively on their county road system. Wyoming had three of its RRFC bridges load tested by a private firm; no posting requirements were recommended for two of the bridges. The state of Arkansas conducted its own research into the concept that included field load tests of two RRFC bridges. California has authorized the use of a bridge system constructed entirely of RRFC for emergency situations; this system has been used on the Interstate highway system. Despite being used frequently, many state bridge engineers have concerns about the RRFC bridge concept, particularly for RRFC bridges used on primary road system.

The main focus of this investigation was on the analytical and experimental evaluation of a RRFC bridge located in Tama County, Iowa. This bridge consisted of two RRFC placed side-by-side that spanned 42 ft from center-to-center of abutments. The primary longitudinal and transverse members of the flatcars were measured and the calculated properties were used to develop a grillage computer model. After a careful inspection, the bridge was instrumented and two field load tests performed. Equivalent loads from the field

test were applied to the analytical model and the theoretical and experimental displacement and strain results compared. The experimental as well as the theoretical results revealed that this particular RRFC bridge was capable of carrying Iowa legal loads.

Because it was known that many different flatcar types exist, two different sets of structural drawings were obtained from railroad industry contacts. These flatcars were then modeled and loaded using the same vehicles as in the Tama County Bridge. These analytical results showed that the two other types of flatcars also have the ability to safely support Iowa legal loads.

Finally, other issues associated with the RRFC bridge concept were addressed, including availability of flatcars, cost, construction, and maintenance. Individuals with vast experience were contacted for information on these subjects. From their accounts, RRFC bridges offer a significant advantage in terms of cost and construction time when compared with the conventional structural steel or reinforced concrete bridge designs.

8.2 Conclusions

Considering all aspects of this investigation, the following conclusions can be made regarding the feasibility of using RRFC bridges on low volume roads:

- With some responsible engineering control, RRFC bridges can be a viable and economical bridge replacement alternative.
- As discussed in the Wyoming load test report, not all flatcar types are acceptable for use as a bridge superstructure. Cross-sections with non-redundant designs and shallow interior girders should be avoided.

- By obtaining flatcars through businesses with experience in supplying RRFC bridges, concerns associated with the unknown condition of the flatcars can be eliminated.
- Because uncertainties do exist in RRFC bridges, more inspections are recommended during the first years of service.
- In low traffic applications, the unknown fatigue life of the RRFC is not considered a concern.
- If RRFC bridges are to be installed by non-engineering personnel (i.e., county crews), guidelines should be established to ensure the structural integrity of the flatcars. This will help avoid some of the problems encountered in some Arkansas counties.
- In regard to the connection between flatcars, a continuous transverse timber deck appears to provide sufficient lateral load distribution characteristics.

9. RECOMMENDED RESEARCH

Additional research is recommended so that Iowa counties can take advantage of the main finding of this report - that is, in certain situations RRFC bridges can be a viable and economical bridge replacement alternative on low volume roads. Specifically, it is recommended that a demonstration project involving the construction and testing of a RRFC bridge be undertaken. As part of this project, design procedures as well as construction guidelines will be established to allow county engineers to consider this type of bridge alternative. After the demonstration bridge is in service, it should be periodically inspected and service load tested for a two year period . With the design criteria established, construction guidelines, and behavior information from the demonstration bridge's first two years of service, counties that desire to take advantage of the economical, quickly installed, safe crossings, provided by RRFC bridges can do so.

APPENDIX A

STATE DEPARTMENT OF TRANSPORTATION QUESTIONNAIRE

Iowa Department of Transportation Research Board

Research Project TR – 421

“Feasibility of Using Railroad Flat Cars for Low Volume Road Bridges”

Questionnaire completed by _____

Title _____

Address _____

City _____ State _____ Zip _____

Phone _____ Fax _____

Please return the completed questionnaire using the enclosed envelope (or fax your response) to:

Prof. F. W. Klaiber
Dept. of Civil and Construction Engineering
Iowa State University
Ames, IA 50011
Phone: (515) 294-8763
Fax: (515) 294-8216

1) Do any RRFC bridges exist in your state? YES NO

If yes, approximately how many? _____

If yes, on what type of roads are they used?

PRIVATE _____ COUNTY _____ STATE _____ OTHER _____

2) Has your state ever conducted research to determine the feasibility and adequacy of RRFC bridges?

YES NO

If yes, what did your research conclude on the use of RRFC bridges? Please list any reports that are available on the subject.

3) Are you aware of other transportation agencies that have experience with RRFC bridges?

YES NO

If yes, please provide the name, phone number, and address of the person who could be reached for more information.

4) Please provide any additional information (positive or negative) on the idea of using RRFC for bridges on secondary roads.

APPENDIX B

RRFC INFORMATION

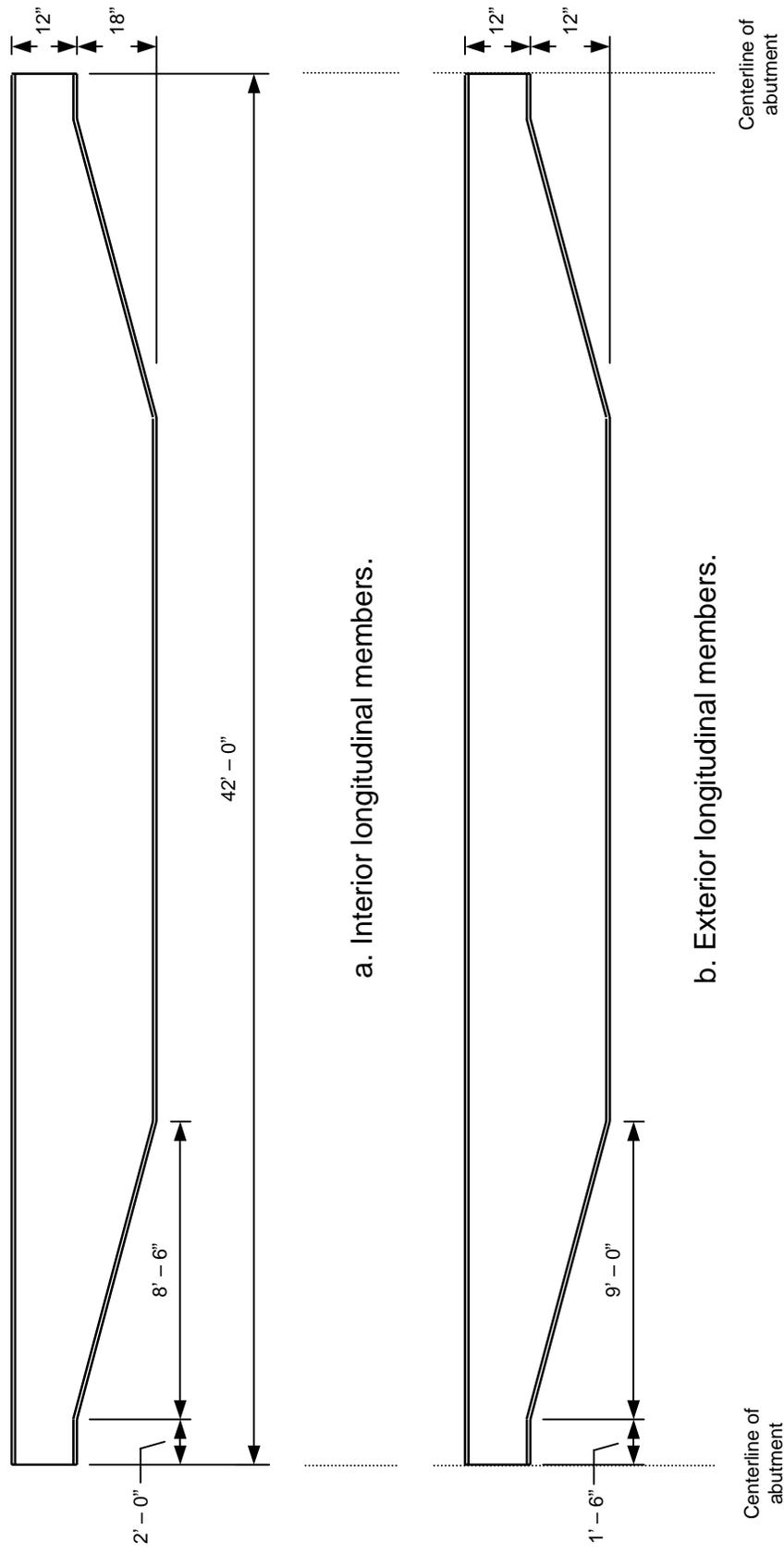


Figure B.1. Tapered regions in the longitudinal members of the Tama County flatcar.

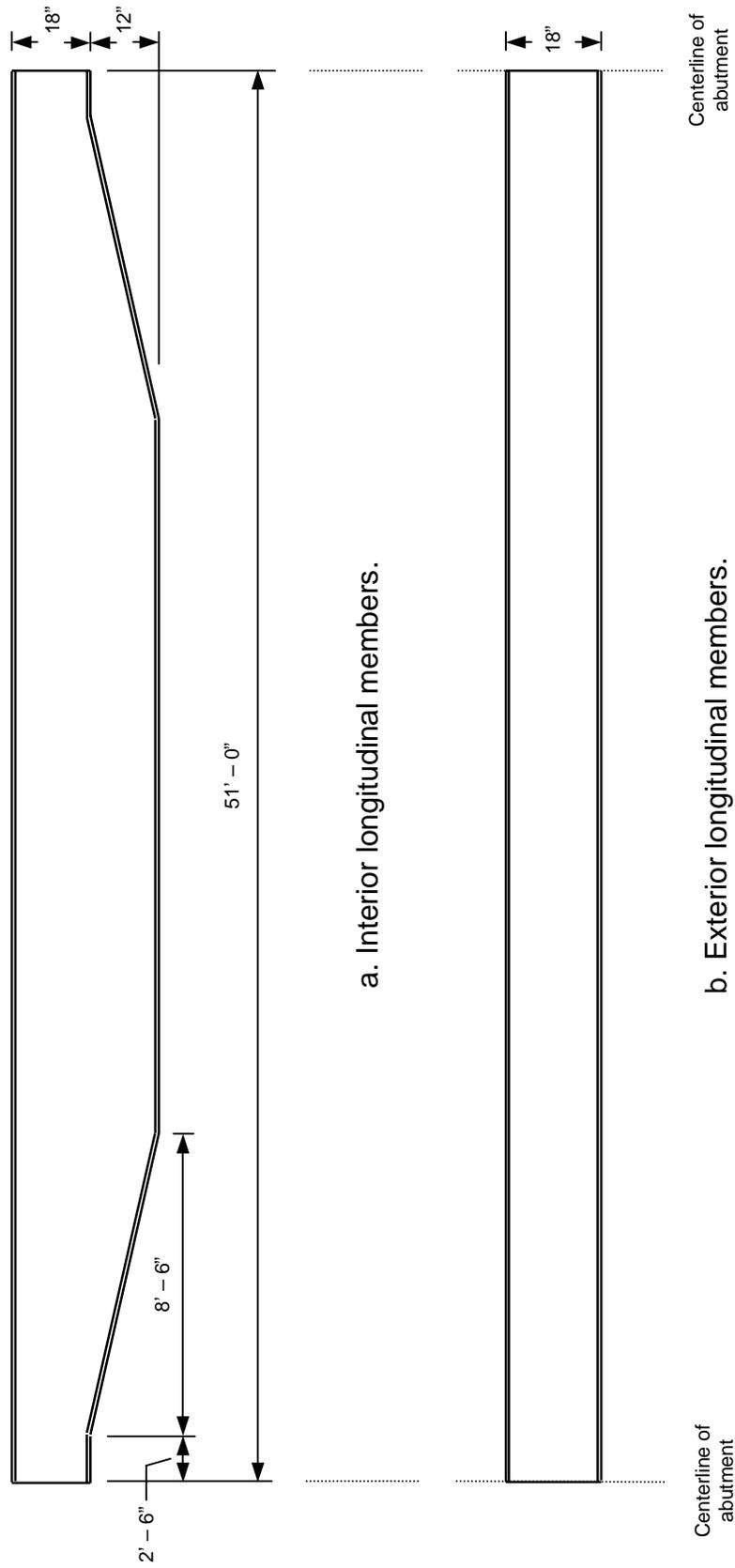


Figure B.2. Tapered regions in the longitudinal members of the Thrall flatcar.

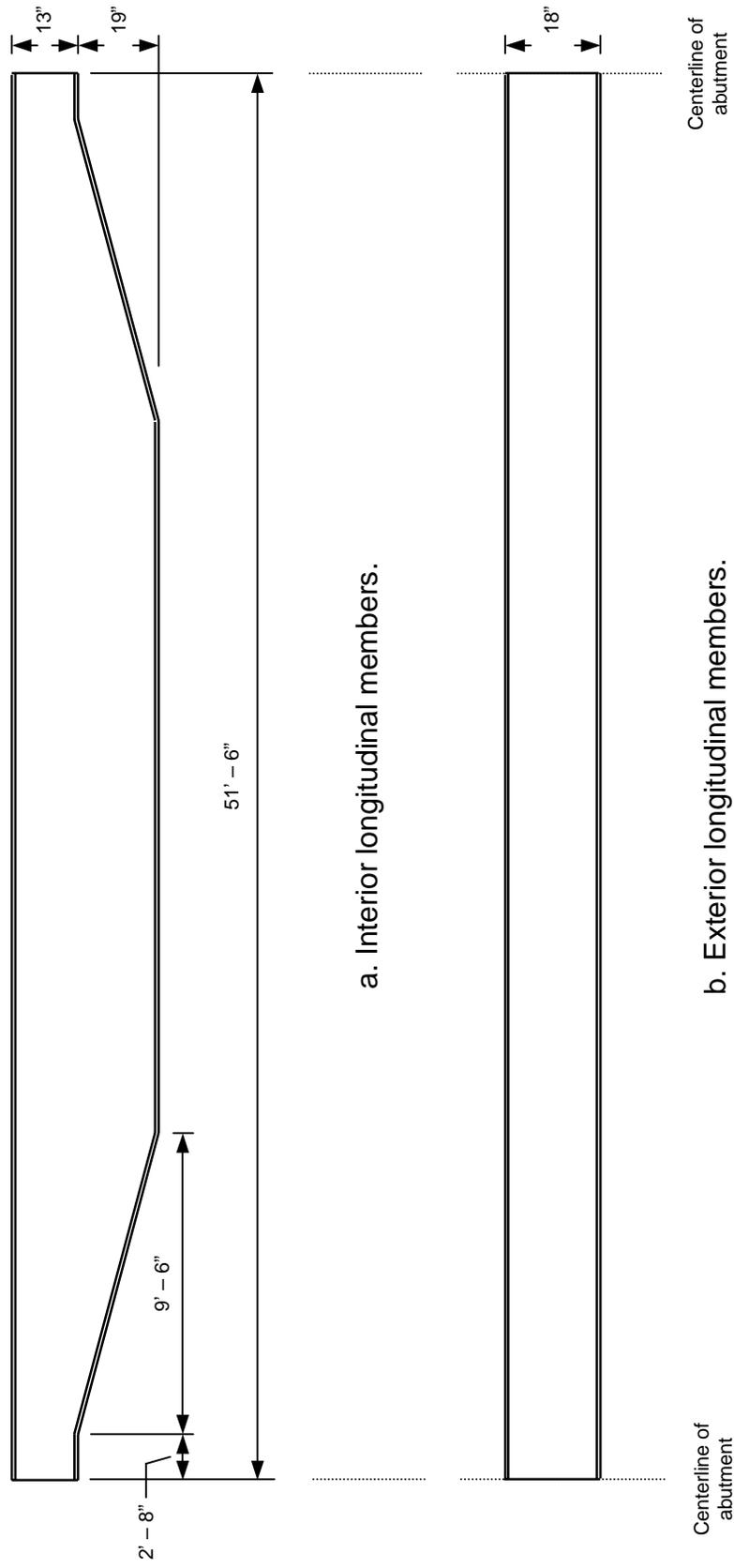


Figure B.3. Tapered regions in the longitudinal members of the Canadian National flatcar.

Table B.1. Member properties of three different RRFC.

	Area (in ²)	I _x (in ⁴)	I _y (in ⁴)
<u>Tama County RRFC</u>			
Exterior Girders			
at supports	12.7	263	19
at midspan	17.2	1,421	22
Main Girder			
at supports	50.2	1,109	2,300
at midspan	63.7	9,597	3,003
Major Transverse Members			
at exterior	10.88	963.9	25
at interior	14.2	2,119	45
<u>Trall RRFC</u>			
Exterior Girders			
at supports	12.6	554	14
at midspan	12.6	554	14
Main Girder			
at supports	61.9	3,470	3,006
at midspan	77.8	11,540	3,679
Major Transverse Members			
at exterior	7.6	156	13
at interior	11.7	951	13
<u>Canadian National RRFC</u>			
Exterior Girders			
at supports	12.6	554	14
at midspan	12.6	554	14
Main Girder			
at supports	78.7	2,757	4,868
at midspan	95.9	17,895	5,687
Major Transverse Members			
at exterior	12.5	655	32
at interior	16.8	2036	32

APPENDIX C

LOAD RATING CALCULATIONS

The following calculations were made to determine the load rating for the C-shaped longitudinal member of the TCB. As noted in Section 6.3, the LRFR load rating method was used in which a theoretical rating factor is described by the following equation:

$$RF = \frac{\phi R_n - \gamma_d D}{\gamma_L L(1+I)}$$

The RF factor determined represents the theoretical rating of the member in question. Using the Load and Resistance Factor Design along with the calculated member properties (see Section 5.1), a nominal flexural strength, R_n , of the C-shaped member was found to be 3785.15 in-kips. This value takes into account the limit states of yielding, lateral-torsional buckling, flange local buckling, and web local buckling. A resistance factor, ϕ , of 0.7 was applied to the nominal strength. To determine the nominal dead load, D , applied to the member, the self-weight of the TCB was applied to the ANSYS model described in Chapter 5. The self-weight was found to cause a maximum dead load moment of 114.17 in-kips in the C-shaped members at center of the bridge. In the analytical portion of this investigation, the live load moment in the members of the TCB were found by applying the tandem-axle test vehicle load from the first load test to the ANSYS model (see Section 5.1); it was determined to be 311.0 in-kips. A dead load factor, γ_d , is assigned a value of 1.2 and is increased by 20% when overlays are present. A live load factor, γ_L , is assigned a value

ranging from 1.3 to 1.8 depending on the average daily traffic and the enforcement of overload restrictions. The live load impact factor, I , ranges from 0.1 to 0.3 depending on the bridge's wearing surface. A value of 1.2, 1.3, and 0.2 were used for γ_d , γ_L , and I , respectively in the rating of the TCB. Combining all factors, a theoretical load rating factor of 5.18 was determined for the C-shaped longitudinal members of the TCB.

As described in Section 6.3, adjustment factors K_a and K_b are applied to the theoretical rating after field load testing has been conducted. K_a accounts for the differences between strains calculated in the ANSYS model and those obtained during a field load test. The following equation is used to determine K_a :

$$K_a = \frac{\epsilon_C}{\epsilon_T} - 1$$

From Figure 6.11, the values of the theoretical strain, ϵ_C , and experimental strain, ϵ_T , were obtained. With $\epsilon_C = 85.02$ MII and $\epsilon_T = 138.77$ MII, K_a was calculated to be - 0.39.

The adjustment factor K_b is determined from three factors; K_{b1} , K_{b2} , and K_{b3} . K_{b1} is a factor that takes into account the behavior of the bridge beyond the test load level. This factor is assigned a value between 1 and 0 where 1 indicates that the behavior of the bridge at a higher load level will be the same as the behavior exhibited at the test load level. This is determined by loading the ANSYS model with a load 1.33 times greater than the rating vehicle load and

ensuring that linear behavior is present in the bridge components at the higher load level. For the TCB, a value of 1.0 was used for K_{b1} .

K_{b2} is a function of the type and interval of inspections. This factor is included to ensure that any change in the condition of the bridge while operating at higher loads will be diagnosed in time to reduce the load ratings that do not endanger the bridge. A value of 0.8 was assigned to K_{b2} and recognizes that the TCB is not likely to have frequent or in-depth inspections performed. K_{b3} is included to account for sudden failure of the bridge due to fracture or fatigue of critical members and the absence of redundant members. Because the TCB flatcars have a redundant geometry (see Section 2.4) and fatigue is not considered a failure mode (see Section 7.2), a value of 1.0 was assigned to K_{b3} .

By multiplying the values of K_{b1} , K_{b2} , and K_{b3} , a value of 0.8 was calculated for K_b . After K_a and K_b were applied to the theoretical rating factor, a modified rating factor, RF_T , was calculated to be 3.57 for the C-shaped longitudinal members of the TCB. The final load rating of 178.7 kips was determined by multiplying RF_T by the weight of the vehicle used to obtain the experimental and analytical results, 52.14 kips.

As mentioned in Section 6.3 of this thesis, these load rating values do not account for the damage that exists in the TCB members. The field load tests performed on the TCB should serve as a 'proof' load test and a load rating of 52.14 kips should be used in practice. For a complete bridge load rating, rating factors for secondary members and abutments should also be considered.

APPENDIX D

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APPENDIX E

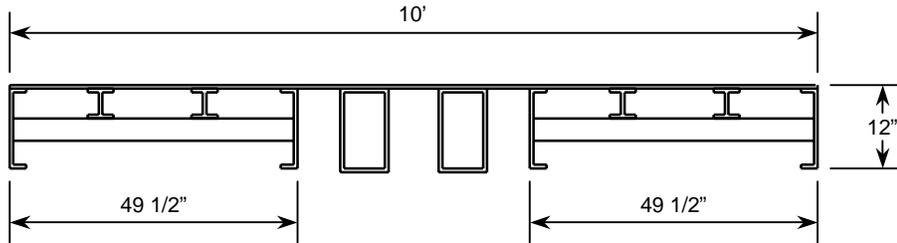
RRFC BRIDGES IN OKLAHOMA

Several other types of RRFC structural frameworks were observed during a field trip to north central Oklahoma (in Grant County). Photographs and cross-section details of several of the RRFC bridges inspected are shown in Appendix E. Illustrated in Figure E.1 are two typical cross-sections that were observed. A bridge that had cross-section 1 (illustrated in Figure E.1a) is shown in Figure E.2. This bridge, which used two RRFC units side by side for a roadway width of 20 ft – 1 in., had an out-to-out span of 40 ft - 10 in. As shown in Figure E.2a, this bridge had a cast-in-place concrete deck. The substructure, shown in Figure E.2d, consisted of stiffened HP sections for the capbeam and circular steel pipe sections for the columns. The two RRFC units were tied together along the bridges' longitudinal centerline at 2 ft. intervals using 8 in. channels.

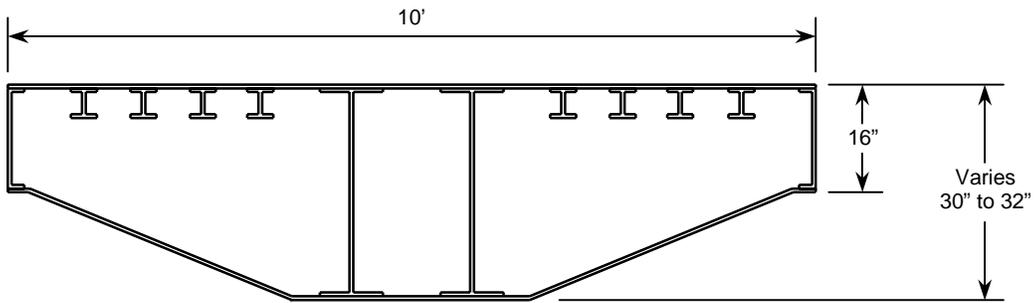
Photographs of another bridge in Grant County, Oklahoma, are presented in Figure E.3. This bridge, which had an asphalt covered deck, and consisted of three RRFC units side by side for a roadway width of 24 ft – 1 in. between curbs, had an out-to-out span of 85 ft - 2 in. Each of the three RRFC in this bridge had the cross section geometry illustrated in Figure E.1b. The bottom flange of one of the three double I-sections is shown prominently in the upper left half of the photographs in Figure E.3c. The bottom flanges for the other two units are also shown in the background. This bridge was one of the bridges inspected that had guardrails (see Figures E.3a and E.3b).

Photographs in Figure E.4 illustrate another bridge that utilized two RRFC's with geometry similar to that shown in Figure E.1b. The two units used had slightly different

cross section geometry and were of different span lengths. The units had out-to-out span lengths of 89 ft - 6 in. and 85 ft - 10 in., respectively. As shown in Figure E.4b, to obtain the desired roadway width, a metal deck grate was used between the two units. Additional structural framing was placed between the two units to support the grating. A number of the bridges observed on the field trip utilized grating to widen the bridge. Although it was over 20 ft from the bridge deck to bottom of the stream channel, there were no guardrails. It should be noted that of the ten bridges that were inspected, only several used connections along the length of the bridge between RRFC units.



a. Cross-section 1



b. Cross-section 2

Figure E.1. Typical cross-sections found in Oklahoma RRFC



b. Side View



a. Top View



d. Abutment details



c. RRFC framework

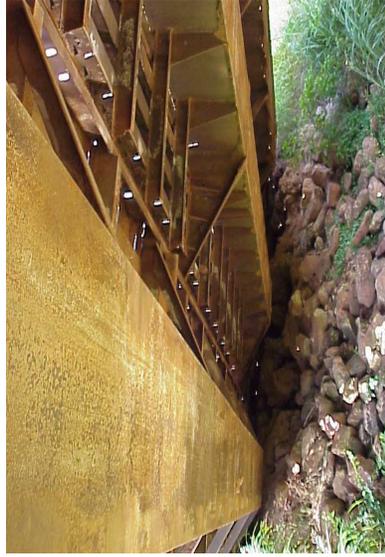
Figure E.2. RRFC Bridge 1: Concrete deck bridge in Grant County, Oklahoma



a. Top view



b. Side view



c. RRFC framework

Figure E.3. RRFC Bridge 2: Asphalt surface deck bridge in Grant County, Oklahoma



a. Side view



b. Top view

Figure E.4. RRFC Bridge 3: Partial grid deck

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