Development of Guidelines for the Rating, Inspection and Acquisition of Railroad Flatcars for use as Highway Bridges on Low-Volume Roads

-Final Report-

July 2011
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Indiana LTAP Center

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Within Indiana, as well as other states, several of these bridge structures have been in service for many years. Although their performance has been satisfactory, there is little to no guidance in the AASHTO Specifications for load rating and inspecting these structures. This inexperience has led to many of these bridges being conservatively posted for traffic loads which are likely less than their actual capacity. Furthermore, inspectors may not be familiar with the details and areas which require special attention during field inspection. County and state highway officials also have little guidance when choosing which railroad flatcars are suitable for use as bridges.
This research is focused on the development of load rating guidelines for railroad flatcar bridges through the use of field instrumentation and controlled load testing. The proposed load rating guidelines intended to be a simple, yet not over-conservative, alternative to existing load rating procedures as well as those developed in previous research studies. Proposed inspection and acquisition guidelines were also developed based on field observations of numerous Indiana railroad flatcar bridges and discussions with county officials who possess a great deal of experience with these structures.</td></tr>
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DEVELOPMENT OF GUIDELINES FOR THE RATING, INSPECTION, AND ACQUISITION OF RAILROAD FLATCARS FOR USE AS HIGHWAY BRIDGES ON LOW-VOLUME ROADS

-FINAL REPORT-

Prepared for

The Indiana Local Technical Assistance Program (LTAP)

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July 2011

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ABSTRACT

Due to budget constraints, many state and county highway agencies are often forced to develop innovative and economical rehabilitation strategies for deteriorated bridges. One such option is to use a retired railroad flatcar for a bridge superstructure. Railroad flatcars can be found in many lengths, making them versatile options for replacing bridges of a range of span lengths up to about 90 feet. Railroad flatcars are also completely modular and lightweight, allowing for quick construction.

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CHAPTER 1. INTRODUCTION

1.1. Background & Organization

Due to budget constraints, many state and county highway agencies are often forced to develop innovative and economical rehabilitation strategies for deteriorated bridges. One such option is to use a retired railroad flatcar for a bridge superstructure. Railroad flatcars (RRFCs) can be found in many lengths, making them versatile options for replacing bridges of a range of span lengths up to about 90 feet. Railroad flatcars are also completely modular and lightweight, allowing for quick construction.

Within Indiana, as well as other states, several of these bridge structures have been in service for many years. Although their performance has been satisfactory, there is little to no guidance in the AASHTO Specifications for load rating and inspecting these structures. This inexperience has led to many of these bridges being conservatively posted for traffic loads which are likely less than their actual capacity. Furthermore, inspectors may not be familiar with the details and areas which require special attention during field inspection. County and state highway officials also have little guidance when choosing which railroad flatcars are suitable for use as bridges.

Chapter 2 of this document will provide an in-depth literature review focusing on the design of RRFCs and previous studies on RRFC bridges. Chapter 3 will describe field visits to many of the RRFC bridges within Indiana. Chapter 4 describes the field instrumentation and controlled load testing, while Chapter 5 presents the results of the testing. Chapter 6 presents the interpretation of the test results and background to the development of the proposed load rating method. Chapter 7 describes the conclusions and recommendations of this research study.

The proposed guidelines for load rating, inspection, and acquisition of railroad flatcars for use as low-volume road bridges can be found in Appendices E, F, and G, respectively. Appendix H presents an example load rating using the proposed guidelines for a typical RRFC bridge and a bridge constructed with a boxcar. Finally, Appendix I presents a comparison of the proposed guidelines for load rating with results obtained from previous research studies by Iowa State University.

1.2. Research Objectives

The research objectives for this project are as follows:

- Develop a load rating procedure for highway bridges made of railroad flatcars. The load rating procedure will focus on evaluating the live load bending stress for longitudinal elements, including the main girders, exterior girders, and stringers.

- Develop inspection guidelines for highway bridges made of railroad flatcars.

- Develop guidelines for the acquisition of railroad flatcars to be used as highway bridges on low volume roads.

CHAPTER 2. CRITICAL REVIEW OF LITERATURE

The objective of the literature review was to collect and review relevant research and experience in regard to railroad flatcars (RRFCs) being used as low-volume road bridges. To complete this task, a comprehensive search of existing literature was undertaken. The search included informal surveys conducted by telephone and email to railroad companies and railroad car manufacturers. Additionally, past studies on RRFC bridges performed at other universities and agencies were reviewed.

This section begins with a brief overview of RRFCs, including their design, in-service use, and reasons for retiring them from the railroad industry. The section then discusses previous research and experience in the load rating, inspection, and acquisition of RRFC bridges.

2.1. Background of Railroad Flatcars

The first step in understanding how RRFCs will behave as bridges is understanding how they behave during their service lives on the railroad. The following sections will attempt to give the reader a general understanding of RRFCs. These sections are, in no way, an attempt to provide a complete and comprehensive detail of RRFCs. They are provided simply to give the reader a general understanding of some of the background information regarding RRFCs in the context of using them as bridges.

2.1.1. Geometry and Structural Features

There are a large variety of types of RRFCs, typically found in lengths of roughly 56 and 89 feet, and approximately 8 to 10 feet wide. Flatcars are typically constructed with one main girder, running longitudinally down the middle of the car, and two exterior girders on either side of the main girder. (It should be noted in the railroad industry the girders are typically referred to as "sills". Since this report is focused on using RRFCs as bridges, and in the bridge industry the term "girder" is more frequently used, they will be referred to as such.) Figure 2.1 shows an elevation and plan view of a typical RRFC.

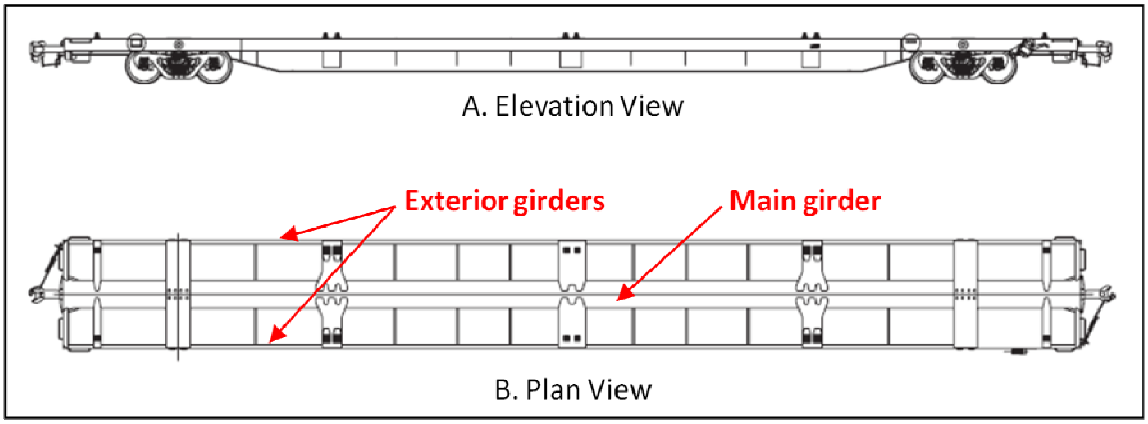

Figure 2.1: Elevation (A) & plan (B) view of typical RRFC (World Trade Ref 2010)

2.1.1.1. Material

Since the 1970's, the main structural elements of flatcars have typically been constructed with high-strength low-alloy steels, with yield strengths ranging from 50-70 ksi. There are RRFCs, however, which are constructed with steels having lower yield strengths (36 ksi) and higher yield strengths (100 ksi). The non-structural elements, such as cross-bracing or transverse beams, of a RRFC might be constructed with a different type of steel. Typically, these non-structural elements are constructed with A36, which has a yield strength of 36 ksi (Mitzenberg 2009).

Unfortunately, many RRFCs being used as bridges were probably designed before the 1970's. It is difficult to find exact information regarding the type of steel used for constructing flatcars during this timeframe. From conducting informal surveys of railroad companies and railroad car manufacturers, engineers estimated steels such as A283, A113, A33, A36, or A7 could have been used. It is also estimated that engineers before the 1970's might have used the typical design practices of today: selecting 50 ksi steel for structural elements and 36 ksi steel for non-structural elements (Lydic 2009).

2.1.1.2. Connections

Prior to roughly 50-60 years ago, most RRFCs were constructed with riveted connections. In the late 1950's and into the 1960's, flatcars began to shift toward welded construction. Since this time most RRFCs have been constructed with welded connections; however, the occasional use of rivets continued and is still being used for different types of cars today (McNally 2009).

2.1.1.3. Supports

RRFCs are designed to be supported at the wheel trucks, the location where the wheels are connected to the flatcar. The wheel trucks are located a few feet from each end of the flatcar. Figure 2.2 shows the location of the wheel trucks on the flatcar.

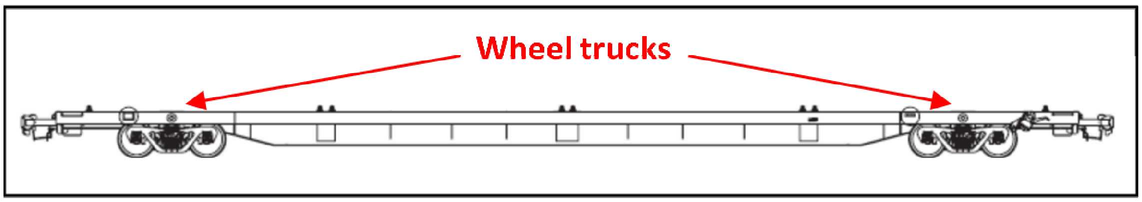

Figure 2.2: Location of wheel trucks (World Trade Ref 2010)

2.1.2. Design Specifications

Railroad flatcars are designed according to the Association of American Railroads (AAR) Specifications, which were issued in 1964. More specifically, flatcars are designed to the standards in AAR Section C (Car Construction – Fundamentals and Details) and AAR Section C, Part II (Design, Fabrication, and Construction of Freight Cars).

Design considerations for RRFCs include vertical, axial, and lateral loads. Only vertical design loads will be presented during this report since these are the loads which will give some indication of how RRFCs will perform when used as bridges on low-volume roads. General information on the fatigue design provisions of RRFCs will also be presented (AAR 2007).

2.1.2.1. Live Load

According to the current AAR Specifications, there are three major classifications of design live loads for flatcars (AAR 2007). These loads are shown in Table 2.1. It was not confirmed if the values in Table 2.1 date back to 1964 or if they were issued in a newer Specification.

Table 2.1: Design live loads for RRFCs

Live Load Limit kips (tons)	**Gross Rail Load** kips (tons)
140 (70)	220 (110)
200 (100)	263 (131.5)
220 (110)	286 (143)

In Table 2.1, the live load limit refers to the maximum live load that can be applied to the flatcar while the gross rail load refers to the maximum vertical load on the flatcar, including the live load plus the self weight of the flatcar. The live load limit and gross rail load must be stenciled onto the side of the flatcar before going into service (AAR 2007). These design live loads must be applied to a RRFC in four different loading patterns, as shown in Figure 2.3.

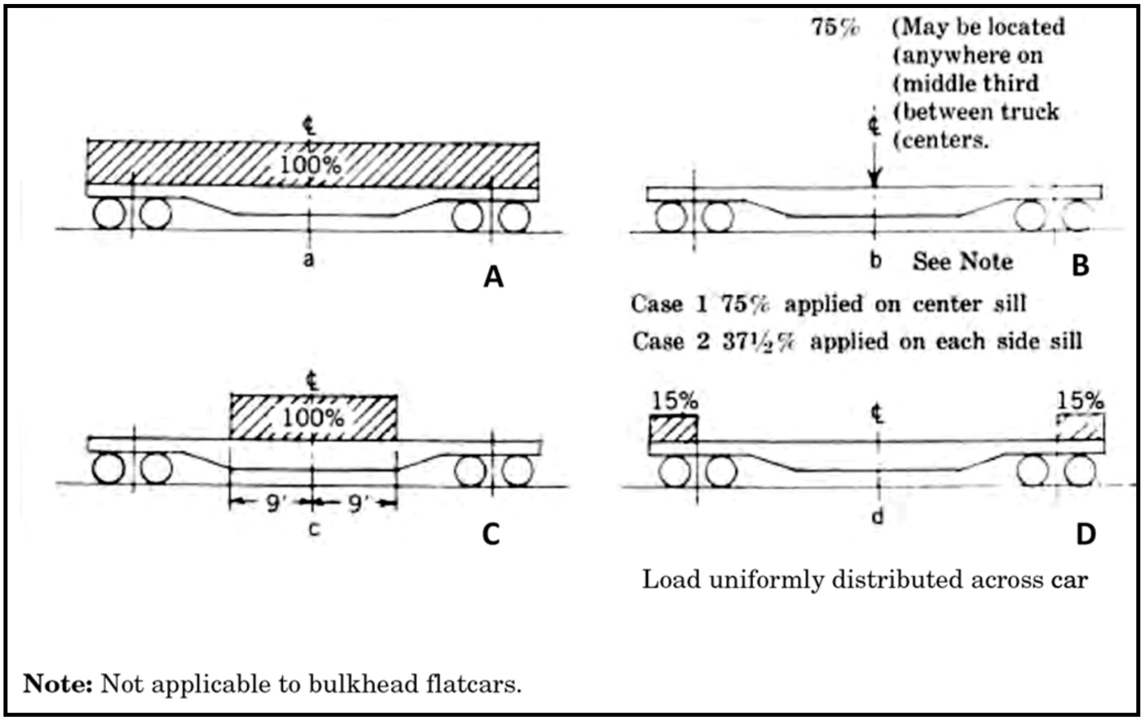

Figure 2.3: Loading patterns for live load on RRFC (AAR 2007)

Load Pattern A involves applying the full live load across the entire length of the flatcar. This is a typical loading pattern, although probably heavier than typical service loads. Load Pattern B has two cases. The first case involves applying a point load, consisting of 75% of the total live load at a location on the middle third of the flatcar between the wheel truck centers. The second case applies a point load of 37.5% of the total live load to each exterior girder, and again may be located on the middle third of the flatcar. Load Pattern C places 100% of the maximum live load over the middle 18 feet of the flatcar, measured 9 feet on each side of the centerline of the car. Load Pattern D applies 15% of the maximum live load to the distance between the wheel trucks to the end of the flatcar, on each side of the car. This load must be applied uniformly over the width of the car, and would create a negative moment at midspan of the flatcar (AAR 2007).

2.1.2.2. Lift Truck Wheel Loads

In addition to live load, flatcars must be designed for another vertical load case: lift truck wheel loads. The purpose of this load case is to simulate loading/unloading of the RRFC by a fork-lift truck. The fork-lift is simulated with front axle loads of 50,000 lb minimum or a wheel load of 25,000 minimum. The treads of the truck are assumed to have a spacing of 32 inches center-to-center with a tire print of 13 $^{1}/_{2}$ x 5 $^{3}/_{8}$ with a 16 inch width of tire. The wheel loads are placed to create the critical design load for the member under consideration (AAR 2007).

2.1.2.3. Design of Structural Elements

Structural elements of RRFCs are designed according to the allowable stress method. The following sections will describe the design of key structural elements related to the bridge performance of RRFCs. Depending on the element (i.e., deck, main girders, etc) one of three cases is used to establish the allowable stress. Case I consists of using a load factor of 1.8 applied to each load with an allowable stress of the yield strength or 80% of the ultimate strength, whichever is lower, or the critical buckling stress. Case II consists of using a load factor of 1.0 applied to each load with an allowable stress of the yield strength or 80% of the ultimate strength, whichever is lower, or the critical buckling stress. Case III consists of using a load factor of 1.0 applied to each load with an allowable stress of the ultimate strength (AAR 2007).

Deck elements of RRFCs can be constructed of wood decking boards, metal, laminated wood panels, composite wood-metal, or other proprietary systems. All deck systems must be designed for the allowable stress specified in Case II (AAR 2007).

Main girders (center sills) of RRFCs must be designed for the following three load cases:

- Dead load, live load, and a tension (also called draft in railroad terminology) or compression (buff) load designed for the allowable stress specified in Case I.

- Dead load, live load, and a compressive end load designed for the allowable stress specified in Case II.

- Dead load, live load, longitudinal impact end load, and vertical forces induced by horizontal impact end load designed for the allowable stress specified in Case III (AAR 2007).

Floor stringers of RRFCs must be designed for two load cases:

- Dead load and lift truck wheel loads designed for the allowable stress specified in Case II.

- Dead load, a uniformly distributed live load (Figure 2.1.C), and the critical longitudinal (end) load designed for the allowable stress specified in either Case II or Case III, as applicable. The portion of the live load on each stringer is computed using the panel area method (AAR 2007).

Larger transverse beams (crossbearers) of RRFCs must be designed for the two following vertical load cases:

- A concentrated load at the center equal to 75% of the load limit multiplied by the percentage of load carried by the exterior girders, designed for the allowable stress specified in Case I.

- A concentrated load on each exterior girder equal to 75% of the load limit multiplied by one-half of the percentage of load carried by the main girder, designed for the allowable stress specified in Case I (AAR 2007).

Smaller transverse beams (crossties) of RRFCs must be designed for the two following load cases:

- Dead load and lift truck wheel loads designed for the allowable stress specified in Case II.

- Dead load, critical live load, and vertical acceleration induced by horizontal impact designed for the allowable stress specified in Case III (AAR 2007).

2.1.2.4. Fatigue & Fracture Provisions

The fatigue design of the main girders of RRFCs has some similarities to the fatigue design of bridge elements as specified in AASHTO (2010). The fatigue provisions of AAR are based on Miner's Rule and a classical S-N (stress vs. number of cycles) curve (AAR 2007).

Experimental data was used to develop the fatigue loading spectra for RRFCs; these loading spectra are provided depending on the type of car, type of load, and whether or not the car is loaded. The spectra provide maximum and minimum loads and the percent occurrence associated with these loads. Other information also included in the loading spectrum includes the average number of cycles/mile and the speed at which the test data was recorded (AAR 2007).

The fatigue resistance is based on the type of detail being evaluated. AAR has detail properties based on the yield strength of the material, which provide a diagram of the detail, description of the detail, effective stress range at a prescribed number of cycles, and the slopes of the two lines on the S-N curve (AAR 2007).

Based on the fatigue loading and the fatigue resistance, a life can be calculated in terms of cycles and miles. The life in miles is then compared to the AAR requirements. Unit train and high utilization cars must have fatigue lives of at least 3,000,000 miles and general interchange cars must have fatigue lives of at least 1,000,000 miles (AAR 2007).

Although not specifically addressed as a fracture requirement, there are toughness requirements for some structural members on RRFCs. The webs, from the wheel trucks to the end of the car, and the entire bottom cover plate(s) of the main girders must be made from steel having a minimum Charpy V-Notch toughness value of 20 ft-lbs at 0°F on a heat lot basis per ASTM A673 (AAR 2007).

2.1.2.5. Design Specification Issues with RRFC Bridges

Unfortunately many of the flatcars being used as bridges were designed prior to 1964, when the AAR Design Specifications were issued. This means many of the RRFCs used as bridges were not designed to any standard loading. These cars were instead designed according to conventional practices established by the railroad companies at that time. From the informal surveys conducted of railroad companies and railroad car manufacturers, it was estimated the design live load limits could have been much lower than the standard practices today (McNally 2009).

2.1.3. In-Service Use

As previously discussed, RRFCs can be relatively long, up to 89 feet, and are able to carry heavy loads. This simple design means flatcars are able to carry bulky and heavy items. Typical items which can be carried on flatcars include pipes, steel products, or heavy machinery. A typical RRFC in service can be seen in Figure 2.4.

Figure 2.4: Typical RRFC in service (TTX Corporation 2010)

2.1.3.1. Inspection & Load Rating

Based on the informal surveys, RRFCs are not currently, and were never, inspected for fatigue prior to being placed into service on the railroad. Flatcars are, however, designed for fatigue based on AAR Specifications. There are no structural inspections performed on flatcars during service, unless the car has been derailed and there is a possibility of structural members being damaged. The majority of in-service inspections are to verify that nonstructural elements, such as the wheels, braking systems, and couplers, are functioning properly (McNally 2009).

RRFCs are typically not load rated once being put in service; however, there are a few cases which may prompt a load rating. One such case would be if the expected applied loads changed such that they exceeded the design loads. Another scenario where a RRFC would be load rated is if a structural member was damaged in an accident. In this case, the damaged member would be rated to see if it would still be fit for service (Sweeney 2009; Unsworth 2009).

2.1.4. Reasons for Retirement

There are three main reasons RRFCs can be retired from service. The first reason to be discussed is age. RRFCs built after 1974 have a service life of 50 years. Flatcars built between 1964 and 1974 must be retired after 40 years of service. This service life could be extended by 10 years if the car was rebuilt according to AAR's Rule 88. For a railcar to be considered rebuilt, a specific set of inspection and repair criteria must be met (AAR 2005). Prior to 1964, there were no service life limits. Railroad companies can petition to the AAR to extend the service life of a car to 65 years if the flatcar can pass inspections and a successful stress analysis is performed. This extension process is rarely performed (Mitzenberg 2009).

The second reason flatcars are retired is due to economics. For example, if a flatcar is derailed and structural members become damaged, it might be more cost effective to purchase a new car rather than repairing the damaged car. Other than major damage from derailment, normal wear-and-tear maintenance costs can also become expensive. If these maintenance costs become too much compared to the cost of a new car, the flatcar could be retired (Sweeney 2009).

The third reason RRFCs could be retired is if they are no longer being used. This could be if there is a lack of need of flatcars or if customers are able to provide their own railcars, making some railroad companies' cars obsolete (Unsworth 2009).

2.2. Using Railroad Flatcars as Bridges

After RRFCs are retired from service in the railroad industry, they can then be used as bridges on low-volume roads. The following sections will present information on past and current research/experience in using RRFCs as bridges. Some key topics to be discussed in these sections include why RRFCs are attractive options for low-volume road bridges, which flatcars can and should be used, and procedures for load rating these types of structures.

2.2.1. Montana State University Research

Dr. Bruce Suprenant of Montana State University conducted a study on RRFC bridges as part of the Montana Rural Technical Assistance Program. The report was published in 1987, and portions of it appeared in the November 1987 issue of *Roads & Bridges* magazine (Suprenant 1987a; Suprenant 1987b). The study is primarily focused on which RRFCs can be used as economical solutions for an aging transportation system, especially for bridges on low-volume roads. Also addressed as part of the study is how these structures are used in other capacities such as for agricultural use, private use, or temporary structures (Suprenant 1987a).

2.2.1.1. Use of RRFCs as Bridges

According to Suprenant, a contractor in Alaska is believed to be the first person to use an old railroad car as a bridge. One of the reasons the contractor used the railroad car as a bridge is the same reason they are used today: RRFCs can be economical alternatives for bridge structures. Oftentimes, a RRFC can be purchased for much cheaper than the purchase and delivery of new steel beams. Retired flatcars, which meet dimensional requirements, were also readily available at the time of the contractor's use. (Suprenant 1987a).

RRFCs are also easy to install. If the flatcar is being used as a replacement superstructure, the contractor or county highway department working on the structure can simply lift the flatcar onto the existing abutment. After a connection is made between the foundation and the railcar, the new bridge system is complete (Suprenant 1987a).

2.2.1.2. Selecting a RRFC

Much of Suprenant's report is focused on the selection process used in choosing an adequate railroad car, because "all railroad cars are not created equal" (Suprenant 1987a). Suprenant makes it clear this is one of the most challenging aspects of using railroad cars as bridges; therefore, engineering judgment must be used in this process. This selection process includes evaluation of the following items:

- **Condition Survey**: Before purchasing a railroad car, a condition survey should be performed. The overall dimensions of the flatcar should be measured to determine whether or not it is suitable for a specific site. The spacing and dimensions of all members should be obtained to determine section properties, which can be used in later bridge inspections. The railcar should also be inspected for damaged members, which would greatly reduce the strength capacity of the car. A visual inspection, including the use of dye penetrant, should be performed on the main members to verify cracks are not present.

- **Strength**: The strength of the material is important because it directly affects the overall resistance of the flatcar. Hardness (Rockwell and Brinell) or yield strength tests can be performed on material from the flatcar itself. Several specimens from railcars were tested and the material behaved similar to A36 steel, with a yield strength of 36 ksi. Older riveted flatcars, however, might have been constructed of A7 steel, with a yield strength of 33 ksi.

- **Fatigue**: During their service lives, RRFCs are subjected to a great deal of cyclic loading. Estimating the remaining fatigue life of a flatcar after years of rail service is extremely difficult. One way to estimate the fatigue adequacy of a RRFC bridge would be to compare the applied stress range to the endurance limit of material, which could be estimated as one-half of the yield strength. In any case, flatcars should be used on low-volume roads to minimize the probability of a fatigue failure. (It should be noted this approach is not consistent with the standards used in the current AASHTO provisions (AASHTO 2010)).

- **Simple Testing Methods**: Two simple testing methods can be used to estimate the adequacy of a flatcar as a bridge. One procedure involves supporting the flatcar on timbers and driving a heavy truck or tractor over it. This proof test would show the flatcar is adequate for carrying the weight of the test vehicle. Another procedure involves supporting the car on timbers and measuring the deflection. A stiffness of the flatcar could then be calculated and used for load rating and strength calculations (Suprenant 1987a).

2.2.2. Arkansas State University Research

Dr. Thomas Parsons conducted a study on using RRFCs as low-volume road bridges for the Arkansas State Highway and Transportation Department in 1991. The study not only included RRFCs but gondola cars and boxcars as well. Since this report is focused on flatcars, these types of railcars will be primarily discussed. Parson's research included the development of a railroad car bridge database, field instrumentation of four railcar bridges, constructing and testing a one-third scale model, and the development of a load rating software program (Parsons 1991).

2.2.2.1. Use of RRFCs as Bridges

As part of the research project, a survey was conducted of Arkansas county officials and performed field visits to existing RRFC bridges to develop an inventory of these structures within the state. As of 1991, Arkansas had approximately 110 RRFC bridges. The majority of these bridges were single span structures, with a handful of two and three span bridges. Many of these bridges were two railcars wide, with or without a "spacer" between the cars; however, a few of the bridges were only one car wide. (Parsons 1991).

The project also included a literature review which examined the AAR Specifications for the design of railcars. The findings were similar to those presented in Section 2.1.2. The review did state that the only change in the AAR Design Specifications between 1964 and 1984 was an increase in the design axial loads; no changes were found in the vertical loads. Also presented during the review were guidelines for selecting a railroad car suitable for bridge structures. These guidelines were very similar to those found in Suprenant's work (Parsons 1991).

2.2.2.2. Field Instrumentation & Load Testing

Four railway cars were instrumented and load tested as part of the study. Each of the four bridges tested, two made of flatcars and two made of boxcars, consisted of two railroad cars connected side-by-side. Each bridge was instrumented with between 30 and 54 strain gages, all of which were located on one of the two railroad cars. Strain gages were placed near midspan and at the one-quarter points of each bridge (Parsons 1991).

Empty and fully loaded test trucks were positioned on the instrumented railroad car to maximize the strain in key structural members. This typically consisted of placing the test truck at midspan, one-third points, or one-quarter points. Both static and dynamic tests were performed. Since the test trucks were positioned only over the railroad cars in which strain gages were installed, transverse load distribution between the two cars was not investigated (Parsons 1991).

Results of the testing on three of the bridges showed that maximum strains in the main girders were recorded at midspan when the test truck was centered transversely on the bridge. In the fourth bridge, maximum strains in the main girders were recorded in the tapered sections (i.e., closer to the ends of the bridge). Smaller strains were recorded at midspan because a large cover plate greatly increased the moment of inertia at this location (Parsons 1991).

2.2.2.3. Scale Model & Destructive Tests

A one-third scale model of a single boxcar was constructed and tested in a laboratory. Strain gages were placed on key members at the midspan, one-quarter and one-third points, and along the centerline of the model. The model was subjected to single axle, tandem, and point loading. These loads were applied both in the center of the car and along one of the edges. The same loads were also applied to the model with a second exterior girder bolted onto one of the existing exterior girders of the model. This was to simulate two railcars bolted together, a common connection observed in the field (Parsons 1991).

Results of the model testing showed the maximum loading case (maximum strains) was when the single axle was positioned along the outside edge of the car at midspan of the bridge. Another key observation was that when the load was centered transversely on the model, the exterior girders which were bolted together each carried roughly the same strain as the single exterior girder (Parsons 1991).

Destructive tests were also performed on the scale model to investigate the behavior of railcars with damaged members. The purpose of damaging the members was to simulate corrosion and cracking in both the exterior and main girders. The destructive tests consisted of loading the model with (a) cuts in an exterior girder, (b) portions of the bottom flange removed on an exterior girder, and (c) cuts in the main girder (Parsons 1991).

The results from the destructive tests revealed that the finite element model, discussed in the next section, could reasonably predict strains if only a flange was cut or removed. When large portions of the web were cut, the model over- or under- predicted strains by as much as 25 percent (Parsons 1991).

2.2.2.4. Load Rating Software

A load rating software program was developed based on a finite element model (FEM) of the railroad car. The FEM was developed to accommodate both RRFCs and boxcars. Required inputs for the program included inspection data such as number, spacing, and sectional dimensions of each structural member. These inputs were then used to develop a finite element model of the railroad car (Parsons 1991).

The analysis program applied a unit inventory or operating vehicle, depending on the type of load rating being conducted, to the FEM of the railcar. A load rating for each structural member was determined by computing the maximum factored resistance, subtracting the factored dead load moment, and dividing by the live load moment produced by the factored unit vehicle. This rating process is similar to the AASHTO Load Factor Rating (LFR) (AASHTO 2008). It should also be mentioned that the stiffness of the railcar deck was neglected in the FEM to be conservative (Parsons 1991).

The finite element analysis (FEA) program was calibrated using the one-third scale model. Strains in the scaled model and predicted strains in the FEA program were generally found to be within 10% of each other. Results from the destructive tests were compared to the FEM to see how well the model could predict strains in damaged members. When there was little damage, the FEA program provided generally good agreement with the testing; however, when there was a large amount of damage, it did not accurately predict strains.

The load rating software was compared to the results from the field instrumentation and testing. The software provided reasonably good agreement with results from the field tests for the RRFCs, but was not as accurate with the boxcars. It was determined that when using the load rating software, it should be used to rate each railroad car as though it was acting independently. Thus the connection between the two railcars could not be assumed to distribute load between railcars (Parsons 1991).

2.2.2.5. Arkansas State DOT Update of Load Rating Software

An informal phone interview was conducted with a representative of the Arkansas DOT regarding the RRFC load rating program developed by Parsons. The employee stated the program was not being used anymore because it was somewhat time-consuming to use and required some background knowledge of finite element modeling. RRFC bridges could be more easily rated by modeling it as a simple beam and using engineering judgment-based assumptions.

2.2.3. California Emergency Bridge System

In response to the 1994 Northridge earthquake, the California Department of Transportation (Caltrans) needed cheap and easy temporary solutions for the many bridges damaged in the earthquake. Although RRFCs were not used in response to the Northridge earthquake, Caltrans discovered they could be used as temporary bridges until permanent bridges could be built. These temporary and re-useable RRFC bridges allowed Caltrans to safely re-open many roads, thereby reducing the economic loss of keeping roads closed (Roberts 1995b).

2.2.3.1. Use of RRFCs as Temporary Bridges

W. H. Wattenburg of the Lawrence Livermore National Laboratory was the individual who suggested using retired RRFCs as temporary bridge structures to Caltrans (Wattenburg 1995). The concept of using RRFCs as bridges was not new in California; as of 1995, there were approximately 83 RRFC bridges on low volume roads in California (Bobb 1995). Wattenburg suggested creating bridge structures made of 10 RRFCs each, with a maximum width of 52.5 feet and a maximum length of 55 feet (Roberts 1995b). Temporary structures could then be placed back-to-back until a desired length was obtained (Wattenburg 1995). A diagram of one of these structures is shown in Figure 2.5.

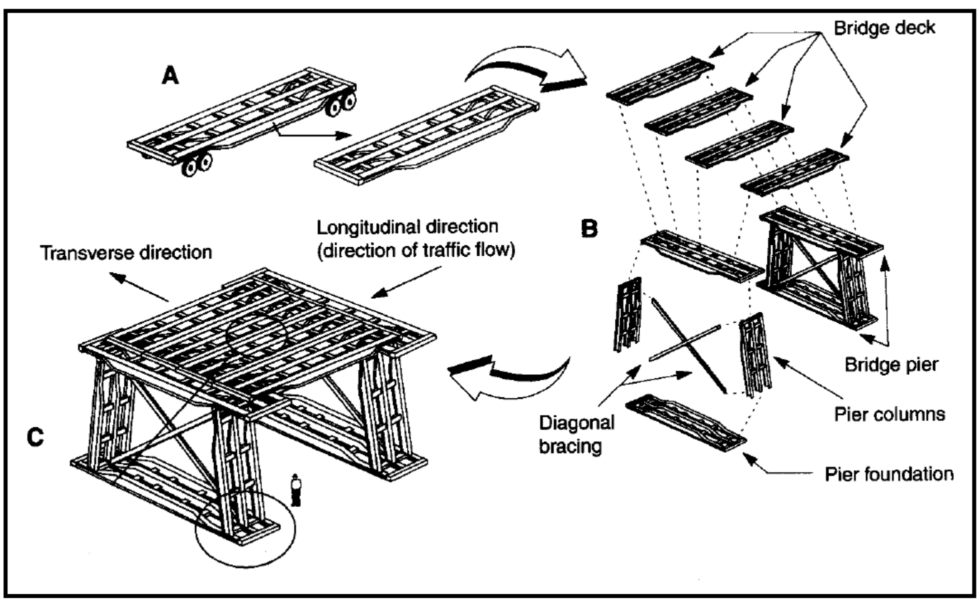

Figure 2.5: Temporary RRFC bridge structure (Wattenburg 1995)

As seen in Figure 2.5.A, the first step in creating one of these structures is to remove the wheel trucks from the flatcars. Figure 2.5.B shows that 10 RRFCs, each with a length of 55 feet, are required to create the temporary bridge structure. Two RRFCs are placed upside down to be used as the pier foundation. Two RRFCs are cut in half and are used as piers, such that the height of each pier column is one-half of a RRFC. Two RRFCs are then placed on top of the pier columns, spanning in the transverse direction. Diagonal bracing, which can be made of the main girders of miscellaneous RRFCs, can be added for stability. Four RRFCs are then placed longitudinally between the piers to form the bridge deck. Figure 2.5.C shows the completed structure (Wattenburg 1995).

2.2.3.2. Finite Element Analysis of Earthquake Loading

Wattenburg constructed a finite element model of the RRFC structure to perform earthquake analyses on it. The analysis was performed to ensure the temporary structures could withstand any aftershocks following the recent earthquake (Wattenburg 1995).

In addition to the dead weight of the structure, a load of approximately 35 metric tons (77 kips) was applied to the FEM to simulate the weight of vehicles and decking material. Ground motions from the 1992 Petrolia-Cape Mendocino earthquake were applied to the FEM. The results of the finite element model analysis showed the main structural members had enough strength to withstand the earthquake (Wattenburg 1995).

2.2.3.3. Prototype of RRFC Temporary Structure

Before putting a temporary RRFC structure into service, Caltrans built and tested a prototype. The prototype was built by a contractor in ten days with the help of Wattenburg. After construction was complete, the structure was loaded with over 100 metric tons (220 kips). The maximum midspan deflection was found to be 3.2 mm (1/8 inch). Roberts, another researcher involved in the California emergency bridge system, also stated RRFCs exhibit more than enough strength to support AASHTO live loads because they are designed to support loads of 45,000 kg (100 kips) (Roberts 1995a).

2.2.3.4. Field Application of RRFC Structures

The first time a RRFC temporary structure was used was in response to the collapse of a 122 ft. bridge on I-5 due to serious flooding. Caltrans deployed three of the RRFC temporary structures, which had been stored in anticipation of emergency situations. Some minor modifications were needed to successfully install the temporary structures. Since the bridge would be located in a river channel, RRFCs were not used for the substructure. Rather, H-piles were used as pier columns, with angle braces used to increase stability (Bobb 1995).

Even with the necessary modifications, the temporary bridge was in service within seven days of the original collapse. Using twelve RRFCs for the superstructure, the temporary bridge cost a total of $228,000. It was estimated that using the RRFC temporary structures saved Caltrans approximately $500,000 for this project. This estimate included the cost of detouring traffic during construction of the permanent bridges (Bobb 1995). Caltrans planned to continue to use these temporary structures for other disasters and had received inquiries from Texas, Louisiana, and several other states regarding their use (Perlman 1995).

2.2.4. Bridge Diagnostics Inc. Load Rating

Bridge Diagnostics Inc. (BDI) has been involved in two projects where field instrumentation was used in conjunction with finite element modeling to determine load ratings of RRFC bridges. As described in an article published in *Roads & Bridges* in 1995, BDI performed a load rating of a RRFC bridge in Wyoming (Bridge Diagnostics Inc. 1995). In 2002, BDI submitted a report to the Bureau of Reclamation describing the use of field instrumentation to determine the load rating of a four-span RRFC bridge, in which each span was its own RRFC (Bridge Diagnostics Inc. 2002).

2.2.4.1. Field Instrumentation & Testing

BDI performed field instrumentation and testing for a single span RRFC bridge in Wyoming as well as a four-span RRFC bridge in California. The Wyoming bridge consisted of two flatcars connected side-by-side (Bridge Diagnostics Inc. 1995). Three of the spans in the California bridge were made of a full length RRFC, while one of the spans was made from half of a flatcar (Bridge Diagnostics Inc. 2002).

For both bridges, strain transducers were mounted at select locations on the superstructure, mainly on the main and exterior girders at midspan. A test truck, loaded to approximately the posted limit, was then driven over each bridge at a crawl speed. Dynamic speed tests were also performed on the Wyoming bridge (Bridge Diagnostics Inc. 1995). From the stress data, it was generally seen the main girders carry the majority of the load. The exterior girders exhibited local behavior in addition to carrying global load (Bridge Diagnostics Inc. 2002).

2.2.4.2. Finite Element Analysis Load Rating

After the field testing, finite element models were constructed for both of the bridges. Modeling parameters, such as the load distribution through the deck and the stiffness of transverse members, were adjusted using an optimization routine so the calculated stress response more closely matched the measured stresses. Results from the FEM load rating programs showed structural members in both bridges were adequate for carrying traffic loads (Bridge Diagnostics Inc. 2002).

2.2.5. Iowa State University Research

Iowa State University (ISU) has conducted a number of studies on the use of retired RRFCs as bridges on low volume roads. Researchers from ISU have published numerous reports and theses on this topic since the late 1990's. The published research includes three reports sponsored by the Iowa Department of Transportation. In their research, ISU has investigated guidelines for selecting a RRFC to be used as a bridge, field and laboratory testing, and load rating procedures.

2.2.5.1. Use of RRFCs as Bridges

Iowa has numerous low volume road bridges in which retired RRFCs are used as the superstructure. These bridges can generally span anywhere from around 20 – 89 feet (Wipf et. al. 1999). Most of these structures were made of either two or three RRFCs side-by-side (Wipf et. al. 2003). A majority of them have a longitudinal connection between adjacent flatcars. These bridges are both single and multi-span RRFCs. The multi-span bridges are typically one RRFC longitudinally, with multiple supports along the length of the flatcar (Wipf et. al. 2007b). Using RRFCs for bridges on low-volume roads can be roughly half of the cost of a more conventional bridge and can be installed in a shorter amount of time (Wipf et. al. 2003).

2.2.5.2. Selecting a RRFC

Selecting an adequate RRFC is critical to the success of RRFC bridges. As a result, ISU developed the following criteria for selecting a RRFC:

- **Structural Element Sizes, Load Distributing Capabilities, and Support Locations:** The members in the RRFC must be sufficient enough to resist legal loads. RRFCs with larger, more closely spaced transverse beams also enable stress to be distributed better between the main girder and exterior girders. The exterior girders of flatcars must be strong enough to carry load, but also be capable of forming longitudinal connections with adjacent flatcars. Areas where the piers or abutments will be placed must also have sufficient bearing strength.

- **Member Straightness/Damage:** Although many flatcars are retired due to age or economics, some flatcars are retired due to damage. These RRFCs could have deformed, buckled, and/or yielded members which cannot carry or distribute load sufficiently. Visual inspections should be performed to determine whether or not damaged flatcars are suitable for use as a bridge.

- **Structural Element Configuration:** The elements of RRFCs are either connected with welds or rivets. It is not recommended that RRFCs connected with rivets be used as bridges because rivets can lose strength over time due to repeated loading and corrosion. RRFCs connected with welds should be inspected for fatigue cracks.

- **Uniform, Matching Cambers:** Since two or more RRFCs will be connected transversely to provide adequate driving width, these RRFCs should have similar cambers. This makes fit-up much easier, as well as creating a smoother driving surface. It was determined that flatcars with cambers of ±1 inch were sufficient during construction.

- **RRFC Availability:** Although somewhat obvious, retired RRFCs must be available if they are to be used as bridges. As implied within these criteria, the available RRFCs must also be structurally sufficient for supporting traffic loads. It is also beneficial if flatcars of the same type are available so construction and installation techniques do not need to be changed if multiple bridges are constructed (Wipf et. al. 2003).

2.2.5.3. Fatigue Considerations

Estimating the remaining fatigue life of a retired RRFC which has been in railroad service can be almost impossible. Fatigue damage depends on the stress range, number of cycles, and the type of detail. Although the type of detail can be seen on the flatcar itself, a county official will have a very difficult, if not impossible, time obtaining loading histories of a RRFC (Wipf et. al. 1999).

It is reasonable to believe the stress ranges and number of cycles a RRFC would experience during its railroad service life would both be much greater than those experienced during its life as a low volume road bridge. Flatcars are designed to support live loads of over 70 tons, which are greater than the majority of traffic crossing a typical RRFC bridge. ISU also contacted agencies who use RRFC bridges on low volume roads, and all of them verified that fatigue had not been an issue. Therefore, ISU concluded fatigue would not be a concern for RRFCs used on low-volume roads (Wipf et. al. 1999).

2.2.5.4. Field Instrumentation & Testing

As part of their numerous research projects involving RRFC bridges, ISU has performed field instrumentation and load testing on ten structures in Iowa. These tests will be briefly described in the following sections.

Tama County Bridge (TCB)

TCB spans 42 feet and is made up of two RRFCs side-by-side. The bridge deck consists of metal grating with timbers placed on top to create a driving surface. Both flatcars used in this bridge contain two main girders and two exterior girders, all of which are made up of built-up members. Several members, both longitudinal and transverse, had large out-of-plane deformations which were there when the bridge was placed into service. The support locations on each RRFC were also different at each abutment. Before testing began, there was no connection between the two flatcars (Wipf et. al. 1999).

Instrumentation consisted of roughly twenty strain gages installed near midspan of the bridge, and displacement transducers placed at the one-quarter and midspan points. A single axle, empty truck (17.1 kips) and a tandem axle, fully loaded truck (52.1 kips) were driven across the bridge in each lane and down the center of the bridge. After the initial load test, angles were used to connect the two RRFCs. The load tests were then repeated to examine the effect of the connection between the two flatcars (Wipf et. al. 1999).

Based on the stress and deflection data, the RRFCs provided adequate strength to carry Iowa legal loads. The maximum stress in the longitudinal members was 4.1 ksi and the maximum deflection was 0.32 inches, as compared with the AASHTO live load limit of 0.63 inches. There was no major difference between the tests with or without the added longitudinal connection between the two flatcars (Wipf et. al. 1999).

Buchanan County Bridge (BCB) and Winnebago County Bridge (WCB)

BCB and WCB were both constructed as demonstration bridges to be tested by ISU. BCB used 56 feet long RRFCs to span 52 feet, while WCB used 89 feet long RRFCs for two end spans of 10 feet and a main span of 66 feet. Both bridges are made up of three RRFCs in the transverse direction. Similar longitudinal connections between the flatcars were also used for both bridges; the exterior girders of adjacent flatcars were connected by concrete beams with longitudinal reinforcement. Threaded rods, cast into the concrete, were also used to connect adjacent exterior girders (Wipf et. al. 2003).

Strain gages were installed near midspan, at the one-quarter points, and near the ends of the bridges on longitudinal and transverse members. Deflection transducers were placed near midspan of the bridges. Load tests were performed on both bridges before and after the longitudinal connections were in place. Tandem axle dump trucks weighing 51-52.5 kips were used in the load testing. Load tests were performed for both a single truck and two trucks on the bridge. Multiple transverse truck locations were examined (Doornink et. al. 2003a).

Stress and deflection data showed both bridges were adequate for supporting Iowa traffic loads. The maximum stress in longitudinal girders in BCB and WCB were approximately 12.7 ksi and 16.7 ksi, respectively. Deflections were also well below the AASHTO requirements. The concrete beam longitudinal connections were shown to be effective in transferring live load between flatcars (Doornink et. al. 2003a).

Buchanan County Bridges 2 & 3 (BCB2 & BCB3), Winnebago County Bridge 2 (WCB2), and Delaware County Bridge (DCB)

Each of these four bridges had single spans with similar span lengths. BCB2 had a span length of 53 feet, while BCB3, WCB2, and DCB all had span lengths of 66 feet. Bridges BCB2 and DCB consisted of two side-by-side RRFCs, while BCB3 and WCB2 were constructed of 3 transverse RRFCs (Wipf et. al. 2007a). The longitudinal connections of BCB2 and WCB2 were made of reinforced concrete beams, similar to those used in BCB and WCB (Palmer 2004). The flatcars of DCB were connected by welding a steel plate to adjacent cars. In BCB3, the exterior girders of adjacent flatcars were bolted together (Wipf et. al. 2007a).

The instrumentation and load tests for these four bridges were similar to those used for the previously discussed bridges. Stress data showed that each of the flatcars tested were adequate for supporting Iowa traffic loads, if the gravel thickness for DCB was slightly reduced. Each of the three different longitudinal connections was deemed sufficient to transfer live load from one flatcar to another, as long as the bolts of the connection used in BCB3 were properly designed. Stress data also showed that the interior girders of RRFCs carry the majority of dead and live load due to its large relative moment of inertia (Wipf et. al. 2007a).

<u>Buchanan County Bridges 4 & 5 (BCB4 & BCB5) and Winnebago County Bridge 3 (WCB3)</u>

These three bridges are all multi-span structures. BCB4 is made of two side-by-side RRFCs and has two roughly equal spans of 40 feet each. BCB5 is also constructed with two RRFCs and has spans of approximately 43 and 44 feet. WCB3 has three RRFCs side-by-side and has three spans: two at 11.5 feet and one at 66 feet. BCB4 and WCB3 have reinforced concrete beam longitudinal connections, while BCB5 has a bolted exterior girder connection (Massa 2007).

The instrumentation and load tests for these three bridges were similar to those used for previously discussed bridges. Strain transducers were generally placed at the ends, midspan, and one-quarter points of each flatcar. String potentiometers were placed near midspan and the one-quarter points. Stresses in each of the bridges were shown to be acceptable for HS-20 loading. ISU determined the critical section to be analyzed for flexure in multi-span RRFC bridges is generally at the shallow end of the tapered sections of RRFCs (Wipf et. al. 2007b).

2.2.5.5. Laboratory Testing of Longitudinal Connection

ISU constructed and performed laboratory tests on one type of longitudinal connection: a reinforced concrete beam with transverse threaded rods placed between the adjacent exterior girders. This connection was meant to simulate the exterior girders of a 56 feet long RRFC. A RRFC of 56 feet typically has larger exterior girders than the 89 feet long RRFCs. Figure 2.6 shows a diagram of this connection.

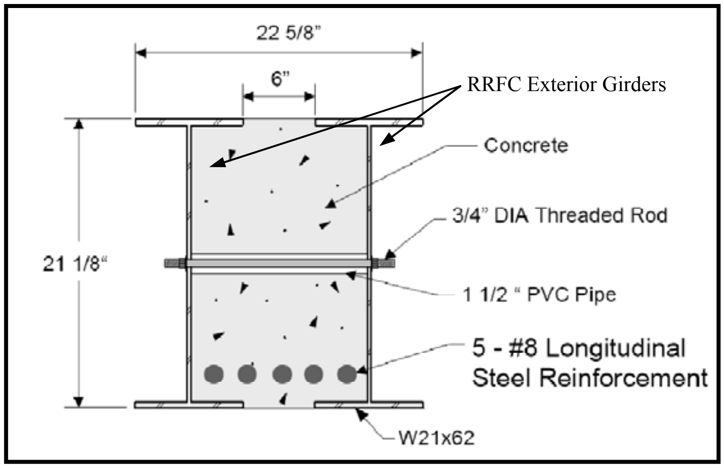

Figure 2.6: Laboratory testing of concrete beam longitudinal connection (Wipf et. al. 2003)

As Figure 2.6 shows, two W21x62 beams, sixteen feet long, were selected to simulate the adjacent exterior girders. The beams had a clear spacing between the edges of the flanges of six inches. Threaded rods were placed between the two beams to provide confinement for the concrete. The threaded rods were placed in poly vinyl chloride (PVC) pipes so the rods could be removed to vary the amount of confinement. Five #8 reinforcement bars were placed near the bottom of the void between the beams and the void was then filled with concrete (Wipf et. al. 2003).

Strain gages and 45° strain rosettes were placed along the length of the beams and along each of the webs and flanges. Strain potentiometer deflection transducers (SPDTs) were also used to measure displacements and inclinometers were used to measure rotations. The connection was tested in torsion and flexure under service loads, and was eventually tested to failure in torsion (Wipf et. al. 2003).

It was determined that flexural stresses in the longitudinal connection can be calculated by conventional composite analysis. The steel warping strains in the torsion tests were more difficult to predict because of the composite interaction. However, the stresses in the steel under extreme torsion, more than a typical RRFC bridge would see, were all much smaller than the yield strength of the steel. The results of the laboratory tests showed this type of connection is suitable for traffic loads on RRFC bridges of 56 feet (Doornink et. al. 2003b).

2.2.5.6. Analytical Modeling through Grillage Analysis

ISU used grillage analysis to model the RRFC bridges which were load tested. Grillage analysis was chosen over finite element modeling because of its ease of model construction, relatively fast results, and economics. The grillage models of RRFC bridges were generally constructed under the following assumptions:

- Using the average spacing between adjacent members rather than the exact spacing will have negligible effects.
- Steel decking and small transverse members do not contribute to the longitudinal stiffness of the bridge.
- Connections between primary longitudinal and transverse members were assumed to be rigid.
- Asphalt and gravel driving surfaces provide negligible load distribution.
- Longitudinal connections between adjacent exterior girders create compatibility constraints (Wipf et. al. 2003).

In general there was good agreement between the field testing results and the grillage model results for each of the RRFC bridges. The grillage models confirmed the load tests results that RRFC bridges are suitable for carrying Iowa traffic loads. ISU also concluded grillage modeling is an acceptable analysis for predicting RRFC bridge behavior (Wipf et. al. 2003).

2.2.5.7. Load Rating Methods

ISU developed two general methods for determining the load rating of RRFC bridges. Each of these methods will be discussed in the following sections.

Load Rating Method #1

Load rating method #1 was based on the load and resistance factor rating (LRFR) method and field testing data. This general approach to rating was proposed in a report by Streeter (1998) and can be found in *The Manual for Bridge Evaluation* (AASHTO 2008). The method is based on the following equations (Wipf et. al. 1999):

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

Equation 1: LRFR load rating factor

where:

RF_C = rating factor from theoretical model

Φ = resistance factor

R_n = nominal resistance, calculated from member properties

γ_d = dead load factor, equal to 1.2 and increased by 20% when overlay is present

D = nominal dead load, determined from applying self weight to grillage model

γ_L = live load factor, ranges from 1.3 – 1.8 depending on ADT and overload restrictions

L = nominal live load, determined from applying tandem axle truck to grillage model

I = live load impact factor, ranges from 0.1 – 0.3 depending on wearing surface

After the theoretical rating factor (RF_c) has been calculated, it can then be modified by the following equation to determine the rating factor to be used when rating the bridge:

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

Equation 2: LRFR theoretical load rating factor modification

where:

RF_T = rating factor after results from load tests have been applied

K_a = factor obtained from comparison of theoretical model results and load test results

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

The K_a factor is determined by the following equation:

$$K_a = \frac{\varepsilon_c}{\varepsilon_t} - 1$$

Equation 3: K_a factor

where:

ε_c = theoretical strain from grillage model

ε_t = experimental strain during load tests

The K_b factor is determined by the following equation:

$$K_b = K_{b1}\, K_{b2}\, K_{b3}$$

Equation 4: K_b factor

where:

K_{b1} = factor accounting for behavior of bridge beyond the test load level, ranging 0 – 1

K_{b2} = factor accounting for type and interval of inspections, ranging 0 – 1

K_{b3} = factor accounting for sudden failure of bridge due to fatigue or fracture of critical members and absence of redundant members, ranging 0 – 1

The Manual for Bridge Evaluation (AASHTO 2008) provides further guidance for determining the values of K_{b1}, K_{b2}, and K_{b3}.

After all variables have been determined and/or calculated, the load rating factor used to determine the load rating of the RRFC bridge can be obtained from using Equation 2. This method was used to show that TCB had sufficient strength to carry Iowa legal loads (Wipf et. al. 1999).

It should be noted this method does not take into consideration any damage on the bridge being rated. Also, this method also only rates primary members; secondary members and abutments would also need to be considered for a complete bridge load rating (Wipf et. al. 1999).

Load Rating Method #2

An allowable stress rating (ASR) was developed and used by ISU in many of their later reports (Wipf et. al. 2007b). The general allowable stress load rating equation, as given in AASHTO (2008) is as follows:

$$RF = \frac{C - A_1\, D}{A_2\, L\, (1 + I)}$$

Equation 5: ASR load rating factor

where:

RF = rating factor

C = allowable stress capacity of the member

A_1 = factor for dead loads = 1.0 for allowable stress

D = dead load effect on the member

A_2 = factor for live loads = 1.0 for allowable stress

L = live load effect on member

I = impact factor used with live load

In Equation 5, the capacity is determined through section and material properties, the dead and live load factors are known, and impact can be determined using AASHTO (2008). The only unknown is the live load effect. The general equations and rules of thumb in AASHTO (2008) cannot be used because RRFCs do not have uniform girders at equal spacing. Therefore, ISU developed the following equation for determining the live load moment effect on the main girder or an exterior girder (Wipf et. al. 2007b):

$$M_{LL} = \lambda\, \psi\, \omega\, M_{SD}$$

Equation 6: ISU live load moment effect on RRFC bridges

where:

M_{LL} = actual maximum live load moment in girder being evaluated

λ = moment fraction value = 2/3 at midspan or 3/5 at tapered section

ψ = design factor to account for live load distribution, longitudinal connection, and load position

ω = inertia ratio of girder being evaluated to total inertia in RRFC

M_{SD} = maximum live load moment at point of interest in RRFC bridge from analysis with vehicle center of gravity at midspan

Equation 6 is used to determine the live load effect on a single member within a RRFC (i.e., either a main girder or exterior girder). The moment fraction (λ) was determined based on the field data obtained through testing the ten RRFC bridges previously discussed. A value of 2/3 is used when determining the live load moment at midspan and a value of 3/5 is used at the tapered section of the flatcar (Wipf et. al. 2007b).

The inertia ratio (ω) represents the load distribution within a RRFC. The inertia ratio can be calculated using the following equation:

$$\omega = \frac{I_D}{I_{int} + 2\, I_{ext}}$$

Equation 7: ISU inertia ratio

where:

I_D = strong axis moment of inertia of girder being evaluated

I_{int} = strong axis moment of inertia of interior girder

I_{ext} = strong axis moment of inertia of exterior girder

The design factor (ψ) was calibrated such to match the field test stresses with the theoretical stresses. ISU presented two different methods to determine a design factor. In each of the two methods, the factors were determined conservatively. (Wipf et. al. 2007b). The first method involved a series of equations which were calibrated depending on the girder and longitudinal connection type. The series of equations is shown below (Wipf et. al 2003):

For interior girders in RRFC bridges with BCB-type connections:

$$\psi = 0.4\,\omega^2 - 0.7\,\omega + 1.1$$

For exterior girders in RRFC bridges with BCB-type connections:

$$\psi = -13.7\,\omega^3 + 10.4\,\omega^2 - 2.6\,\omega + 1.2$$

For interior girders in RRFC bridges with WCB-type connections:

$$\psi = -1.7\,\omega^3 + 4.8\,\omega^2 - 4.1\,\omega + 1.9$$

For exterior girders in RRFC bridges with BCB-type connections:

$$\psi = 1800\,\omega^3 - 270\,\omega^2 + 20.1\,\omega + 0.4$$

Equation 8: ISU design factor equations

The second method ISU used to determine the design factors was done in tabular form. This table was used in later reports and is shown below (Wipf et. al. 2007b):

Table 2.2: ISU table for determining design factor (Wipf et. al. 2007b)

	RRFC Connection			
	Type 1	**Type 2**	**Type 3**	**Type 4**
Brief Description of Connection	Concrete Beam w/threaded rods	Concrete beam w/timber planks	Bolted through exterior flange	Welded plates
RRFC Bridge where connection was used	BCB1, BCB2, BCB4	WCB1, WCB2, WCB3	BCB3, BCB5	DCB
Referenced Report / **Type of RRFC**	TR-444 [4] TR-498 Vol 1&2 [11]	TR-444 [4] TR-498 Vol 1&2 [11]	TR-498 Vol 1 [11]	TR-498 Vol 1 [11]
<u>56-ft RRFC Single Span</u>				
Midspan Exterior Girder	0.8 - 1.1 *	----	----	----
Midspan Interior Girder	0.8	----	----	----
<u>89-ft RRFC Single Span</u>				
Midspan Exterior Girder	----	0.6	0.4	0.4
Midspan Interior Girder	----	1.2	1.2	0.9
<u>89-ft RRFC Multiple Span</u>				
Midspan Exterior Girder	0.5	0.8	0.8	----
Midspan Interior Girder	1.5	1.0	2.0	----
Pier Exterior Girder	0.9	----	0.8	----
Pier Interior Girder	0.8	----	0.8	----
Taper Exterior Girder	----	----	----	----
Taper Interior Girder	0.9	----	1.0	----

As shown in Table 2.2, the design factor depends on the type of girder, number of spans, length of RRFC, and the longitudinal connection between RRFCs (Wipf et. al. 2007b).

When multiplied together, the moment fraction, inertia ratio, and design factor represent a distribution factor. As shown in Equation 6, once these factors are determined, they can be multiplied by the moment due to a load rating vehicle on a simply supported span. This result yields the live load moment to be used for determining the allowable stress rating factor in Equation 5 (Wipf et. al. 2007b).

The second method for load rating RRFC bridges was used by ISU on single and multi-span RRFC bridge structures, as well as bridges with widths of two or three

RRFCs. This load rating method produced conservative results and showed reasonably good agreement with the load test results (Wipf et. al. 2007b).

2.3. Summary

The primary intent of this literature review was to provide a basic understanding of the design of RRFCs and their use as bridges on low-volume roads. As presented, there is little standard guidance when it comes to load rating, inspecting, and acquiring RRFCs as bridges. These tasks are made increasingly difficult, specifically in this context, because RRFCs are being used as bridges, an application much different than their intended use in the railroad industry.

An abundance of valuable information was learned during the literature review. However, there appears to be two areas which require additional guidance to ensure the safe use of RRFCs as bridges on low volume roads. The first area is in regard to the selection and acquisition of RRFCs for use as bridges. During the literature review a few basic "rules of thumb" were found for selecting and acquiring a RRFC to be used as a bridge. These were presented in a couple of the prior studies performed on RRFCs. Through this research these existing guidelines will be built upon to develop clear and concise implementable guidelines to be used when selecting and acquiring a RRFC for a bridge application.

The second area found to need additional guidance was in regard to load rating and these structures. During the literature review several methods for load rating RRFC bridges developed during past studies were presented. These included the use of finite element modeling and beam line analysis in conjunction with a distribution factor. One downfall of many of these methods presented was the complexity and need for finite element or grillage models. Thus, the current research will also attempt to develop simple, yet accurate, load rating method to be used for RRFC bridges. The proposed load rating method will use engineering principles and be aimed similar to the current AASHTO Specifications, which will provide engineers familiarity with the methods in hopes of making them more comfortable load rating these structures. By further developing these two areas, RRFC bridges will continue to be a safe, economical solution for bridges on low volume roads.

CHAPTER 3. FIELD VISITS

To gain a better understanding of the RRFC bridge inventory within Indiana, field visits were conducted between November – December 2009 and March – April 2010. Visiting all 133 RRFC bridges found in Indiana was not a feasible task due to time and budget constraints. Therefore, parameters were selected to determine which bridges would be subject to field visits. The parameters were identified based on a review of a database containing all of the RRFC bridges in Indiana. The following sections describe the Indiana inventory of RRFC bridges and how a representative sample of bridges was selected for field visits.

3.1. Indiana RRFC Bridge Inventory

A database of the RRFC bridges in Indiana was provided by the Indiana Local Technical Assistance Program (LTAP). The database contains 124 RRFC bridges and includes information such as the location of the bridge, year it was built, geometric features, inspection data, maintenance data, etc. In addition to the database there are nine newer RRFC bridges located in Clay County, Indiana. These newer bridges were constructed since 2007. Using the database and information regarding these nine newer bridges, a map of the 133 RRFC bridges in Indiana was developed and is shown in Figure 3.1.

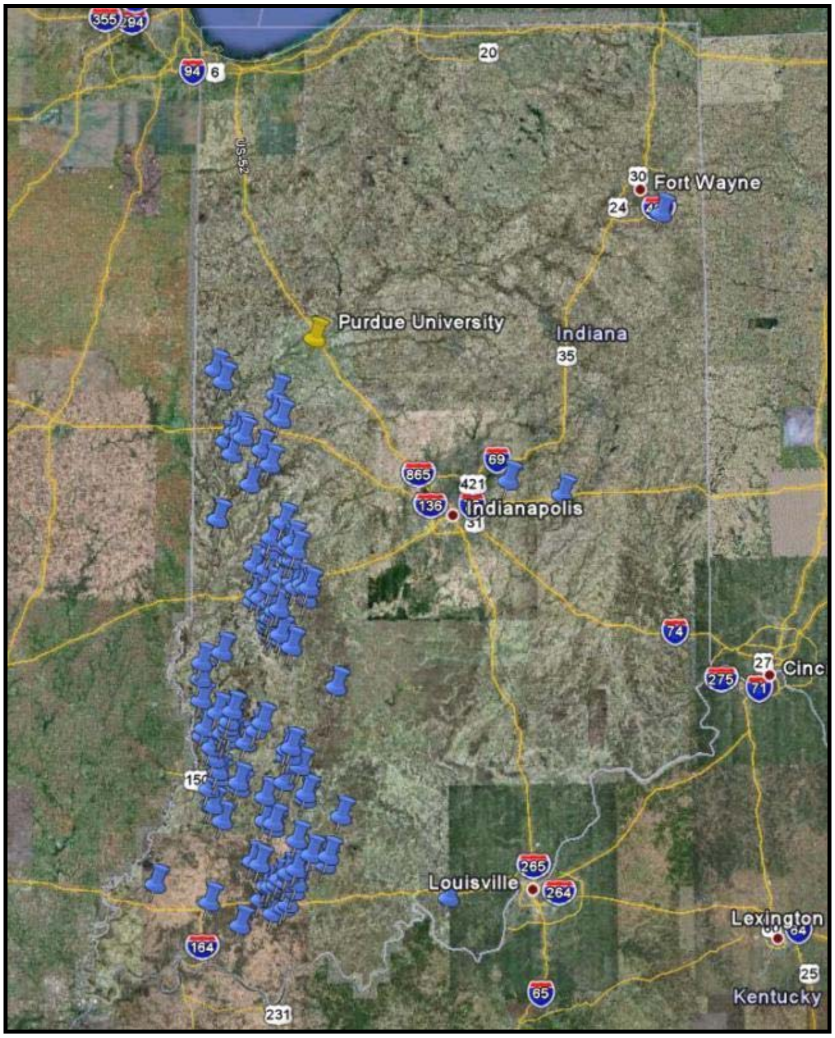

Figure 3.1: Map of RRFC bridges in Indiana (Google Earth)

As seen in Figure 3.1, of the 133 RRFC bridges in Indiana, the majority are located in the southwestern portion of the state. Using this map and other information from the database, several key parameters were identified to aid in selecting a representative sample of RRFC bridges to visit. These parameters consisted of structure length, bridge width, deck type, number of spans, and county of residence. Each parameter is discussed in the following sections. It should be noted that only the 124 RRFC bridges included in the database were used when evaluating these parameters. This is because some of the information about the nine newer bridges was still unknown at the time of the selection process.

3.1.1. Bridge Length

The bridge length provides an indication of what type of RRFC was used for the bridge. As stated in Section 2.1.1. the majority of flatcars come in lengths of either 56 or 89 feet. Although all RRFCs can be constructed differently, it was decided that flatcars of similar lengths (either 56 or 89 feet) would probably have similar design and behavior characteristics as opposed to flatcars of variable lengths. It was assumed that bridges with lengths of 56 feet or less would most likely be constructed of 56 foot long RRFCs. Similarly, bridges with lengths greater than 56 feet would most likely be constructed of 89 foot long RRFCs. Table 3.1 shows the percentage of RRFC bridges less than and greater than 56 feet.

Table 3.1: Indiana inventory RRFC bridge length

Structure Length	# of Bridges	Percentage
≤ 56 feet	45	36%
> 56 feet	79	64%
Total	**124**	**100%**

Note: Percentages rounded to nearest whole number.

3.1.2. Bridge Width

The width of the bridge was considered to estimate how many RRFCs were placed side-by-side when constructing each bridge. As discussed in the Iowa State University research, many counties use either two or three flatcars placed in the transverse direction to provide an adequate driving width (Wipf, 1999). It was estimated bridges with a width smaller than 24 feet were most likely made of two flatcars. However, bridges wider than 24 feet could potentially to be made of two or three flatcars depending on the connection between the cars.

The width of 24 feet was selected based on using three RRFCs, each with a width of 8 feet, placed side-by-side for a bridge structure. The individual flatcar minimum width of 8 feet was chosen based on findings from the ISU studies. Table 3.2 shows the number of RRFC bridges having widths less than and greater than 24 feet.

Table 3.2: Indiana inventory RRFC bridge width

Bridge Width	# of Bridges	Percentage
≤ 24 feet	112	90%
> 24 feet	12	10%
Total	**124**	**100%**

Note: Percentages rounded to nearest whole number.

3.1.3. Deck Type

The type of deck on any bridge has an effect on how load is distributed. This is no different for RRFC bridges. Stiffer decks, such as those constructed of concrete, typically distribute load better than more flexible decks, such as the steel plate decks found on some RRFC bridges. The number of bridges and their corresponding deck types are shown in Table 3.3. It is noted that all of the timber decks were found in Fountain County.

Table 3.3: Indiana inventory RRFC bridge deck type

Deck Type	# of Bridges	Percentage
Steel Plate	77	62%
Concrete Cast-in-Place	34	27%
Timber	8	7%
Corrugated Steel	4	3%
Open Grating	1	1%
Total	**124**	**100%**

Note: Percentages rounded to nearest whole number.

3.1.4. Number of Spans

The number of spans directly impacts the load rating of a bridge. This is especially true if a distribution factor is being used for the rating rather than some form of modeling. The number of spans and corresponding number of RRFC bridges for each number of spans is presented in Table 3.4.

Table 3.4: Indiana inventory RRFC bridge number of spans

# of Spans	# of Bridges	Percentage
1	113	91%
2	2	2%
3	8	6%
4	1	1%
Total	**124**	**100%**

Note: Percentages rounded to nearest whole number.

3.1.5. County

One of the parameters selected was the county in which a given RRFC bridge was located. This parameter was selected based on two factors. The first factor was the construction practices of each county. It was understood that each county has their own construction practices when installing RRFC bridges. These construction practices might or might not have an effect on how the bridge performs while in service.

The second factor involved the types of RRFCs being used. It was also proposed that different counties might have received retired RRFCs from different sources. This could lead to many different types of RRFCs being used as bridges within Indiana. The number of RRFC bridges in each county can be seen in Table 3.5. Additionally, the percentage of the entire inventory in the state is also listed. Any county not listed in the table did not have any RRFC bridges listed in the database.

Table 3.5: Indiana inventory bridge county table

County	# of Bridges	Percentage
Adams	1	1%
Clay	30	24%
Daviess	10	8%
Dubois	11	9%
Fountain	9	7%
Greene	2	2%
Hancock	1	1%
Harrison	1	1%
Knox	21	17%
Parke	4	3%
Pike	8	6%
Posey	1	1%
Rush	1	1%
Spencer	1	1%
Sullivan	5	4%
Vermillion	2	2%
Warren	2	2%
Warrick	14	11%
Total	**124**	**100%**

Note: Percentages rounded to nearest whole number.

3.2. Selection of Representative Sample

Based on the key parameters identified (bridge length, bridge width, deck type, number of spans, and county), a representative sample of RRFC bridges to visit was selected. A total number of 25 bridges were selected. The 25 bridges represent roughly 20% of the Indiana RRFC bridge inventory. It was decided that 20% of the total sample was sufficient to draw some conclusions about the Indiana inventory. Additionally, there was hope that a handful of these 25 bridges would be candidates for field instrumentation, which will be discussed later in the document.

3.2.1. Bridges Selected for Representative Sample

The bridges which were selected as the representative sample for field visits are shown in Table 3.6. When selecting the representative 25 bridges, percentages of the bridge length, bridge width, deck type, and number of spans similar to those of the Indiana inventory were used. Also included in the table is the maximum span length for each bridge.

In addition to the 25 RRFC bridges identified as a representative sample, another bridge, located in Harrison County, was selected for a visit. The reason for visiting the sole RRFC bridge in Harrison County was due to its unique nature. Harrison County Bridge # 84 (HA-84) is a four span RRFC bridge with a total length of 240 feet. It was unknown if the bridge consisted of three or four flatcars along its length. Table 3.6 also contains additional information on this bridge.

Table 3.6: RRFC bridges selected as representative sample

Name of County	County Designation	County Bridge #	Bridge Length (ft)	Bridge Width (ft)	Deck Type[2]	# of Spans	Length of Max Span (ft)
Clay	CL	33	34	18.1	SP	1	28
Clay	CL	53	45.8	18.1	SP	1	33.9
Clay	CL	163[1]	79.2	19.7	SP	1	59
Clay	CL	191	41.5	18.2	SP	1	28.2
Clay	CL	205	29	18	SP	1	25.8
Clay	CL	214	86.5	18.1	SP	1	69
Clay	CL	270	82.5	20	SP	3	42
Daviess	DA	153	90	18.3	SP	3	63
Daviess	DA	242	86	17.5	SP	1	68.3
Dubois	DU	145	54	25.1	CCIP	1	50
Dubois	DU	146	60	20.9	CCIP	1	48
Fountain	FO	20	85	21	T	1	82
Fountain	FO	51	56	21.5	T	1	55
Greene	GR	142	51	19.6	SP	1	31
Knox	KN	27	56	21.3	CCIP	1	50
Knox	KN	28	62	20.8	CCIP	1	56.8
Knox	KN	162	61	21	CCIP	1	55
Knox	KN	182	86	18	SP	1	67.5
Parke	PA	194	78.3	18.1	CS	1	55.3
Pike	PI	62	85	19.4	SP	1	60
Spencer	SP	281	75	20.5	SP	1	72
Sullivan	SU	58	62	22.2	CCIP	1	53
Warrick	WK	34	71.8	21.6	SP	2	40
Warrick	WK	165	56	24.6	CCIP	1	49
Warrick	WK	200	85	27.5	SP	1	74
Additional Bridge Selected For Field Visit							
Harrison	HA	84	240	20	OG	4	60

[1]Bridge CL-163 was not found during the field visits, therefore a similar bridge located in the near proximity was visited.
[2]For Deck Type: SP = steel plate, CCIP = concrete cast-in-place, T = timber, CS = corrugated steel, and OG = open grating.

In addition to being selected based on the parameters listed in the previous sections, DU-146 was selected because it was closed to traffic. Based on the low condition rating of the substructure, it was estimated the bridge was closed due to problems with one or both of the abutments. Nevertheless, DU-146 was selected for a field visit to see if issues with the RRFC were the cause of the closure. (As it turned out, the bridge did appear to be closed due to deterioration of both abutments. There was no noticeable major damage to the RRFCs.)

3.3. Field Visits Findings

Field visits to the 26 bridges listed in Table 3.6 allowed a better overall understanding of RRFC bridges in Indiana, particularly in terms of their geometry and construction. The following sections will describe some of the findings from the field visits.

3.3.1. General Overview

Although, there are many types of RRFCs, most used as bridges in Indiana are constructed with one main girder and two exterior girders on either side of the main girder. A system of stringers is located between the main girder and each exterior girder. A typical RRFC bridge in Indiana can be seen in Figure 3.2. An elevation view of the bridge is shown in Figure 3.2.A. The longitudinal members of a flatcar can be seen in Figure 3.2.B, while the transverse members are shown in Figure 3.2.C (the main girder and exterior girder are labeled for reference in this figure).

Figure 3.2: Typical RRFC, elevation view (A), longitudinal members from underneath (B), & transverse members from side (C)

3.3.2. Main Girders

The main girder of a RRFC runs longitudinally down the middle of the flatcar. This member is typically a box girder consisting of a bottom flange, top flange, and two webs spaced at approximately 1–1.5 feet. These girders typically have depths of approximately 2–2.5 feet. A typical box girder is shown in Figure 3.3.

Figure 3.3: Typical box girder

Although most RRFCs have one main box girder, this is not always the case. Some RRFCs have two large I-shaped girders that run down the middle of the car, as seen in Figure 3.4. These can be similar to the box girders, expect with a separate bottom flange for each web (Figure 3.4.A). Conversely, the two large girders can also be spaced at a much larger distance (Figure 3.4.B). When there is a large spacing, the RRFC will not typically have any stringers, but will have large transverse floor beams between the main girders.

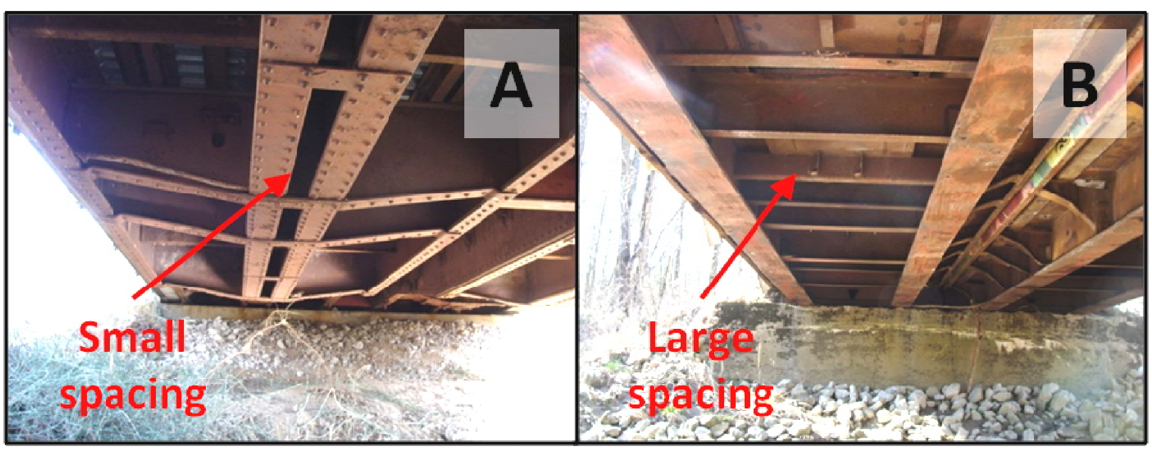

Figure 3.4: Two large main girders with small spacing (A) & large spacing (B)

3.3.3. Exterior Girders

The exterior girders are located on the outside of a RRFC. Most of the RRFC bridges in Indiana had exterior girders made of channels (C-shapes). It was found that these channels are normally much shallower than the main girder. A typical channel exterior girder is shown in Figure 3.5.

Figure 3.5: Typical channel exterior girder

Other variations of the exterior girders on RRFC bridges include rolled steel shapes (Figure 3.6.A) and larger I-beam sections (Figure 3.6.B). The large beam exterior girders are typically as deep as the main girder; however, there were cases where these large exterior girders were larger than the main girder.

Figure 3.6: Rolled steel shape (A) & large beam (B) exterior girders

3.3.4. Stringers

The stringers are secondary elements located between the main girder and the exterior girders. Stringers on RRFCs come in many shapes and sizes. In general, the stringers are not as deep as either the main girder or the exterior girders. A typical set of stringers consists of three inverted T-shapes spaced at approximately 6-8 inches. Generally, there are three stringers on each side of the main girder. A typical set of three stringers is shown in Figure 3.7.

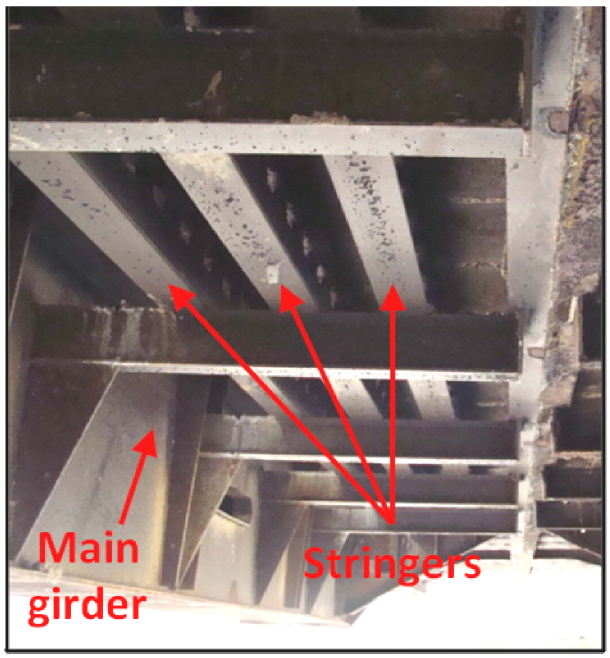

Figure 3.7: Typical inverted T stringers

Several additional variations of stringers were also found. These included other structural shapes such as small I-beams, Z-shapes, and a rolled steel plate in place of the stringers. The rolled steel plate is shown in Figure 3.8.

Figure 3.8: Rolled steel plate stringers

3.3.5. Transverse Members

Similar to the other structural elements of RRFCs, the transverse members varied greatly from flatcar to flatcar. Most RRFCs have both large and small transverse members. Typically, the larger transverse members are located at midspan of the flatcar and at the locations where the main girder begins to taper to the end of the car. However, these larger transverse members can also be spaced along the entire length of the flatcar. Typically a large transverse member extends from the bottom of the main girder and supports the stringers. A typical large transverse member is shown in Figure 3.9.

Figure 3.9: Typical large transverse member

The smaller transverse members are typically steel shapes such as I-beams. For some flatcars, the small transverse members are accompanied with a stiffener on the main girder web. Other times, there is a knee-brace constructed from an angle to provide support at the end of the transverse member. These two typical small transverse members are shown in Figure 3.10.

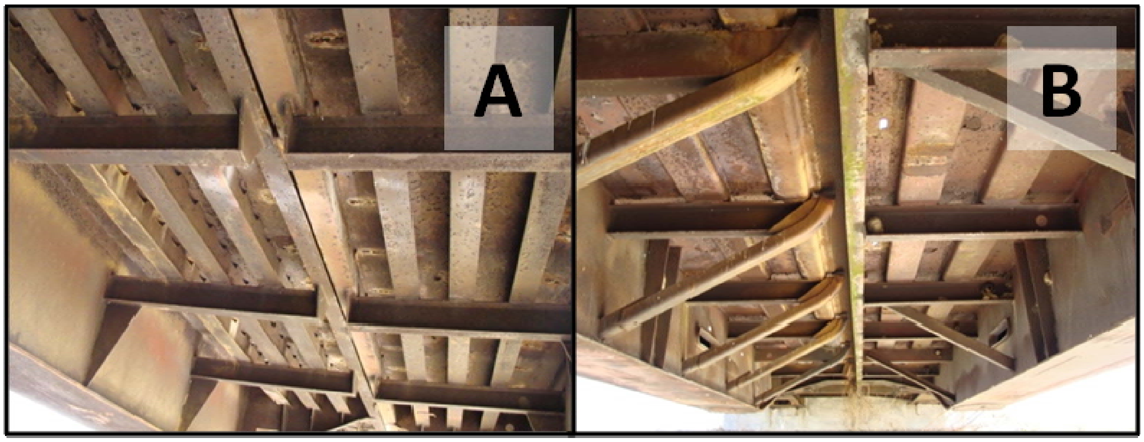

Figure 3.10: Typical small transverse members with stiffeners (A) & knee braces (B)

3.3.6. Connections

In general RRFC members are connected with welds. There were a few flatcars, however, connected with rivets. These flatcars had built-up members as main girders and exterior girders. No RRFCs were found containing bolted connections.

3.3.7. Deck Type/Surface

Although the deck type was given in the database of RRFC bridges, the actual driving surface varied from the deck type on a few occasions. The main difference was that an asphalt wearing surface had been applied to some of the original decks. These decks were most likely steel plating as seen from the database. Table 3.7 shows the types of deck surfaces seen in the representative sample.

Table 3.7: RRFC bridge deck surface

Deck Type	# of Bridges	Percentage
Asphalt	10	40%
Concrete	7	28%
Steel	6	24%
Timber	2	8%
Total	25	100%

3.3.8. Number of RRFCs

Of the 25 RRFC bridges visited in Indiana, only one of the bridges was made up of three RRFCs. The remaining 24 bridges all consisted of having two RRFCs placed side-by-side to obtain an acceptable driving width. Figure 3.11 shows WK-200, which was constructed with three RRFCs.

Figure 3.11: Bridge constructed of 3 RRFCs side-by-side

3.3.9. Longitudinal Connection Type

As previously discussed, RRFCs are oftentimes placed side-by-side to provide an adequate driving width. The adjacent exterior girders of the two (or more) flatcars are then usually connected to better distribute traffic loads from one flatcar to the other. As seen from the field visits, RRFC bridges in Indiana have many different types of longitudinal connections. The specific type of longitudinal connection used at any given bridge appeared to be generally consistent within a particular county. Different types of connections were grouped into broad categories and are presented in Table 3.8. As shown in the table, the three general groups of longitudinal connections used on Indiana RRFC bridges are threaded rods, steel connection, or not connected.

Table 3.8: RRFC bridge longitudinal connection types

Connection Type	# of Bridges	Percentage
Threaded Rods	1	4%
Steel Connection	16	64%
No Connection	7	28%
Connection Inaccessible	1	4%
Total	**25**	**100%**

3.3.9.1. Threaded Rods Connection

Only one of the visited bridges, GR-142, was connected with threaded rods. The rods were spaced infrequently along the length of the structure. In this type of connection, the exterior girders of the RRFCs are basically butted up against each other and a threaded rod is placed through the web of each girder. Figure 3.12 shows an example of this type of connection.

Figure 3.12: Threaded rod longitudinal connection

3.3.9.2. Steel Connection

The most frequently seen type of connection was a steel plate. For this longitudinal connection, the two RRFCs are typically spaced with a gap, ranging anywhere from several feet to a few inches. A steel plate was placed over this gap and was typically welded to the top of the each flatcar. It should also be noted that for most of the larger gaps between flatcars, steel sections were also installed in both the longitudinal and transverse directions. These sections were generally welded to the steel plates and flatcars. Examples of various steel plate connections are shown in Figure 3.13.

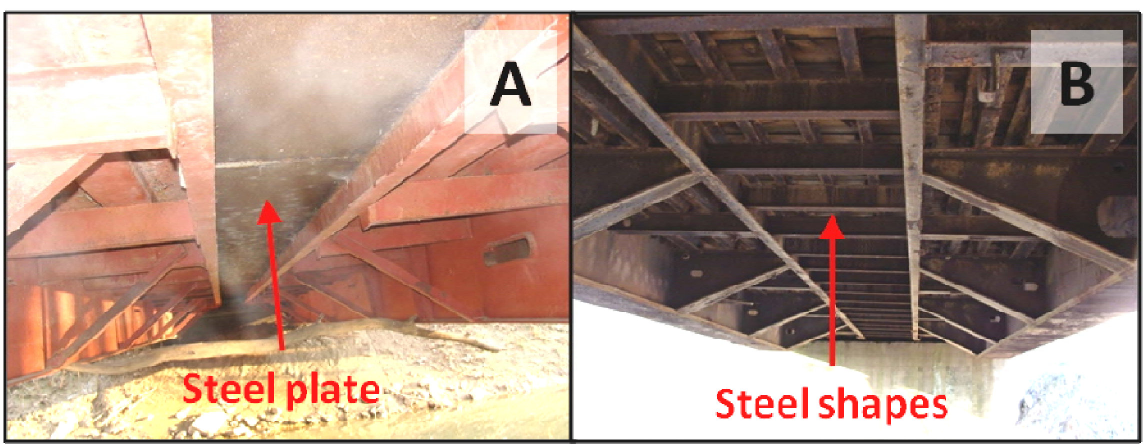

Figure 3.13: Steel longitudinal connection made of steel plate (A) & steel shapes (B)

3.3.9.3. No Connection

Another somewhat common "connection" between two RRFCs is no connection at all. In this case, the exterior girders of adjacent flatcars are butted up next to each other and no connection is made between the two flatcars. Most of the RRFC bridges with no longitudinal connections have a concrete deck, which would provide some continuity between the two cars. An example of the two flatcars butted up against each other is shown in Figure 3.14.

Figure 3.14: No connection between RRFCs

3.3.10. Damage

Although the majority of the RRFCs had relatively little damage, a few had major damage to various structural elements. Damage in the transverse members was more common; however, some flatcars did have major damage to the longitudinal members. Most of the damage consisted of either bent or deformed members. Examples of damage seen in the field are shown in Figure 3.15.

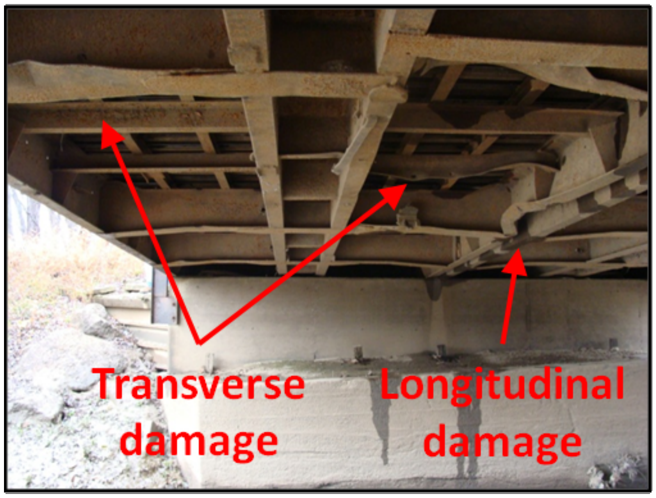

Figure 3.15: Damaged members in RRFC bridges

3.3.11. Load Rating

Out of the 25 representative bridges, five had a load posting. This is 20% of the bridges visited. The postings varied between 8-15 tons: one bridge was posted at 13 tons, two were posted at 10 tons, one was posted at 8 tons, and one bridge had postings of 12 or 15 tons depending on which direction the bridge was approached. After the field visits were conducted, the database was reviewed and revealed 20 of the total 124 RRFC bridges (roughly 16%) had load postings ranging from 3-15 tons. This agreed quite well with the representative sample bridges.

3.3.12. Integral Abutments

During the field visits it was observed that some of the RRFC bridges, including the majority of those found in Clay County, had integral abutments. It appeared as though when the flatcars were installed as bridges, they were cast directly in place on an existing abutment. By using integral abutments, a concrete connection was also made between the flatcars on the support. An example of a RRFC bridge with an integral abutment is shown in Figure 3.16.

Figure 3.16: Integral abutment on RRFC bridge

3.3.13. Location of Wheel Trucks

As discussed in Section 2.1.1.3. RRFCs are designed to be supported on the wheel trucks. While performing the field visits, however, this was not always the case. In many instances, the flatcars were supported at the very end of the car rather than at the wheel trucks.

There were generally two cases in the field where this was observed. One of these cases was when flatcars were spanning a large distance. If the span was greater than the distance between wheel trucks, the flatcars were usually supported at the ends of the car. There were a few instances seen where supporting the flatcar at the ends created a “kink” in the flatcar. An example of this is shown in Figure 3.17. In the figure, it appears as though the flatcar is bent at the location where the main girder tapers up to a shallower section.

Figure 3.17: RRFC bridge supported outside of wheel truck

The other case where RRFCs were not supported on the wheel trucks was for very short spans, such as 30-40 feet. In these cases, the wheel trucks were generally located near midspan. To create this scenario, it appeared as though a single flatcar may have been cut in half and each half was placed side-by-side to create a bridge. One issue with this is the section is greatly reduced near midspan. However, since the spans are short, the maximum moments are significantly less than with longer spans. An example is shown in Figure 3.18.

Figure 3.18: Wheel trucks located near midspan

3.3.14. Four Span, 240 Feet Long RRFC Bridge

As stated in Section 3.2.1. HA-84 was selected for visitation in addition to the 25 representative bridges. This structure is 240 feet long and appeared to have four spans of roughly the same length. Each of the four spans was constructed from two RRFCs placed side-by-side, for a total of 8 RRFCs. The deck surface of the bridge was made of steel open grating. This open grating also served, somewhat, as a connection between adjacent flatcars because it was welded to the flatcars in intermittent locations. An elevation view of HA-84 is shown in Figure 3.19. Three of the spans can be clearly seen, while the fourth span extends from the pier on the right side of the picture, hidden by trees.

Figure 3.19: Four span RRFC bridge

3.4. Additional Field Visits

After performing the initial field visits, it was determined that field instrumentation would be used in the development of a load rating procedure for RRFC bridges. (This will be discussed more in-depth in the following Chapter.) Unfortunately, many of the 26 bridges were not adequate for field instrumentation due to poor access and proximity to Purdue University. Therefore, a total of 49 more RRFC bridges were visited in Indiana.

3.4.1. Additional Bridges Selected for Field Visits

Since one of the goals of the field instrumentation was to find candidate bridges relatively near to Purdue University, only those bridges with close proximity were visited. Of the 49 additional bridges, 32 were located in Clay County, 7 in Fountain County, 3 in Parke County, 3 in Sullivan County, 2 in Vermillion County, and 2 in Warren County. A complete list of all the RRFC bridges visited can be found in Appendix A.

3.4.2. Additional Field Visit Findings

Since the primary purpose of the additional field visits was to find adequate candidate bridges for field instrumentation, these visits were less in-depth than the visits to the representative sample of bridges. The findings from these visits were essentially the same as those described in Section 3.3. with two additional findings described in the following sections.

3.4.2.1. "Car Haulers" or Boxcars

As stated in Section 3.1. there were nine newer RRFC bridges installed in Clay County since 2007. Of these nine newer bridges, seven were a different type of railroad car which has been referred to as a "car hauler". Of the seven car haulers, six of them had load postings, ranging from 4-11 tons. After reviewing the Arkansas State University study (Parsons 1991), the car haulers appear to be similar to the pictures of boxcars taken from their field visits.

Car haulers differ from RRFCs in that they do not have a main box girder. Instead, they contain two Z-shapes running longitudinally close to the middle of the car. These Z shapes are placed back-to-back spaced at a few inches. A channel, which is deeper than the Z-shapes, serves as an exterior girder on either side of the car. The car haulers have a deck which appears to be similar to nailable steel flooring. A picture of the underneath side of a car hauler is shown in Figure 3.20.

Figure 3.20: Car hauler

3.4.2.2. Missing Bottom Flange

As discussed in Section 3.3.10. some of the RRFCs found in the field were damaged. One of the additional bridges visited was missing the entire bottom flange of the main girder. A Structural Inventory and Appraisal (SI&A) Report of this bridge was reviewed after the field visits. The report stated the bottom flange had been removed due to clearance issues. It should be noted that the bridge was load posted for 6 tons and was closed to traffic during the field visits. A picture of the missing bottom flange is shown in Figure 3.21.

Figure 3.21: RRFC bridge with missing bottom flange

CHAPTER 4. FIELD INSTRUMENTATION & LOAD TESTING

Field instrumentation and load testing were performed to aid in the understanding of the behavior of RRFC bridges. Instrumenting a bridge allows researchers to get stress/strain results directly as a result of applying load. Analytical or finite element models were not selected because of the large number of variables and assumptions needed to construct and analyze these structures. The following sections will describe the field instrumentation and load testing procedures.

4.1. Selection of Bridges to be Instrumented

The first step in the field instrumentation process was to identify how many RRFC bridges would be instrumented with strain gages and load tested as part of this study. Secondly, the specific bridges to be instrumented needed to be identified. Bridge selection was performed using the Indiana inventory data and the criteria described below.

4.1.1. Criteria for Selection of Bridges

4.1.1.1. Simple, Longer Spans

Since more than 90% of the RRFC bridges in Indiana are simple spans, the decision was made to test only these types of structures. It was also determined that testing longer spans would yield “better” results. This is because longer spans develop larger moments than shorter spans which, in turn, mean larger stresses are developed in the members. Larger stresses can be easier to work with because noise in the stress data and small experimental errors will be much smaller compared to the actual stresses.

Additionally, due to these larger stresses, bridges with long spans are more likely to have problems.

4.1.1.2. Deck Type

As stated in Section 3.1.3. the most common deck types for RRFC bridges are steel plates, concrete, and timber. The decision was made to select one timber deck bridge because all of the timber deck bridges were located in Fountain County, which is close to Purdue University. One concrete bridge deck was also selected. This would allow a comparison to be made between the load distribution of different deck types. The remaining bridges had a steel plate deck.

4.1.1.3. Cross-Section

As discussed in Section 3.3.3. the exterior girders of RRFCs can range from being shallower than the main girder to being deeper than the main girder. Therefore, the decision was made that one of the bridges selected would have deeper exterior girders, but the majority of the bridges would have shallower exterior girders. It was also decided to select one car hauler bridge since almost all of them located in Indiana had low load postings. These different cross-sections would provide some indication of which members were carrying load and which were transferring it to another member.

4.1.1.4. Longitudinal Connection Type

As discussed in Section 3.3.9. there are many different types of longitudinal connections found on RRFC bridges in Indiana. Keeping this in mind, the decision was made to select a variety of longitudinal connections. This would allow an investigation of the load distribution between flatcars through different types of longitudinal connections.

4.1.1.5. Low Load Posting

Since one of the objectives of the project is to develop load rating guidelines, the decision was made that at least one of the bridges selected should have a low load posting. By studying a bridge with a low load posting, stress measurements could be compared to the current posting to determine how conservatively the bridge had been load posted.

4.1.1.6. Access

Access was a considerable challenge, as seen by the need to visit 49 additional bridges. Even if a bridge fulfilled the other criteria, if sufficient access was not available, the bridge could not be considered. Access included considerations such as ability to work underneath the bridge, parking, safety, and proximity to Purdue University.

4.1.2. Bridges Selected for Field Instrumentation

Based on the Indiana inventory and the criteria determined, seven RRFC bridges were selected for field instrumentation and load testing. The seven bridges chosen are shown in Table 4.1. Each of these bridges displayed adequate access for testing.

Table 4.1: Bridges selected for field instrumentation

Bridge	Span Length	Deck Type	Exterior Girder Size	Longitudinal Connection	Load Posting (tons)
CL-53	34'-0"	Asphalt	Small	Welded steel plate	None
CL-179	31'-6"	Asphalt	Small	Welded steel plate	None
CL-406	42'-0"	Asphalt	Car hauler	Large steel beam & plate	4
FO-25	70'-0"	Timber	Small	Steel beams	None
FO-54	81'-0"	Steel	Small	Steel beams & plate	None
FO-256	82'-0"	Steel	Small	Steel beams & plate	4
VE-24	50'-0"	Concrete	Large	One steel beam at midspan	None

4.2. Field Instrumentation

Field instrumentation was chosen to aid in the development of a load rating procedure for the longitudinal members of a RRFC bridge. In particular, the proposed load rating guidelines will focus on the longitudinal members of a RRFC bridge. These include the main girders, stringers, and exterior girders. The specific strain gages used for testing and the location of these strain gages will be discussed in the following sections.

4.2.1. Strain Gages

Strain gages were used to find the live load stresses in response to controlled load testing. The measured stresses include both the global response of the bridge as a system, and the local response of individual elements. All the strain gages installed on the RRFC bridges were produced by Vishay Micro-Measurements model LWK-06-W250B-350 and model CEA-06-W250A-350, both with an active grid length of 0.25 inches. Both strain gages are uniaxial weldable resistance-type strain gages and were selected because of their easy installation in the field. Both strain gage types have a resistance of 350 ohms and were used with an excitation voltage of either five or ten volts depending on the data logger used for the monitoring.

These strain gages come pre-bonded to a metal strip by the manufacturer. The strain gage installation process begins by first grinding the steel surface of the flatcar smooth and cleaning it with degreaser. Multiple pinprick sized resistance spot welds gages were then used to attach the strain gages to the bridge steel. The final step in the installation process consisted of covering the strain gages with a multi-layer weatherproofing system to protect them from the environment. Figure 4.1.A shows a strain gage after welding installation, and Figure 4.1.B shows the gage after the weatherproofing system has been applied.

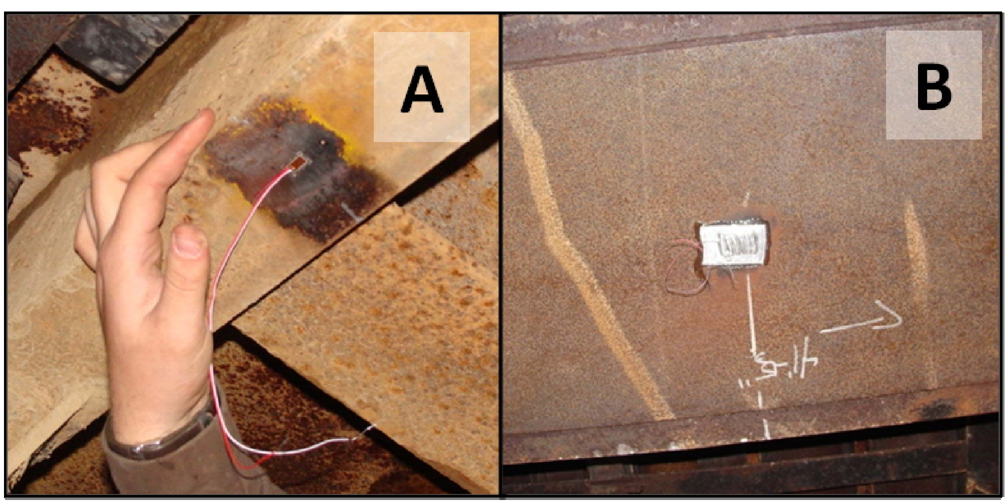

Figure 4.1: Strain gage after installation (A) & after protective system (B)

4.2.2. Locations of Strain Gages

A total of 109 strain gages were installed on the seven RRFC bridges. As stated previously, the proposed load rating guidelines are focused on the main girders, stringers, and exterior girders. Strain gages were typically placed on each of these elements at midspan, where the moment is at a maximum. Where possible, strain gages were placed on the top and bottom flanges of the main girder and exterior girder. Typically, strain gages were installed on one or two stringers per bridge. Since many stringers do not have top flanges, only the bottom flanges were instrumented. Detailed instrumentation plans containing exact stain gage locations for all seven bridges can be found in Appendix B. Figure 4.2 shows a general view of the strain gages installed at midspan on CL-53.

Figure 4.2: General view of strain gages installed at midspan of CL-53

4.3. Controlled Load Testing

A series of controlled load tests were performed on each of the seven RRFC bridges. The tests were performed using tandem axle dump trucks provided by the county where each respective bridge was located. During each of the load tests, traffic control was performed to ensure the safety of all persons present during the testing. Specifics about the test trucks and load tests will be described in the following sections.

4.3.1. Test Trucks

An empty and fully loaded truck was present during the testing at each bridge, except for VE-24 in which only a fully loaded truck was present. The empty truck was driven across the bridge first to ensure it could safely cross the bridge without introducing any damage. Stresses were recorded while the empty truck crossed the bridge. Using this data, stresses were estimated for the fully loaded truck to determine whether or not excessive stresses would be reached. If excessively large stresses were not expected, the fully loaded truck would be driven across the bridge and stresses would be recorded.

The dimensions and axle weights were measured for each test truck. These dimensions can be found in Figure 4.3 and Table 4.2. Only one set of dimensions is shown for each county test truck since the empty and fully loaded trucks had the same dimensions. Additionally, the axle weights of the empty and fully loaded trucks can be found in Table 4.3.

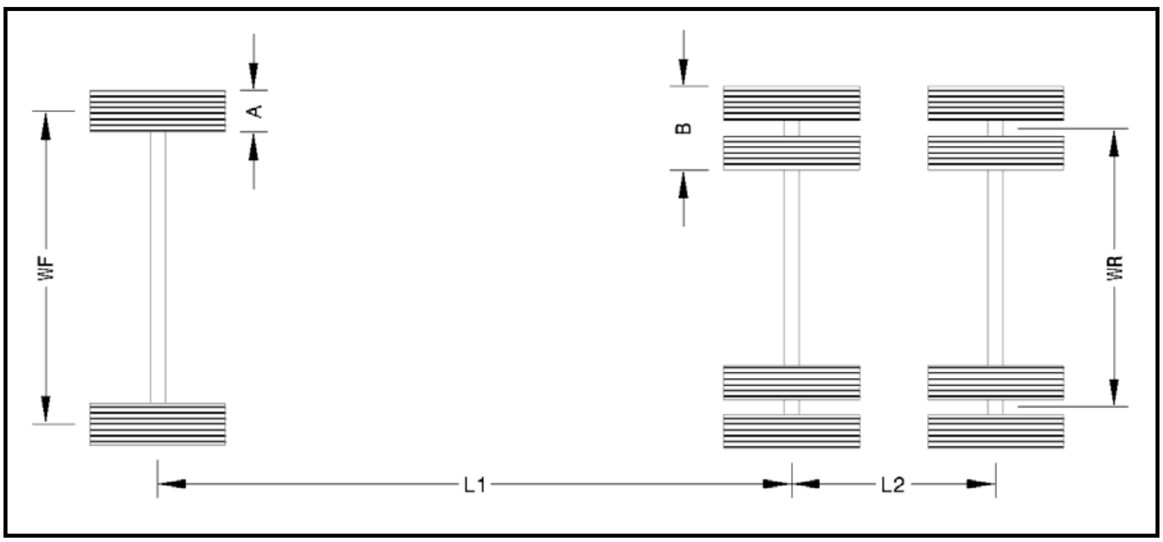

Figure 4.3: Diagram of test truck axles

Table 4.2: Dimensions of test truck axles

County	L1	L2	WF	WR	A	B
Clay	14'-0"	4'-6"	6'-11"	6'-2"	11"	1'-10"
Fountain	13'-2"	4'-0"	6'-10"	6'-2"	11"	1'-10"
Vermillion	10'-7"	4'-5"	7'-0"	6'-1"	10"	1'-10"

Table 4.3: Weights of test truck axles

County	Full or Empty	Axle Weights (kips)		
		Front	Front Tandem	Rear Tandem
Clay	Empty	6.6	7.9	7.9
	Full	15.7	18.9	18.9
Fountain	Empty	5.6	8.5	8.5
	Full	14.0	20.9	20.9
Vermillion	Full	13.1	22.3	22.0

4.3.2. Load Tests

The load tests typically consisted of driving the test trucks in three lanes: upstream, downstream, and on the centerline. The upstream and downstream positions referred to the direction of the current in the creek or stream below the test bridge. The upstream and downstream lanes were typically in the right- or left-most position transversely on the bridge. This typically resulted in roughly one foot off of the guardrail. In some cases, geometric constrictions of the roadway or previous damage to the bridge deck prevented the test truck from driving this close to the guardrail. For these instances, the test truck was moved transversely toward the center of the bridge. It should also be noted that for some bridges, other transverse positions were tested as well. All test positions for each of the seven bridges can be found in Appendix B. An example of a load test location is shown in Figure 4.4.

Figure 4.4: Test truck in upstream lane on FO-25

Each of the transverse load positions had two tests performed: static park test and crawl test. The crawl test consisted of driving the test truck across the bridge at a slow speed (~5 mph) to reduce the dynamic response of the bridge. The crawl tests were performed twice for repeatability purposes. The static test consisted of parking the centerline of the tandem at the midspan of the bridge for a few seconds while any dynamic amplification settled out of the bridge. Additionally, dynamic testing was performed for all tested bridges. During the dynamic test, the driver was asked to cross the bridge at a "typical" speed. For most tests, this was around 30-35 mph and was generally within the posted speed limit where it was posted. Generally, the transverse position during the dynamic test was down the center of the bridge. The static and dynamic tests were each performed at least once for every bridge. These tests were then compared then compared with the measurements recorded during the crawl tests.

CHAPTER 5. RESULTS OF CONTROLLED LOAD TESTING

As described in Chapter 4, controlled load testing was performed on seven RRFC bridges. Trucks of known weight and geometry were used for the controlled testing that consisted of park tests and crawl tests. Testing was performed at three different transverse positions (i.e., lanes) for each bridge. Results from the crawl tests for each of the three lanes (left, right, and center) provided the most valuable information, and thus will be presented in the following sections. The crawl tests were primarily used because much of the bridges' dynamic responses were removed, making the results more representative of the bridges' static responses. It should also be mentioned that static and dynamic tests were confirmed to have reasonably good agreement with the crawl tests.

As stated in Section 4.2.2. strain gages were placed on the main girders, exterior girders, and stringers of each of the seven RRFC bridges. (The exact locations of all strain gages can be found in Appendix B.) The measured strains were converted to stresses by multiplying by an assumed elastic modulus value for steel of 29,000 ksi. These stresses then were recorded during the controlled load testing. Results from the controlled load testing will be presented in two groups: main girders, and secondary elements, including exterior girders and stringers. The interpretation of these test results and how they were used to develop the load rating guidelines will be presented in Chapter 6.

5.1. Main Girder Results

The following sections will present selected results from the top and bottom flange stresses from the main girders measured during various load tests. These members undergo global bending in response to the test truck driving across the bridge. As stated previously, the locations of all strain gages and load tests can be found in Appendix B. Also, as to be stated in the following sections, all stress results can be found in tabular form in Appendix C.

5.1.1. FO-54: CL Crawl Test, Fully Loaded Truck

Figure 5.1 presents the resulting stresses at the top and bottom flanges of the main girders on bridge FO-54 as the fully loaded test truck drove in the center-lane at a crawl speed. Each of the channels (gages) shown in the figure were installed near midspan. The numbers shown in boxes on the figure (1-6) represent the trace number used to help identify individual channel numbers. It should be noted the trace number is not the same as the channel number. Each trace number refers to a specific channel number as shown directly above the stress vs. time plot, and below the load test position. Trace numbers are shown in all remaining figures similar to Figure 5.1.

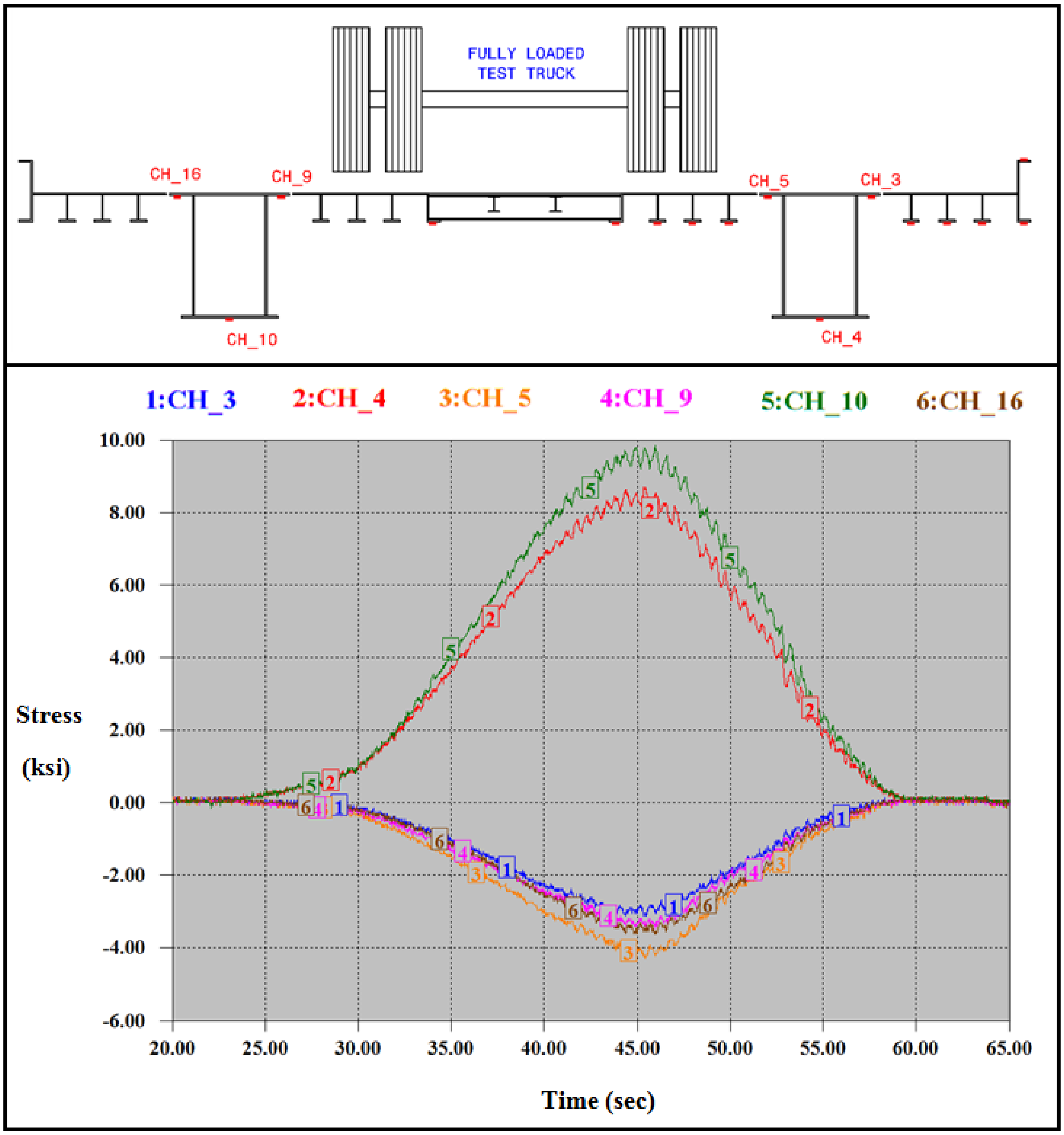

Figure 5.1: FO-54 main girder CL crawl test results

In general, the results of the above load test were as expected. When comparing the bottom flange stresses (CH_4 and CH_10), the resulting stresses are relatively similar. This indicates good load distribution between the two main girders. One reason the stresses were different could be if the test truck was not exactly centered transversely on the bridge. In this case a higher stress would be measured in one girder over the other.

Similar observations can be made about the top flange stresses. As seen in the figure, the top flange stresses (CH_3, CH_5, CH_9, and CH_16) are relatively similar. This again shows good load distribution between the two main girders. When comparing the top flange stresses in a single flatcar (CH_3 and CH_5), a small difference in stress can be seen. This difference can be attributed to some out of plane bending of the main girder.

The results from Figure 5.1 are shown in tabular form rounded to the nearest tenth of a ksi in Table 5.1. These stress values were obtained by trying to remove the dynamic response of the bridge. As seen in Figure 5.1, the stresses oscillate as the bridge bounces up and down as the test truck crosses the bridge. The stresses in the table were obtained by drawing a line approximately through the mean of the stress oscillations. Each of the stresses shown in the table occurred at the same point in time. Positive values represent tensile stresses, while negative values represent compressive stresses. As stated previously, each of the load tests were performed twice. Both load tests produced reasonably similar results; thus only results from one of the tests are shown in the table. All remaining stress result tables, included those in Appendix C, were constructed using these assumptions.

Table 5.1: Main girder stress results for CL crawl test on FO-54

Bridge	**FO-54**
Load Test	**CL, Crawl**
Channel	**Stress (ksi)**
3	-3.0
4	8.5
5	-4.2
9	-3.3
10	9.7
16	-3.5

5.1.2. FO-54: UP Crawl Test, Fully Loaded Test Truck

Figure 5.2 presents the measured stresses for the top and bottom flanges of the main girder during the UP crawl test. Again, each of the channels shown were installed near midspan.

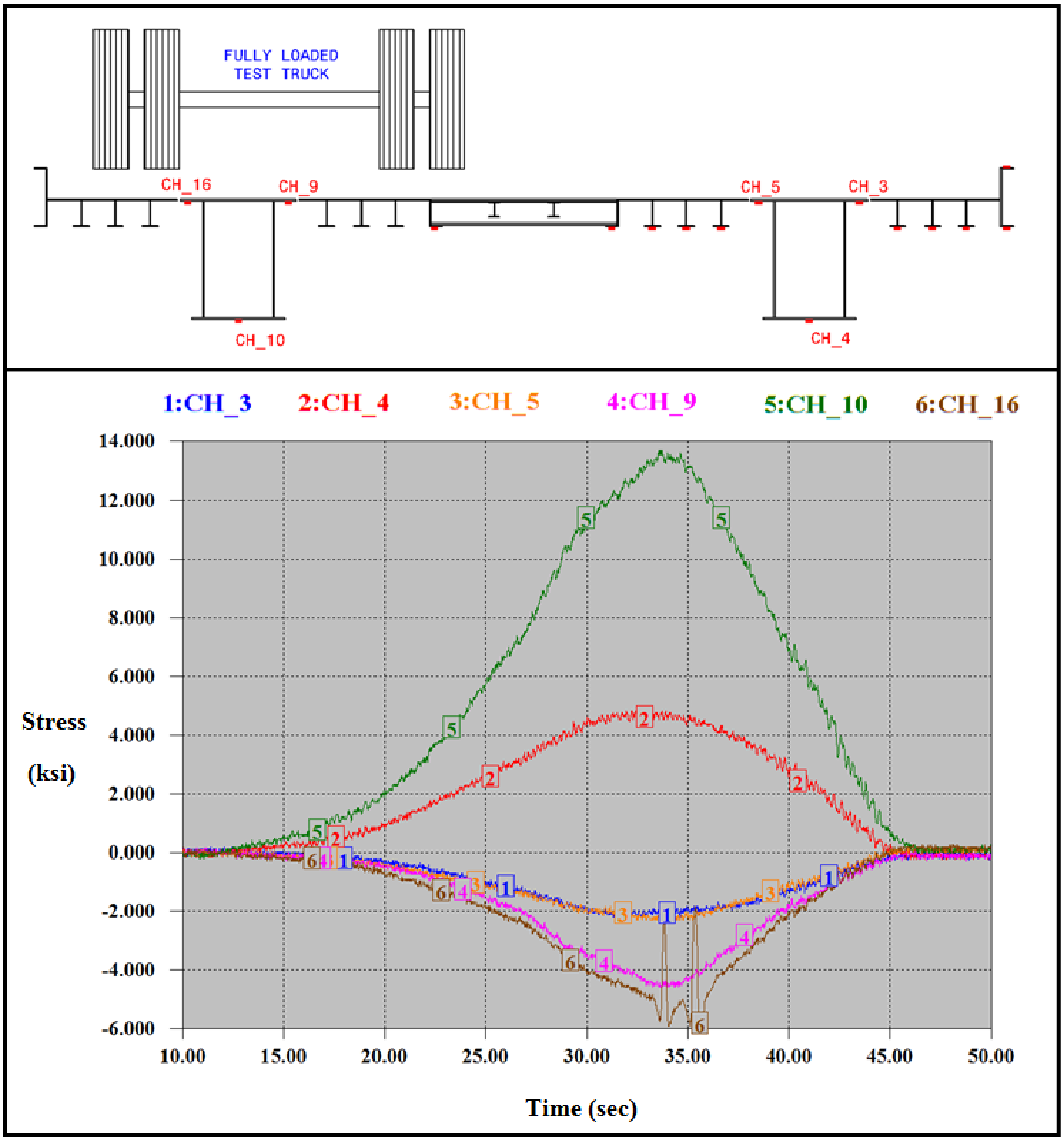

Figure 5.2: FO-54 main girder UP crawl test results

The stresses shown in Figure 5.2 present a general view of the load distribution between the two main girders. As clearly seen, the stresses in the left main girder are greater for both the top and bottom flanges. This is as expected because the test truck is located directly over the left flatcar.

One interesting thing to note from the figure is the local tension spikes in CH_16 between the times of approximately 33-36 seconds. These two tension spikes represent the local bending that occurs in the top flange when the tandem axles were located directly over the location of the strain gage. Such spikes provide information as to where the test truck was located during the time of maximum stress. As seen in the figure, the maximum tension and compression stresses occur at roughly the same instant as the first tension spikes, or when the first tandem axle was at midspan. The stress results from Figure 5.2 can be summarized in Table 5.2.

Table 5.2: Main girder stress results for UP crawl test on FO-54

Bridge	FO-54
Load Test	**UP, Crawl**
Channel	**Stress (ksi)**
3	-2.2
4	4.5
5	-2.3
9	-4.5
10	13.8
16	-4.8

The results from the remaining crawl test, DOWN, on FO-54 can be found in tabular form in Appendix C.

5.1.3. CL-53: UP Crawl Test, Fully Loaded Test Truck

Stresses measured during the UP crawl test on CL-53 are presented in Figure 5.3. The results include the top and bottom flange stresses from both main girders. All of the channels shown were installed near midspan.

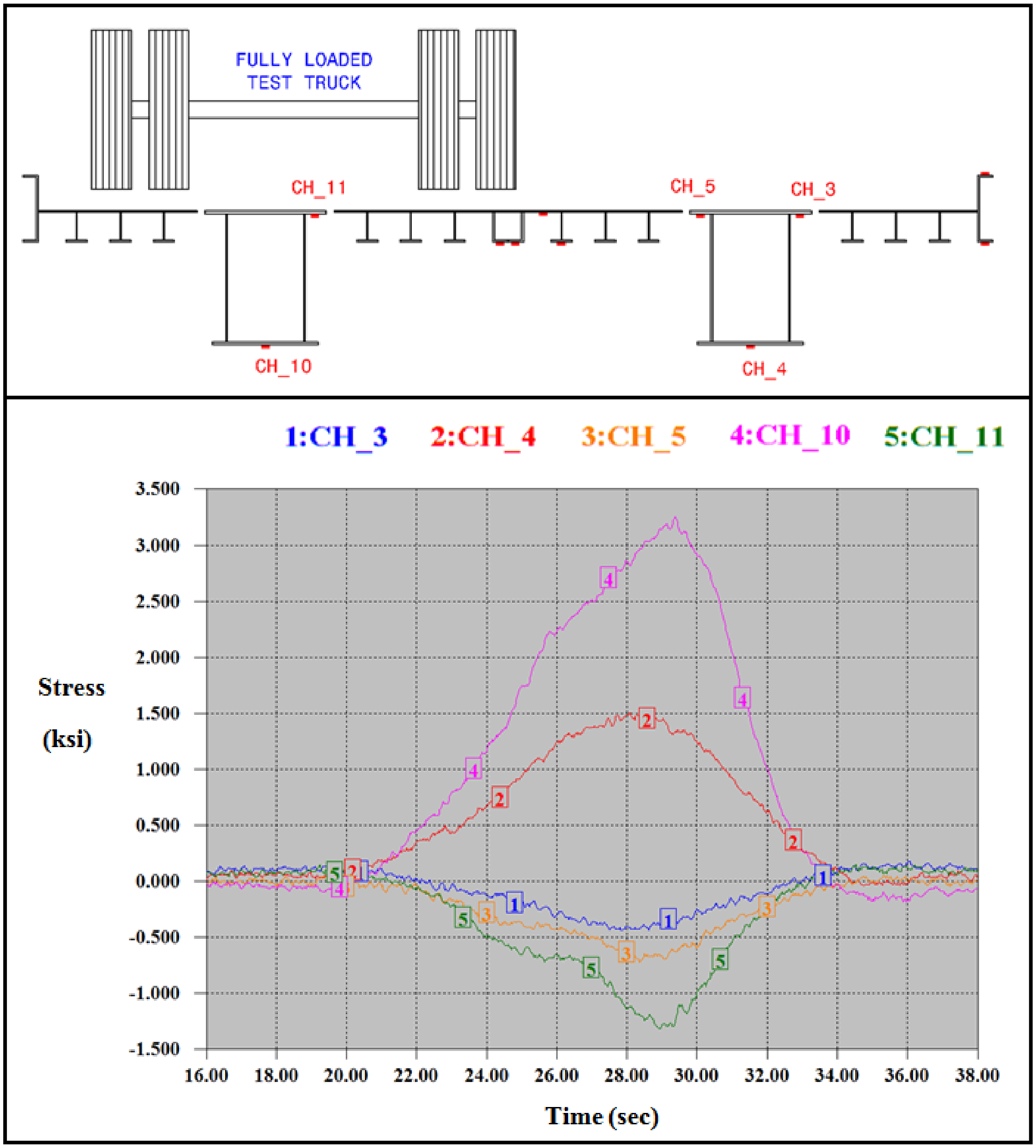

Figure 5.3: CL-53 main girder UP crawl test results

The stresses shown in Figure 5.3 provide a similar load distribution to that seen in FO-54. However, smaller overall stresses were measured due to the shorter span length. As seen in the figure, the left main girder carries the majority of the load with a portion of it being distributed to the right main girder. This load distribution can be seen in both the top and bottom flange stresses. The remaining crawl load test results, CL and DOWN, are provided in tabular form in Appendix C.

5.1.4. CL-179: DOWN Crawl Test, Fully Loaded Test Truck

CL-179 is slightly different from the rest of the seven instrumented bridges in that the entire RRFC cross-section could not be visually observed in the field. As seen in Appendix B and in Figure 5.4, the stringers consisted of rolled steel plates. These plates prevented the access required to verify what (if any) structural members were located above the bottom plate. The stress results from the DOWN tests performed on CL-179 are presented in Figure 5.4. As seen in the figure, the top and bottom flange stresses were recorded for the right main girder, while only the bottom flange stresses were recorded for the left main girder. All labeled channels were installed near midspan.

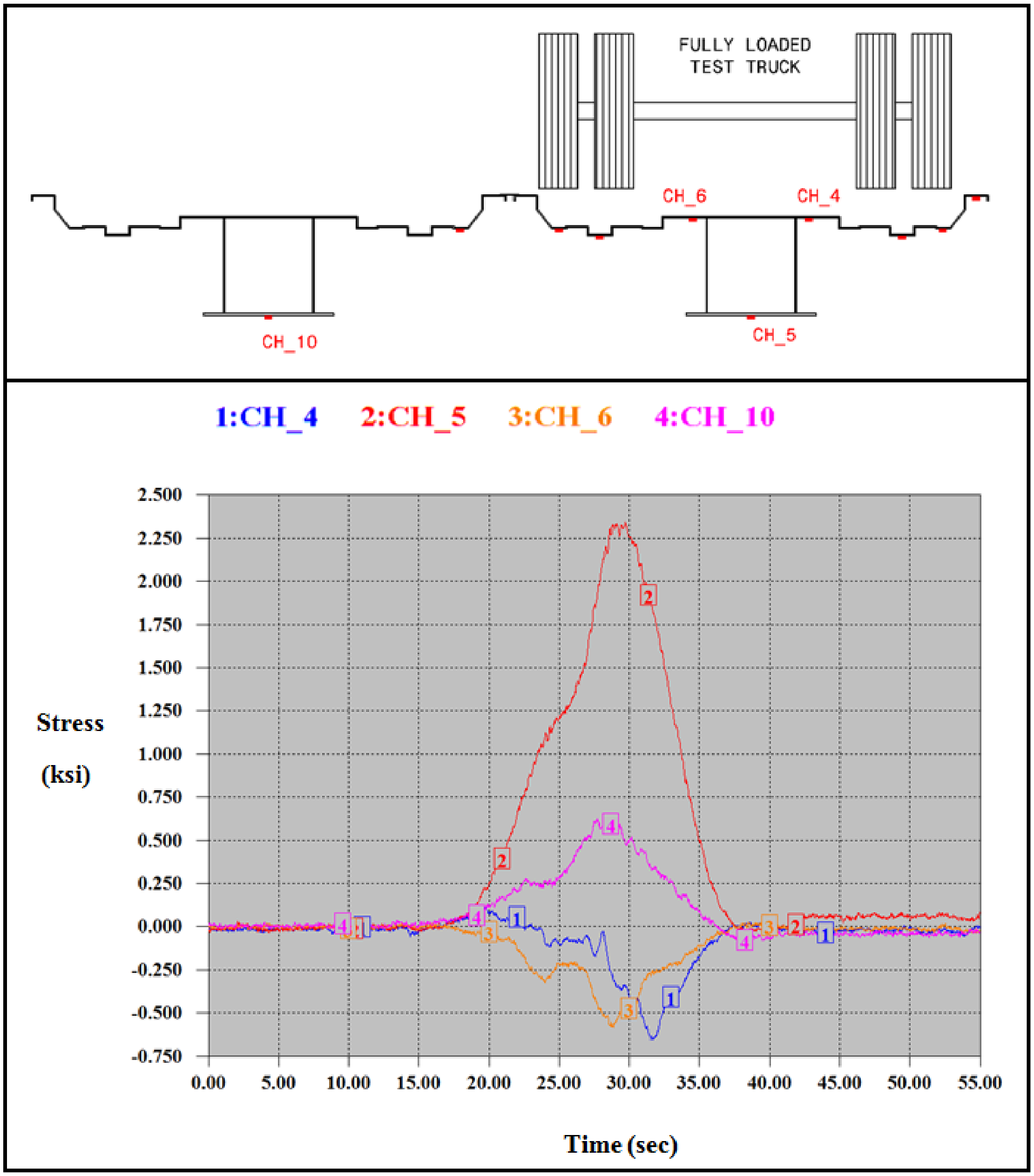

Figure 5.4: CL-179 main girder DOWN crawl test results

When comparing the load distribution of the bottom flange stresses, it appears as though a smaller percentage of the load is being distributed from the loaded girder (right) to the unloaded girder (right) than observed in the main girders of other RRFC bridges. Another interesting thing to note is the relatively large local bending stresses in the top flange of the loaded girder (CH_4 and CH_6). This local bending behavior can be seen at approximately 24-33 seconds. These spikes represent the times when each of the three truck axles crossed the locations of the strain gages. The local bending spikes for CL-179 are much less pronounced than those measured in the top flange of FO-54. The remaining crawl test results for the main girders on CL-179 can be found in tabular form in Appendix C.

5.1.5. FO-256: UP Crawl Test, Fully Loaded Test Truck

The measured stress results from the UP crawl test with the fully loaded truck are presented in Figure 5.5. As seen in the figure, both top and bottom flange stresses were recorded for each of the main girders. All of the channels shown were installed near midspan.

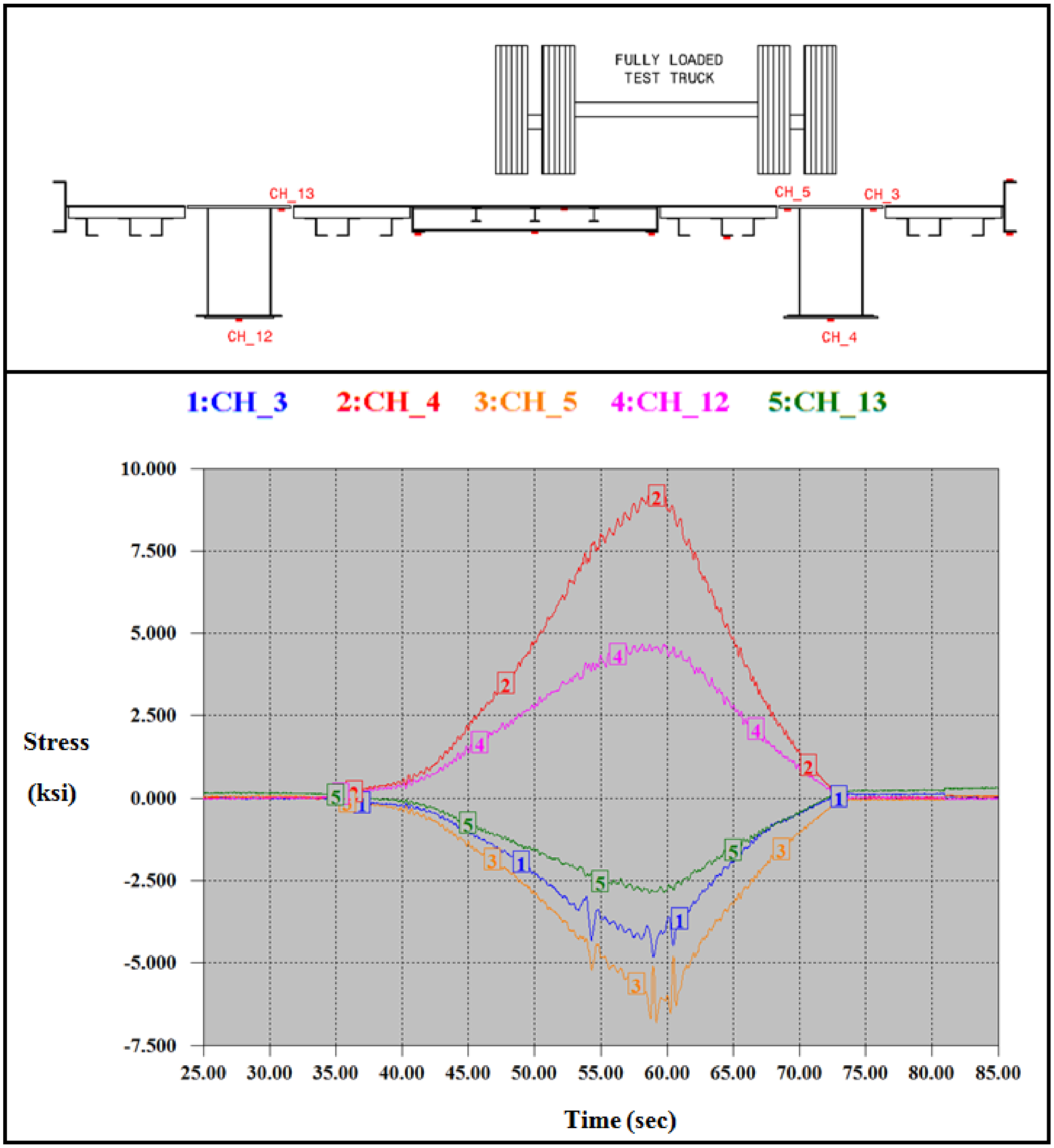

Figure 5.5: FO-256 main girder UP crawl test results

As shown in the cross section and test truck position in the figure, the UP test did not consist of placing the truck up against the guardrail as was done for the other bridges. This was because the deck was damaged against the guardrail, and any further damage to the bridge deck wanted to be avoided. The results for both top and bottom flange stresses seen in Figure 5.5 show good load distribution for the main girders similar to that seen in

FO-54 and CL-53. Again, the local tension stress spikes can be seen in the top flange of the loaded girder (right) for each of the three truck axles.

The remaining crawl test results for the main girders of FO-256 can be found in tabular form in Appendix C. As shown in Appendix B, both empty and fully loaded test trucks were used in the load tests for FO-256. Results from each of the trucks (empty or fully loaded) produced reasonably similar load distribution regardless of the weight of the test truck.

5.1.6. FO-25: CL Crawl Test, Fully Loaded Test Truck

FO-25 differed from the previously presented bridges due to its timber deck. Figure 5.6 shows the stress results from the top and bottom flanges of the right main girder and the bottom flange stresses from the left main girder. Although CH_11 is labeled in the figure, it is not included in the plot because it was not functioning properly during the time of testing. All channels shown were installed near midspan.

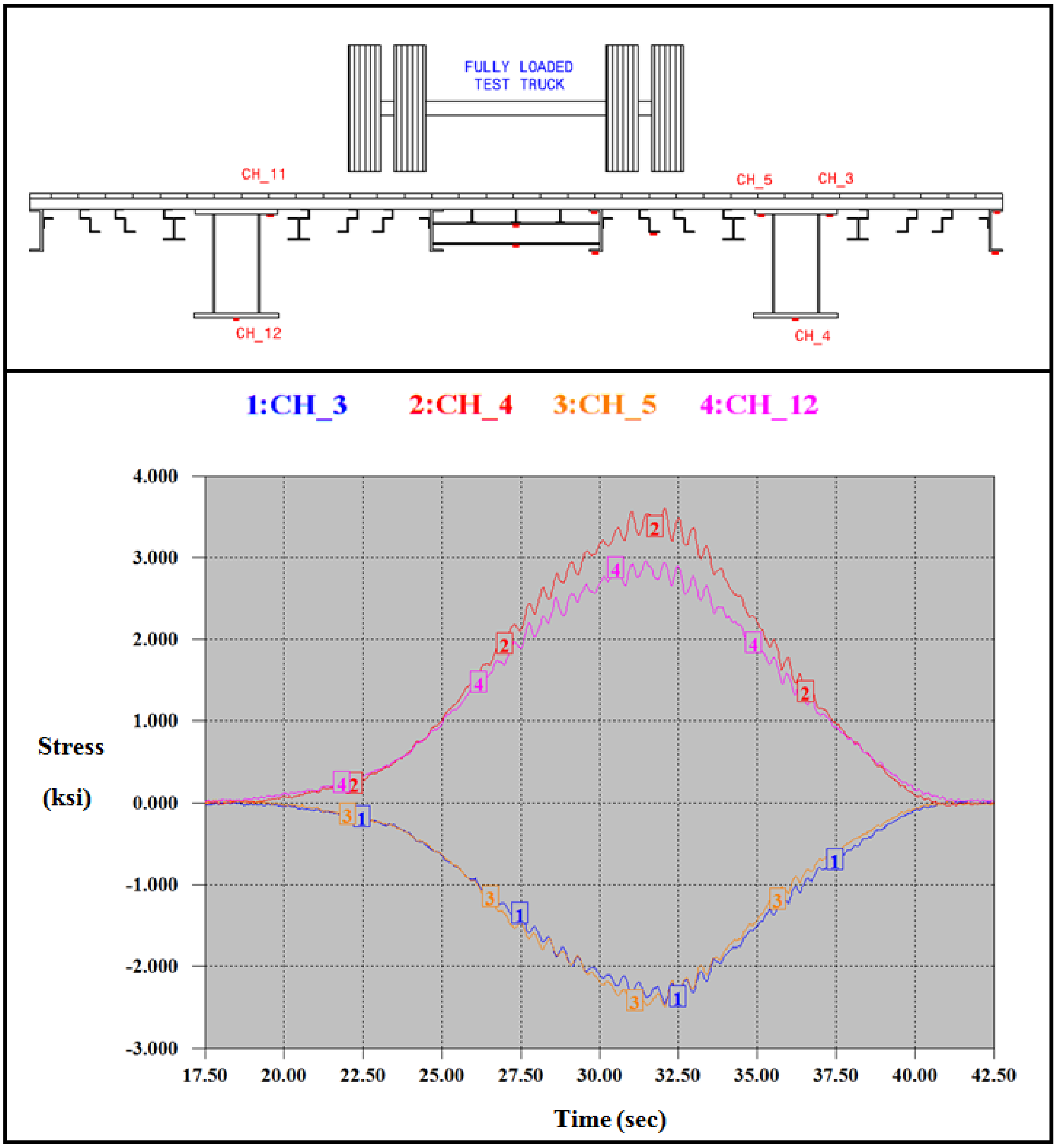

Figure 5.6: FO-25 main girder CL crawl test results

As seen with the CL crawl test on FO-54, relatively good load distribution was found between the two bottom flange stresses (CH_4 and CH_12) of the main girders. The right girder (CH_2) did measure a slightly greater stress than the left girder (CH_12), but generally speaking, the two had very similar results. It should be noted the difference between the two girders could be due to the test truck not being exactly centered on the bridge.

5.1.7. FO-25: UP Crawl Test, Fully Loaded Test Truck

Results for the UP crawl test on FO-25 can be found in Figure 5.7. As seen in the figure, the top and bottom flange stresses are shown for the loaded girder (right), and only the bottom flange stresses are shown for the unloaded girder (left). As stated previously, CH_11 was not functioning properly during the load tests. All channels shown were installed near midspan of the bridge.

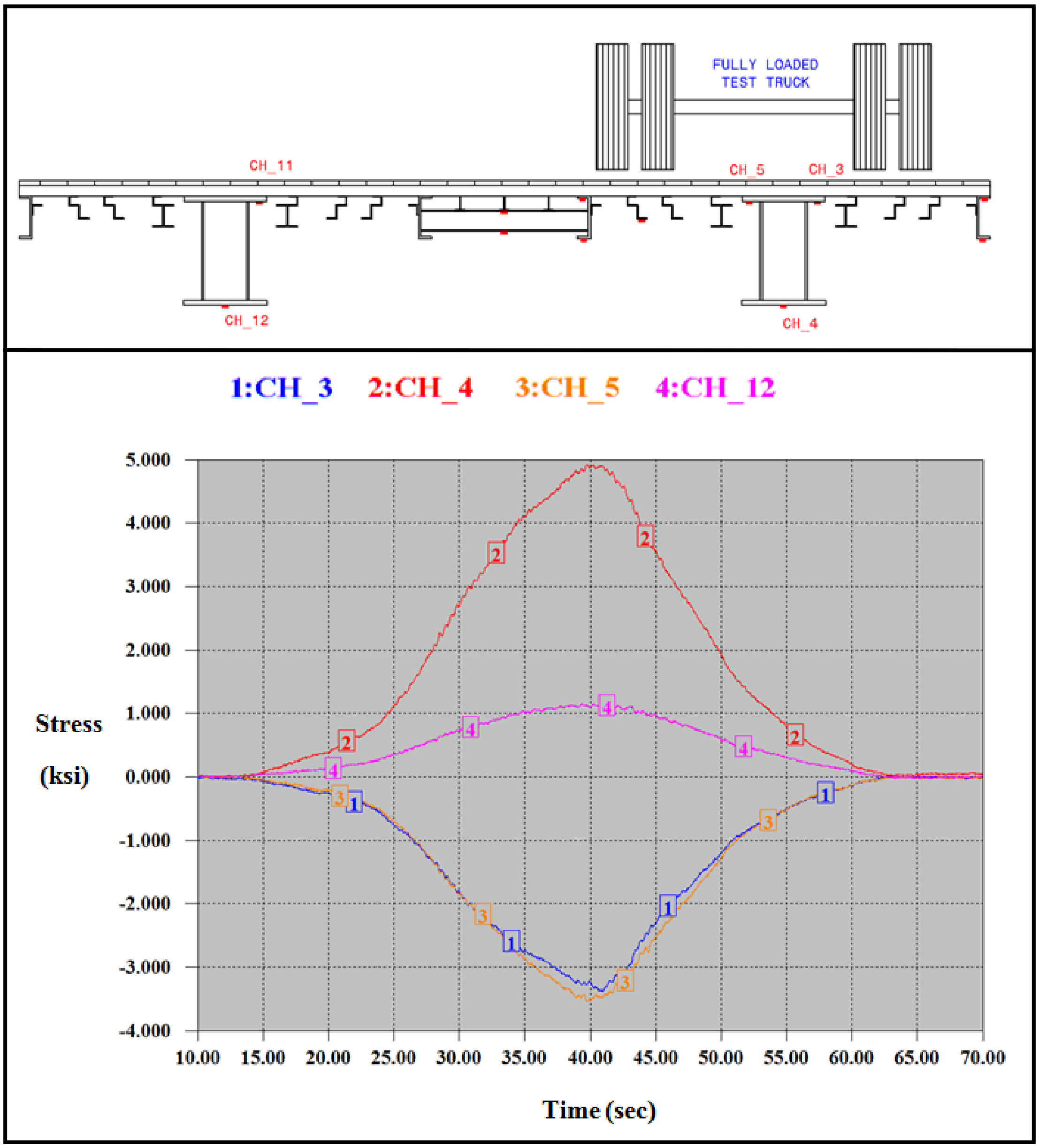

Figure 5.7: FO-25 main girder UP crawl test results

As shown in the figure, there is good load distribution between the two main girders, shown by the bottom flange stresses. The loaded girder carries the majority of the load, with some load being transferred to the opposite girder. The results from the remaining crawl test, DOWN, on FO-25 can be found in Appendix C.

5.1.8. VE-24: CL Crawl Test, Fully Loaded Test Truck

VE-24 differs from the rest of the bridges previously presented because it has a concrete deck. This bridge is also constructed from a RRFC made of riveted built-up members. The stresses resulting from the CL crawl test with a fully loaded test truck are presented in Figure 5.8. Top and bottom flange stresses are shown for the right main girder, while only a single bottom flange channel is shown. All channels plotted were installed near midspan.

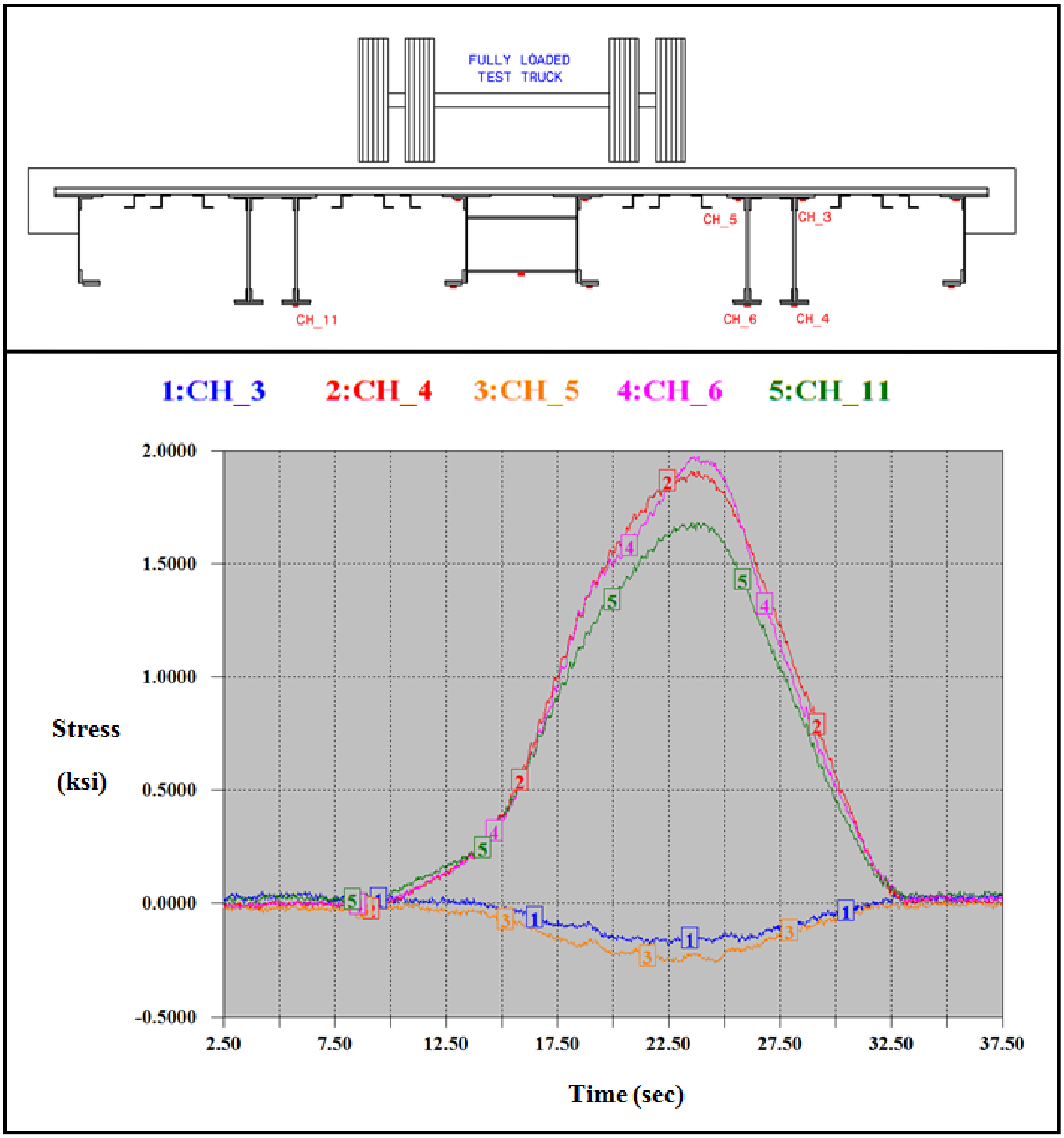

Figure 5.8: VE-24 main girder CL crawl test results

As seen in the figure, the top flange stresses are very similar even though the channels are located on different elements. The same can be said regarding the bottom flange stresses. When comparing the bottom flange stresses between the two main girders, good load distribution can be seen. As with some of the previous bridges, the right girder is carrying slightly more load than the left. Again, this could be due to imperfect test truck positioning.

5.1.9. VE-24: DOWN Crawl Test, Fully Loaded Truck

Figure 5.9 presents the stress results from the DOWN crawl test in which a fully loaded test truck was driven across the bridge in the position as shown in the figure. The top and bottom flange stresses were recorded in the loaded girder (right), while only the bottom flange stress was recorded in the unloaded girder (left). All channels shown were installed near midspan.

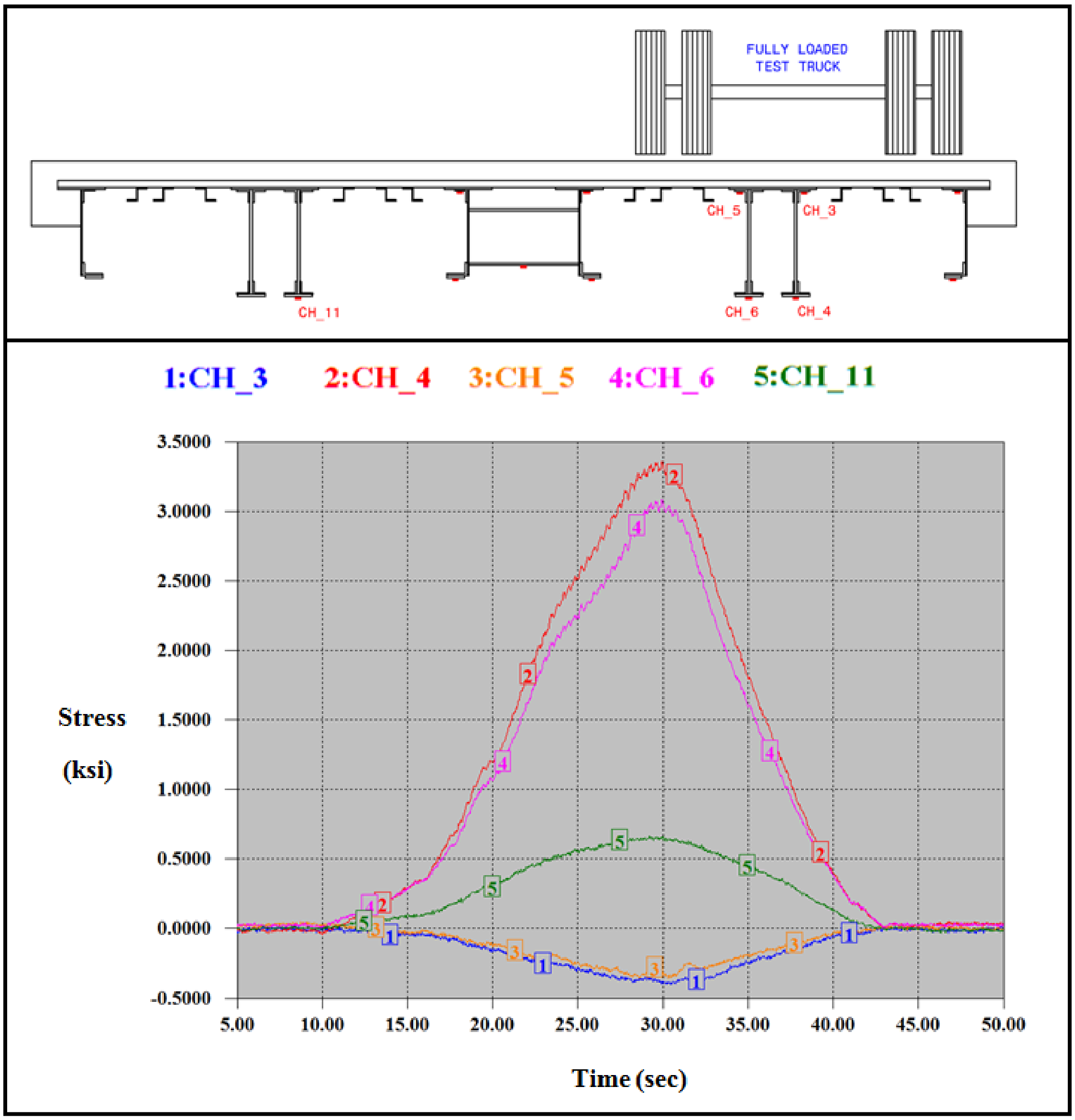

Figure 5.9: VE-24 main girder DOWN crawl test results

The results of Figure 5.9 show good load distribution between the two main girders when comparing the bottom flange stresses. One interesting feature to note is the stresses seen in VE-24 were somewhat similar to those seen in CL-53 (Figure 5.3) and CL-179 (Figure 5.4) for similar load tests, even though VE-24 has a much longer span length (50'-0") than CL-53 (34'-0") and CL-179 (31'-6"). This difference is most likely attributed to the presence of the additional stiffness provided by the concrete deck. The remaining load test, UP, results can be found in Appendix C.

5.1.10. CL-406: FULL Crawl Test, Fully Loaded Test Truck

CL-406 differs from the other bridges previously discussed in that it is a car hauler, as described in Section 3.4.2.1. As seen in Appendix B, CL-406 has a much different cross-section than a typical RRFC. For CL-406, the typical box girder is replaced with what appears to be two Z-shapes welded back to back. These welded Z-shapes are to be considered the main girders of car hauler type RRFC bridges.

Figure 5.10 presents the results of the FULL load test performed on CL-406. This load test consisted of the fully loaded test truck driving down the center of the bridge. Top and bottom flange strain gages were installed on the right main girder, while only bottom flange strain gages were placed on the left main girder. All channels installed on the bridge were placed near midspan.

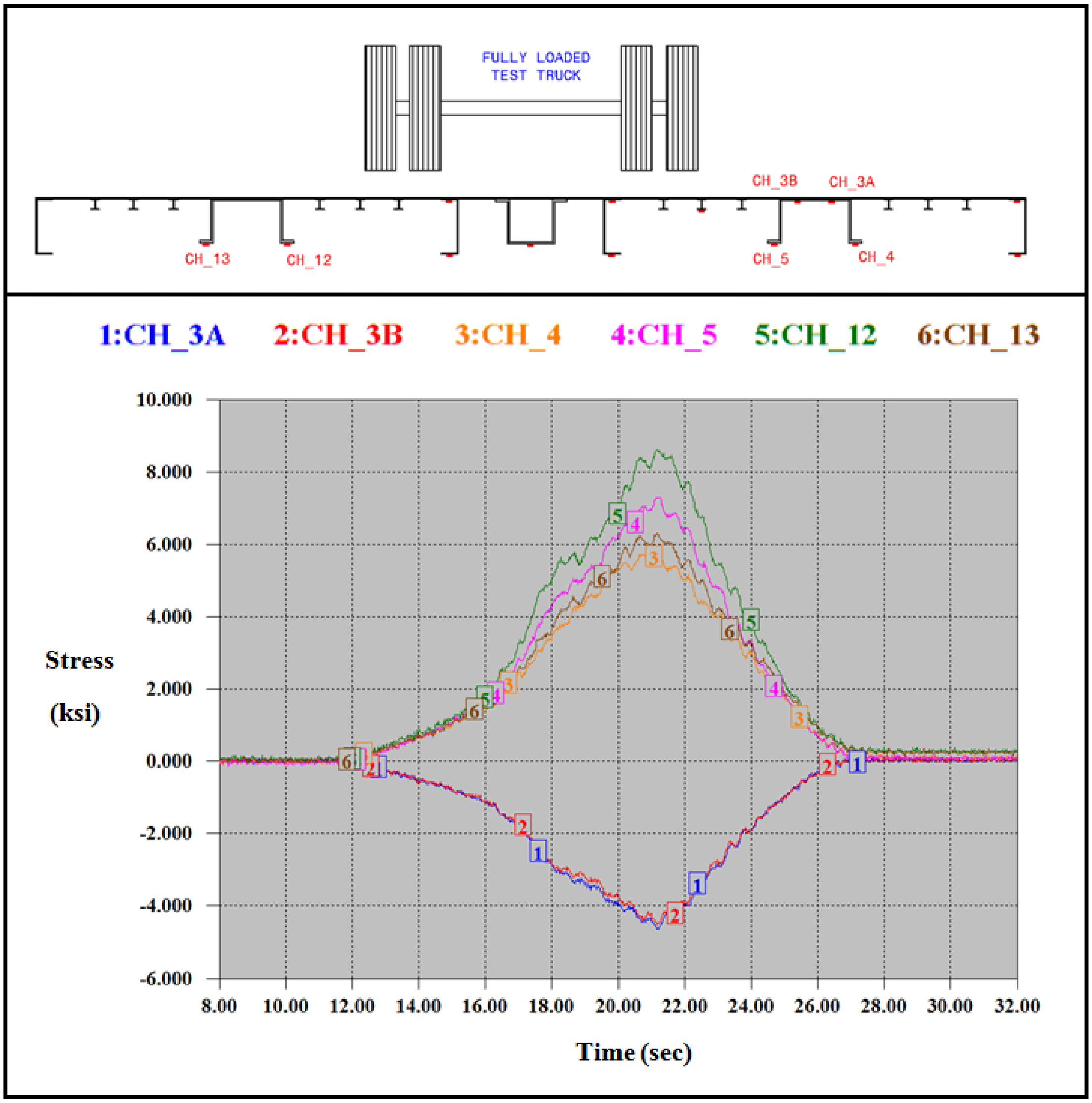

Figure 5.10: CL-406 main girder FULL crawl test results

As seen in the figure, CL-406 shows good load distribution, similar to the other bridges. Roughly half of the load is carried by each girder, although it appears the left girder is carrying slightly more. The top flange stresses (CH_3A and CH_3B) on the right girder show good similarity, while the bottom flange stresses are slightly more variable. This variability could be due to some out-of-plane twisting of the girders toward the middle of the bridge. Out-of-plane twisting seems reasonable since the inner bottom flanges (CH_5 and CH_12) exhibit slightly more stress than their respective outer bottom flange stresses (CH_4 and CH_13).

5.1.11. CL-406: DOWN Crawl Test, Empty Test Truck

Figure 5.11 shows the stress results for the DOWN crawl test on CL-406. This test consisted of an empty test truck driving on one side of the bridge. Top and bottom flange channels were placed on the right main girder, while only bottom flange channels were placed on the left main girder. All channels installed on the bridge were placed near midspan.

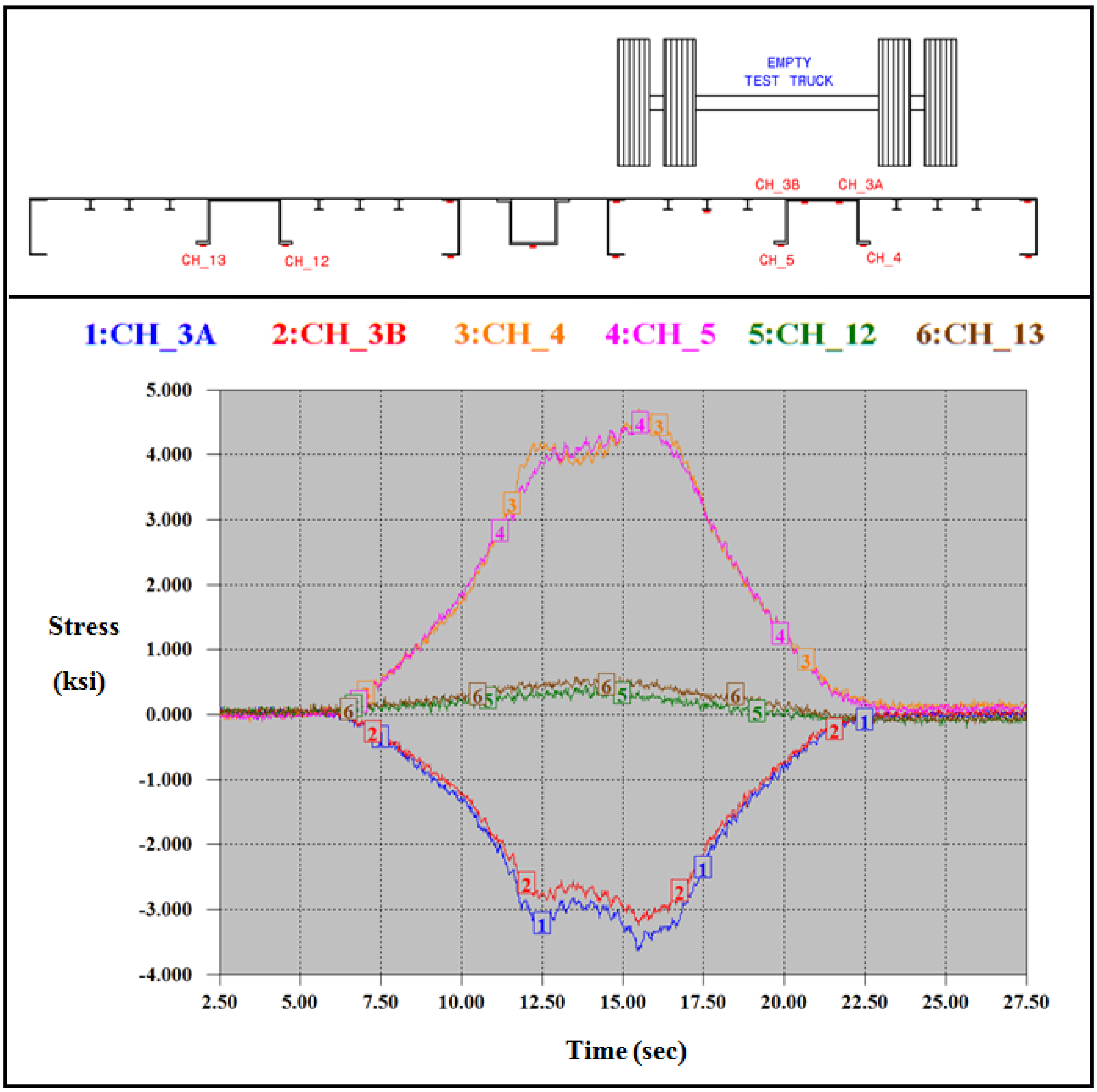

Figure 5.11: CL-406 main girder DOWN crawl test results

As seen in the figure, there is very little load distribution between the two main girders. The loaded (right) main girder carries almost the entire load, while a small amount is carried by the unloaded girder (left). Small local effects can be seen in the top and bottom flanges of the loaded girder; however, this effect is not nearly as pronounced as in some of the other bridges. The remaining load test results for CL-406 can be found in Appendix C.

5.2. Exterior Girder & Stringer Results

The following sections will present the stress results for the secondary elements of RRFC bridges, including the exterior girders and stringers. These elements, in addition to experiencing global bending, undergo local bending due to the presence of each axle, or each wheel load. Selected stress results will be presented in the following sections. Tabulated stress results are presented in Appendix C.

5.2.1. FO-54: CL Crawl Test, Fully Loaded Test Truck

Figure 5.12 presents the resulting stresses measured for the secondary elements under the right wheel loads in the CL load test on bridge FO-54. The bottom flange stress in the right main girder was also included as a reference. All of the channels shown in the figure were installed near midspan.

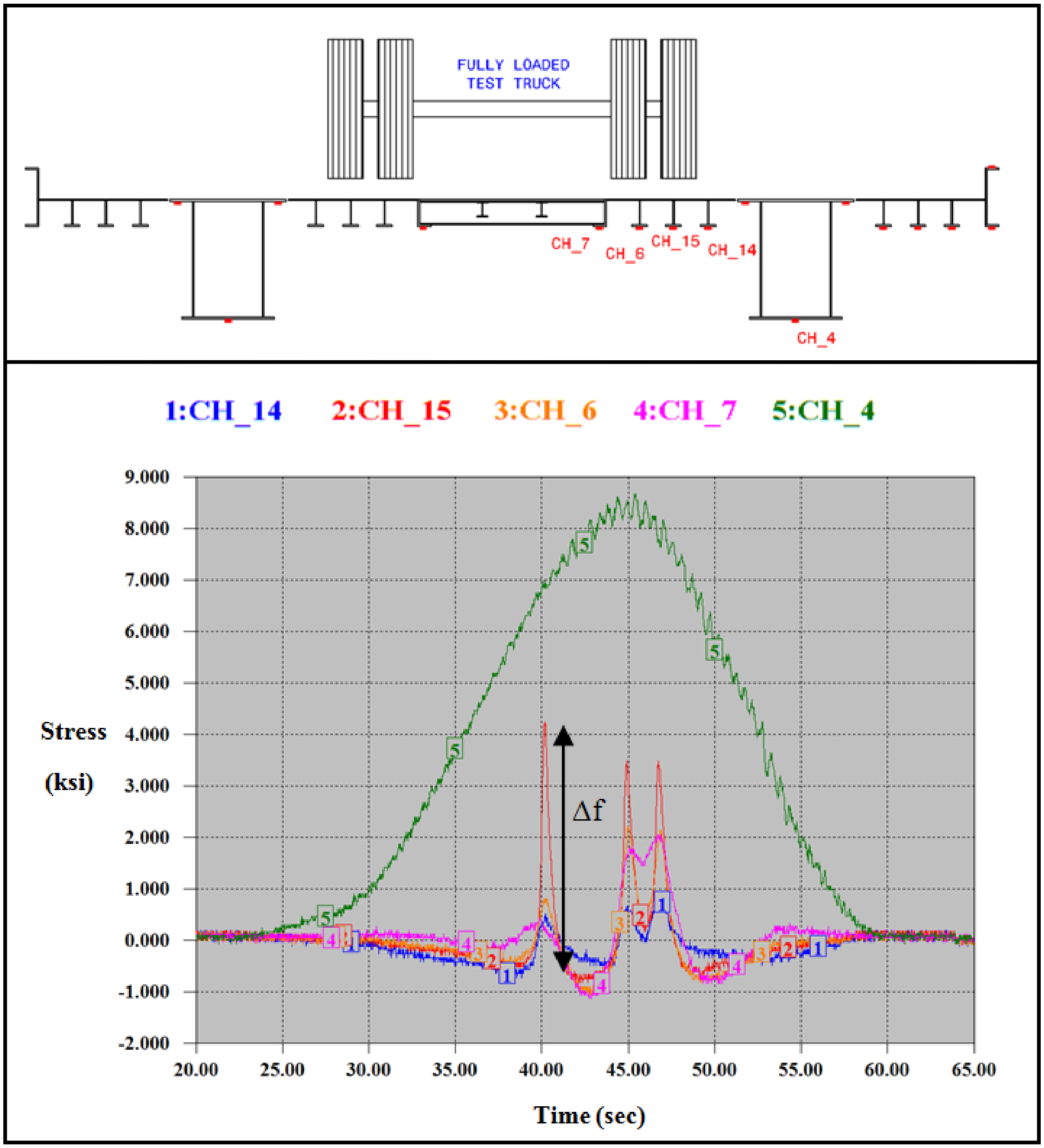

Figure 5.12: FO-54 secondary members CL crawl test results

As shown in the figure, all of the stringers and the exterior girder experience local bending effects for each of the three truck axles. These local bending effects are signified by the three local tension spikes in stress. The change in stress, labeled in the figure as Δf, represents the local bending stress. It should be noted that all of the secondary elements on one side of the main girder experience some local stress, showing load distribution of the wheel loads. The stress, however, is concentrated on the stringer labeled with CH_15. This is reasonable since the wheels are almost directly over that particular stringer. Another feature to be noted is all of the local stress spikes are less than the maximum global tension stress.

The results from Figure 5.12 are also shown in Table 5.3. All stresses were rounded to the nearest tenth of a ksi. As stated previously, each of the load tests were performed twice. These two load tests typically produced reasonably similar results; therefore, only one set of the load test results are shown in Table 5.3. All remaining stress result tables, included those in Appendix C, were constructed using these assumptions.

Table 5.3: Local stress results for CL crawl test on FO-54

Bridge	**FO-54**	
Load Test	**CL, Crawl**	
	Δf (ksi)	
Channel	**FA**	**STA**
14	1.1	1.3
15	4.4	3.9
6	1.4	3.1
7	1.1	2.7

FA = front axle & STA = single tandem axle

It should be noted that in order to determine the local tension spikes, the global stress of each element had to be estimated as if there was no local bending. Thus, the local stress was then found by taking the difference between the maximum local stress and the estimated global stress. Figure 5.13 shows an example of how the global stress was estimated and the local stress was determined.

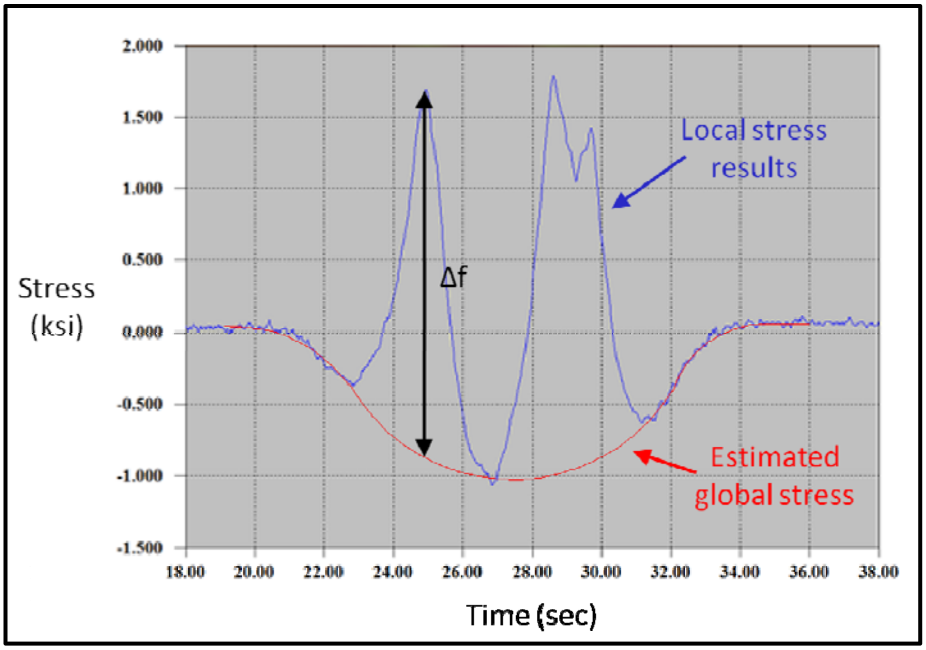

Figure 5.13: Example of global stress estimation

Other load tests performed on FO-54 in which the wheel lines were located over the stringers are presented in Appendix C.

5.2.2. CL-53: DOWN Crawl Test, Fully Loaded Test Truck

Figure 5.14 presents the local stress results for one of the left stringers and exterior girder in the loaded flatcar. The stress in the exterior girder in the unloaded flatcar (CH_9) was also included in the figure for the possibility of the truck being in the opposite lane. The bottom flange stress in the loaded main girder (CH_4) was also included as a reference. All channels shown were installed near midspan.

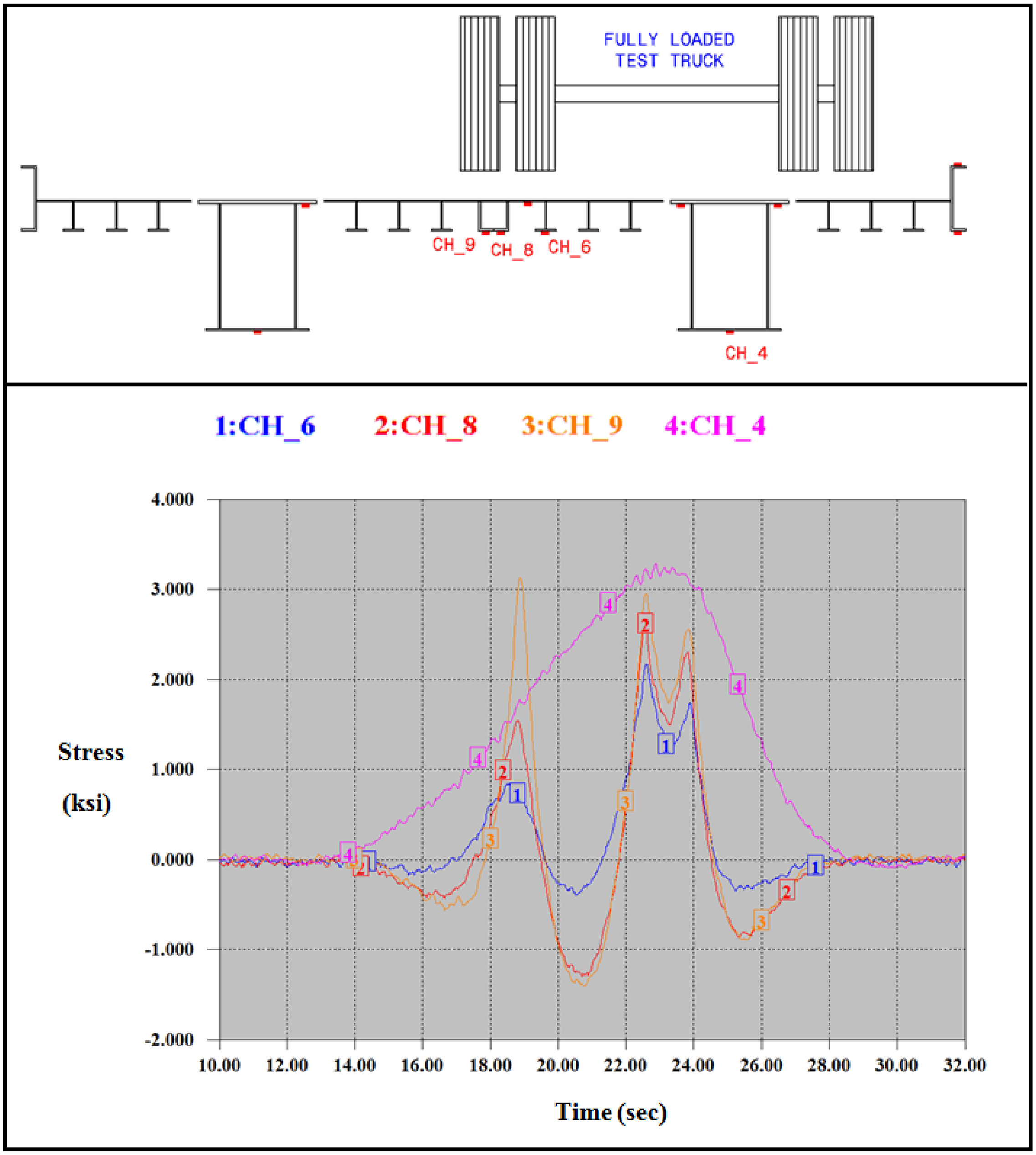

Figure 5.14: CL-53 secondary members UP crawl test results

Similar to FO-54, the local bending is distributed among the stringers and exterior girders. The largest local stress occurred at CH_9, the member directly beneath the wheel loads. As seen in the figure, the maximum local stress occurs at approximately 19 seconds, and is larger than the global tension stress in CH_4 at that time. However, it is still less than the maximum global tension stress seen by the bottom flange. Tabular results for the UP test are displayed in Appendix C.

5.2.3. CL-179: DOWN Crawl Test, Fully Loaded Test Truck

Figure 5.15 presents the local stresses measured for CL-179 during the DOWN load test. As seen in the figure, this bridge was constructed from a rolled steel plate deck, as opposed to a more typical stringer system. CH_2 was not functioning properly at the time of testing and therefore was not included in the plot. The results from the channels placed on the steel plate are shown. All channels shown were installed near midspan.

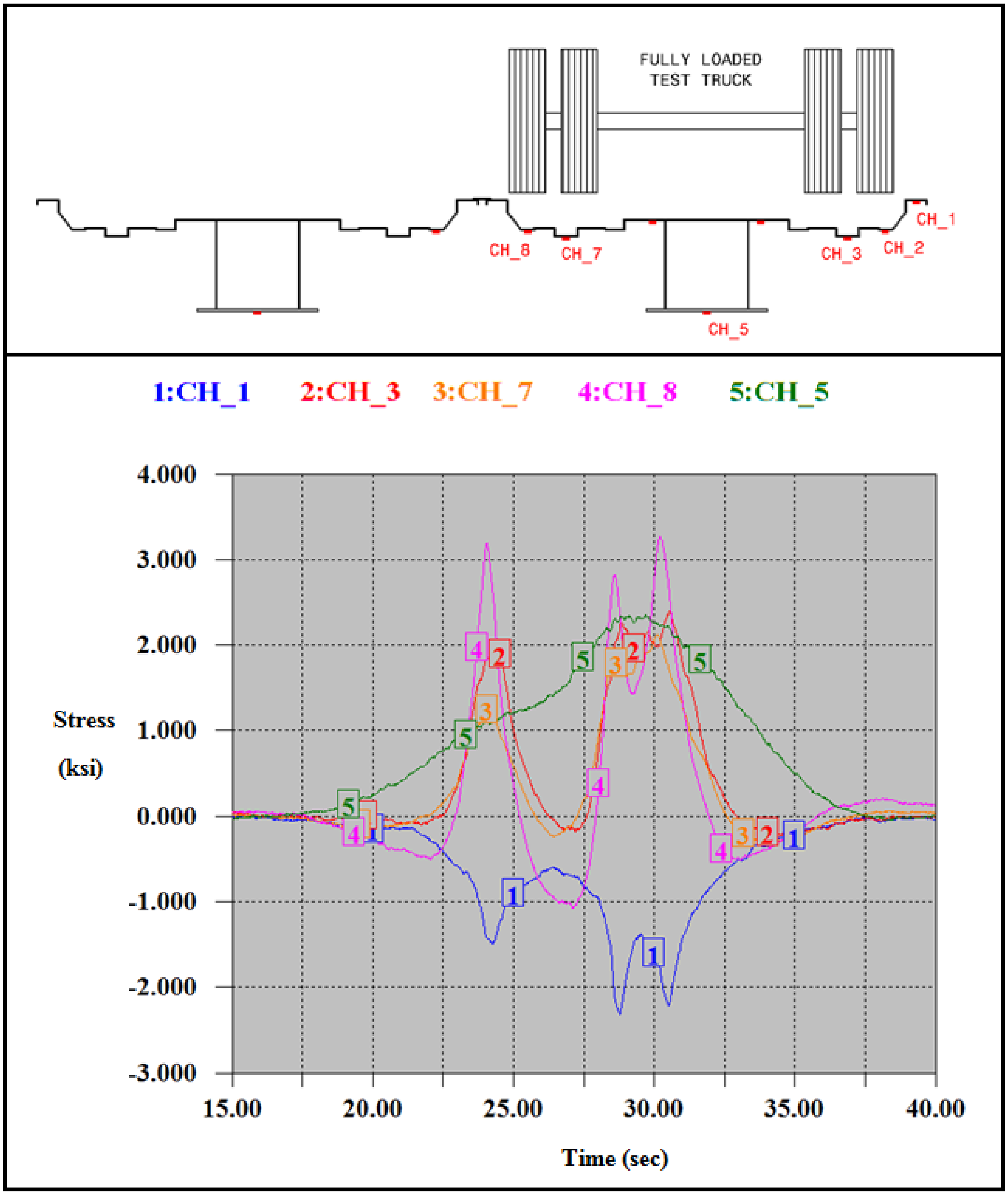

Figure 5.15: CL-179 secondary members DOWN crawl test results

As shown in Figure 5.15, the steel plate does undergo local bending, similar to the more typical stringers. CH_1, located in the same position as a normal exterior girder, exhibits compression local bending as a result of each of the three axles. The other strain gages experience local tension spikes, with the maximum local stress occurring in CH_8.

As opposed to the prior results presented, this local stress is greater than the maximum global tension in the bottom flange of the main girder.

5.2.4. FO-256: CL_2 Crawl Test, Fully Loaded Test Truck

Figure 5.16 presents the results for the local stresses in a middle stringer and an exterior girder on bridge FO-256 during the CL_2 crawl test. The bottom flange stress in one of the main girders is also shown as a reference. All channels plotted were installed near midspan of the bridge.

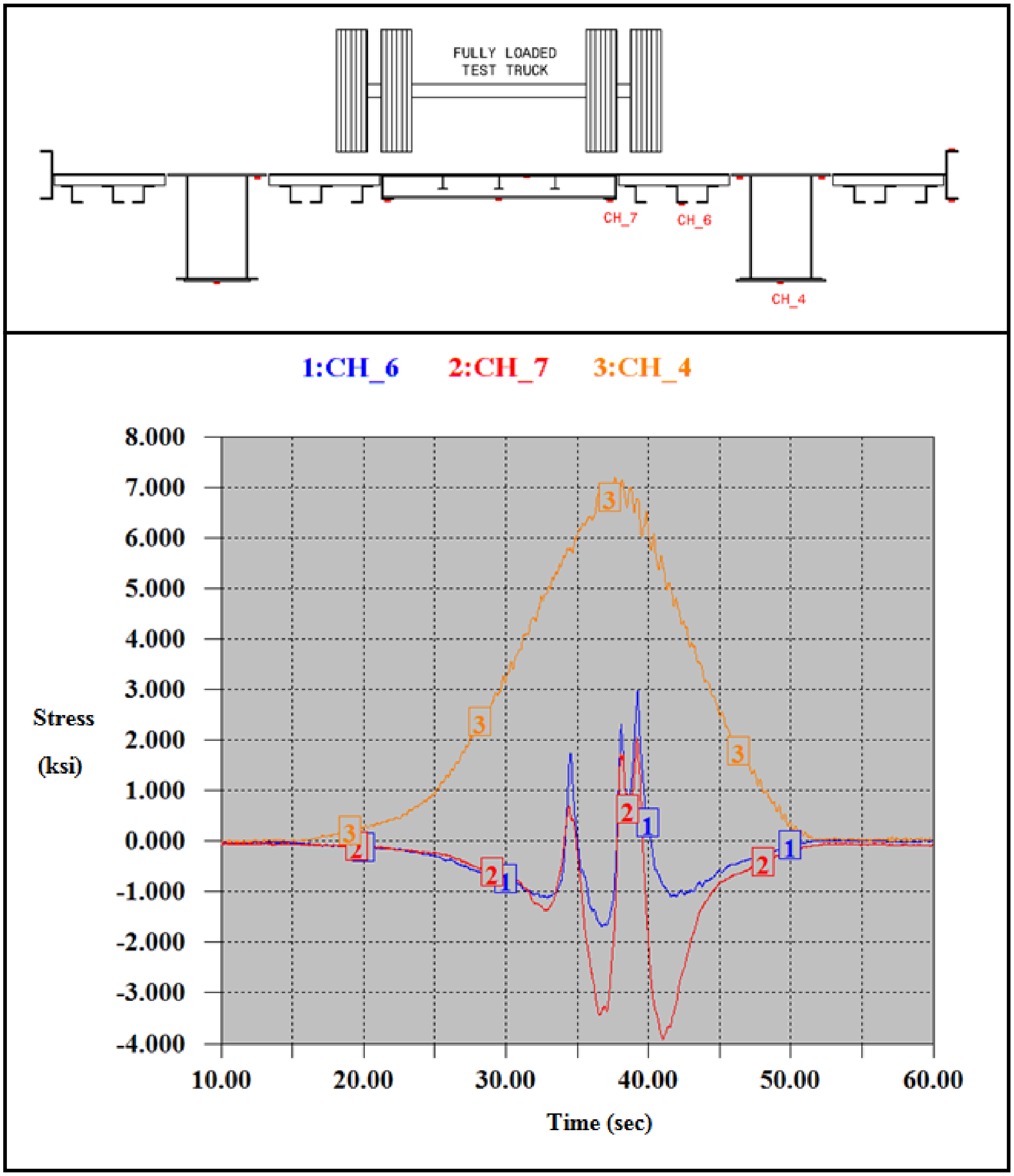

Figure 5.16: FO-256 secondary members CL_2 crawl test results

Similar to the other bridges discussed, each of the three truck axles can be seen in the figure. Although the wheel loads appear to be directly over CH_7, some stress is measured for CH_6. Also, the maximum local stress value occurs in CH_6; however, the largest change in stress occurs in CH_7, which is as expected due to its proximity to the wheel loads. The maximum local stresses are less than the maximum global stress, similar to the majority of the other bridges examined. Other load test results for FO-256 are presented in Appendix C.

5.2.5. FO-25: UP Crawl Test, Fully Loaded Test Truck

Figure 5.17 presents the local stress results for the UP test performed on FO-25. As shown in the figure, one stringer and an exterior girder were instrumented. It should be noted the inner exterior girders in FO-25 are not cut to form the longitudinal connection, as is the case with some of the other RRFC bridges. The bottom flange stress in the loaded main girder is also included for reference. All channels shown were installed near midspan of the bridge.

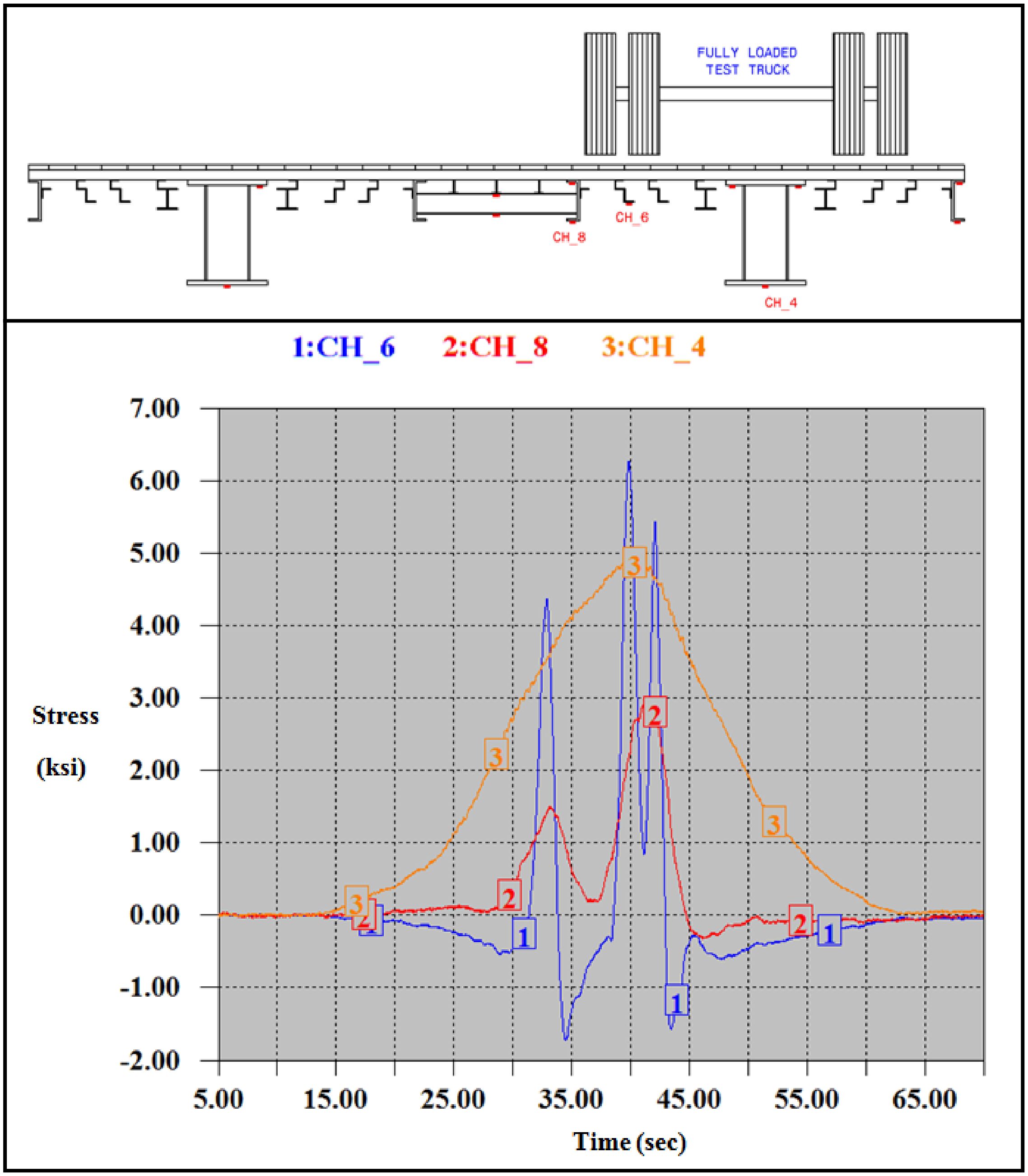

Figure 5.17: FO-25 secondary members UP crawl test results

As seen in Figure 5.17, the presence of all three axles can be seen in CH_6; however, only the front and the combination of the tandem axles can be seen in CH_8. An explanation for this could be that the bottom flange of the exterior girder (CH_8) is slightly further away from the wheel loads than the bottom flange of the stringer (CH_6); thus, less local bending is occurring. It should also be noted the local stresses seen in FO-

25 were larger than the maximum global tension stresses. Other test results from FO-25 can be found in Appendix C.

5.2.6. VE-24: DOWN Crawl Test, Fully Loaded Test Truck

Stress results for the secondary elements in VE-24 from the DOWN test are presented in Figure 5.18. The bottom flange strain gage (CH_2) on one of the exterior girders in the loaded flatcar is shown in the figure. Unfortunately, the bottom flange strain gage (CH_8) on the opposite exterior girder was not functioning properly at the time of testing and therefore no results from that channel are shown. Also, the bottom flange gage on the main girder is also shown for reference. All channels plotted were installed near midspan.

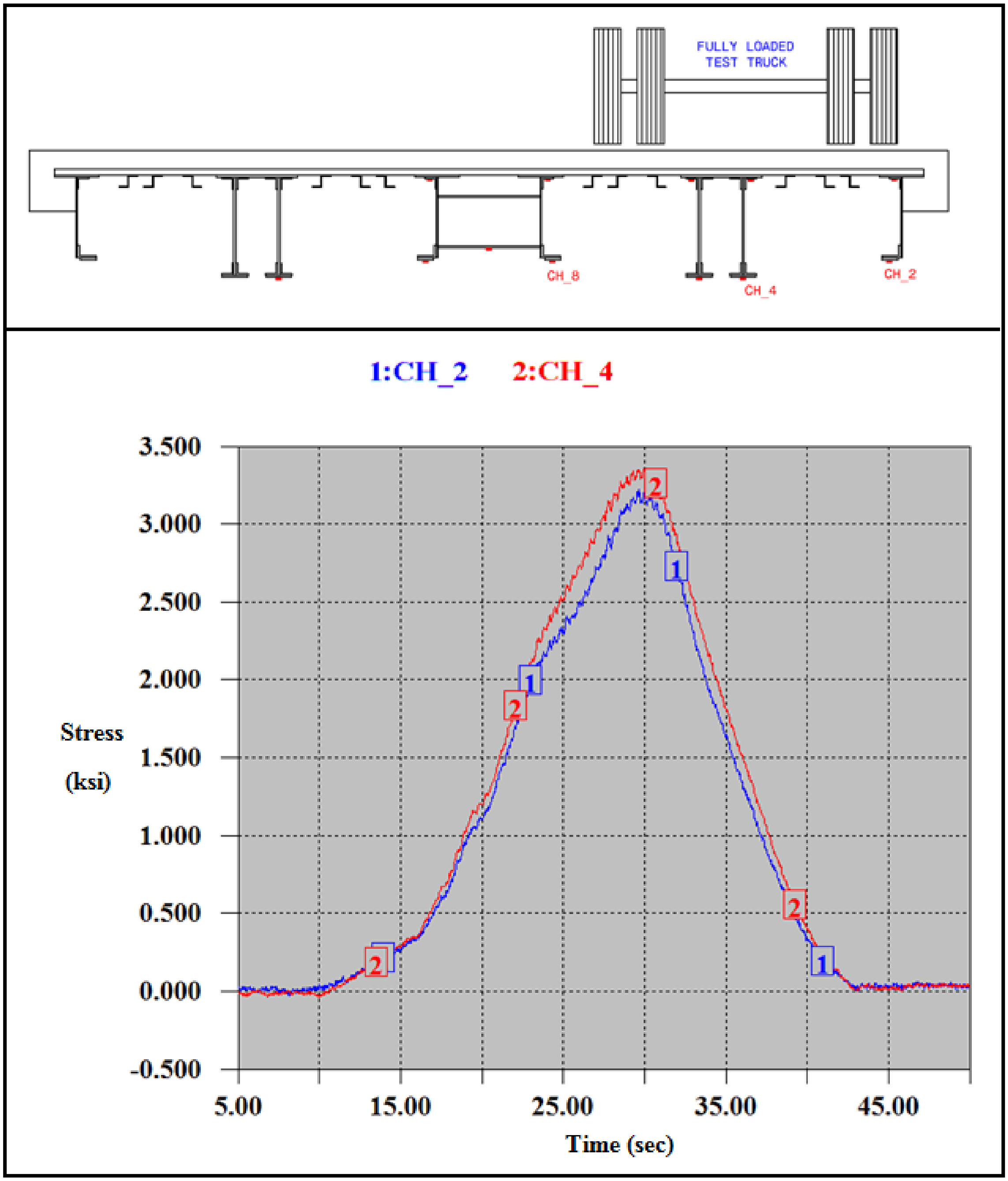

Figure 5.18: VE-24 secondary members DOWN crawl test results

As shown in Figure 5.18, the bottom flange of the exterior girder does not undergo any local bending since no local tension spikes are present in the stress data. It appears the global response of the loaded flatcar is distributed between the main girder and the exterior girder. The stress results show the main girder (CH_4) is carrying slightly more load than the exterior girder (CH_2).

5.2.7. CL-406: DOWN Crawl Test, Empty Test Truck

Figure 5.19 presents the local stress results for the exterior girders and one stringer of the loaded flatcar on CL-406. The test results are from the DOWN crawl test using an empty test truck. All channels shown were installed near midspan.

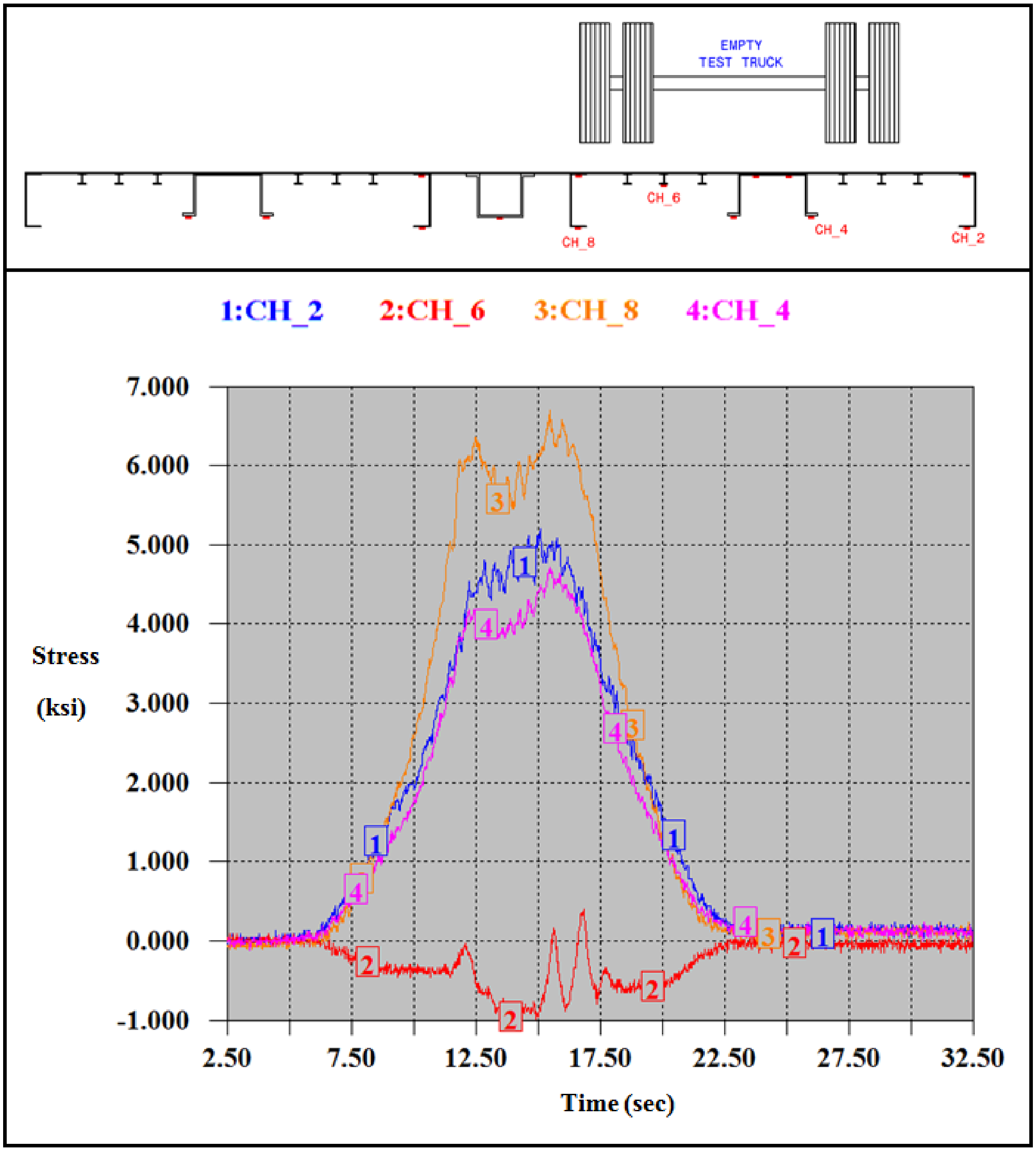

Figure 5.19: CL-406 secondary members DOWN crawl test results

As shown in Figure 5.19, the exterior girders exhibit primarily global bending; however, some local bending is measured. Similar to the results shown in Figure 5.18, the global response appears to be distributed among the main girder and exterior girders. However, the presence of each axle can be seen in the stringers (CH_6). Other local stress results for CL-406 are presented in Appendix C.

CHAPTER 6. DEVELOPMENT OF LOAD RATING GUIDELINES

The proposed load rating guidelines were developed based on the stress data from single lane load tests performed on each of the seven bridges. These guidelines are intended to be a simple, yet accurate, alternative to load rating procedures developed in previous research studies. The guidelines are aimed at establishing the maximum positive live load bending stress to be used when load rating RRFC bridges for a single lane loaded. This live load bending stress can then be used in conjunction with the allowable stress load rating method presented in *AASHTO The Manual for Bridge Evaluation* (AASHTO 2008).

Similar, but separate, load rating guidelines were developed for the primary elements (i.e., main girders) and secondary elements (i.e., exterior girders and stringers). Each of these procedures is presented in the following sections, as well as the background to the development of the procedures. A separate guideline was developed for car hauler type RRFCs and is presented as well.

6.1. Main Girders

The guidelines for determining the live load bending stress on the main girders of typical RRFC bridges can be generally summarized as follows. First, the total live load moment must be calculated for a given bridge. This total moment can be determined based on a beam line model of the bridge or using simple statics. Bridge engineers use this same step when performing load ratings; thus, it is not uncommon to the standard practice.

Once the total moment has been calculated, it must be distributed to each main girder based on an appropriate distribution factor, which was developed based on a single lane loaded. After the total live load moment has been distributed to each main girder, the resulting stresses in the main girder must be evaluated. These stresses are found by using basic bending equations and the section properties of a particular effective section for the main girder. The resulting stress must then be multiplied by a stress modification factor to obtain the final live load bending stress in the main girder. The development of each of these steps, which are specific to typical RRFC bridges, will be discussed in the following sections.

6.1.1. Distribution Factor

As seen in Figure 5.1 - Figure 5.11, and the stress results tables found in Appendix C, the moment (or stress) produced by the test truck is distributed between the two main girders of RRFC bridges. This load distribution was most notable during the load tests where the test truck was positioned in either the left or right driving lane. An example of this distribution can be seen for the UP load test on FO-54 (Figure 5.2). Results from that test are shown in Table 6.1, which is a replica of Table 5.2 repeated for convenience.

Table 6.1: Load distribution on FO-54 for UP load test

Bridge	**FO-54**
Load Test	**UP, Crawl**
Channel	**Stress (ksi)**
3	-2.2
4	4.5
5	-2.3
9	-4.5
10	13.8
16	-4.8

The stresses in Table 6.1 show the loaded girder carries a large portion of the load (or stress); however, some load is distributed to the unloaded girder. In order to determine the portion of the live load moment which is applied to each main girder, a simpler version of Table 6.1 has been created and is presented in Table 6.2.

Table 6.2: Top & bottom flange stresses for loaded & unloaded girders

Bridge	**FO-54**	
Load Test	**UP, Crawl**	
	Loaded Girder	**Unloaded Girder**
Top Flange Stress (ksi)	-4.6	-2.2
Bottom Flange Stress (ksi)	13.8	4.5

The stresses in Table 6.2 were determined based on the results in Table 6.1. Single values for the top flange stresses were obtained for each girder by averaging the two values for each respective top flange and rounding the result to the nearest tenth of a ksi. The actual load distribution factor for the loaded girder can be calculated by the ratio of stress in the loaded girder to the total of stress between the two girders. The actual distribution factor for the bottom flange stresses is presented below.

$$DF_{act,bot} = \frac{13.8\ ksi}{13.8\ ksi + 4.5\ ksi} = 0.75$$

Similarly, the actual distribution factor for the top flange stresses is as follows:

$$DF_{act,top} = \frac{4.6\ ksi}{4.6\ ksi + 2.2\ ksi} = 0.68$$

The actual distribution factor for the loaded girder could then be taken, conservatively, as $DF_{act} = 0.75$.

It should be noted that using a stress ratio, as done here, is not always the same as using a moment ratio. The ratio of moments is the actual distribution factor, not the stress ratio. In this case, a stress ratio can be used because the stress ratio and the moment ratios are the exact same. This is because the two flatcars have the same dimensions, and thus the same section modulus. However, if the two flatcars were of different cross-section, the stress ratio and moment ratio would differ.

As will be shown, the lever rule, as presented in the AASHTO Specifications (AASHTO 2010), can be used to reasonably and conservatively predict the distribution factor between the two main girders. A diagram to be used for calculating the lever rule distribution factor for the loaded girder for the same load case is shown in Figure 6.1.

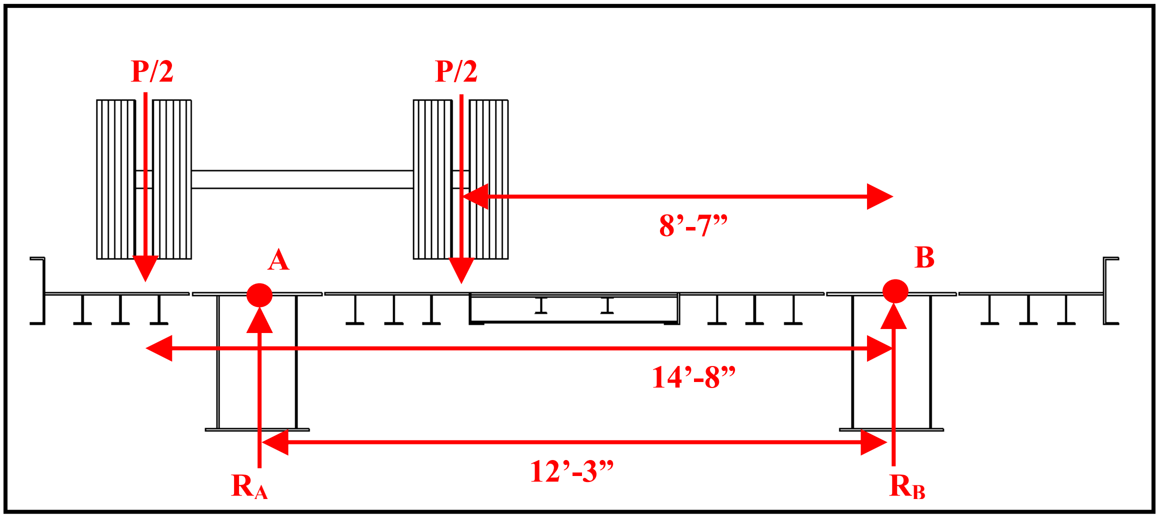

Figure 6.1: FO-54 UP load test used for calculating lever rule

Figure 6.1 shows the necessary dimensions to be used when calculating the lever rule for the UP test for FO-54. The reactions at each of the main girders are located at the centerline of the box girders. For this case, the test truck is assumed to have a total load of P, with half of the load going to each wheel line. The test truck location and the distance between the two main girder reactions are shown in Appendix B.

The lever rule distribution factor is determined by calculating the reactions at each of the main girders. The reaction at the left main girder, R_A, is calculated by summing moments about point B, shown as follows:

$$R_A = \frac{\left(\frac{P}{2}\right)(14' - 8") + \left(\frac{P}{2}\right)(8' - 7")}{(12' - 3")} = 0.95\,P = DF_{LR}$$

When comparing the actual distribution factor, $DF_{act} = 0.75$, and the distribution factor determined by the lever rule, $DF_{LR} = 0.95$, the results are relatively similar. The lever rule result is approximately 21% greater than the measured result, which is reasonably similar. It should also be noted the lever rule result is greater than the actual distribution factor. This shows the lever rule is a reasonably conservative method for determining the live load distribution factor of the main girders. Table 6.3 presents the comparison of the actual and lever rule distribution factors for the loaded girders of FO-54 and the five other “typical” RRFC bridges.

Table 6.3: Comparison of actual & lever rule DF

Bridge	Loaded Girder DF		% Difference
	DF_{act}	DF_{LR}	
FO-54	0.75	0.95	+21%
CL-53	0.69	0.87	+21%
FO-256	0.67	0.72	+7%
FO-25	0.84	0.96	+13%
VE-24	0.82	1.0	+18%
CL-179	0.88	0.86	-2%

The actual distribution factors shown in Table 6.3 are the maximum distribution factor values in each respective bridge. In each of the six bridges shown in the table, except for CL-179, the lever rule distribution factor is reasonably conservative. As seen in the table, for CL-179 the lever rule slightly under predicted the distribution factor measured from the controlled load testing. It is noted that using the lever rule for this case would be slightly unconservative; however, this unconservative result will be accounted for with the combination of the effective section and stress modification factor, which will be discussed in the following sections.

6.1.2. Effective Section

After the total live load moment has been distributed to the main girders, the stresses in those girders must be calculated. In order to calculate the stresses, an effective section must be selected for the main girder. When determining the effective section, it must be known how many, if any, of the stringers are participating in the global bending of the main girders. To evaluate how many stringers are participating, section properties were calculated for various effective sections of a RRFC bridge. An example of this calculation will be shown for FO-54.

Using the dimensions of FO-54 at midspan, found in Appendix B, the moment of inertia, neutral axis, and corresponding section modulus were calculated using simple mechanics of materials for the following effective sections: (a) only main box girder, (b) main box girder and 1 stringer on each side, (c) main box girder and 2 stringers, (d) main box girder and 3 stringers, (e) entire flatcar. These section properties for each effective section are shown in Table 6.4.

Table 6.4: Section properties for various effective sections on FO-54

Section	I (in^4)	y_{bot} (in)	y_{top} (in)	S_{bot} (in^3)	S_{top} (in^3)
Main box	8,672	17.5	13.0	497	665
1 stringer	10,294	19.9	10.6	516	974
2 stringers	11,200	21.3	9.2	526	1,217
3 stringers	11,885	22.3	8.2	533	1,446
Entire car	13,002	23.4	7.1	556	1,823

As seen in Table 6.4, the moment of inertia gradually increases as the effective section increases, as expected. When looking at the neutral axis location with respect to the bottom flange, its position is relatively close to the bottom flange of the stringers. This explains why the moment of inertia does not increase rapidly as additional stringers are added to the effective section.

To obtain moments, the bottom flange main girder stress results from one of the load tests were then multiplied by the section modulus for each of the previously discussed effective sections. The moments in both girders were then summed to come up with the total moment acting on the bridge. This process was repeated for each of the three load tests performed on FO-54. The same procedure was then repeated for the top flange stress results. The results from these operations are shown in Figure 6.2.

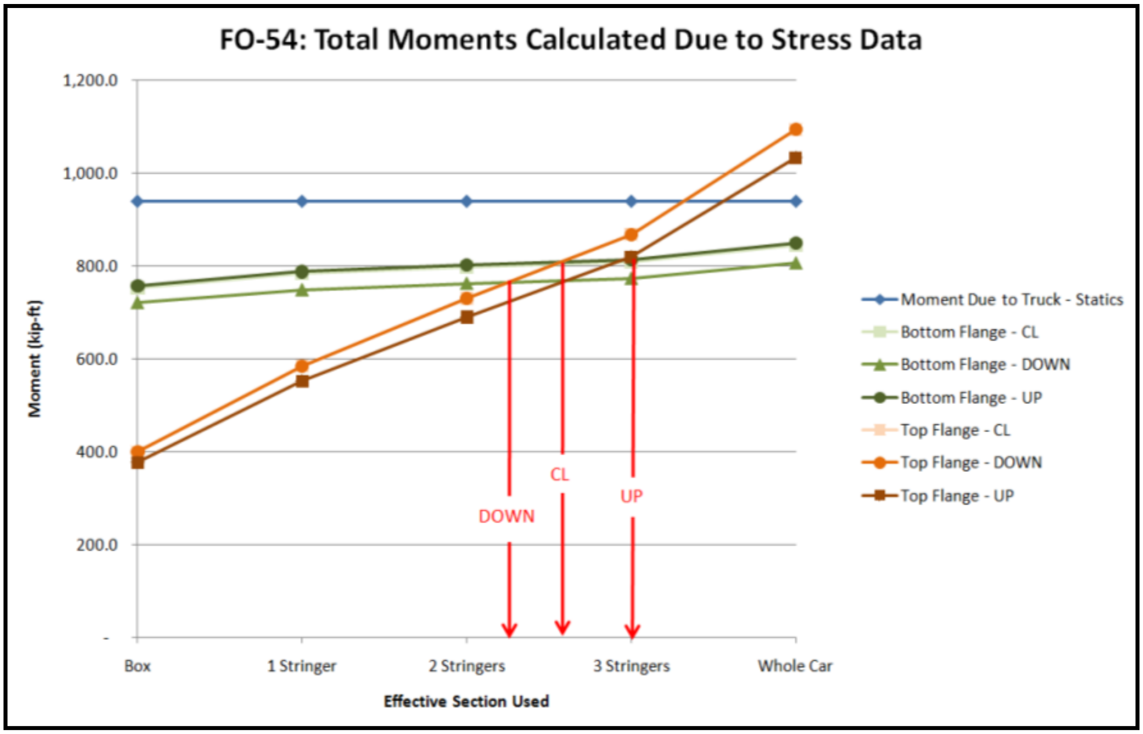

Figure 6.2: Total moments due to varying effective sections on FO-54

As seen in Figure 6.2, connecting lines were then drawn between each of the total moments calculated for a given load test. An interesting feature to note is the total moments calculated from the bottom flange stresses remain more constant as the effective section increases than those compared to the top flange stress results. This difference can be attributed to the relative difference in section modulus when referenced from either the top or bottom flange.

Another feature to note in the figure is although the moments calculated from the CL load test results, for both top and bottom flanges, appear to be missing, they are actually hidden behind the results from the UP load test. This is because the two load tests produced extremely similar moment results.

The downward (red) arrows included on Figure 6.2 are located where, for a given load case, the moments calculated from the top and bottom flange stresses are equal. For the three load cases performed on FO-54, these locations occurred when the effective section included the main box girder plus either 2 or 3 stringers. To be conservative, an effective section of the main girder plus 2 stringers is recommended. This would result in slightly larger stresses being calculated when performing a load rating; hence, a reasonably conservative approach.

The results presented in Figure 6.2 provided reasonably good agreement with those found in all of the other typical RRFC bridges with smaller exterior girders, except for VE-24 which has large exterior girders and includes a composite concrete deck. In some cases, the effective sections were shown to be larger than the “2 stringer” sections; however, it is suggested this effective section be used for all RRFC bridges without composite concrete decks. In one bridge (FO-25), an effective section was shown to be equal to the box girder only. In this case, using the “2 stringer” effective section would seem unconservative. This difference will be accounted for in the section modification factor, presented Section 6.1.3. Figures similar to Figure 6.2 and resulting rationale for each of the other RRFC bridges, except for VE-24, are presented in Appendix D.

For RRFC bridges with large exterior girders, such as VE-24, a different effective section is recommended. Figure 6.3 presents results similar to those found in Figure 6.2 but for VE-24.

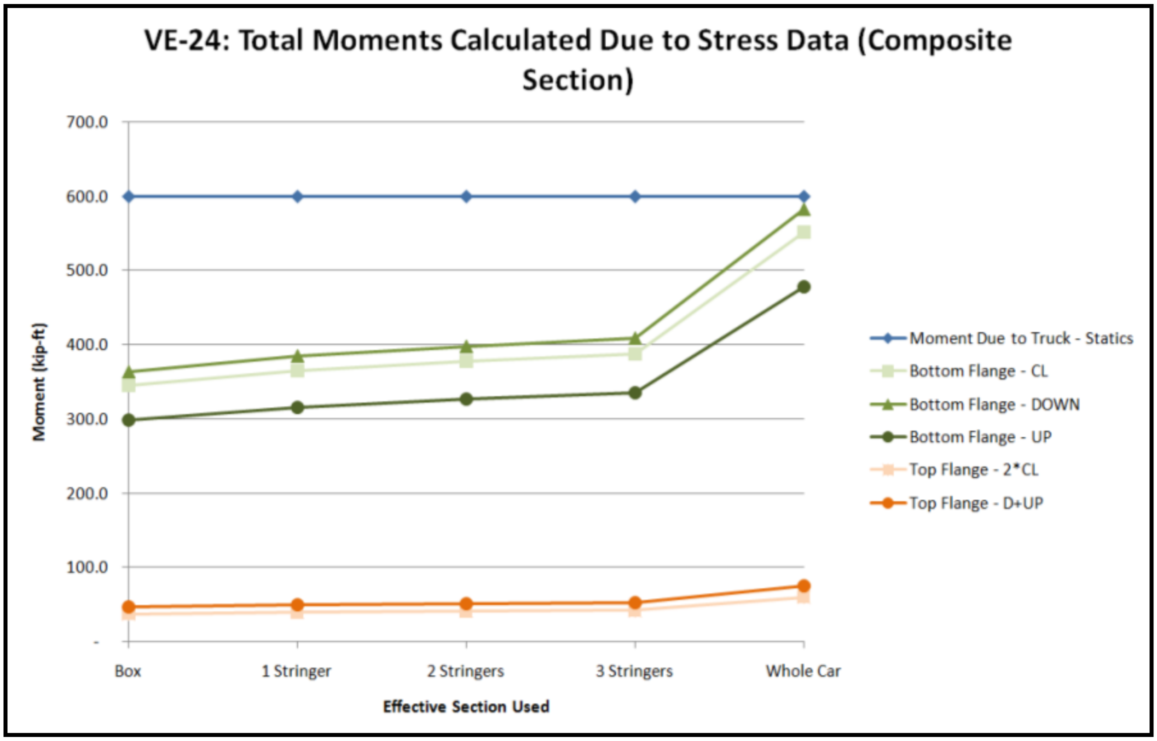

Figure 6.3: Total moments due to varying effective sections on VE-24

In Figure 6.3, the effective sections shown include the composite section properties of the concrete deck. An assumed, and widely accepted, modular ratio of 8 was used to transform the concrete deck to an equivalent steel section. As shown in the figure, the moments calculated due to the top flange stresses are never equal to those produced by the bottom flange stresses. This suggests some composite interaction is occurring between the RRFC and the concrete deck.

An effective section of the entire flatcar, including the composite concrete deck, was selected for VE-24 based on the moments produced from the bottom flange stresses. As shown in Figure 6.3, the moments produced from the bottom flange stresses (green lines) are relatively similar to the maximum moment produced from the test truck (blue line) when the entire section is included. Therefore, the entire flatcar plus the composite concrete deck would be the recommended effective section for RRFC bridges which contain large exterior girders. Large exterior girders are defined as those having a moment of inertia of at least 15% of the moment of inertia as the main girder. The rationale for having a different effective section is that when large exterior girders are present, load is distributed outward towards those girders instead of the main girder carrying the majority of the load.

The 15% limit was chosen based on comparing the moments of inertia of the exterior girders to the main girder for five of the RRFC bridges tested. CL-179 was not included because it contained the rolled steel plate deck stringers instead of exterior girders. Since only one bridge with large exterior girders was instrumented, six other single span bridges tested by Iowa State University (Wipf et. al. 1999; Wipf et. al. 2003; Wipf et. al. 2007a) are also included in Table 6.5. The moments of inertia for each of the Iowa State University bridges were determined using drawings provided in their respective reports.

Table 6.5: Moment of inertia comparison of large exterior girders

University	Bridge	Inertia (I_x, in^4)		I_{ext} / I_{main}
		Main Girder	Exterior Girder	
Purdue	**FO-54**	8,672	231	3%
Purdue	**CL-53**	8,390	305	4%
Purdue	**FO-256**	10,863	203	2%
Purdue	**FO-25**	16,619	222	1%
Purdue	**VE-24**	15,513	3,585	23%
Iowa State	**BCB1**	8,159	2,552	31%
Iowa State	**BCB2**	8,670	2,552	29%
Iowa State	**TCB1**	9,784	1,421	15%
Iowa State	**DCB**	8,999	344	4%
Iowa State	**BCB3**	8,999	344	4%
Iowa State	**WCB2**	8,999	344	4%

When comparing ratio of the moments of inertia of the exterior girders to the main girders of the eleven bridges, there is a clear distinction between large and small exterior girders relative to their main girder. Four bridges (Purdue: VE-24, Iowa State: BCB1, Iowa State: BCB2, and Iowa State: TCB1) all have exterior girders with a moment of inertia of at least 15% that of the main girder. Each of the other bridges shown in Table 6.5 have much smaller moment of inertia ratios, with the maximum being 4%.

(It should be noted that when visually examining a RRFC bridge, either in the field or in a photograph, it can easily be determined whether or not an exterior girder should be considered large or small.) Using the results of Figure 6.3 and Table 6.5, it is recommended that for RRFC bridges which contain exterior girders with a moment of inertia of at least 15% of the main girder, the effective section should consist of the entire flatcar.

In addition using the entire flatcar as the effective section in VE-24, the concrete deck properties were also included in the section. In order to utilize the composite deck properties, it must be shown that the deck is, in fact, composite with the flatcar. As seen in Appendix B, VE-24 is constructed from a RRFC made of riveted built-up members. The rivet heads on the top flanges of the flatcar extend up into the concrete deck. In previous research studies, it has been shown that even the presence of rivet heads extending into a concrete deck is enough to make it composite with the superstructure (Bowman et. al. 2010).

6.1.3. Stress Modification Factor

The stress modification factor is applied to the main girder stresses as calculated through the use of the appropriate distribution factor and effective section. The purpose of the stress modification factor is to more accurately match the stresses measured during the load tests with the calculated stresses (i.e., a calibration factor).

The development of the stress modification factor will be explained through the example of FO-54. The difference in moments to be corrected with the stress modification factor is shown in Figure 6.4, which presents similar results to those shown in Figure 6.2 and Figure 6.3.

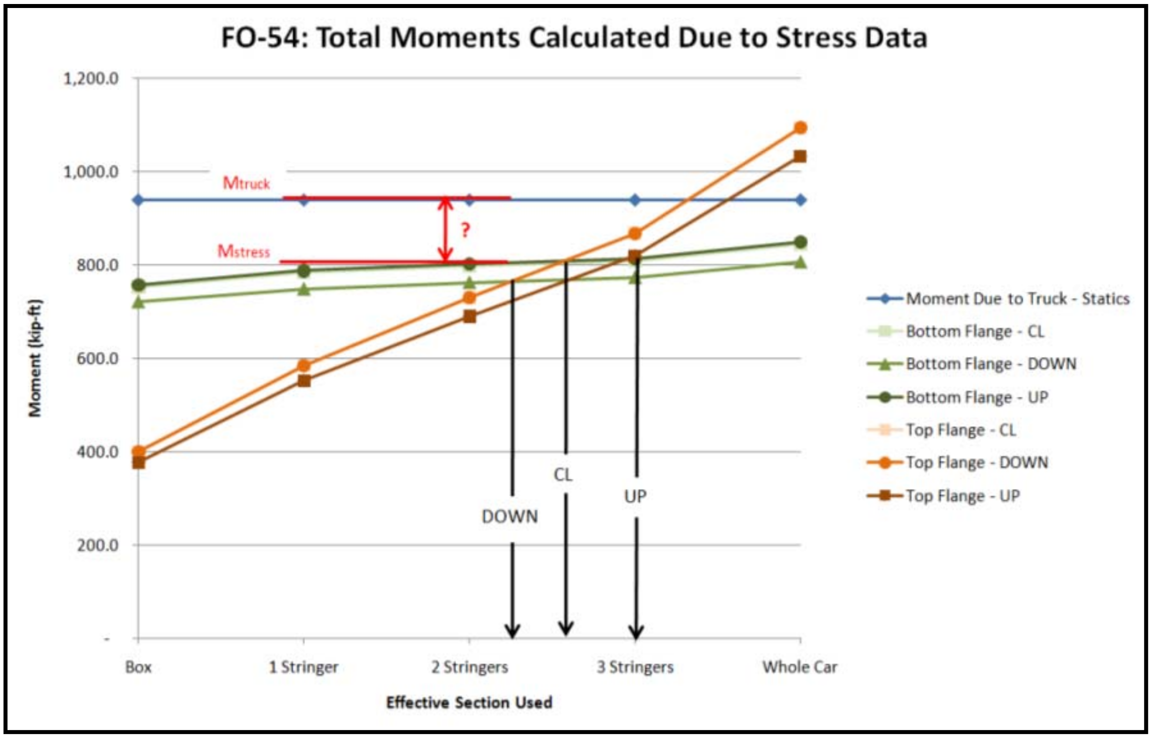

Figure 6.4: Stress modification factor correction for FO-54

As seen in Figure 6.4, the maximum moment due to the test truck, M_{truck}, used in FO-54 is always greater than the moment calculated from the actual stress results, M_{stress}, both of which are shown (in red) on the figure. The difference between the moments is most likely due to the distribution of moment to additional bridge elements. In other words, if stresses were calculated due to the moment produced by the test truck, these stresses would be greater than the actual measured stresses. This behavior would be expected due to the previously discussed load distribution. These results were typical of the other RRFC bridges, as seen in Appendix D.

Figure 6.4 shows the difference between M_{truck} and M_{stress} depends on which effective section is used. As more stringers are included in the effective section, the difference between these two values decreases; therefore, the effective section must be known when determining the stress modification factor. As stated Section 6.1.2. for typical RRFC bridges with small exterior girders, the recommended effective section is the main girder plus 2 stringers on each side of the main girder, and for typical RRFC bridge with large exterior girders, the recommended effective section is the entire flatcar. Using these recommended effective sections, along with using the lever rule as the distribution factor, stresses were calculated for each of the typical RRFC bridges for loading cases similar to those used during the field tests. The calculated stresses in the top and bottom flanges were then compared to the stresses measured in the field for the critical loading conditions on each bridge (i.e., the loading conditions which produced the highest stresses). The results are shown in Table 6.6. It should be noted CL-406 was not included because it is a car hauler. The loaded bottom flange gages in FO-25 and CL-179 were not functioning properly during testing and thus are not shown.

Table 6.6: Stress modification factors for typical RRFC bridges

Bridge	Controlling Test	Loaded Flange	Flange Stress (ksi)		σ_{meas} / σ_{calc}
			Calculated	Measured	
FO-54	UP	Bottom	20.4	13.8	0.68
		Top	8.8	4.6	0.52
CL-53	DOWN	Bottom	6.2	3.3	0.53
		Top	2.6	0.9	0.35
FO-256	DECK	Bottom	5.5	3.3	0.61
		Top	4.2	2.0	0.48
FO-256	UP	Bottom	13.5	9.1	0.67
		Top	10.3	5.1	0.50
FO-25	DOWN	Bottom	8.3	5.6	0.68
		Top	5.4	-	-
CL-179	UP	Bottom	6.4	2.9	0.45
		Top	4.6	-	-
VE-24	DOWN	Bottom	5.4	3.2	0.60
		Top	1.9	0.4	0.21

As seen in Table 6.6, for each of the bridges, the calculated stress values are always greater than the stresses measured in the field. Therefore, a stress modification factor can be applied to the calculated stresses to decrease them so they more closely match the measured stresses. The last column in the table shows the ratio between the measured and calculated stress. In other words, these are the values which the calculated stresses can be multiplied by to obtain the measured stresses. As seen in the table, the controlling value for this ratio is 0.68, which was found for the loaded bottom flange in FO-54, and the loaded bottom flange in FO-25. It is recommended that a stress modification factor of 0.75 be used for all types of RRFCs. The value of 0.75 was chosen to provide a slightly conservative, but still reasonably accurate, stress value.

The stress modification factor shown for the loaded bottom flange in Table 6.6 for CL-179 (0.45) is much less than the other bridges shown in the table. The large difference is most likely due to CL-179 having an “unknown” geometry. As can be seen in the bridge drawings found in Appendix B, the stringers and exterior girders were constructed of a rolled steel plate. There was also an asphalt deck surface on the bridge. The combination of these two components made parts of the RRFC inaccessible to inspection and thus making it a structure of “unknown” geometry. The difference could also be due to some transverse load distribution through plate action due to the rolled steel plate. Even though this bridge displayed a different value for the stress modification factor than other bridges, it was not justified to use a different factor for only this bridge. Using a stress modification factor of 0.75 would simply result in a more conservative stress result.

6.1.4. Comparison with Iowa State University Results

The proposed load rating guidelines were used to calculate live load bending stresses for the six simple span RRFC bridges tested by Iowa State University. Three of the bridges consisted of typical RRFCs with small exterior girders three of the RRFCs contained large exterior girders. For each of the six typical RRFC bridges, the proposed guidelines calculated stresses which were reasonably similar to the measured stresses. Tables containing the calculated and measured stresses are presented in Appendix I. The live load stresses calculated using the proposed guidelines produced reasonably conservative results for each of the critical loading cases (i.e., trucks positioned in one lane to produce highest stresses) for each of the six bridges.

6.2. <u>Exterior Girders & Stringers</u>

The guidelines for determining the live load bending stress in secondary elements, including the exterior girders and stringers, of typical RRFC bridges was developed to be similar to those presented in Section 6.1. for the main girders. First, the total live load moment on the secondary elements must be established. Then the total moment must be distributed to each of the secondary elements in the section being evaluated. Finally, stresses must be calculated based on an appropriate effective section. The development of each of these steps will be discussed in the following sections.

6.2.1. Total Live Load Moment & Effective Section

The total live load moment and effective section were dependent on each other, and therefore their development will be discussed simultaneously. Their development will be discussed through the use of an example performed on FO-54.

The total live load moment on FO-54 was estimated in two ways using basic bending equations, shown as follows:

$$M_{LL,pin} = \frac{P\ L}{4}$$

$$M_{LL,fix} = \frac{P\ L}{8}$$

where:

P = wheel load weight; taken as one half of the axle weight

L = span length of secondary elements; taken as the center to center distance between transverse floor beams

Both pin-pin and fix-fix conditions were considered because although the exterior girders were continuous along the length of the bridge, the stringers were only continuous on either side of the large transverse member located at the center of the RRFC. It was also unknown whether or not the transverse members were stiff enough to act as a support to the secondary elements. The results of the pin-pin and fix-fix conditions for both the front and a single tandem axle are shown in Table 6.7. The span length of the secondary elements can be found in Appendix B.

Table 6.7: Live load moments on secondary elements on FO-54

Axle	Wheel Loads (kip)	Moment (kip-in)	
		$M_{LL,pin}$	$M_{LL,fix}$
Front	7.0	66.8	33.4
Single Tandem	10.6	100.3	50.1

These live load moments were then compared to moments calculated by multiplying selected stress range results by an appropriate section modulus. An example will be shown for the CL test on FO-54, pictured in Figure 6.5.

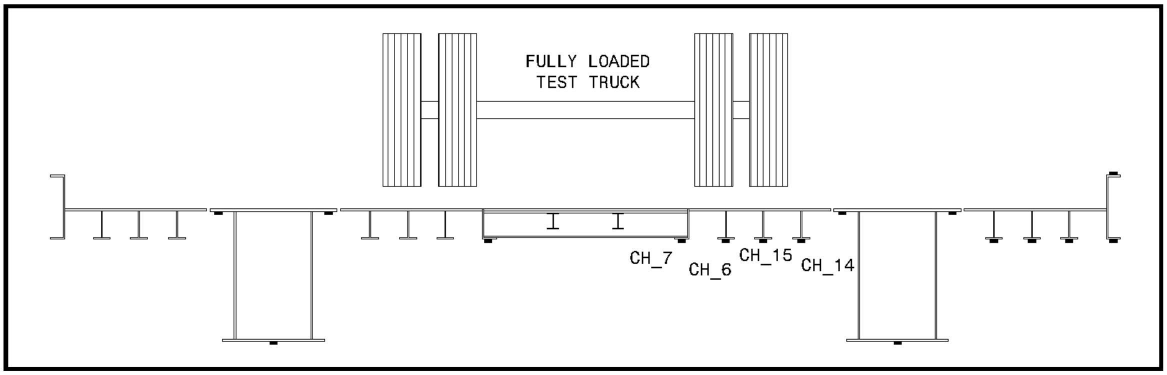

Figure 6.5: CL load test on FO-54

The entire live load moment produced by the right wheel load was assumed to be carried by the three stringers and the exterior girder directly under the wheel. The stress ranges found for each of these four secondary elements were then multiplied by an appropriate section modulus, which will be discussed, to obtain moments for each element. These moments were then summed across the four elements to come up with a total moment. According to the previously mentioned assumption, this moment should be equal to the moment produced by the wheel load.

To determine the section modulus of each element, an appropriate effective section was selected. For the stringers, the effective section consisted of the inverted T-shape and a portion of the steel deck with a width equal to the width of the bottom flange of the T-shape. For the exterior girder, which was cut to form the longitudinal connection, the effective section consisted of the cut channel plus a portion of the steel deck with a width equal to the width of the bottom flange of the channel.

Using these effective sections, the section modulus of each element was calculated. These values were then multiplied by the stress ranges due to both the front axle and a single tandem axle in each respective channel during the CL load test. The results are shown in Table 6.8 and Table 6.9.

Table 6.8: Total moment on secondary elements due to front axle in CL load test on FO-54

	CH 7	CH 6	CH 15	CH 14	
Δf_{FA} (ksi)	1.1	1.4	4.4	1.1	
Inertia (in^4)	38.5	39.6	39.6	39.6	
y_{bot} (in)	3.7	3.6	3.6	3.6	
S_{bot} (in^3)	10.5	11.1	11.1	11.1	**Total**
Δf * S (kip-in)	11.6	15.6	48.9	12.2	**88.3**

Table 6.9: Total moment on secondary elements due to single tandem axle in CL load test on FO-54

	CH 7	CH 6	CH 15	CH 14	
Δf_{STA} (ksi)	2.7	3.1	3.9	1.3	**Total**
Δf * S (kip-in)	28.4	34.5	43.4	14.5	**120.7**

In both the front axle and single tandem axle cases, the moments calculated from the stress data were greater than either the pin-pin or fix-fix conditions, shown in Table 6.7. Since the moments calculated from the measured stresses had better agreement to the pin-pin conditions, the pin-pin condition is recommended to be used when calculating the total live load moment on secondary elements. It is understood that using the pin-pin condition yields a smaller moment on the section than was determined based on the stress results, which is unconservative. However, this difference is accounted for when developing the distribution factor, which will be discussed in Section 6.2.2.

A similar process was performed using the results from the DOWN load test on FO-54. One important note for the DOWN test is that the exterior girder was not cut since it was located on the outside of the bridge. Hence, for this case, the entire structural channel was chosen as the effective section. None of the steel deck was included since it was located near the centroid of the section and therefore would not contribute much resistance.

The DOWN test results produced similar results to the CL test, in that the moments determined from the stress data were larger than those calculated from the wheel loads. Since the results were similar, it is recommended that the pin-pin equation be used for calculating the total live load moment, and the appropriate effective section be used for determining section properties.

Unfortunately, FO-54 was the only bridge in which all secondary elements were instrumented on one side of the main girder. In the other bridges, typically an exterior girder and one stringer were instrumented so the total moment over the appropriate secondary elements could not be calculated. For these other bridges, the live load moment and effective sections were chosen to be the same as FO-54 since no justification could be made for using a different method.

6.2.2. Distribution Factor

The distribution factors for the secondary elements were developed using procedures as discussed in the previous section. For each of the typical RRFC bridges, the total live load moment acting on a group of secondary elements was calculated using the pin-pin moment equation. Stresses were then calculated for each element using the section properties from the appropriate effective section. The stress in each element was calculated based on the total moment acting on each element. Ratios of the measured stress divided by the calculated stress ranges were then calculated. This procedure was done for the moments produced by the front axle and a single tandem axle loading. The single tandem axle loading controlled, and therefore will be discussed.

The ratios were grouped into two categories. The first category consisted of secondary elements which were "much stiffer" than the other elements within the group of elements. "Much stiffer" was defined as having a moment of inertia at least three times greater than each of the other elements. The rationale for this category was that if one element was much stiffer than the remaining elements, it would attract the majority of the load.

The second category consisted of secondary elements having approximately equal stiffness. These elements all had moments of inertia which were less than three times that of the other elements within the group. The rationale for this category was if all elements had a similar stiffness, the load would be better distributed between the elements. The results for these two categories are shown in Table 6.10 and Table 6.11.

Table 6.10: Distribution factors for "stiff" secondary elements

Bridge	Load Test	Flatcar, Stringer Group[1]	CH	Secondary Element Type	$I_{element}$ / I_{others}[2]	σ_{meas} / σ_{calc}[3]
CL-53	DOWN	R, R	2	Uncut Exterior Girder	6	0.40
FO-25	CL	R, L	8	Uncut Exterior Girder	3[4]	1.00
	UP	R, L	8	Uncut Exterior Girder	3[4]	0.80
	UP	R, R	2	Uncut Exterior Girder	3[4]	0.65
FO-54	DOWN	R, R	2	Uncut Exterior Girder	5	0.40
FO-256	CL	R, L	7	Cut Exterior Girder	3	0.60
	CL_2	R, L	7	Cut Exterior Girder	3	0.70

[1]Refers to either right or left flatcar shown in load test positions, and either right or left group of secondary elements.
[2]Moment of inertia of element in question divided by moment of inertia of other secondary elements within group, rounded down to nearest whole number.
[3]Rounded up to nearest 0.05.
[4]FO-25 had three different sizes of elements within a group. The controlling inertia ratio was found to be three for one other element type, and was equal to 10 for another element type.

Table 6.11: Distribution factors for "typical" secondary elements

Bridge	Load Test	Flatcar, Stringer Group[1]	CH	Secondary Element Type	σ_{meas} / σ_{calc} [2]
CL-53	DOWN	R, L	8	Cut Exterior Girder	0.50
	UP	L, R	9	Cut Exterior Girder	0.45
	DOWN	L, R	9	Cut Exterior Girder	0.55
	DOWN	R, L	6	Cut Exterior Girder	0.45
FO-25	CL	R, L	6	Stringer	0.45
	UP	R, L	6	Stringer	0.40
FO-54	CL	R, L	15	Stringer	0.45
	DOWN	R, R	12	Stringer	0.30
FO-256	CL	R, L	6	Stringer	0.20
	CL_2	R, L	6	Stringer	0.25

[1]Refers to either right or left flatcar shown in load test positions, and either right or left group of secondary elements.
[2]Rounded up to nearest 0.05.

The stress ratios in each of the tables can be taken as the distribution factors applied to the total live load moment, since that is how the calculated stresses were determined. As shown in Table 6.10, the maximum stress ratio for "stiff" secondary elements was found to be 1.0. A distribution factor of 1.0 is therefore recommended for all secondary elements which are at least three times as stiff as other elements within the group. As shown in Table 6.11, the maximum stress ratio for "typical" secondary elements was found to be 0.55. A distribution factor of 3/5 is therefore conservatively recommended for all typical secondary elements. These distribution factors would be applied to the moments produced by a single tandem axle.

In cases where there are at least one "stiff" and at least one "typical" secondary element within a group, both would need to be load rated using their appropriate distribution factors. This is because the local effects are highly dependent on knowing the exact location of the wheel load. Therefore, even though a "stiff" element is present, if the wheel load is directly over the "typical" element, the "typical" element could still experience 3/5 of the moment produced by the single tandem axle.

6.2.3. Non-Typical RRFC Bridge Issues

6.2.3.1. Concrete Bridge Decks

For bridges with composite concrete decks, such as VE-24, it is not recommended that local effects be evaluated when performing a load rating. There were no local tension spikes present in the load test data on VE-24. It is believed the concrete deck provided enough stiffness to prevent any local bending of the secondary elements in this case.

6.2.3.2. "Unknown" Cross Sections

Load rating guidelines for secondary elements were not developed for bridges with cross-sections similar to those used in CL-179. As discussed in Section 6.1.1. CL-179 had an "unknown" cross-section, since it was constructed with a steel plate deck instead of the more typical stringer system. There is a possibility stresses in the deck could be determined using orthotropic plate procedures in the AASHTO Specifications (2010). There is also a possibility the CL-179 cross-section consisted of hollow steel box stringers, rather than the steel plate deck. If this is the case, structural drawings of the RRFC would be needed to evaluate the effective sections and distribution factors for this type of cross-section.

6.3. Car Haulers

Different guidelines were developed for load rating car haulers because they presented unique behavior not seen in typical RRFC bridges. As shown in Figure 5.19, the exterior girders participated in mostly global bending and not the local bending typical to other types of RRFCs. Therefore, instead of only distributing the global live load moment to the main girders, it can be distributed to the exterior girders as well. The guidelines for car haulers were meant to be similar to that of typical RRFCs for the sake of simplicity.

Like typical RRFC bridges, the total live load moment is distributed using the lever rule. However, the difference when performing this operation with a car hauler is the moment is distributed to the loaded car, not only the main girder. Similar to the typical RRFCs, the lever rule provided reasonably good agreement with the measured distribution factor for the car hauler.

Table 6.12: Comparison of actual & lever rule DF for car hauler

Bridge	Loaded Car DF		% Difference
	DF_{act}	DF_{LR}	
CL-406	0.93	0.94	+1%

After the live load moment is distributed to each car, it is distributed within the car to the main girder and exterior girders. These distribution factors within the car, DF_{car}, were developed by first summing the moment over the main girder and the two exterior girders in one car. These moments were calculated by multiplying the stress measured for each element by its section modulus. Section modulus values were obtained using an effective section consisting only of the shape of each element (i.e., welded Z-shapes for main girder and channels for the exterior girders). No stringers were included in any of the effective sections. The moment fraction of each element was then calculated as the moment found in each element divided by the total moment across the car. The results for the bottom and top flange moments are presented in Table 6.13 and Table 6.14.

Table 6.13: Distribution factor within car for bottom flange moments on CL-406

Test	Car	Moments Fractions		
		Inside Exterior Girder	Main Girder	Outside Exterior Girder
FULL	Right	0.49	0.47	0.04
CL	Right	0.48	0.49	0.03
UP	Right	0.56	0.44	0.00
DOWN	Right	0.27	0.51	0.23
FULL	Left	0.47	0.53	0.00
CL	Left	0.44	0.56	0.00

Table 6.14: Distribution factor within car for top flange moments on CL-406

Test	Car	Moments Fractions		
		Inside Exterior Girder	Main Girder	Outside Exterior Girder
FULL	**Right**	0.19	0.72	0.09
CL	**Right**	0.19	0.71	0.10
UP	**Right**	0.35	0.65	0.00
DOWN	**Right**	0.11	0.70	0.19

The moment fractions shown in the tables were used to come up with the distribution factors within a railcar, DF_{car}. For the main girders, a DF_{car} value of 3/4 is recommended. This value is limited by the top flange moment fraction for the FULL load test, and was rounded up to reach a conservative value. For the exterior girders, a DF_{car} equal to 3/5 is recommended. This value is limited by the bottom flange moment fraction in the UP load test, and was also rounded up to be conservative. Similar to the typical RRFC bridges, a stress modification factor of 0.75 was included to more reasonably, and conservatively, match the calculated with measured stresses.

The local effects were studied on CL-406, based on only one strain gage (CH_6). The total live load moment on the stringer was determined based on procedures described in 6.2.1. The effective section consisted only of the stringer itself. Based on these conditions, a conservative distribution factor of the live load moment was found to be 0.06. Since this was determined based on only one strain gage, and the distribution factor seemed interestingly low, it is not recommended to be used when performing a load rating of the secondary elements in a car hauler. The test results for CL-406 showed the stress in the instrumented stringer was always less than the stresses of either the main girder or exterior girders.

CHAPTER 7. RESULTS, CONCLUSIONS, & RECOMMENDATIONS

7.1. Results

As a result of this research, guidelines for the load rating, inspection, and acquisition of RRFCs to be used as low-volume road bridges have been developed and are presented in Appendix E – Appendix G, respectively. The load rating guidelines are focused on determining the live load bending stress on the main girders and secondary members. These guidelines, which are based on field instrumentation and controlled load testing of seven RRFC bridges, are intended to be used in conjunction with the allowable stress method as described in *AASHTO The Manual for Bridge Evaluation* to determine a load rating. The load rating guidelines are meant to be simple, yet accurate, and are intended to be similar to AASHTO Specifications to provide engineers with a degree of familiarity when performing load ratings on RRFC bridges. Example load ratings were also performed on a typical RRFC and a boxcar (car hauler) bridge. These examples are presented in Appendix H.

The inspection and acquisition guidelines were developed based on field visits to 75 RRFC bridges in Indiana and discussions with county personnel having a numerous years of experience with these types of structures. The inspection guidelines are meant to provide bridge inspectors with information about details and areas of RRFC bridges which require special attention during an inspection. The acquisition guidelines are intended to provide county personnel with information such that they can make an informed decision regarding whether or not a RRFC is suitable to be used as a low-volume road bridge.

7.2. Conclusions

Key conclusions as a result of this research have been found and are discussed below:

- The main box girders of typical RRFC bridges experience global bending due to live load moment. These main box girders carry the majority of the load on typical RRFC bridges.

- The lever rule can be used to reasonably and conservatively distribute live load moment between RRFCs connected with a longitudinal connection. Comparisons of the distribution factor determined by field measurements and computed using the lever rule for seven RRFC bridges provided reasonably good agreement.

- The effective section which resists global live load bending for typical RRFC bridges with small exterior girders can reasonably and conservatively be assumed to consist of the main box girder and two stringers on each side of the main girder.

- The effective section which resists live load bending for RRFC bridges with large exterior girders can reasonably and conservatively be assumed to consist of the entire RRFC.

- For RRFC bridges which contain a composite concrete deck, the deck properties can reasonably and conservatively be included in the effective section to resist global live load bending.

- The live load bending stress, as calculated using the proposed load rating guidelines, provided reasonably good agreement to stress results measured by Iowa State University for six single span bridges constructed with typical RRFCs.

- The secondary members of typical RRFC bridges consist of the stringers and exterior girders. These elements experience local bending caused by individual wheel loads.

- Secondary members which are much stiffer (i.e., at least three times as stiff) than other secondary members will attract the majority of the local moment acting on a group of secondary members.

- When used properly, typical RRFCs are a suitable option to be used as low-volume road bridges. The results of the field instrumentation and controlled load testing showed generally good behavior and load distribution on each of typical RRFC bridges tested. It is recommended, however, that in order to be suitable as bridges, RRFCs be supported at the wheel trucks (locations where wheels attach to flatcar).

- "Car haulers" or boxcars are not recommended to be used as bridges. This type of structure, CL-406, did not perform as well during the controlled testing. CL-406 showed larger deflections relative to the other RRFC bridges tested. These types of railcars also exhibit much smaller and less stiff main girders than the typical RRFCs.

- Various types of decks can be effective when used properly on RRFC bridges. Steel plate decks appear to be the most commonly used in Indiana and were observed to generally exhibit good performance. A timber deck was shown to exhibit characteristics similar to that of a steel plate deck. It is unknown if additional stiffness or load distribution was gained through the use of timber decking. Composite concrete decks can provide additional stiffness through the composite section properties. The additional dead load associated with the addition of a concrete deck must be considered.

- Longitudinal connections should be made stiff enough to effectively transfer load between adjacent flatcars. Stiff longitudinal connections can greatly improve the load distribution, and thus the performance of RRFC bridges.

7.3. Recommendations for Future Work

As a result of this research, two recommendations for future research have been discovered. These recommendations are as follows:

- Additional research, including laboratory testing, is required to obtain information regarding the ultimate strength of RRFCs. Results from the field instrumentation and controlled load testing suggest RRFC bridges possess a significant amount of reserve capacity beyond their current load ratings. Laboratory tests could determine the ultimate strength of RRFCs, providing engineers with an additional level of confidence when load rating these structures.

- As a result of speaking to engineers regarding this research project, the question of whether or not RRFC bridges should be labeled as fracture critical has been raised. If RRFC bridges are to be labeled as such, a hands-on inspection would be required of these structures every two years, thereby making them a far less economical bridge structure for county transportation agencies. Laboratory testing would offer a controlled environment to perform testing on damaged and undamaged specimens to assess the level of redundancy RRFCs possess, in turn, answering the question about whether or not these bridges are fracture critical.

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APPENDICES

Appendix A. List of RRFC Bridges in Indiana

County	County Bridge #	Newly Installed Since 2007	Representative Sample	Additional Visits	Field Instrumentation & Load Testing
Adams	1				
Clay	2			X	
Clay	3			X	
Clay	7			X	
Clay	33		X		
Clay	41			X	
Clay	53		X		X
Clay	80			X	
Clay	83			X	
Clay	91			X	
Clay	121	X			
Clay	131			X	
Clay	155			X	
Clay	162			X	
Clay	163		X		
Clay	166			X	
Clay	179			X	X
Clay	191		X		
Clay	192			X	
Clay	204			X	
Clay	205		X		
Clay	207			X	
Clay	213			X	
Clay	214		X		
Clay	226			X	
Clay	228			X	
Clay	232			X	
Clay	237			X	
Clay	247			X	
Clay	269			X	
Clay	270		X		
Clay	311	X			
Clay	400	X			
Clay	401	X			

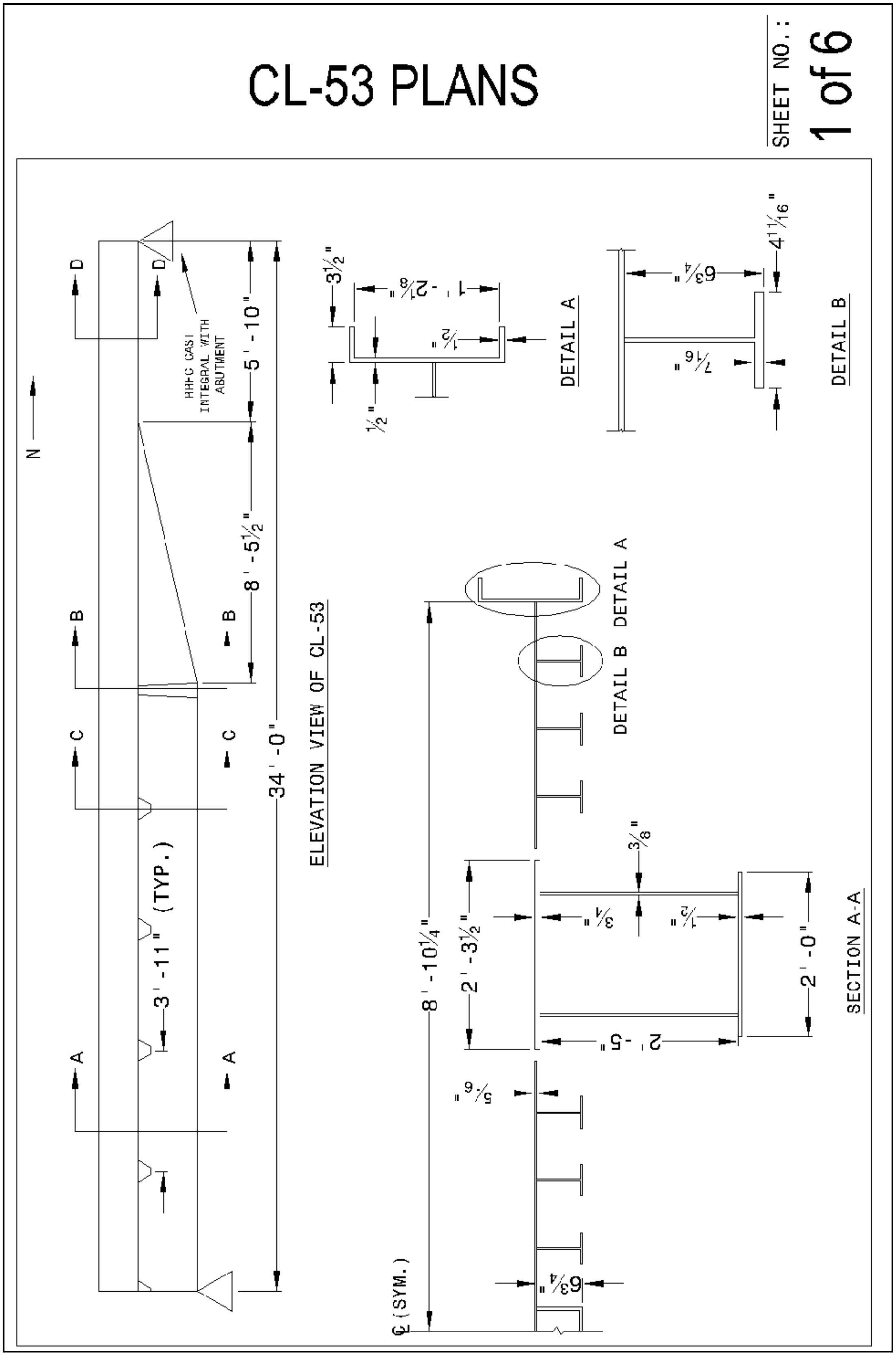
County	County Bridge #	Newly Installed Since 2007	Representative Sample	Additional Visits	Field Instrumentation & Load Testing
Clay	402	X			
Clay	404	X			
Clay	405	X			
Clay	406	X			X
Clay	745			X	
Daviess	118				
Daviess	133				
Daviess	149				
Daviess	153		X		
Daviess	171				
Daviess	176				
Daviess	193				
Daviess	195				
Daviess	217				
Daviess	242		X		
Dubois	14				
Dubois	45				
Dubois	94				
Dubois	110				
Dubois	143				
Dubois	144		X		
Dubois	145		X		
Dubois	146				
Dubois	147				
Dubois	149				
Dubois	259				
Fountain	6			X	
Fountain	14			X	
Fountain	20		X		
Fountain	25			X	X
Fountain	30			X	
Fountain	51		X		
Fountain	54			X	X
Fountain	88			X	

County	County Bridge #	Newly Installed Since 2007	Representative Sample	Additional Visits	Field Instrumentation & Load Testing
Fountain	**256**			X	X
Greene	**49**				
Greene	**142**		X		
Hancock	**56**				
Harrison	**84**		X[1]		
Knox	**27**		X		
Knox	**28**		X		
Knox	**36**				
Knox	**84**				
Knox	**91**				
Knox	**97**				
Knox	**131**				
Knox	**158**				
Knox	**162**		X		
Knox	**182**		X		
Knox	**196**				
Knox	**206**				
Knox	**212**				
Knox	**230**				
Knox	**312**				
Knox	**355**				
Knox	**358**				
Knox	**386**				
Knox	**411**				
Knox	**416**				
Knox	**418**				
Parke	**8**			X	
Parke	**194**		X		
Parke	**214**			X	
Parke	**226**			X	
Pike	**4**				
Pike	**5**				
Pike	**16**				
Pike	**59**				

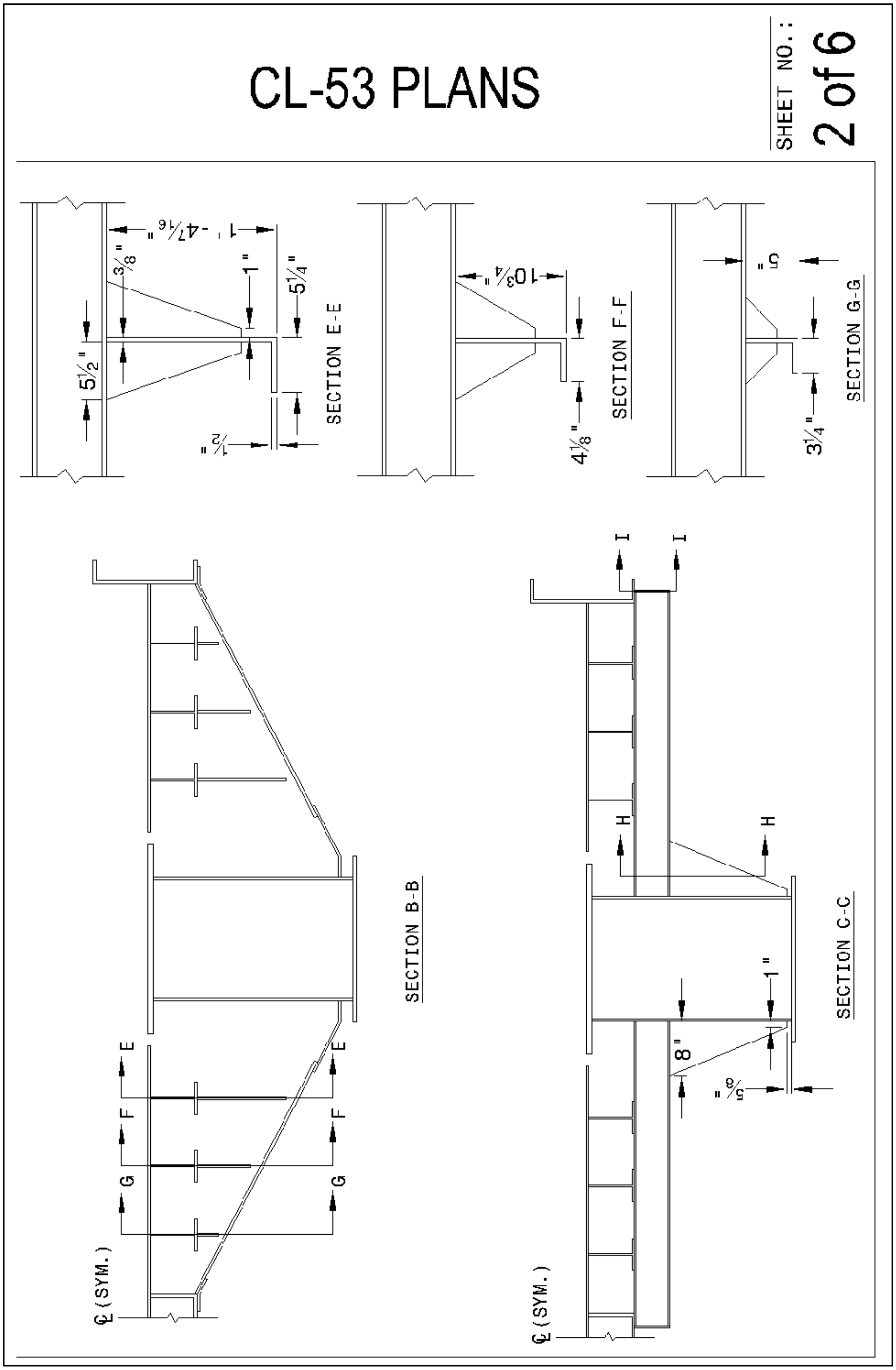
County	County Bridge #	Newly Installed Since 2007	Representative Sample	Additional Visits	Field Instrumentation & Load Testing
Pike	**62**		X		
Pike	**124**				
Pike	**138**				
Pike	**174**				
Posey	**151**				
Rush	**1**				
Spencer	**281**		X		
Sullivan	**8**				
Sullivan	**58**		X		
Sullivan	**72**			X	
Sullivan	**192**			X	
Sullivan	**217**			X	
Vermillion	**24**			X	X
Vermillion	**25**			X	
Warren	**18**			X	
Warren	**21**			X	
Warrick	**34**		X		
Warrick	**123**				
Warrick	**124**				
Warrick	**155**				
Warrick	**164**				
Warrick	**165**		X		
Warrick	**183**				
Warrick	**187**				
Warrick	**200**		X		
Warrick	**204**				
Warrick	**221**				
Warrick	**251**				
Warrick	**373**				
Warrick	**380**				

[1]Bridge was visited as part of Representative sample, but was not included in actual Representative Sample

Appendix B. Bridge Plans, Instrumentation Plans, & Load Test Locations



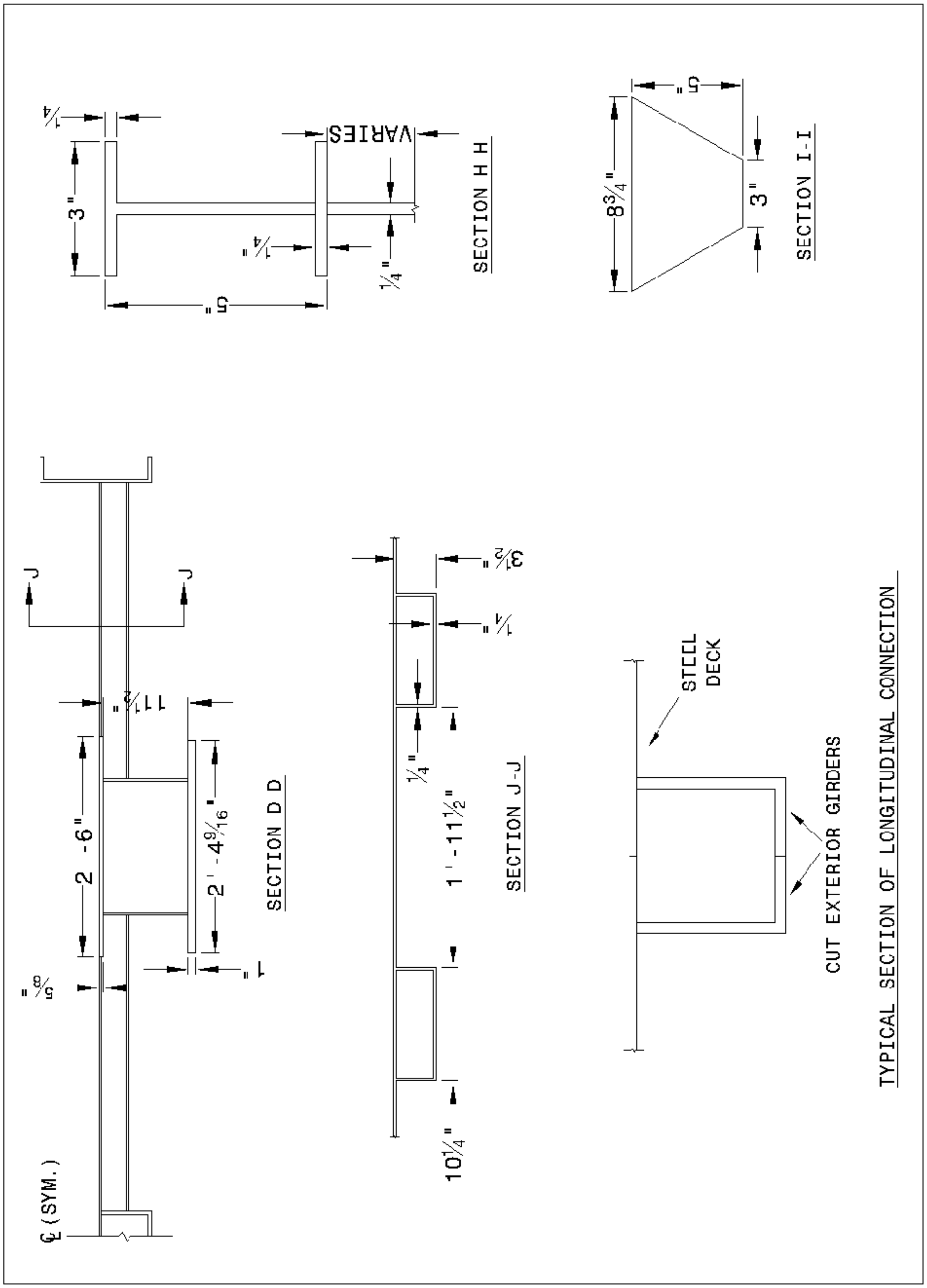
CL-53 PLANS
SHEET NO.:
1 of 6
ELEVATION VIEW OF CL-53
HHFC CAST INTEGRAL WITH ABUTMENT
34'-0"
5'-10"
8'-5½"
3'-11" (TYP.)
N
DETAIL A
DETAIL B
SECTION A-A
8'-10¼"
2'-3½"
2'-0"
2'-5"
℄ (SYM.)

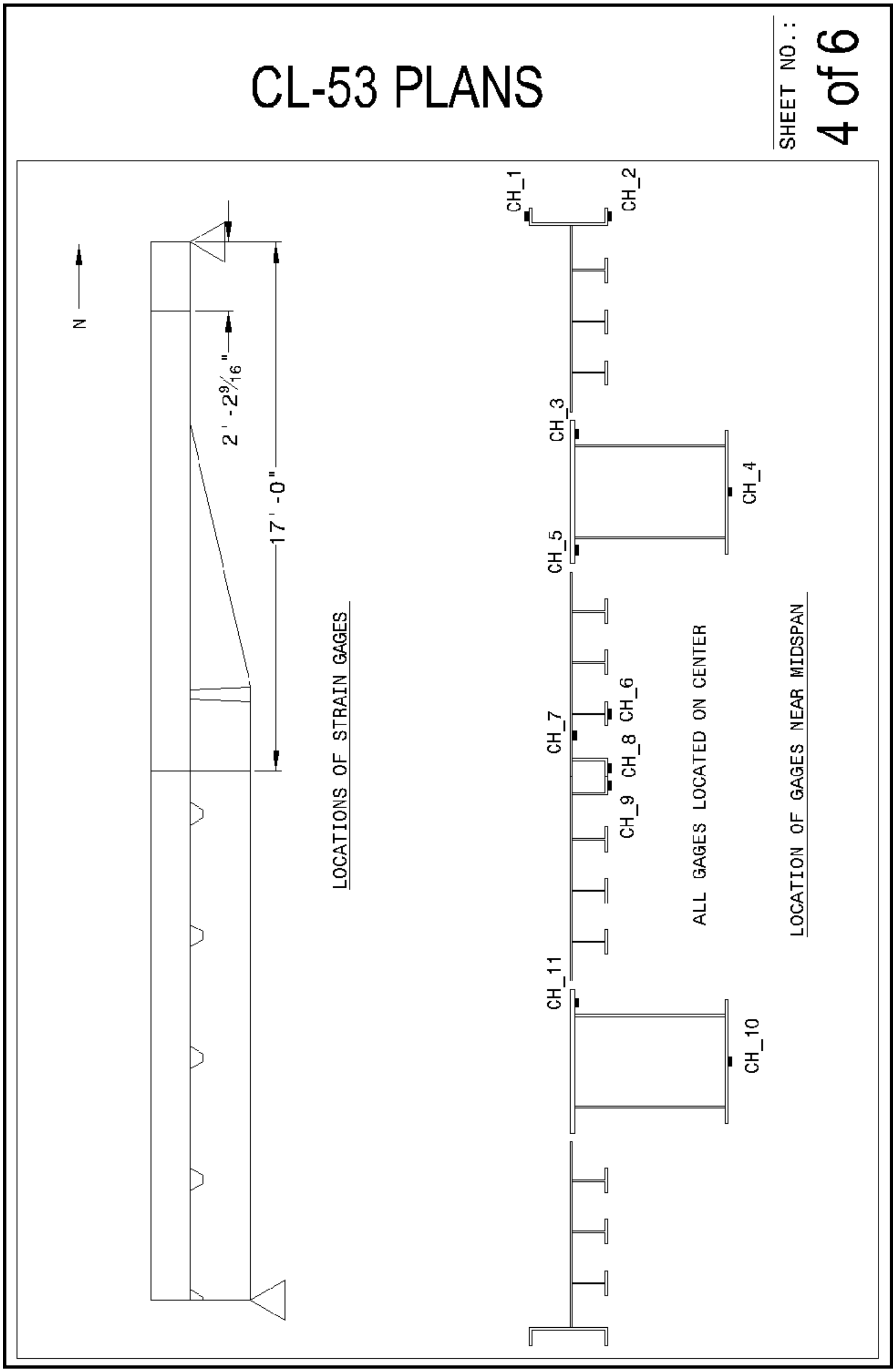


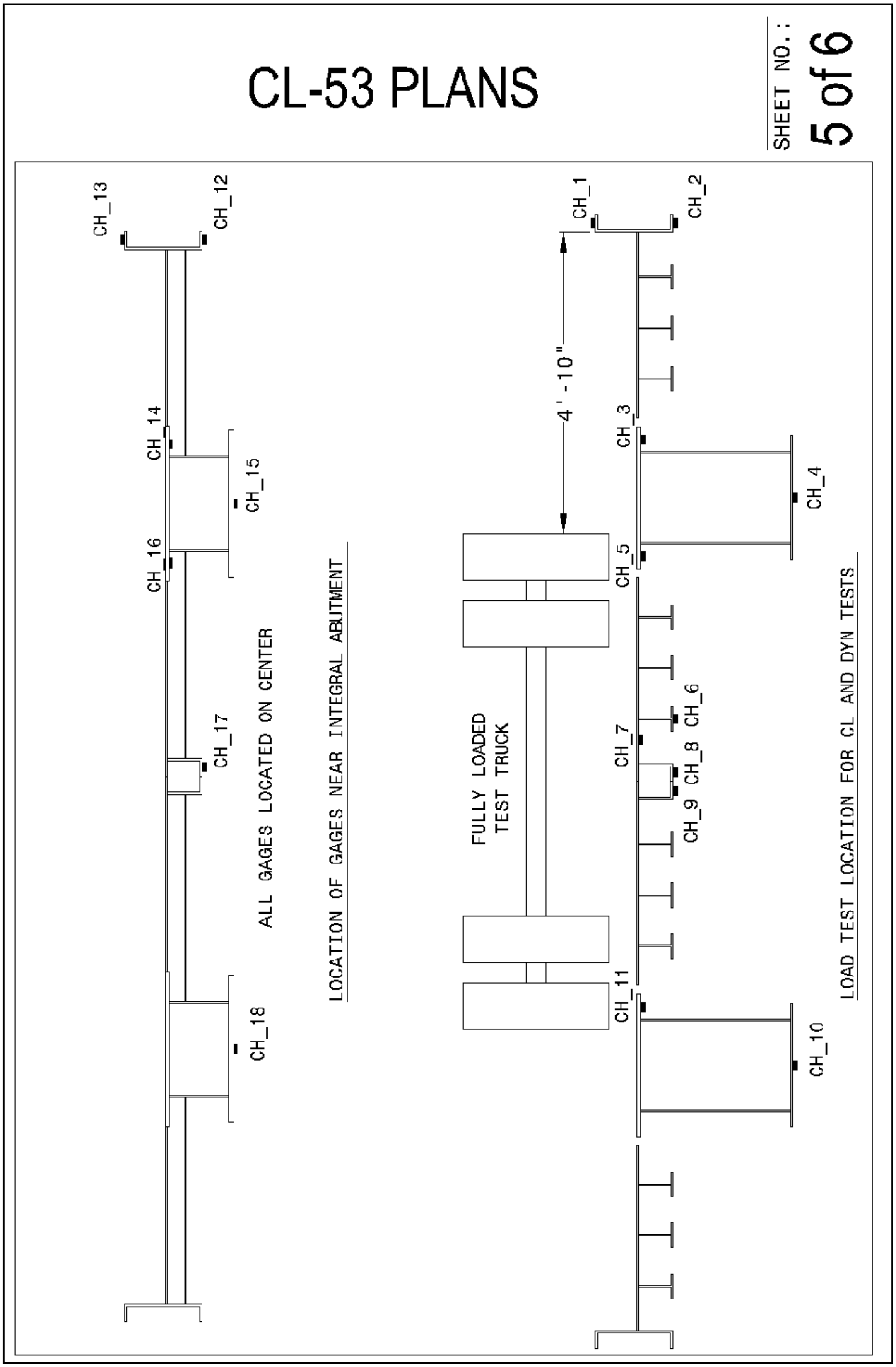
CL-53 PLANS
SHEET NO.:
2 of 6
SECTION E-E
SECTION F-F
SECTION G-G
SECTION B-B
SECTION C-C
1'-4 7/16"
3/8"
1"
5 1/4"
5 1/2"
1/2"
10 3/4"
4 1/8"
5"
3 1/4"
8"
5/8"
℄ (SYM.)
E
F
G
H
I

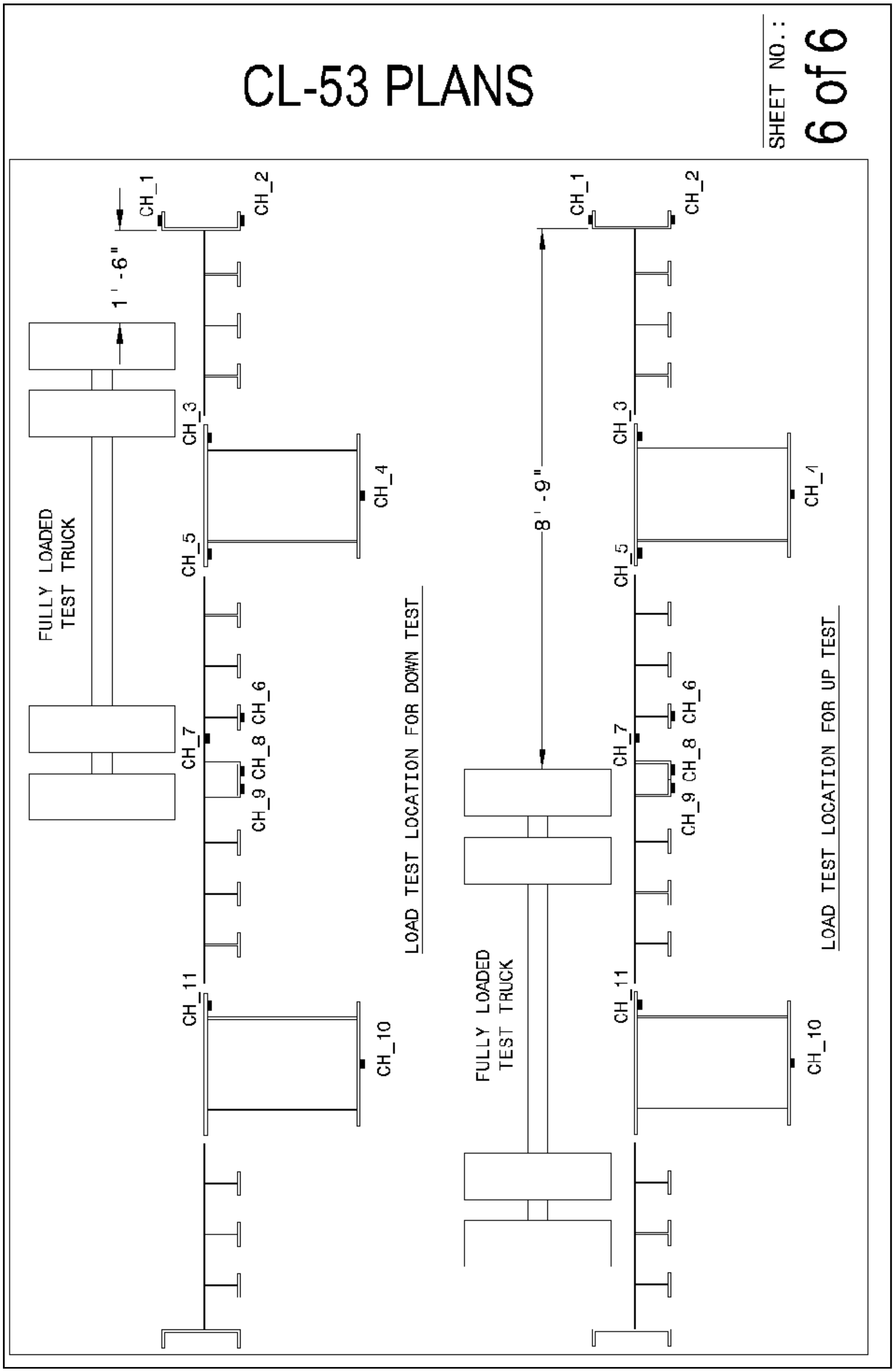
CL-53 PLANS

SHEET NO.: 3 of 6



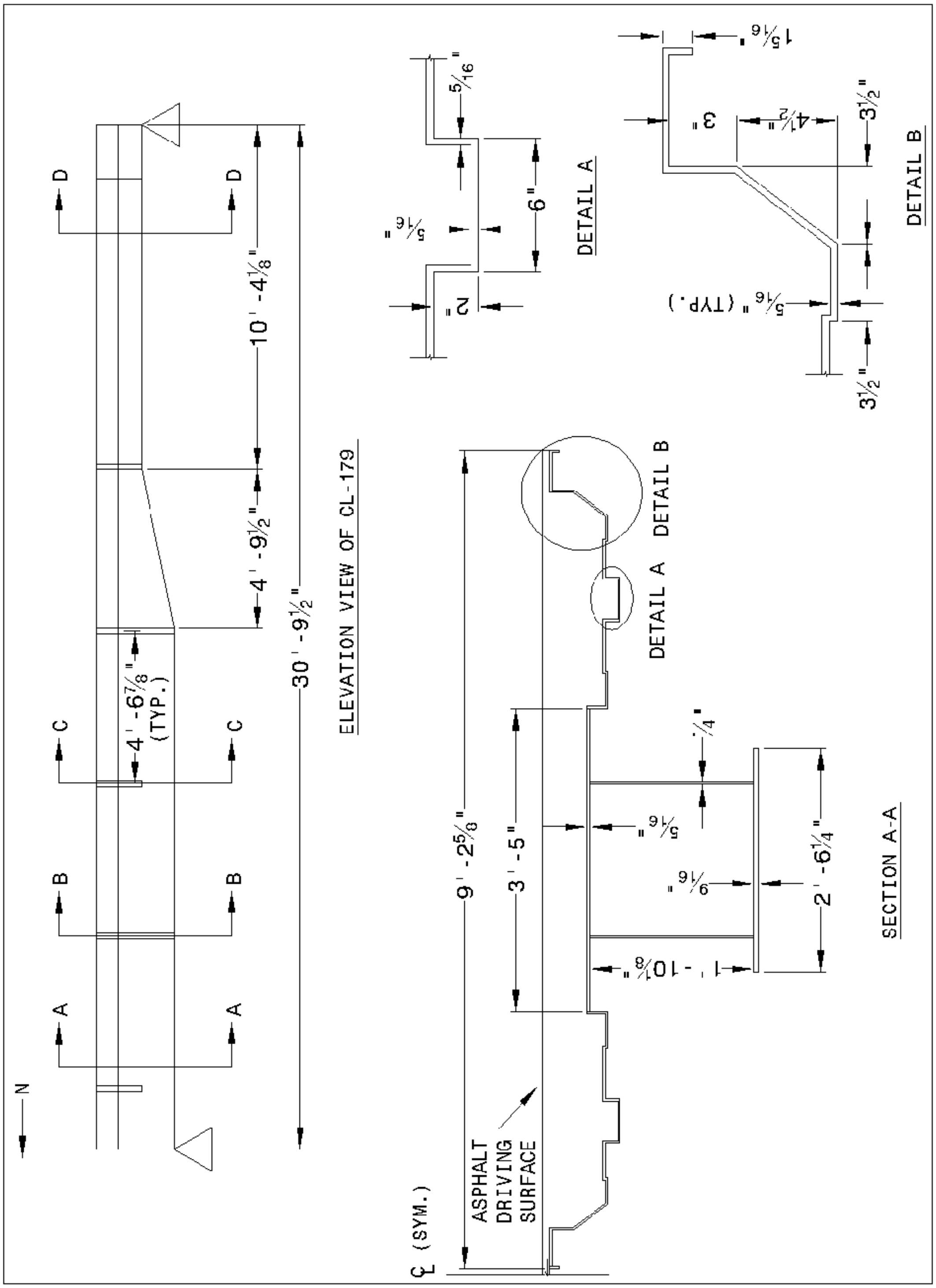

CL-53 PLANS
SHEET NO.:
4 of 6
N
2'-2 9/16"
17'-0"
LOCATIONS OF STRAIN GAGES
CH_1
CH_2
CH_3
CH_4
CH_5
CH_6
CH_7
CH_8
CH_9
CH_10
CH_11
ALL GAGES LOCATED ON CENTER
LOCATION OF GAGES NEAR MIDSPAN


CL-53 PLANS
SHEET NO.:
5 of 6
CH_1
CH_2
CH_3
CH_4
CH_5
CH_6
CH_7
CH_8
CH_9
CH_10
CH_11
CH_12
CH_13
CH_14
CH_15
CH_16
CH_17
CH_18
4'-10"
FULLY LOADED
TEST TRUCK
LOAD TEST LOCATION FOR CL AND DYN TESTS
LOCATION OF GAGES NEAR INTEGRAL ABUTMENT
ALL GAGES LOCATED ON CENTER


CL-53 PLANS
SHEET NO.:
6 of 6
FULLY LOADED TEST TRUCK
1'-6"
CH_1
CH_2
CH_3
CH_4
CH_5
CH_6
CH_7
CH_8
CH_9
CH_10
CH_11
LOAD TEST LOCATION FOR DOWN TEST
FULLY LOADED TEST TRUCK
8'-9"
CH_1
CH_2
CH_3
CH_4
CH_5
CH_6
CH_7
CH_8
CH_9
CH_10
CH_11
LOAD TEST LOCATION FOR UP TEST

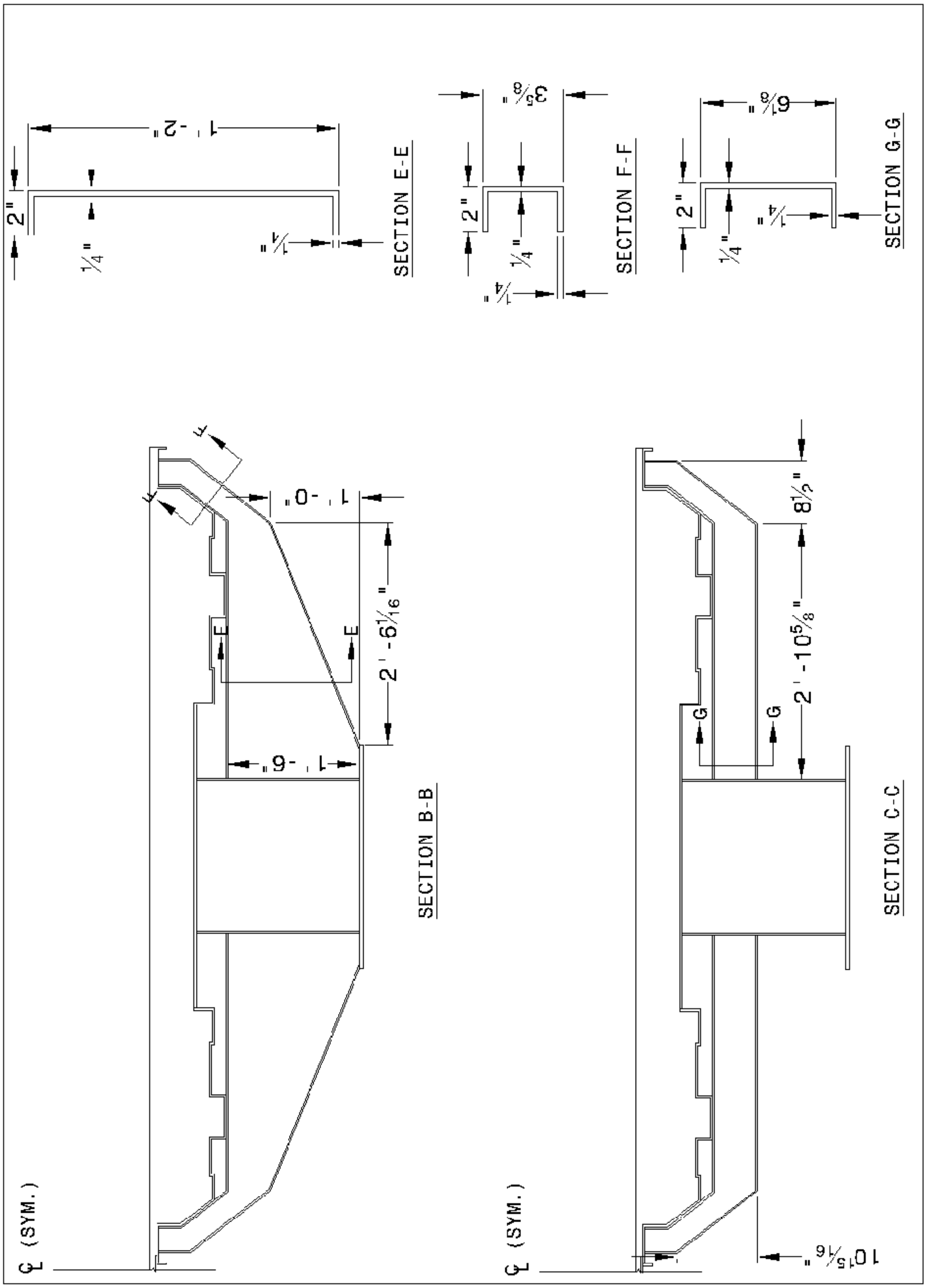
CL-179 PLANS

SHEET NO.: 1 of 6



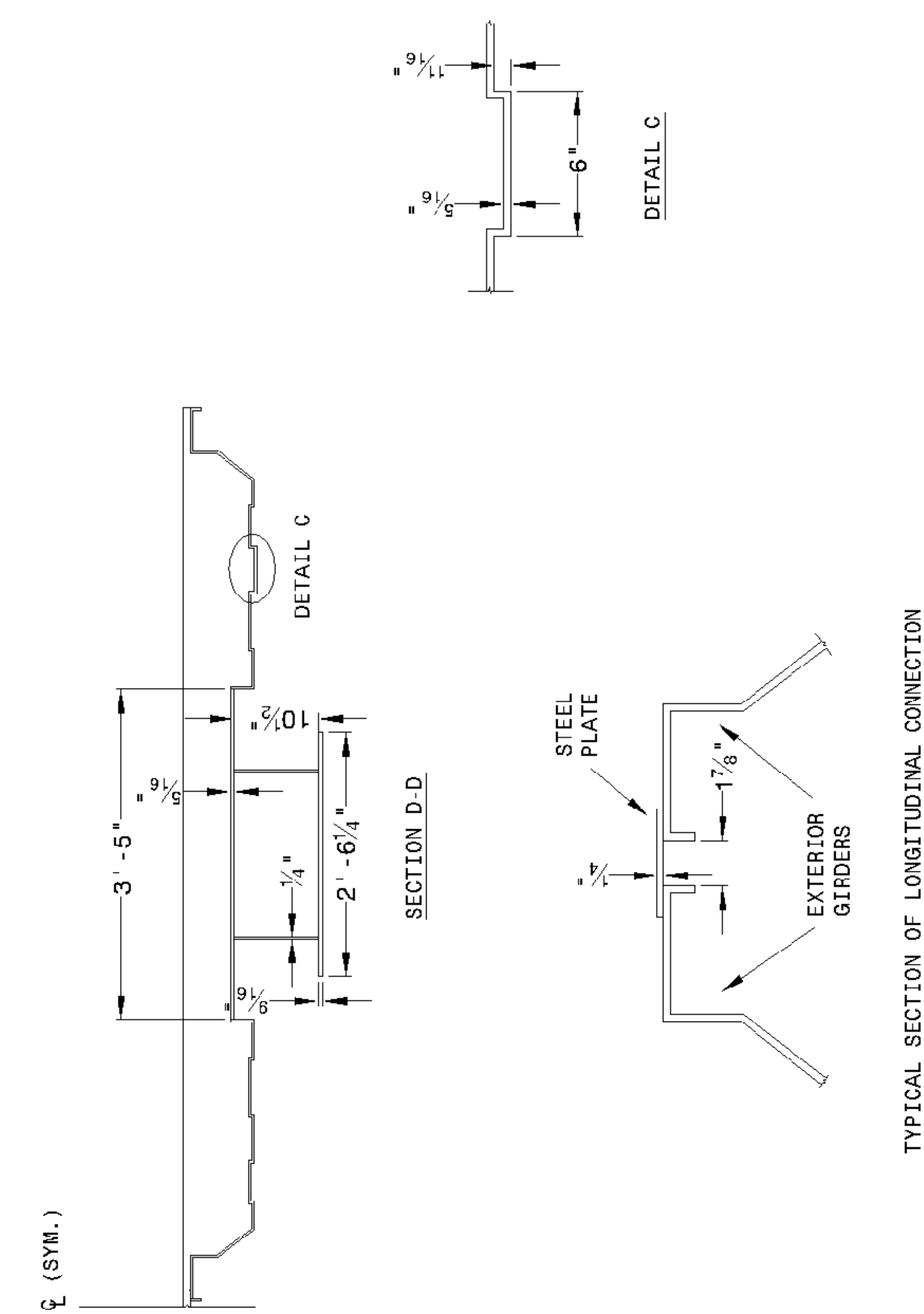
CL-179 PLANS

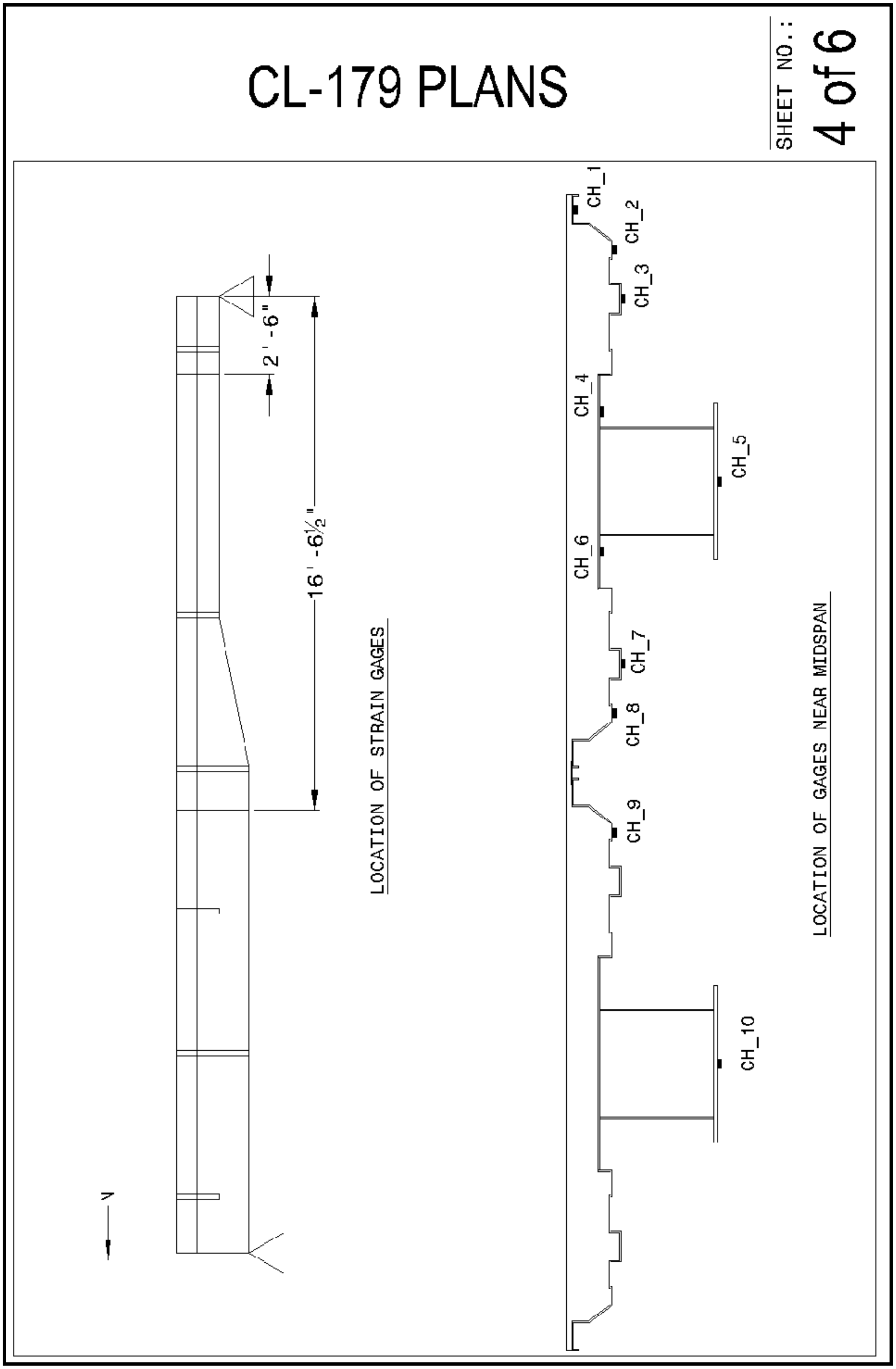
SHEET NO.: 2 of 6



CL-179 PLANS

SHEET NO.: 3 of 6

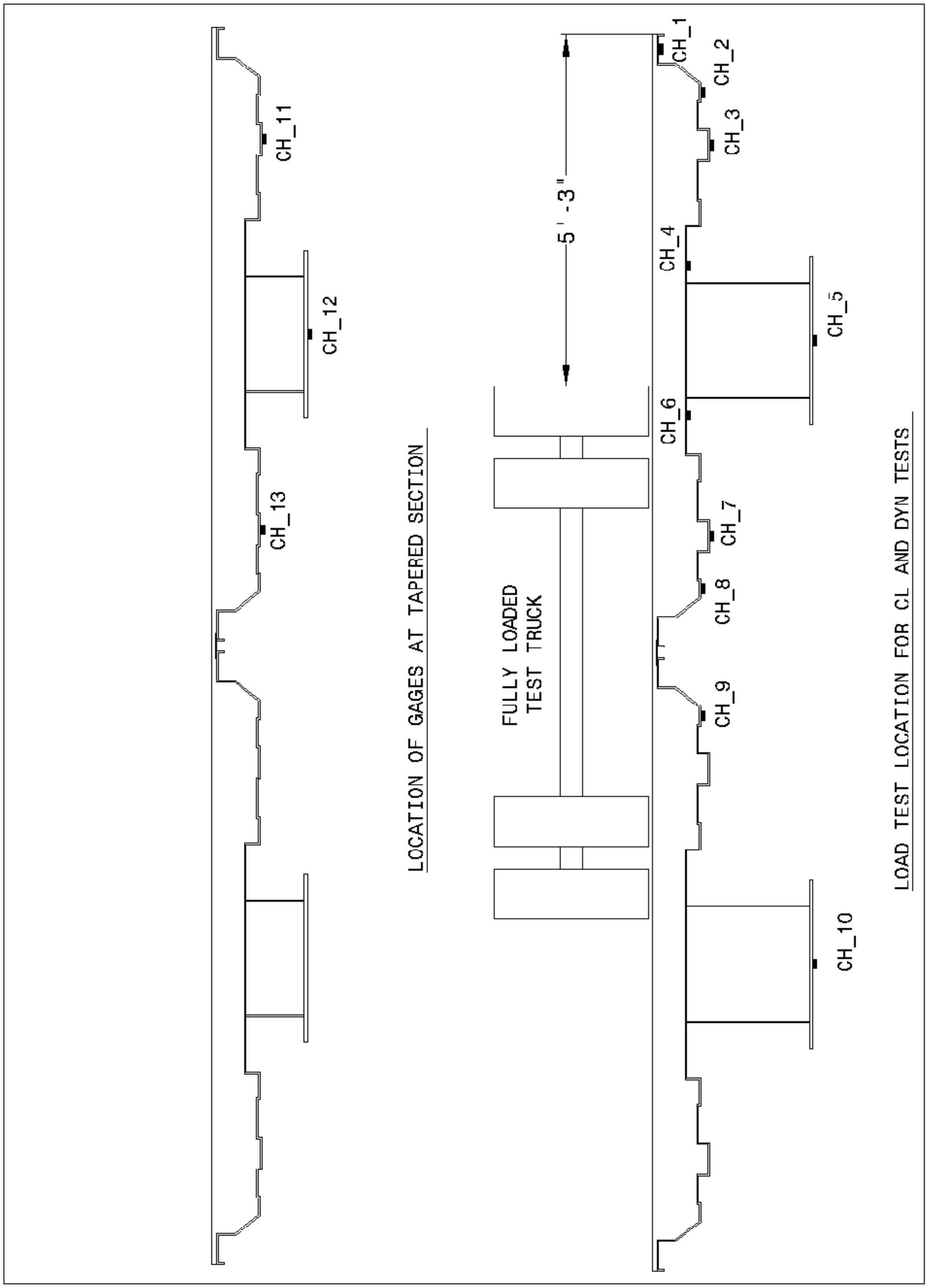


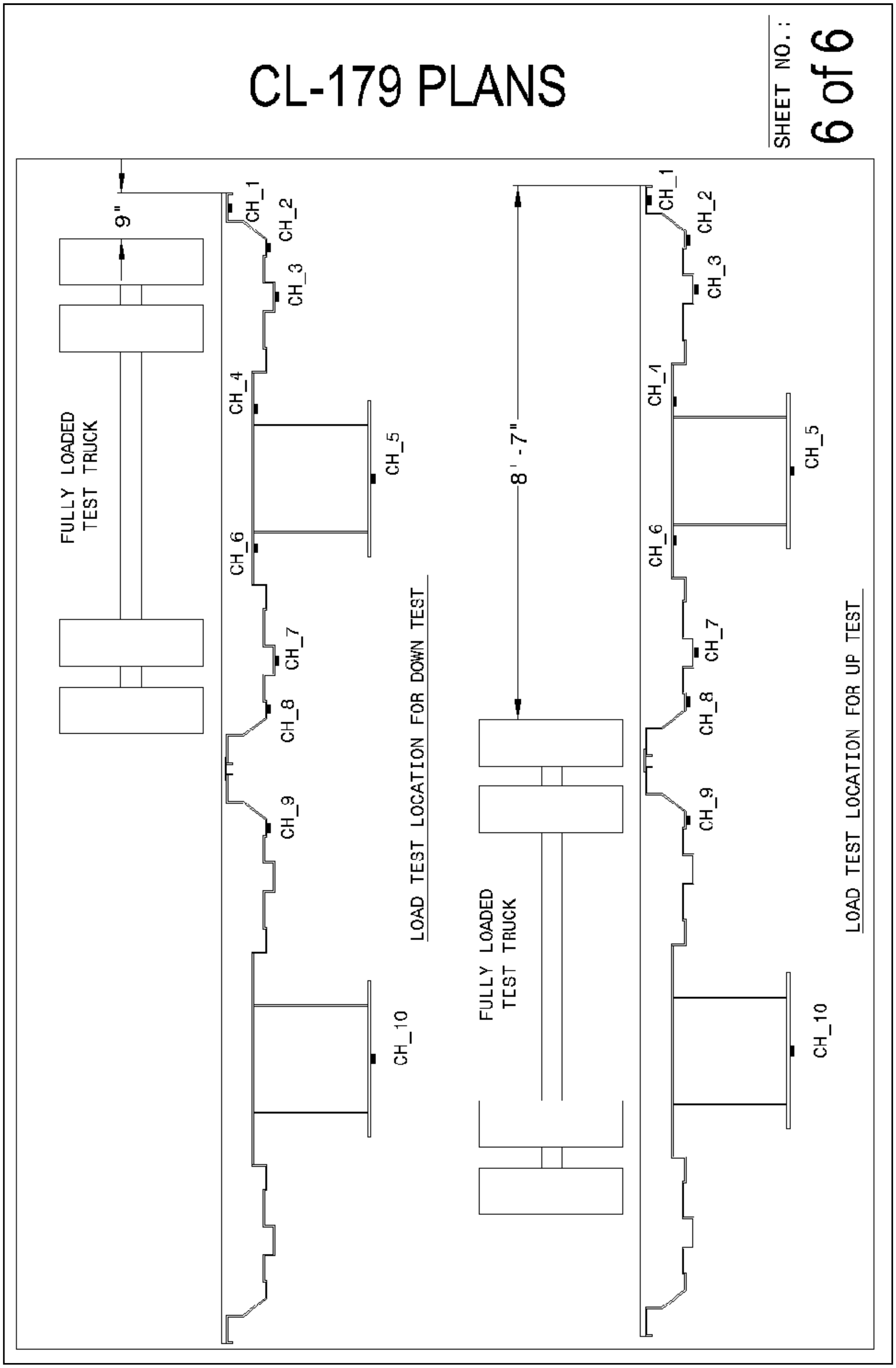


CL-179 PLANS
SHEET NO.:
4 of 6
2' -6"
16' -6½"
LOCATION OF STRAIN GAGES
CH_1
CH_2
CH_3
CH_4
CH_5
CH_6
CH_7
CH_8
CH_9
CH_10
LOCATION OF GAGES NEAR MIDSPAN

CL-179 PLANS

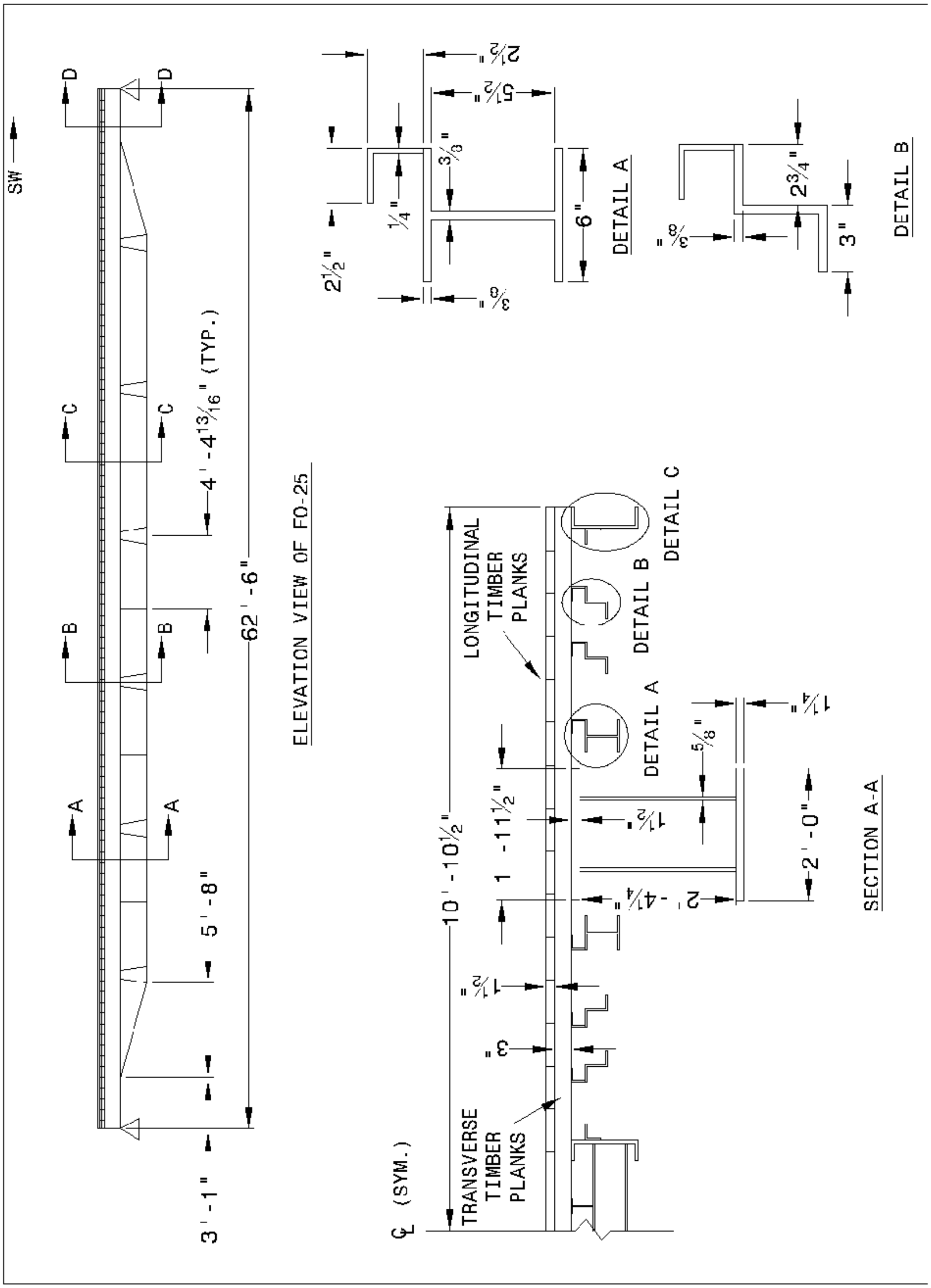
SHEET NO.: 5 of 6




CL-179 PLANS
SHEET NO.:
6 of 6
FULLY LOADED TEST TRUCK
9"
CH_1
CH_2
CH_3
CH_4
CH_5
CH_6
CH_7
CH_8
CH_9
CH_10
LOAD TEST LOCATION FOR DOWN TEST
8'-7"
FULLY LOADED TEST TRUCK
CH_1
CH_2
CH_3
CH_4
CH_5
CH_6
CH_7
CH_8
CH_9
CH_10
LOAD TEST LOCATION FOR UP TEST

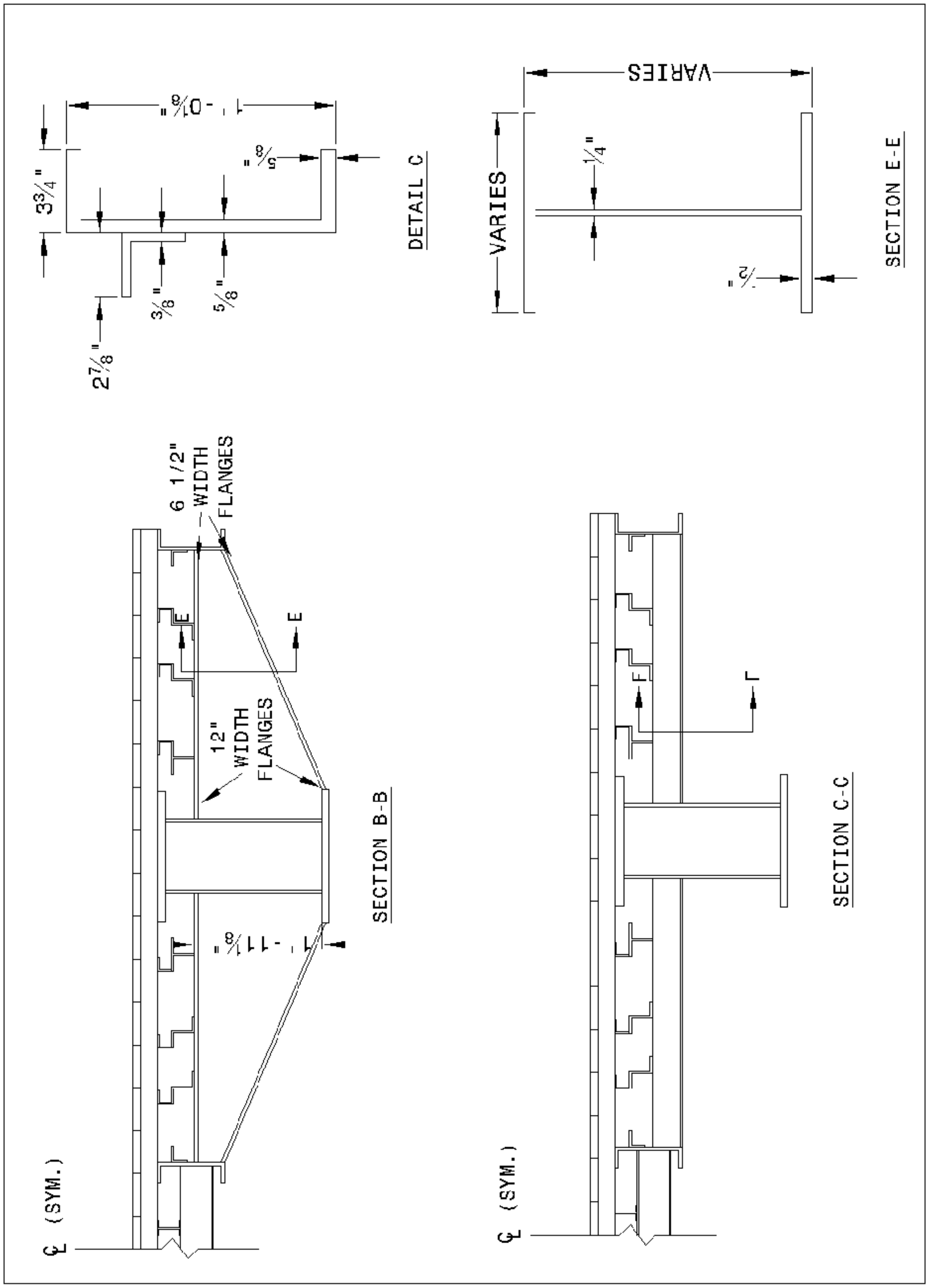
FO-25 PLANS

SHEET NO.: 1 of 6



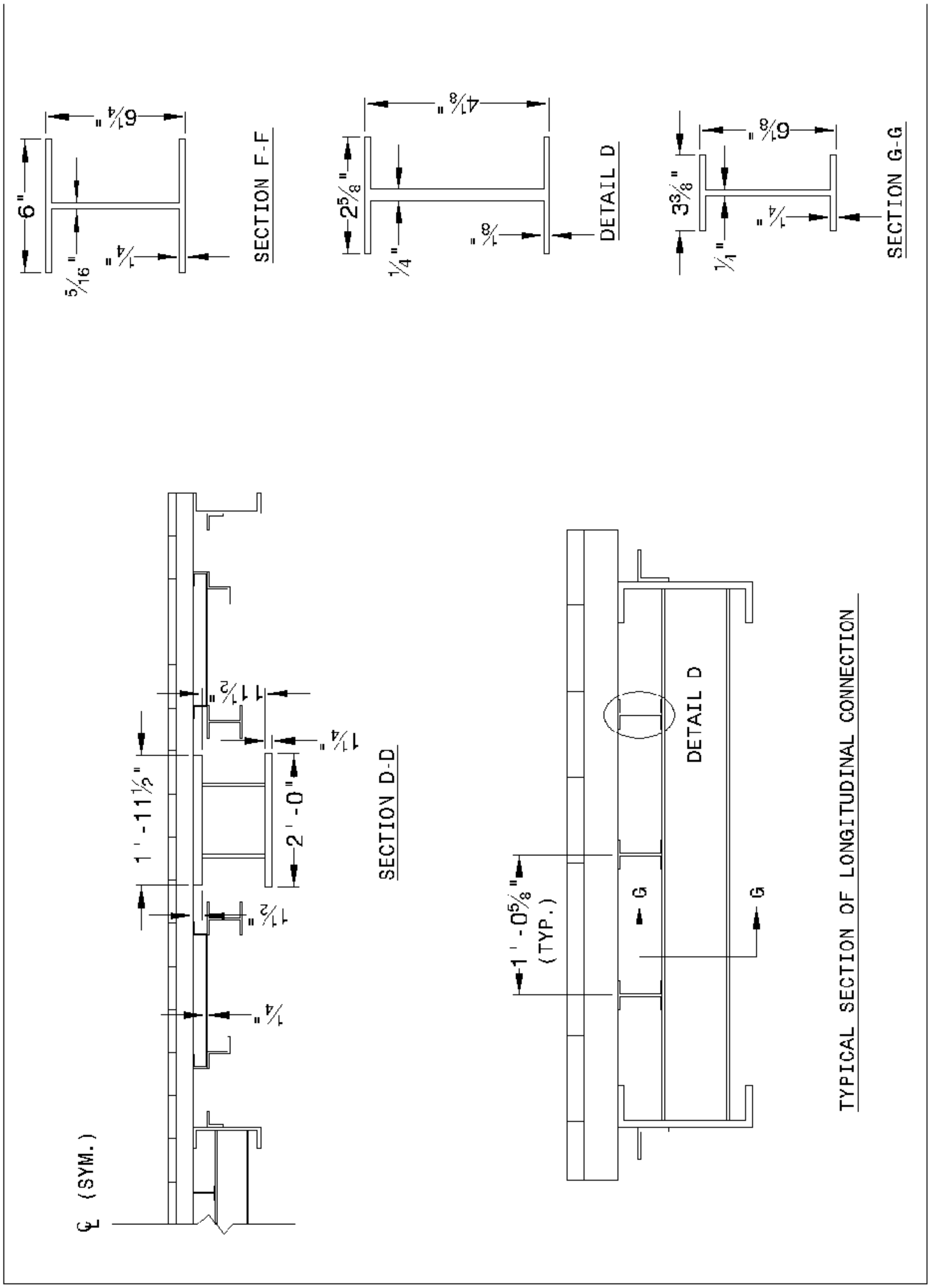
FO-25 PLANS

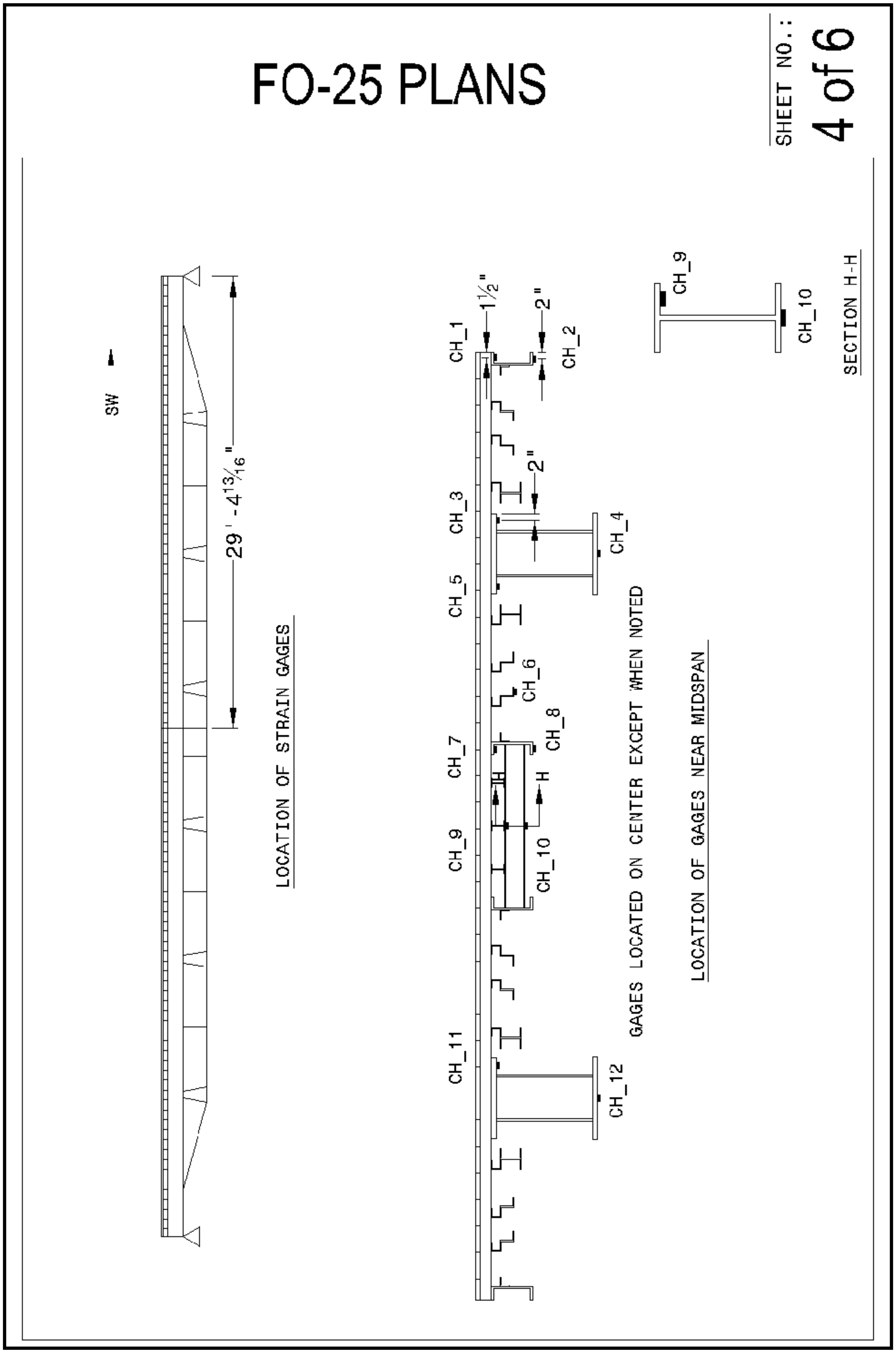
SHEET NO.: 2 of 6



FO-25 PLANS

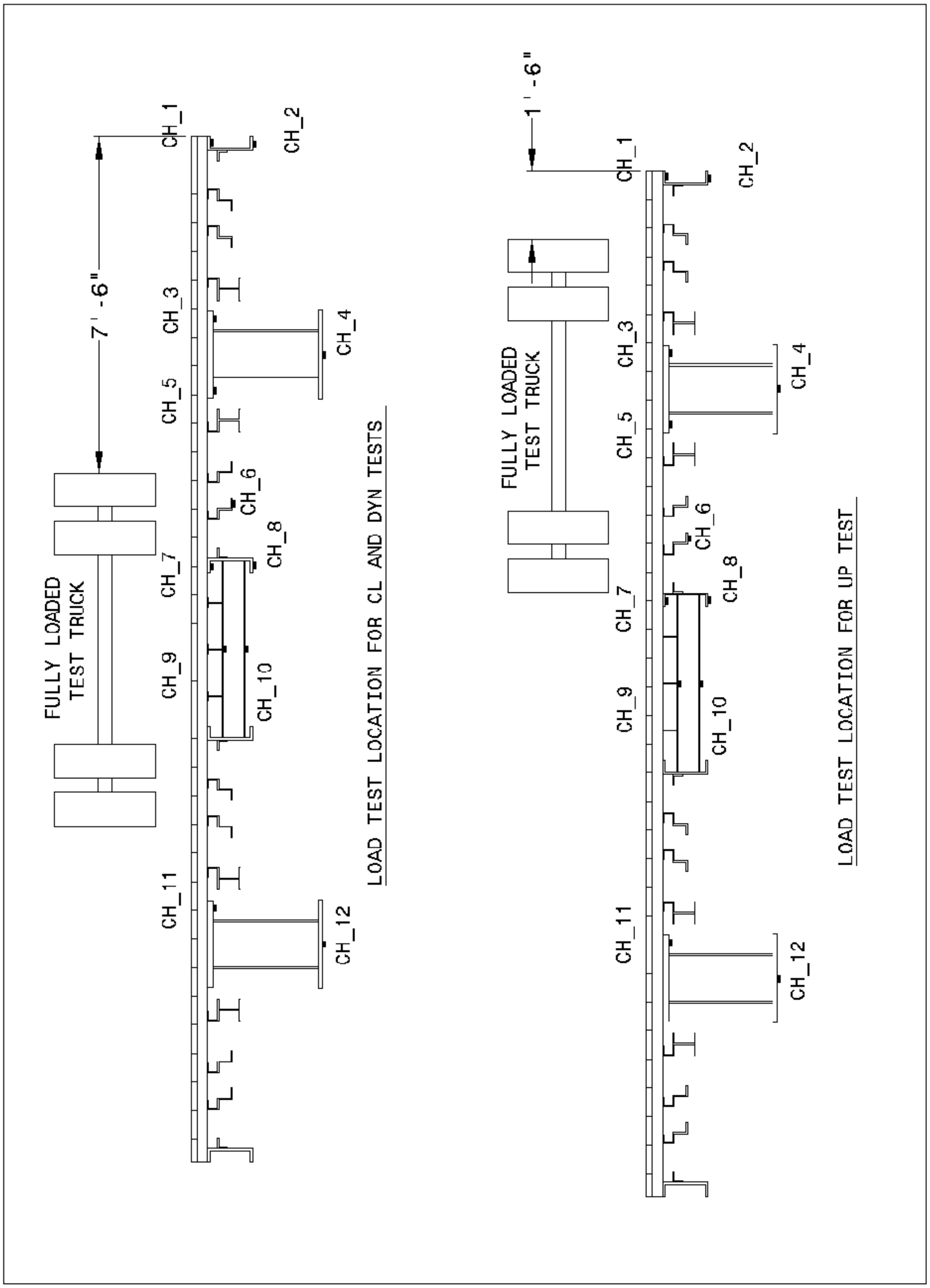
SHEET NO.:
3 of 6

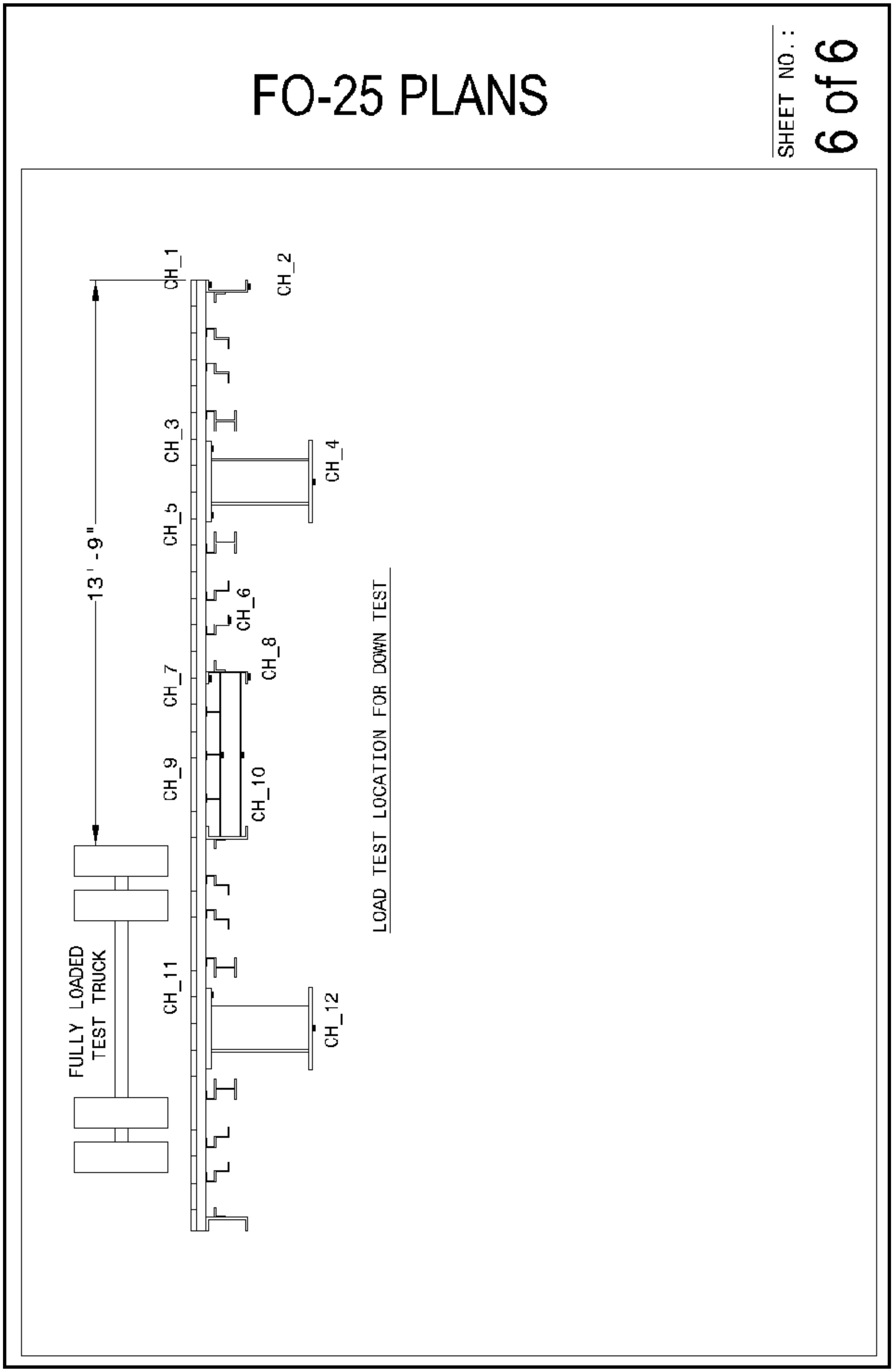



FO-25 PLANS
SHEET NO.:
4 of 6
SW
29' - 4 13/16"
LOCATION OF STRAIN GAGES
CH_1
1½"
2"
CH_2
CH_3
2"
CH_4
CH_5
CH_6
CH_7
CH_8
H
H
CH_9
CH_10
CH_11
CH_12
GAGES LOCATED ON CENTER EXCEPT WHEN NOTED
LOCATION OF GAGES NEAR MIDSPAN
CH_9
CH_10
SECTION H-H

FO-25 PLANS

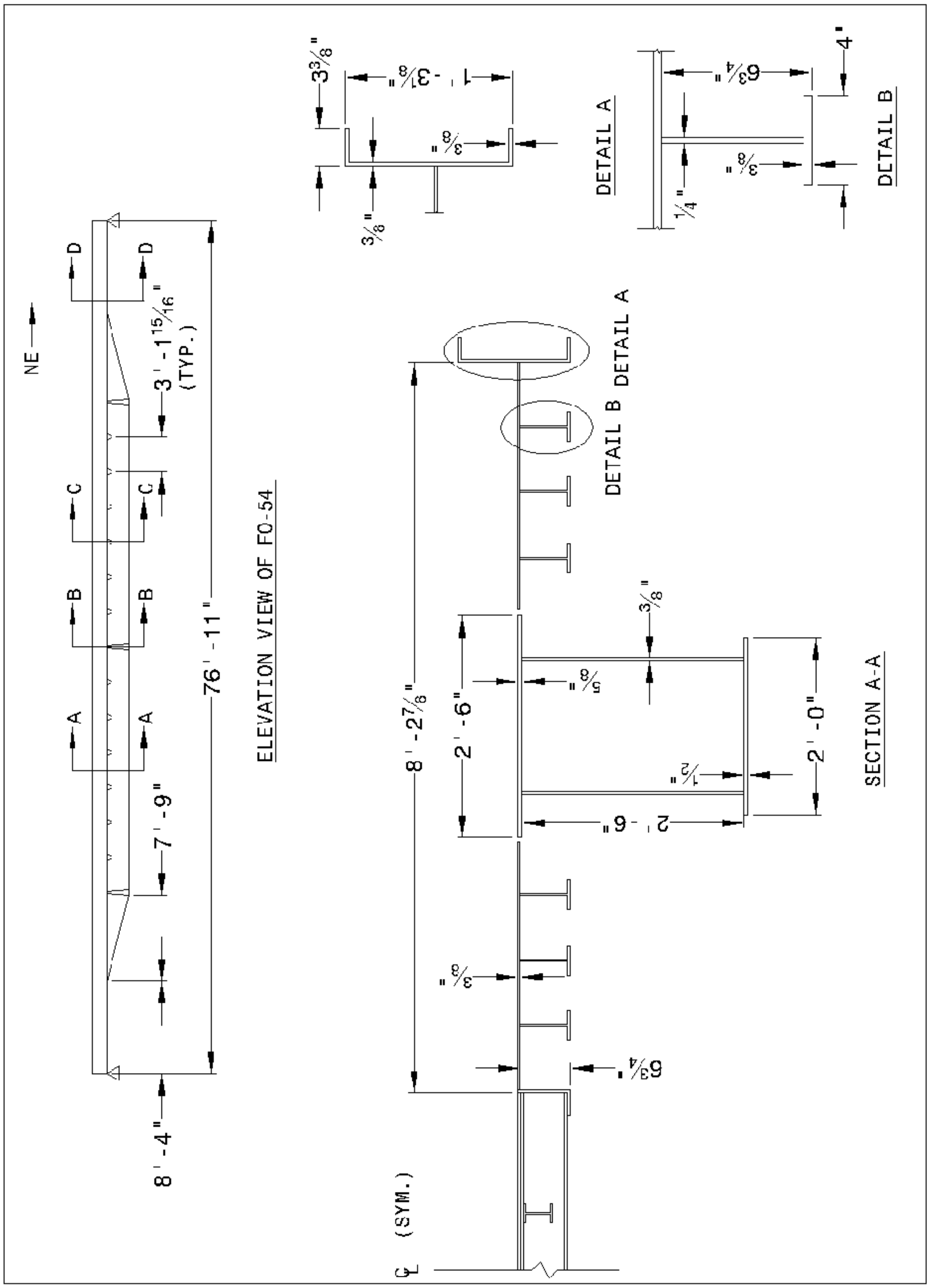
SHEET NO.: 5 of 6

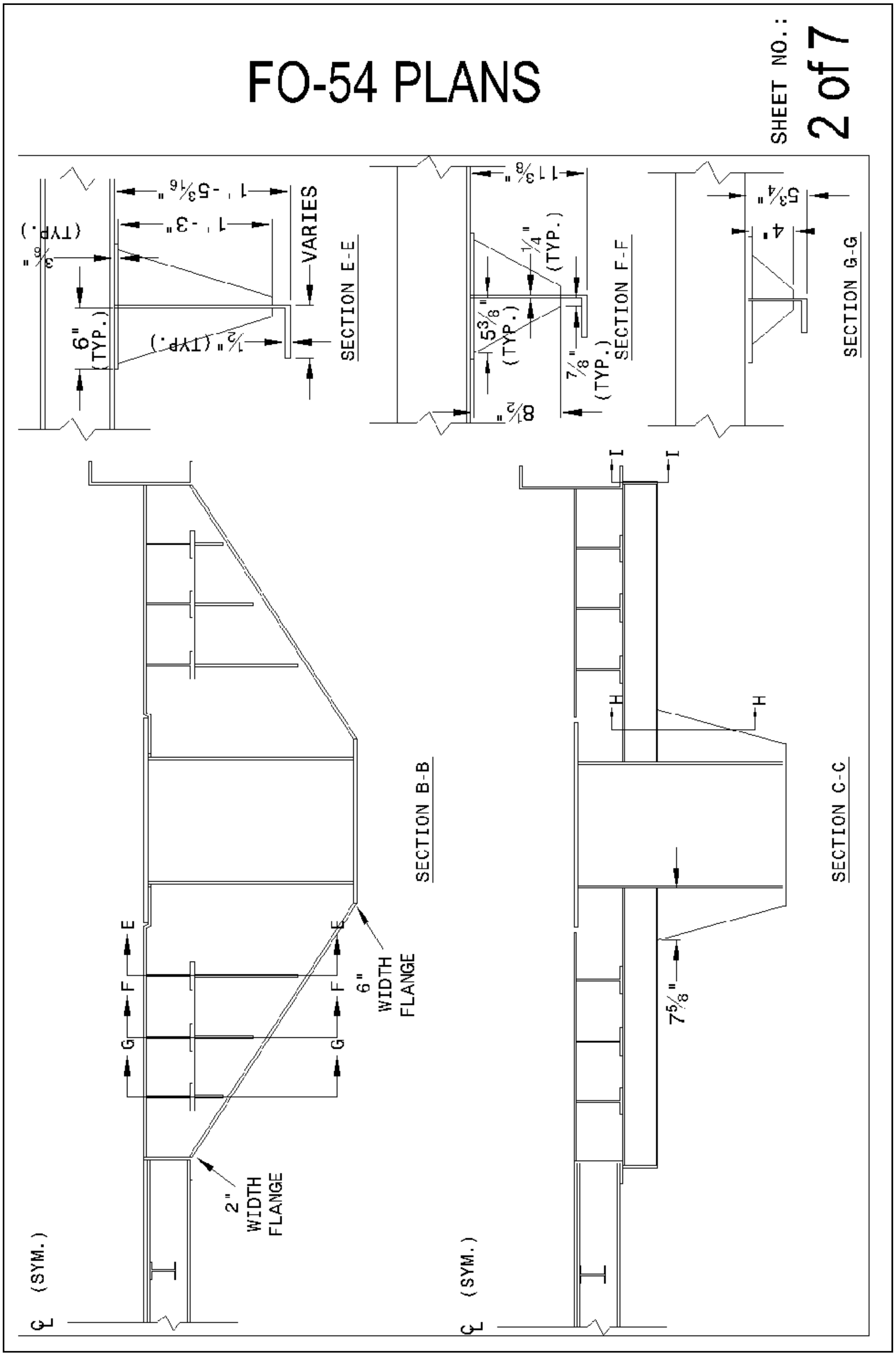



FO-25 PLANS
SHEET NO.:
6 of 6
13'-9"
FULLY LOADED
TEST TRUCK
CH_1
CH_2
CH_3
CH_4
CH_5
CH_6
CH_7
CH_8
CH_9
CH_10
CH_11
CH_12
LOAD TEST LOCATION FOR DOWN TEST

FO-54 PLANS

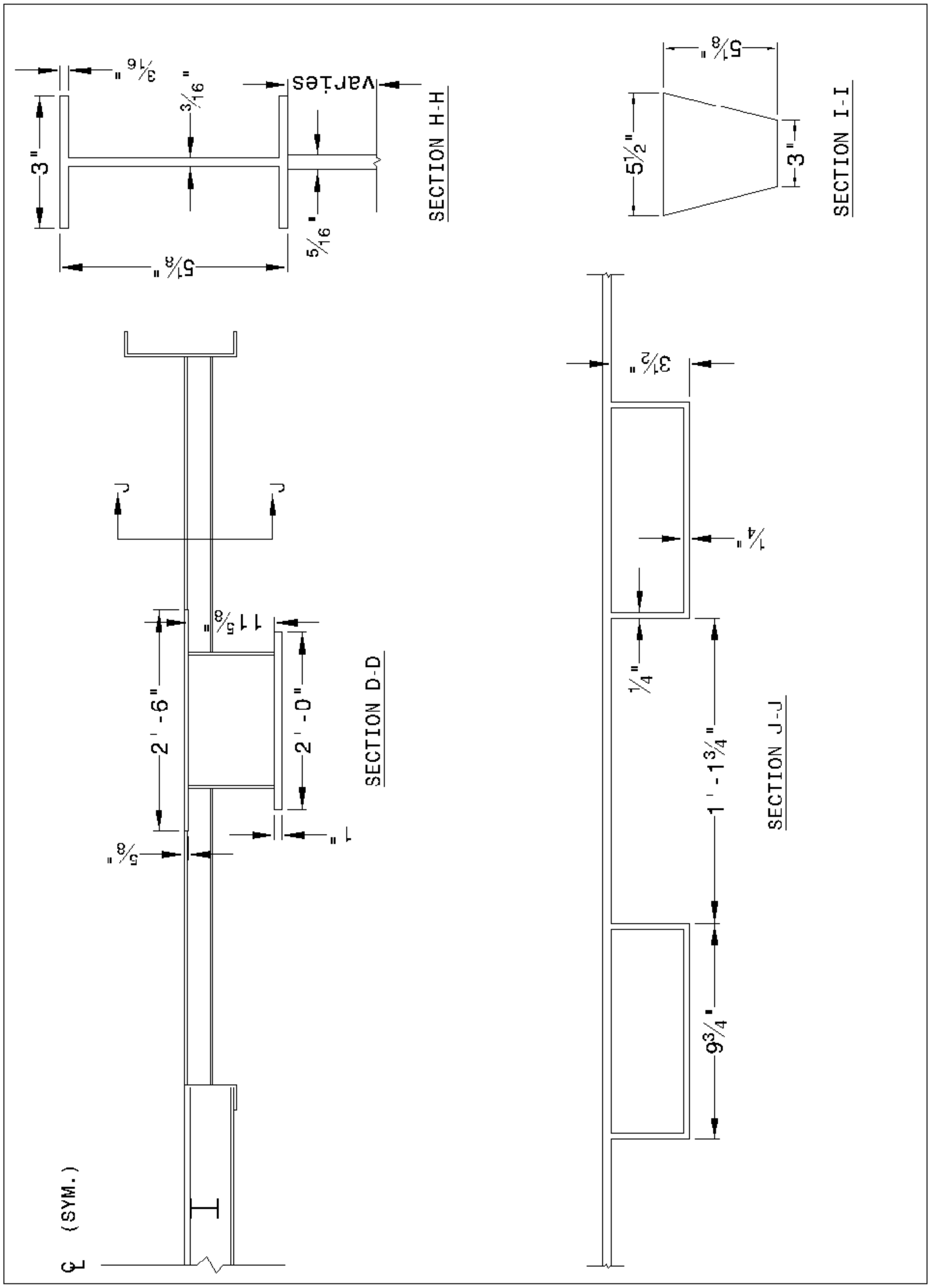
SHEET NO.: 1 of 7




FO-54 PLANS
SHEET NO.:
2 of 7
SECTION E-E
VARIES
6" (TYP.)
SECTION F-F
SECTION G-G
SECTION B-B
6" WIDTH FLANGE
2" WIDTH FLANGE
(SYM.)
SECTION C-C

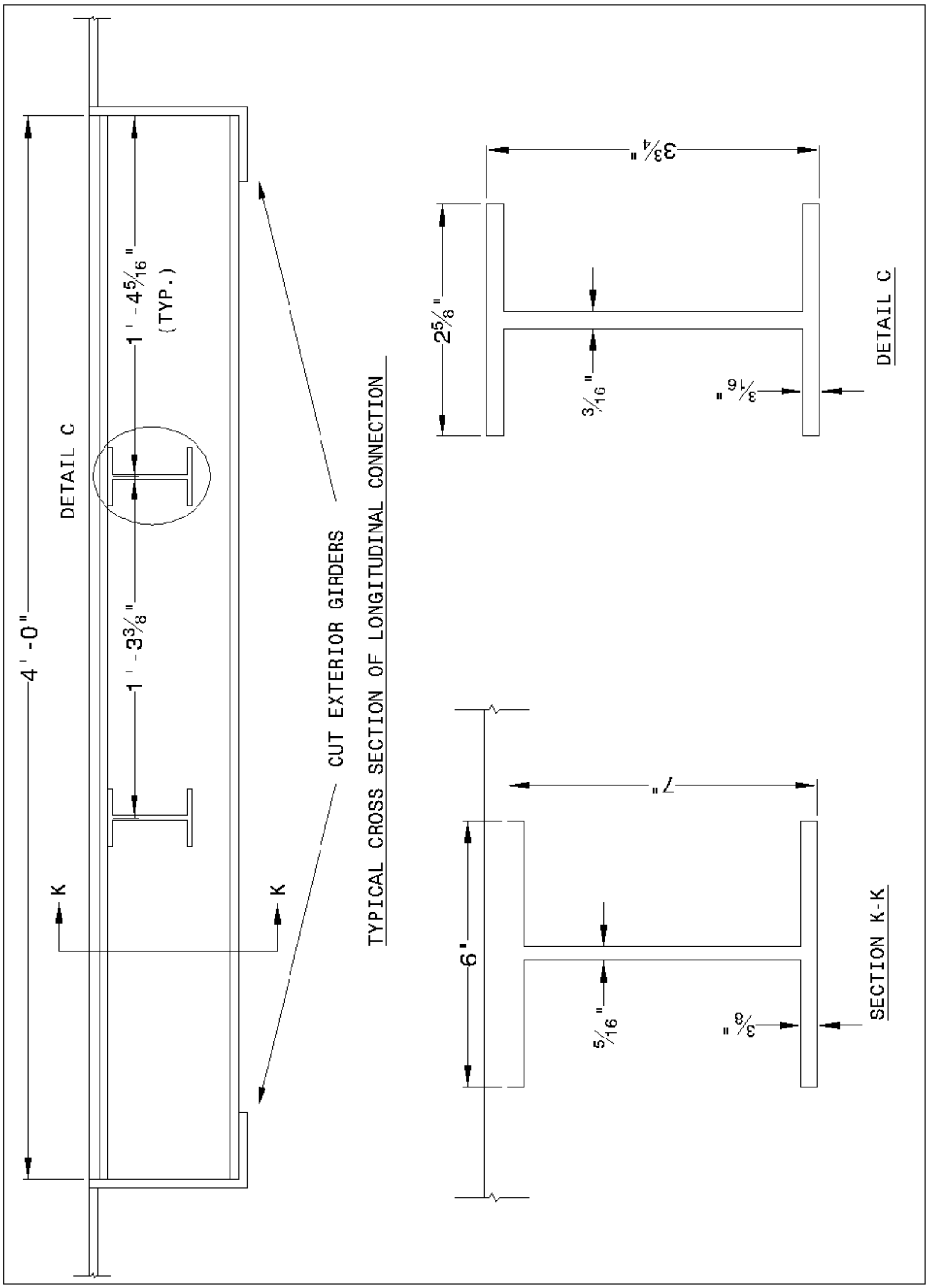
FO-54 PLANS

SHEET NO.: 3 of 7



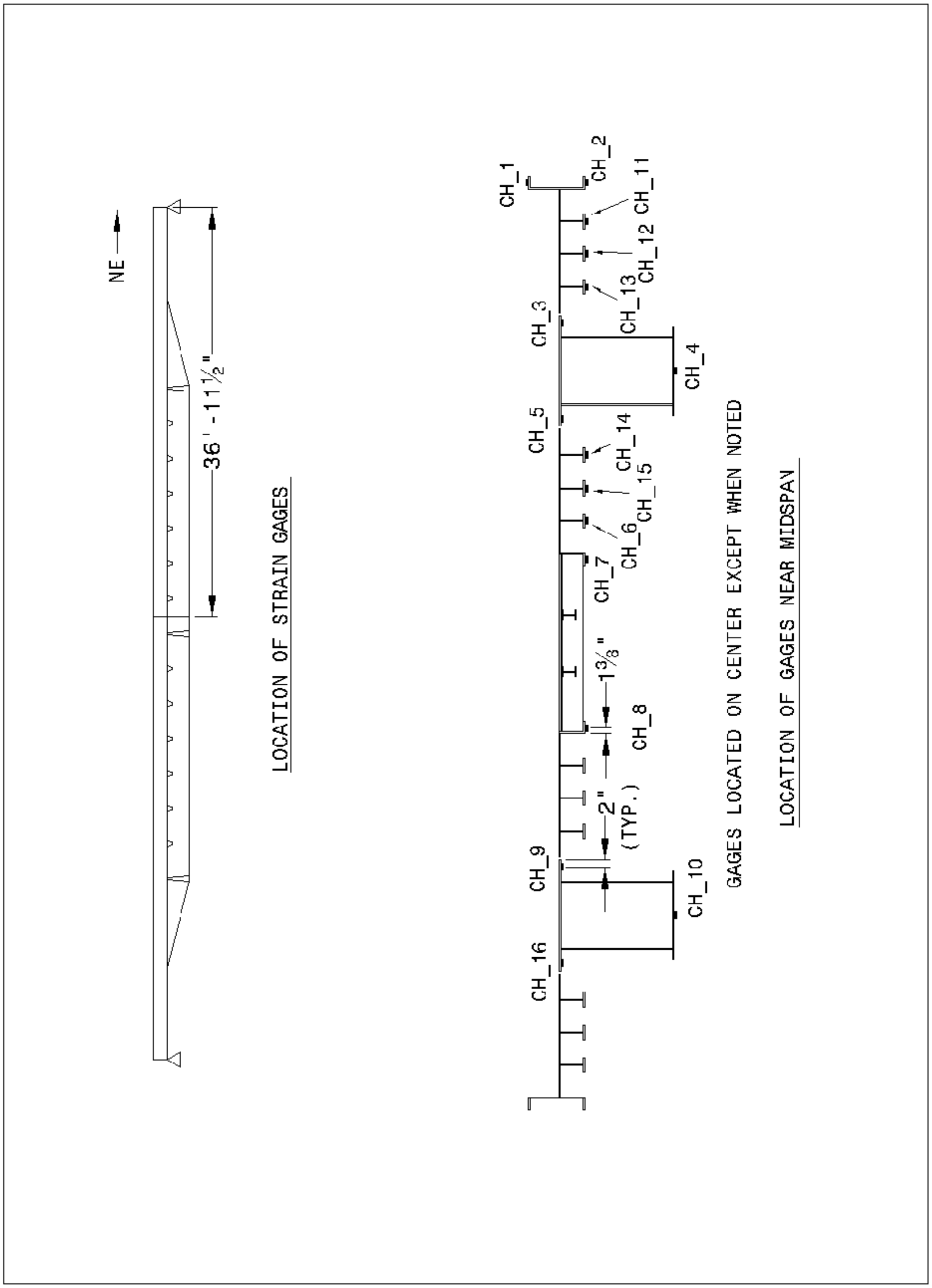
FO-54 PLANS

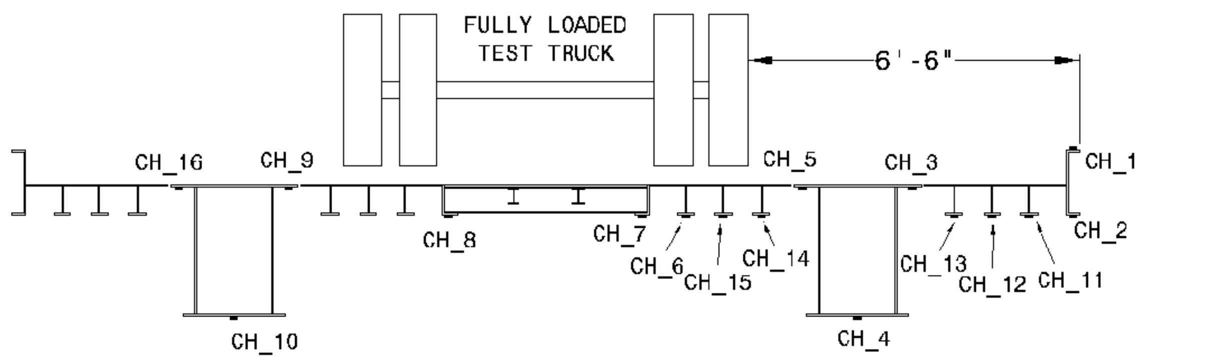
SHEET NO.: 4 of 7



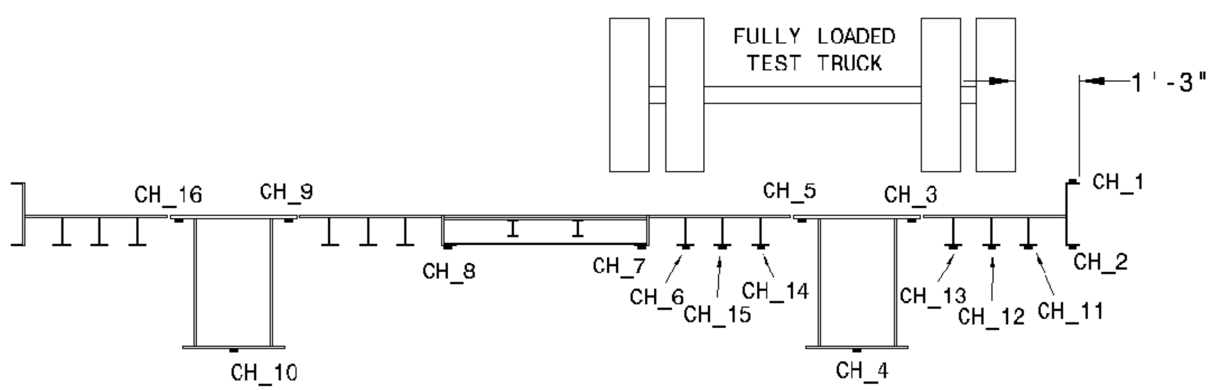
FO-54 PLANS

SHEET NO.: 5 of 7





LOAD TEST LOCATION FOR CL AND DYN TESTS



LOAD TEST LOCATION FOR DOWN TEST

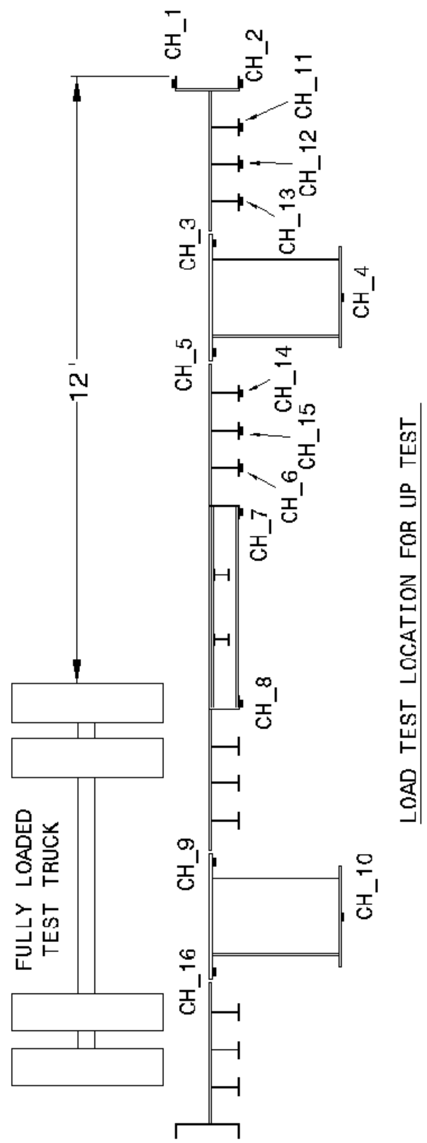
FO-54 PLANS

SHEET NO.:

6 of 7

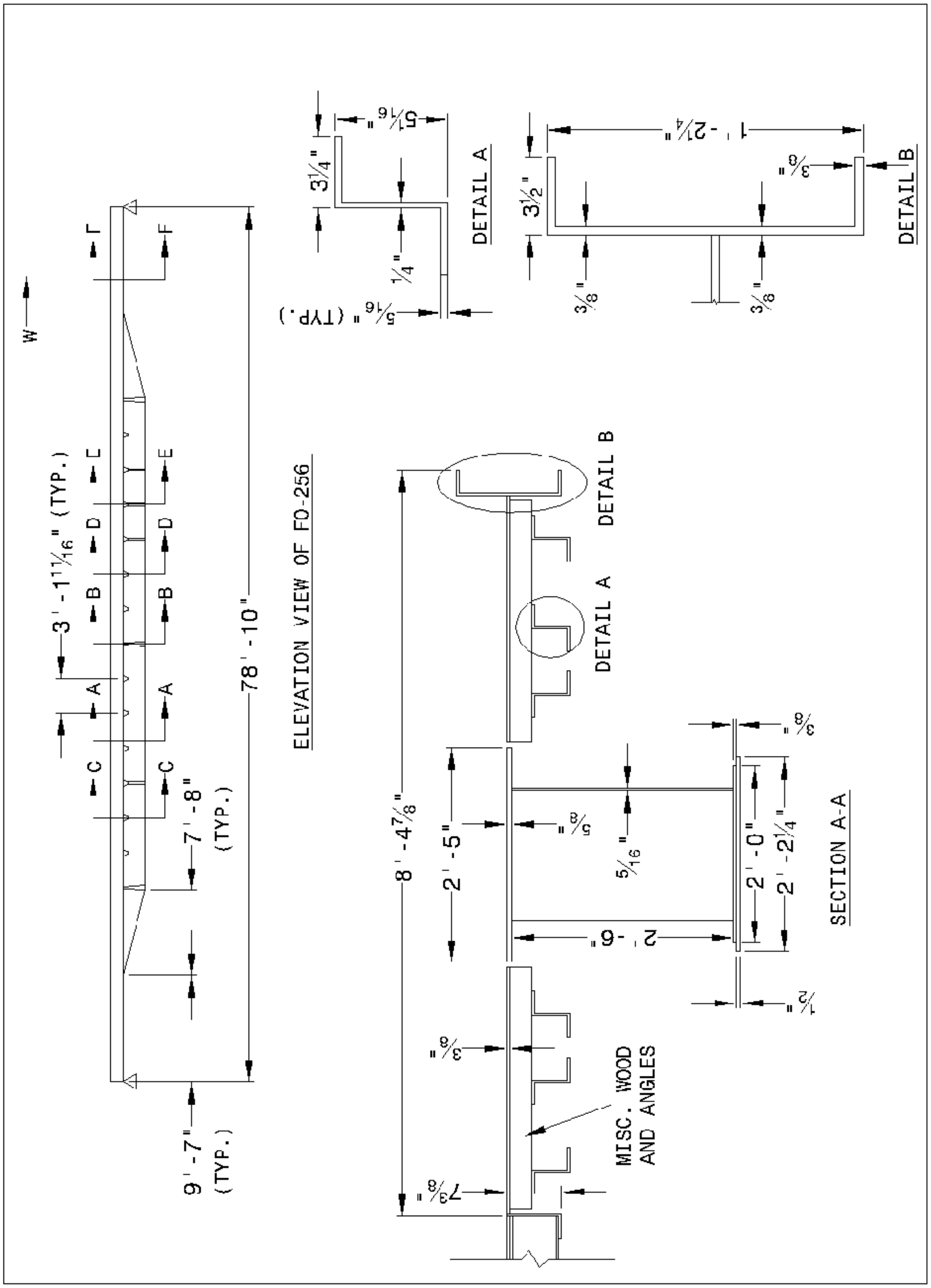
FO-54 PLANS

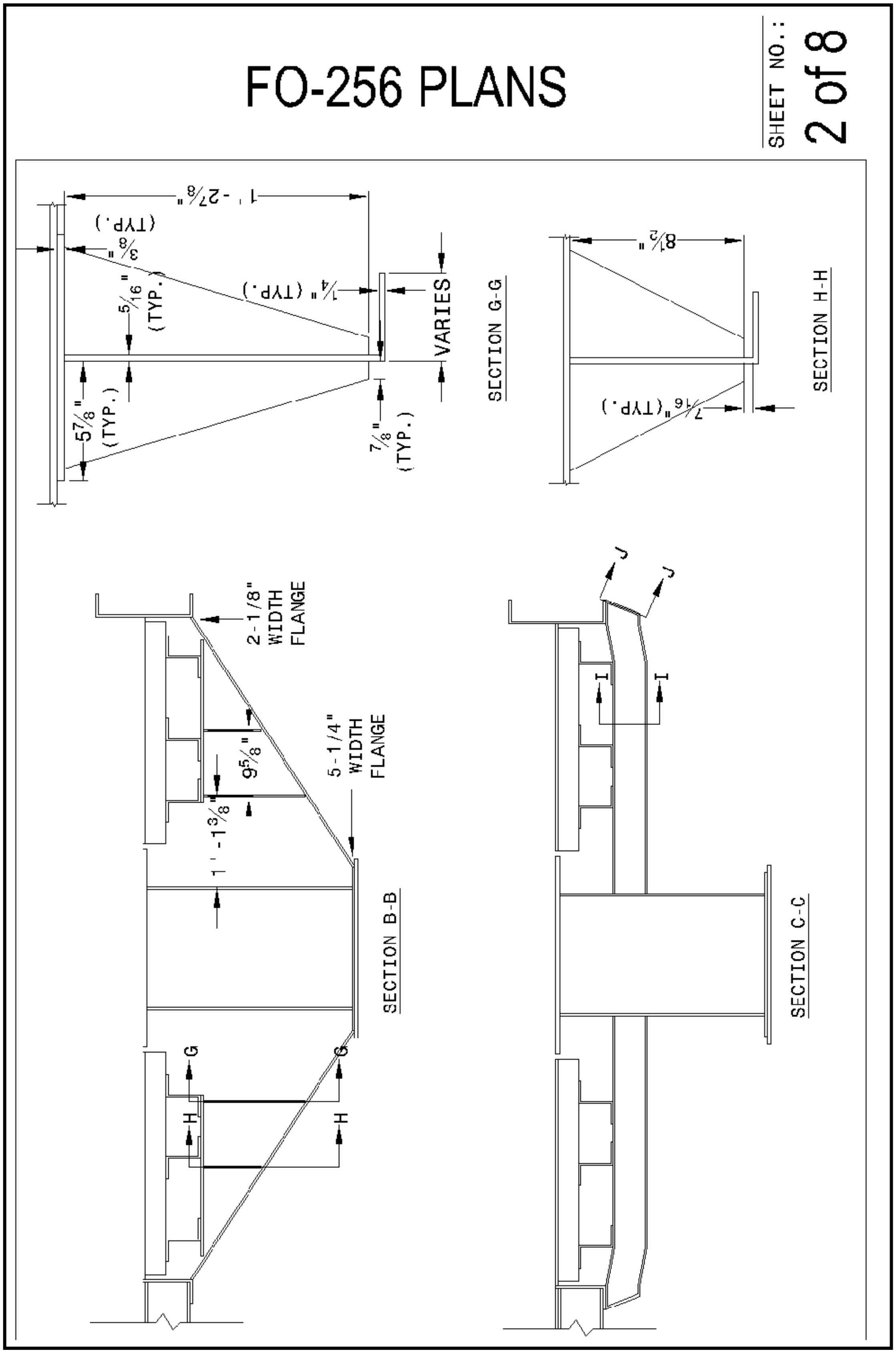
SHEET NO.:
7 of 7



FO-256 PLANS

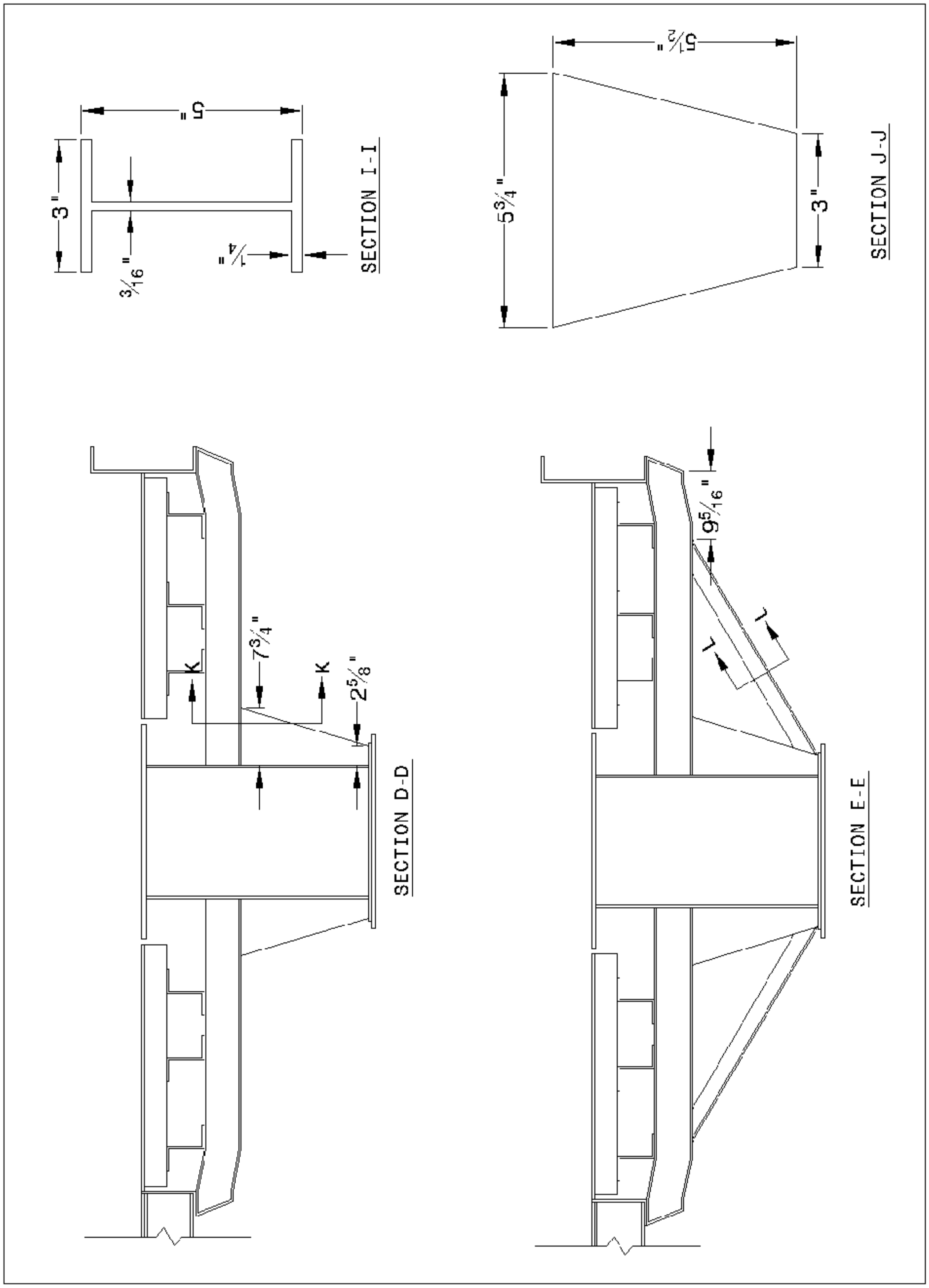
SHEET NO.: 1 of 8




FO-256 PLANS
SHEET NO.:
2 of 8
1'-2 7/8"
3/8" (TYP.)
5/16" (TYP.)
1/4" (TYP.)
VARIES
5 7/8" (TYP.)
7/8" (TYP.)
SECTION G-G
8 1/2"
7/16" (TYP.)
SECTION H-H
2-1/8" WIDTH FLANGE
5-1/4" WIDTH FLANGE
9 5/8"
1'-1 3/8"
SECTION B-B
SECTION C-C

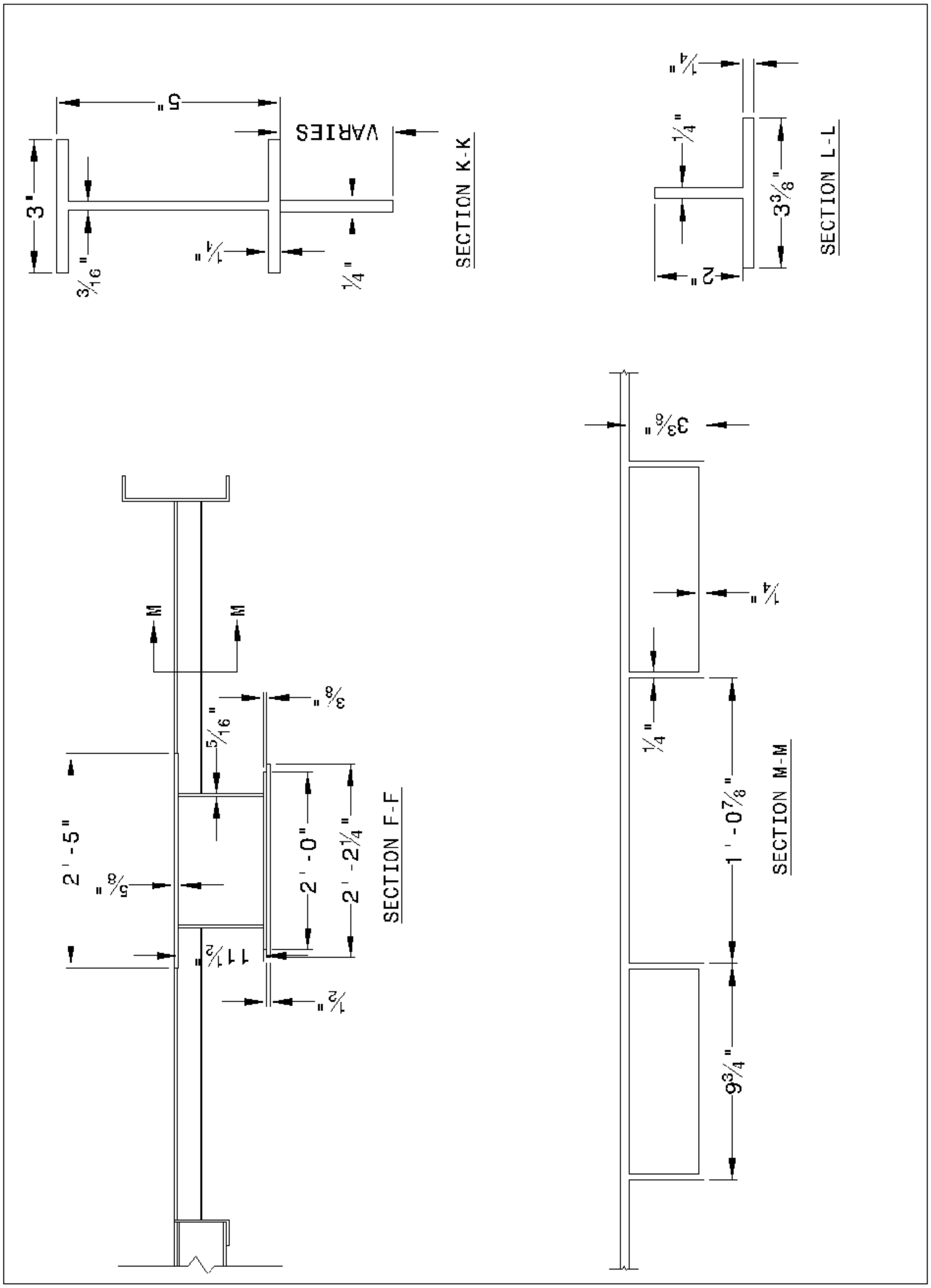
FO-256 PLANS

SHEET NO.: 3 of 8



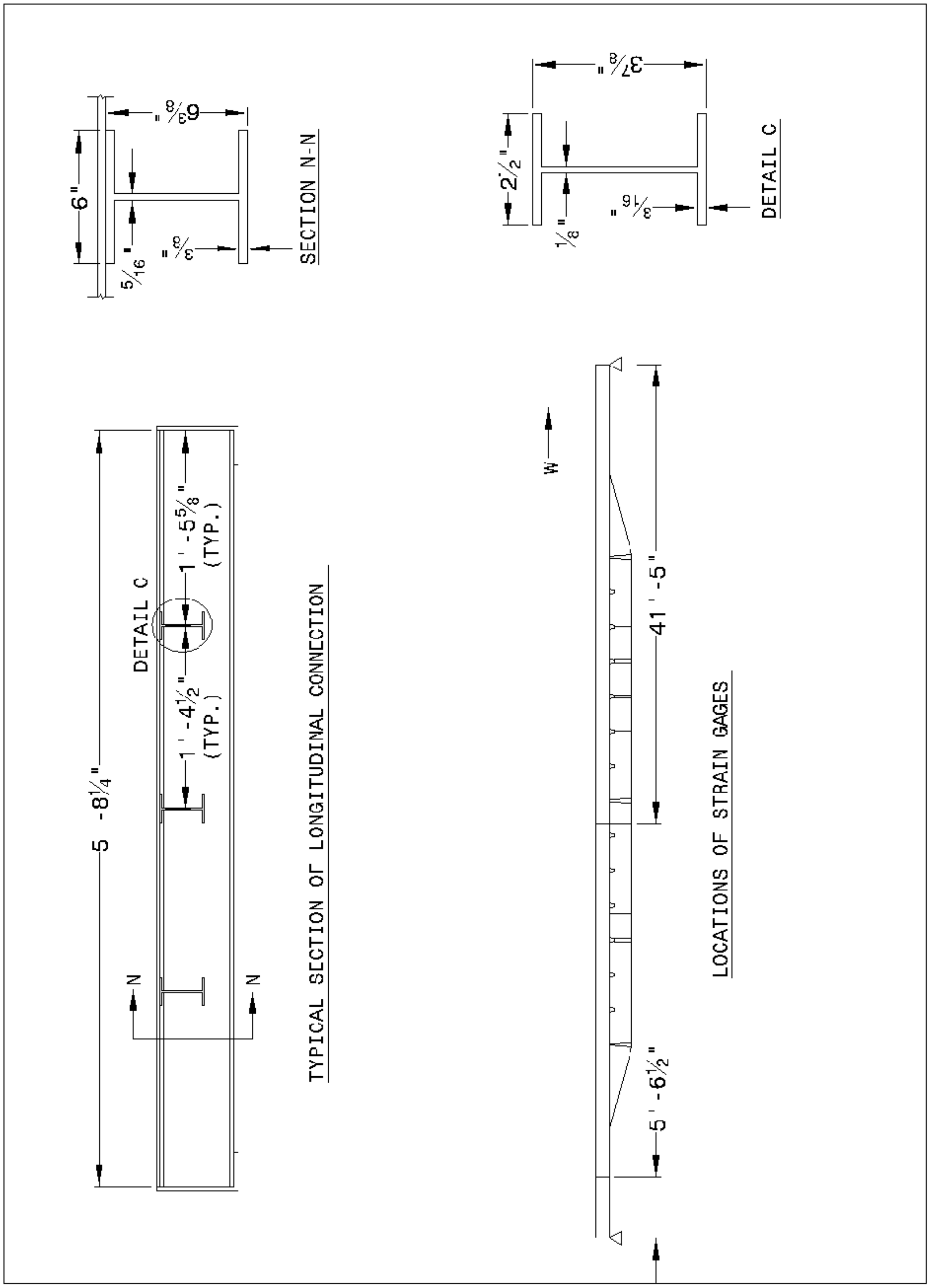
FO-256 PLANS

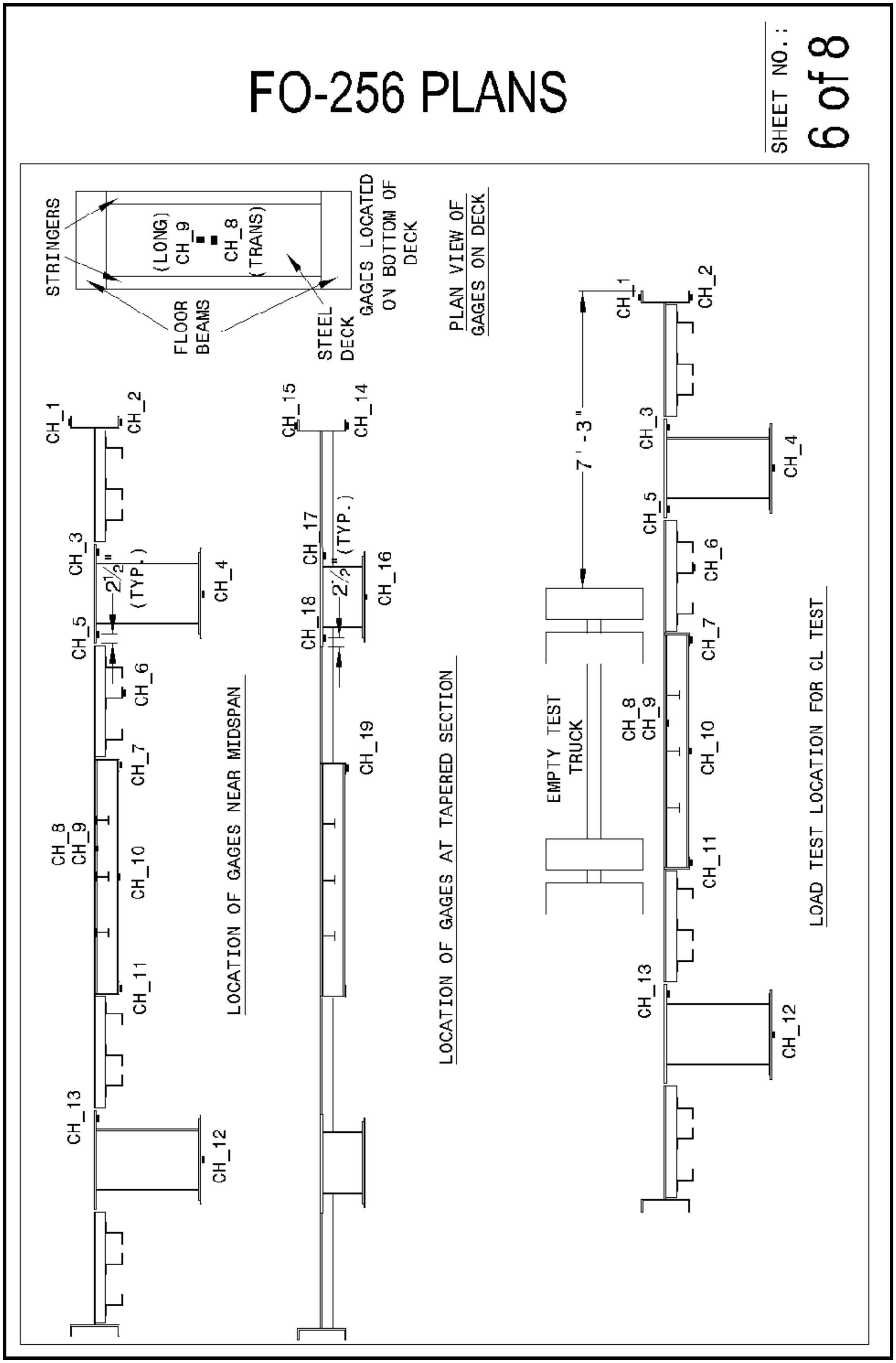
SHEET NO.:
4 of 8

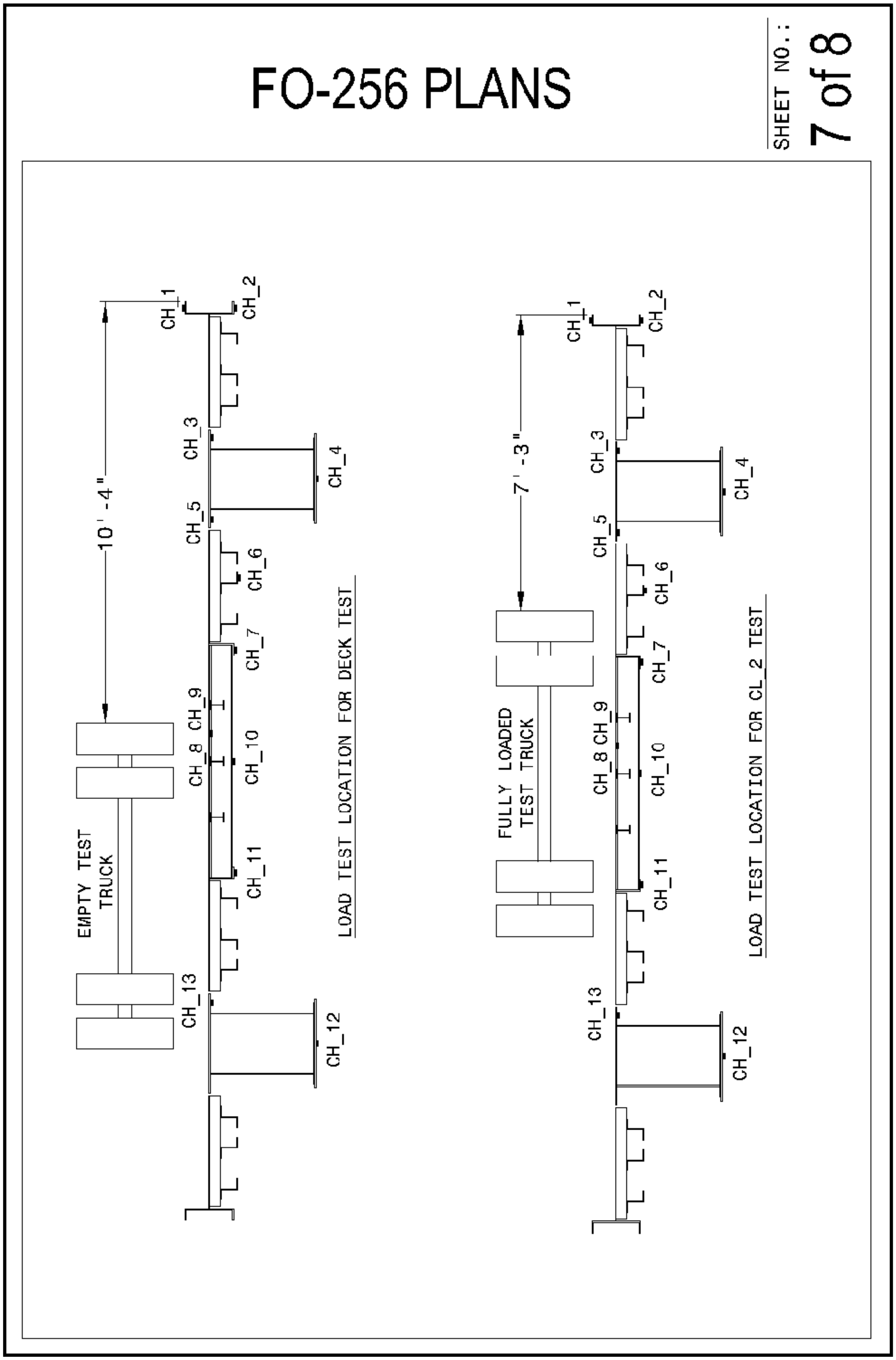


FO-256 PLANS

SHEET NO.: 5 of 8

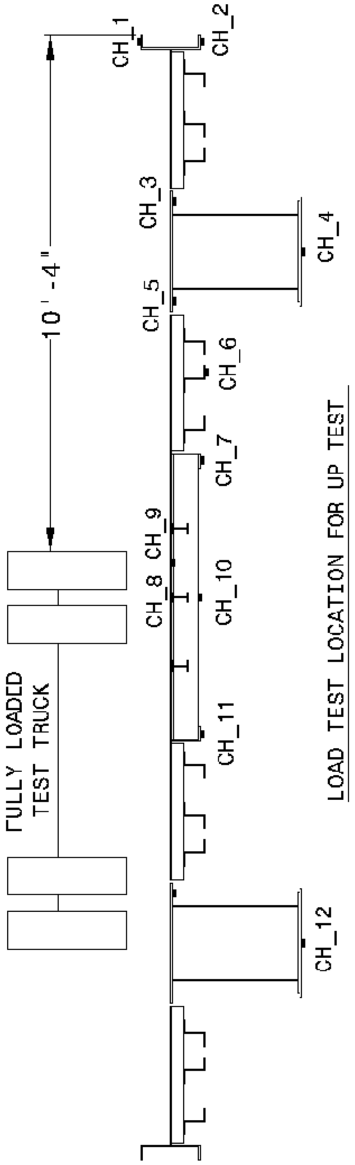



FO-256 PLANS
SHEET NO.:
6 of 8
STRINGERS
FLOOR BEAMS
STEEL DECK
(LONG) CH_9
CH_8 (TRANS)
GAGES LOCATED OV BOTTOM OF DECK
PLAN VIEW OF GAGES ON DECK
CH_1
CH_2
CH_3
CH_4
CH_5
CH_6
CH_7
CH_8
CH_9
CH_10
CH_11
CH_12
CH_13
2½" (TYP.)
LOCATION OF GAGES NEAR MIDSPAN
CH_14
CH_15
CH_16
CH_17
CH_18
CH_19
2½" (TYP.)
LOCATION OF GAGES AT TAPERED SECTION
7'-3"
EMPTY TEST TRUCK
LOAD TEST LOCATION FOR CL TEST


FO-256 PLANS
SHEET NO.:
7 of 8
10'-4"
EMPTY TEST TRUCK
LOAD TEST LOCATION FOR DECK TEST
7'-3"
FULLY LOADED TEST TRUCK
LOAD TEST LOCATION FOR CL_2 TEST
CH_1
CH_2
CH_3
CH_4
CH_5
CH_6
CH_7
CH_8
CH_9
CH_10
CH_11
CH_12
CH_13

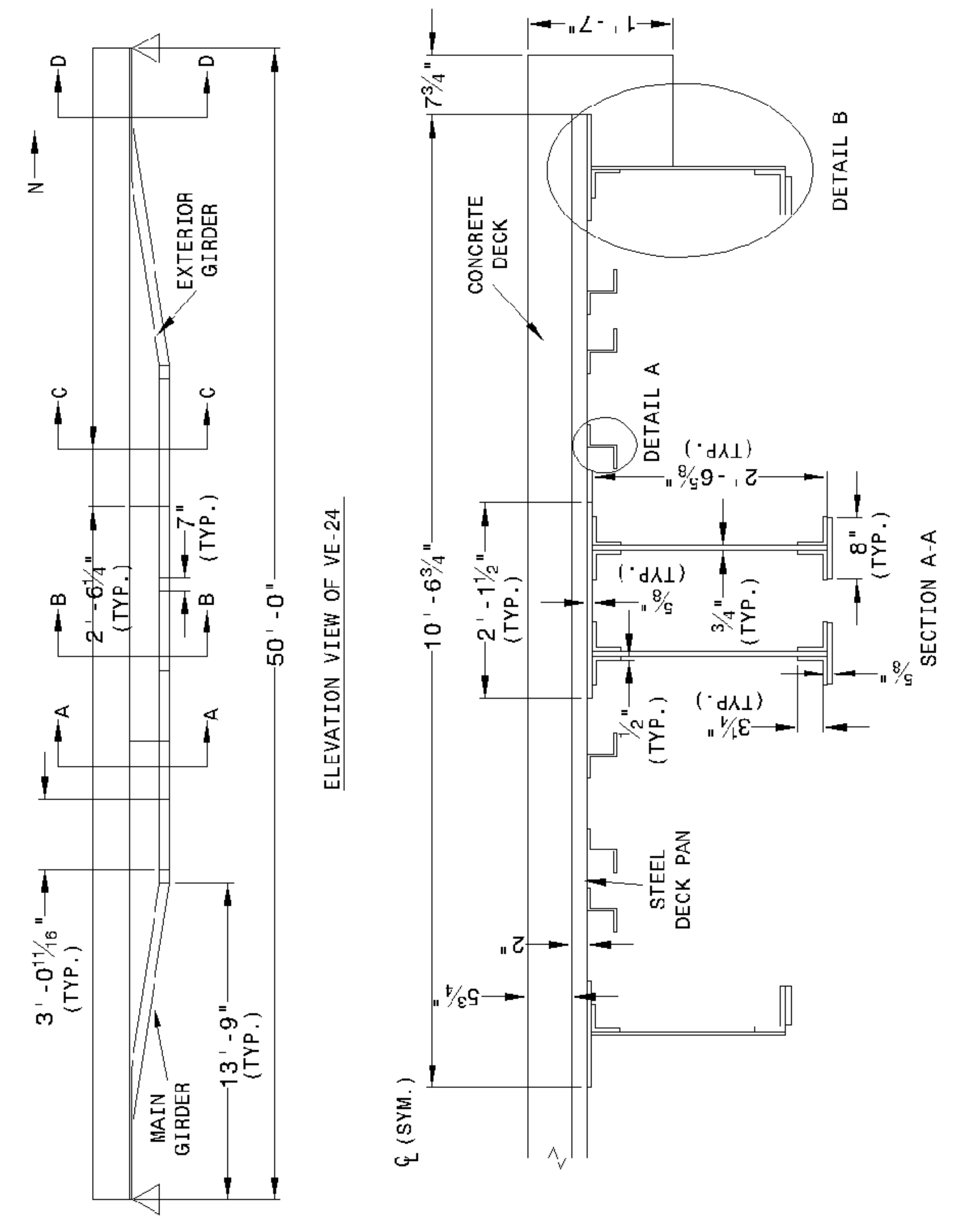
FO-256 PLANS

SHEET NO.: 8 of 8



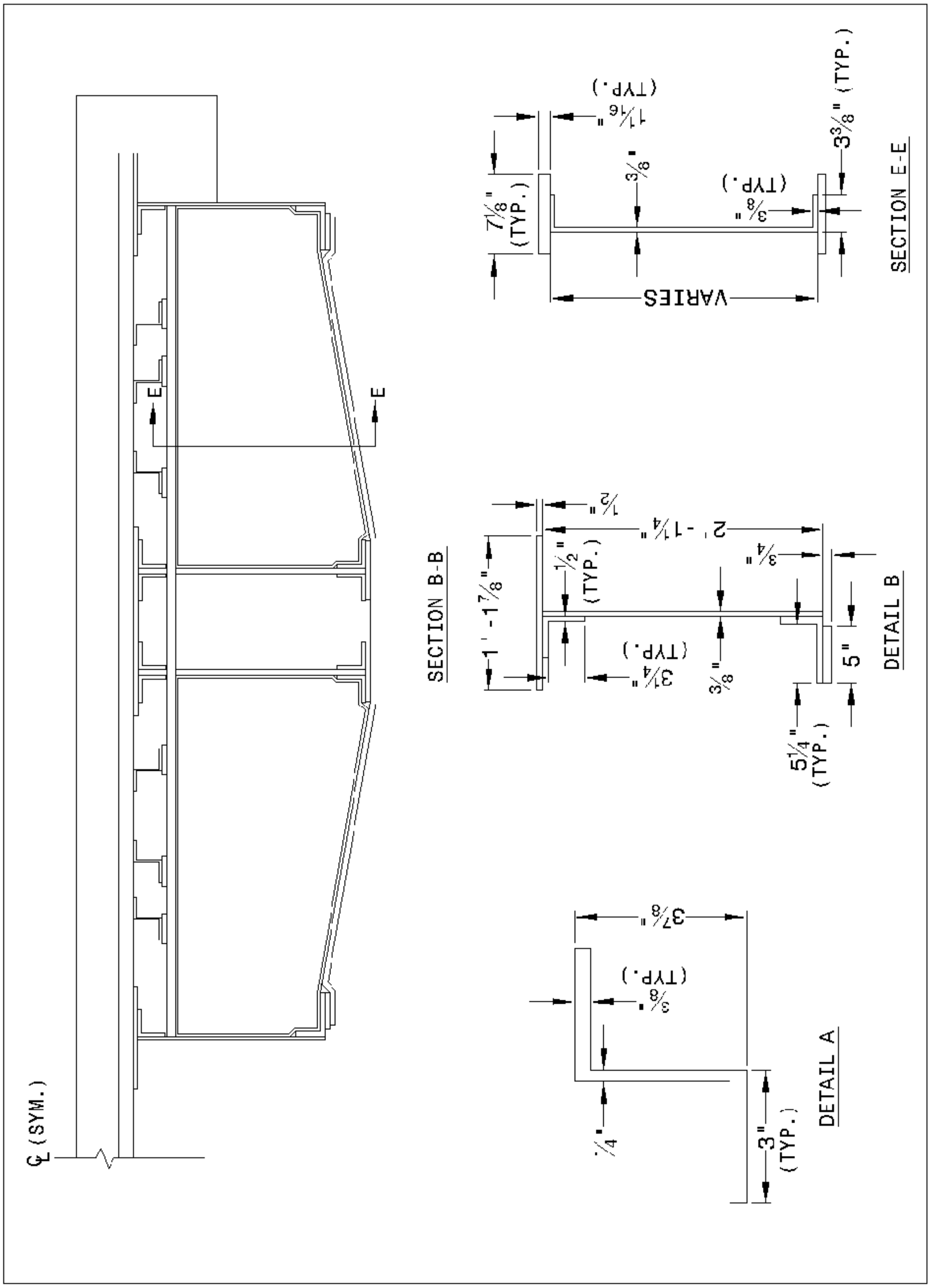
VE-24 PLANS

SHEET NO.: 1 of 7



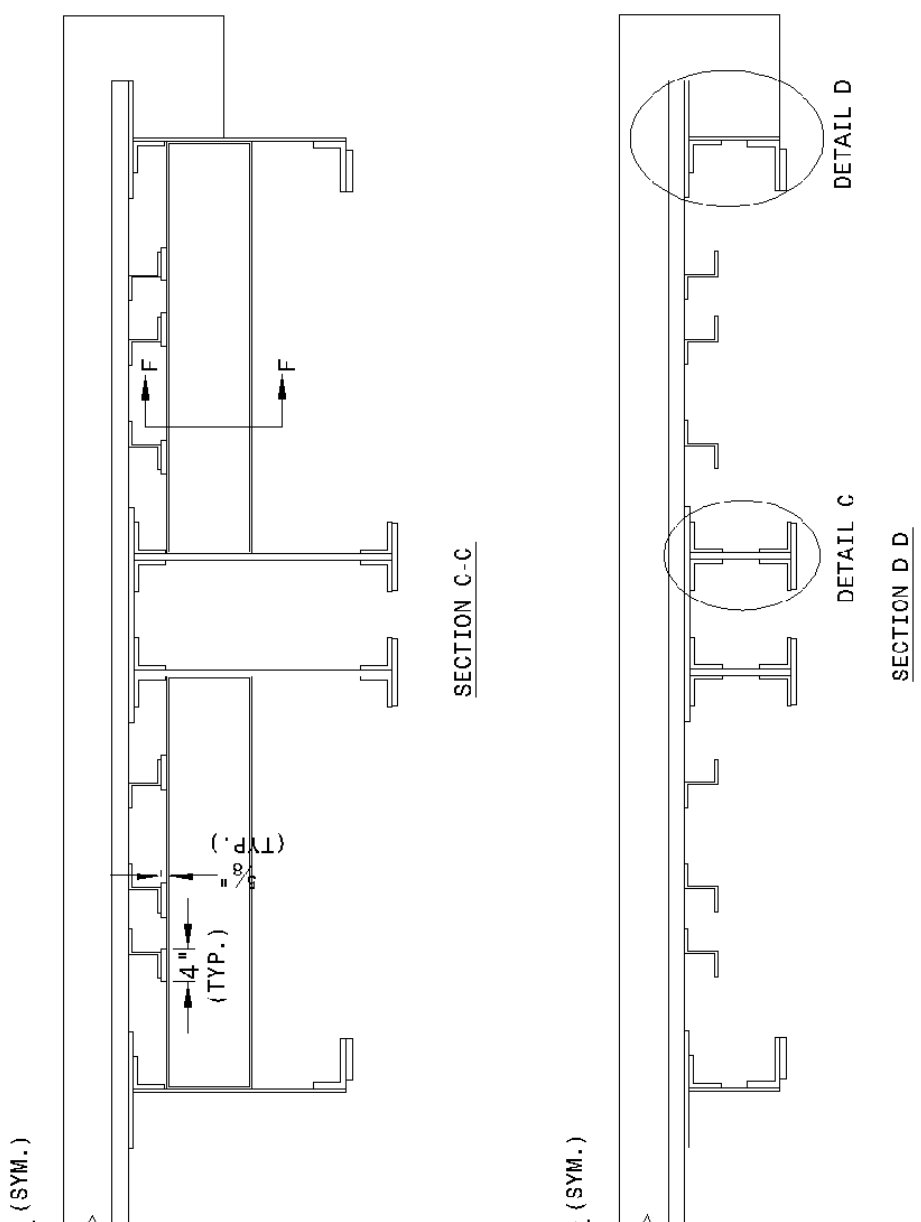
VE-24 PLANS

SHEET NO.:
2 of 7



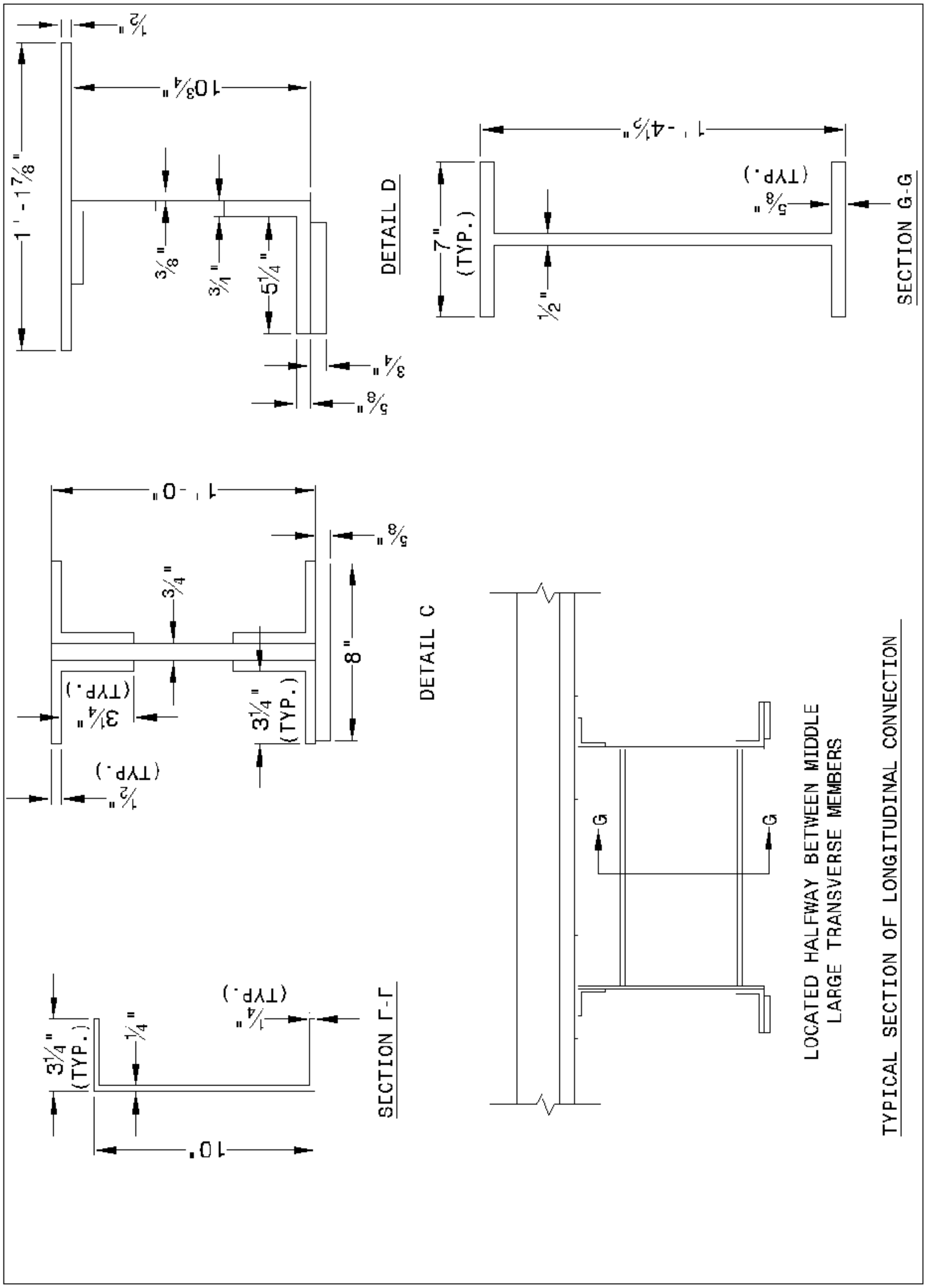
VE-24 PLANS

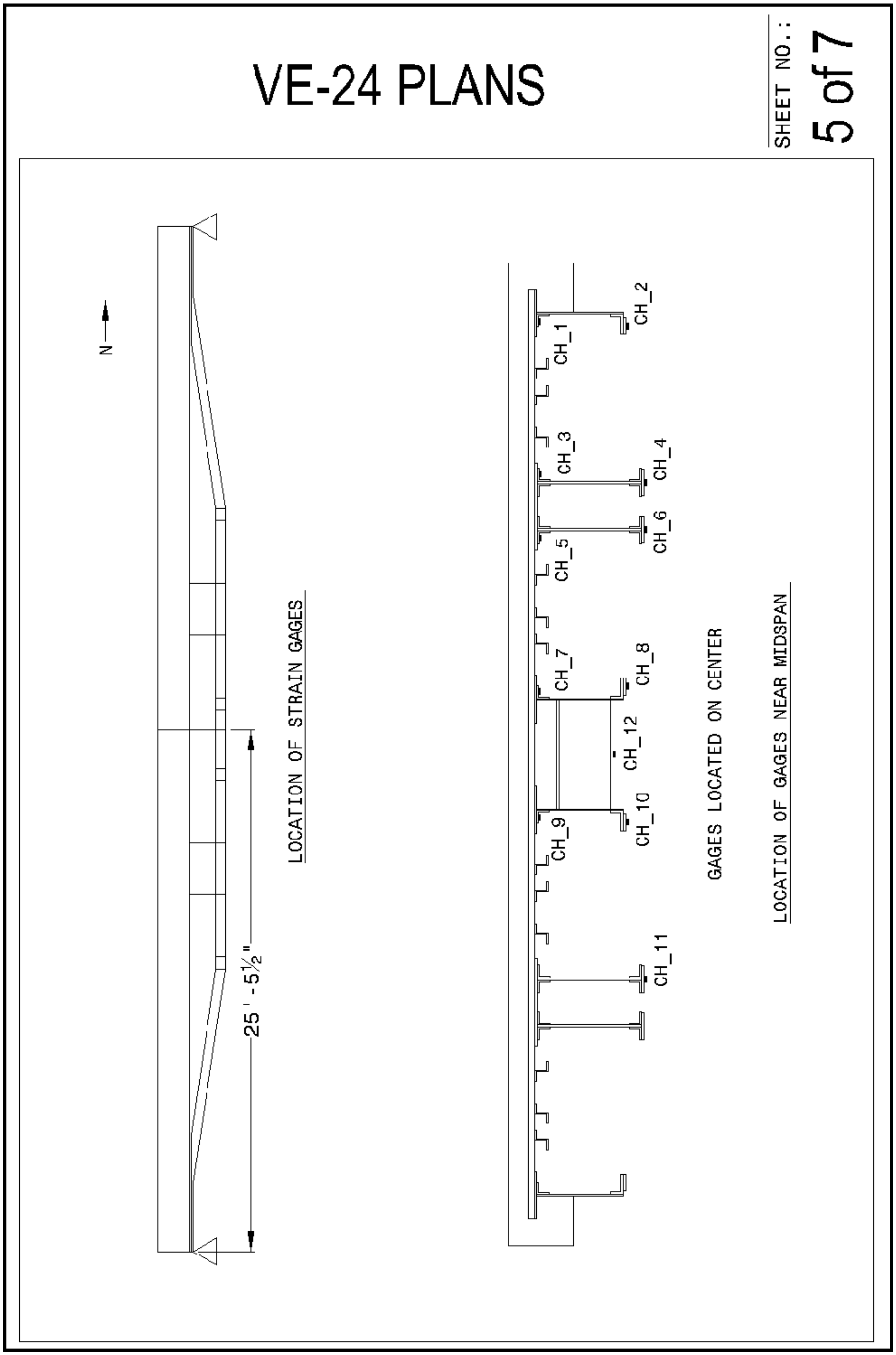
SHEET NO.: 3 of 7



VE-24 PLANS

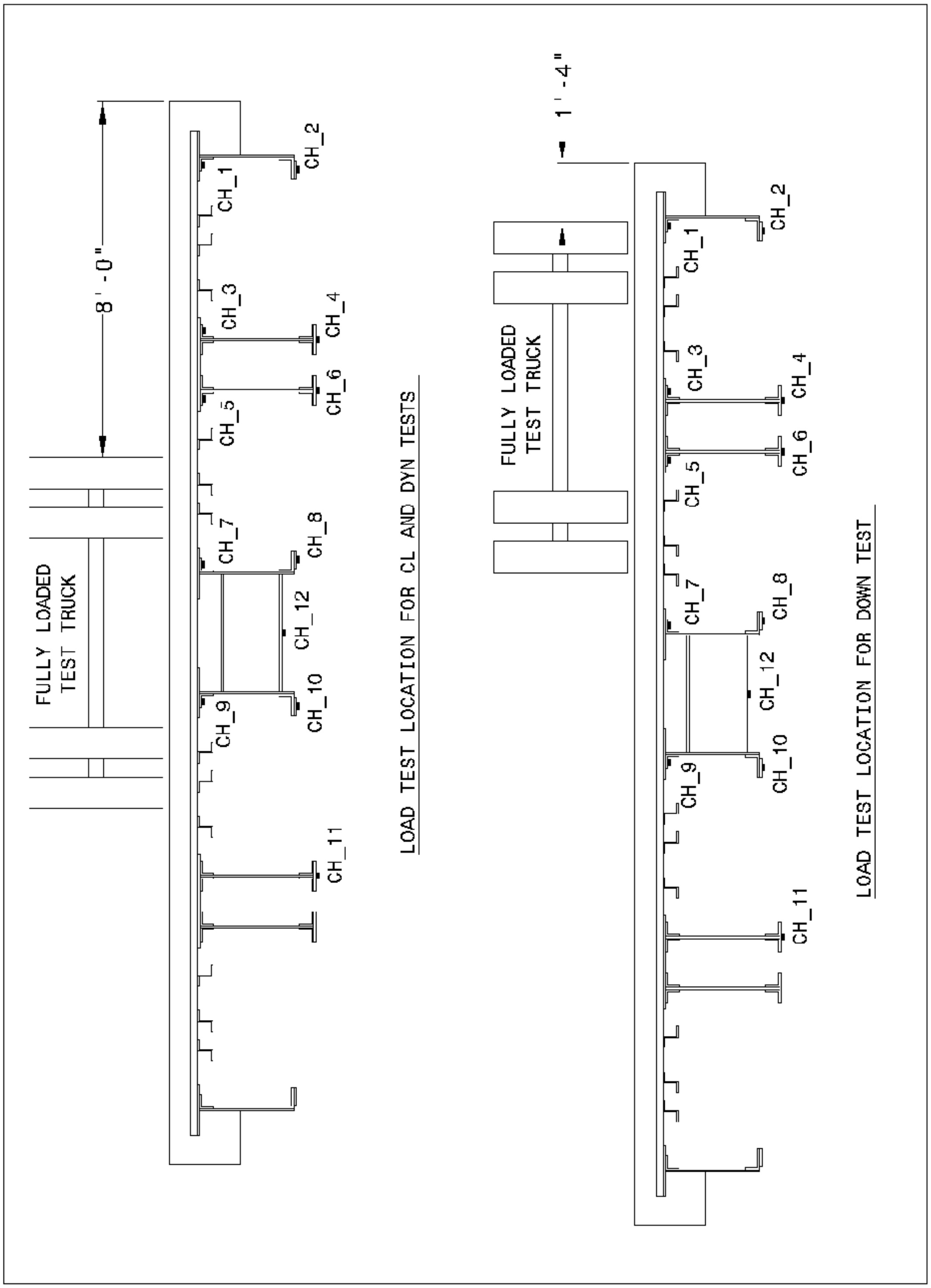
SHEET NO.: 4 of 7

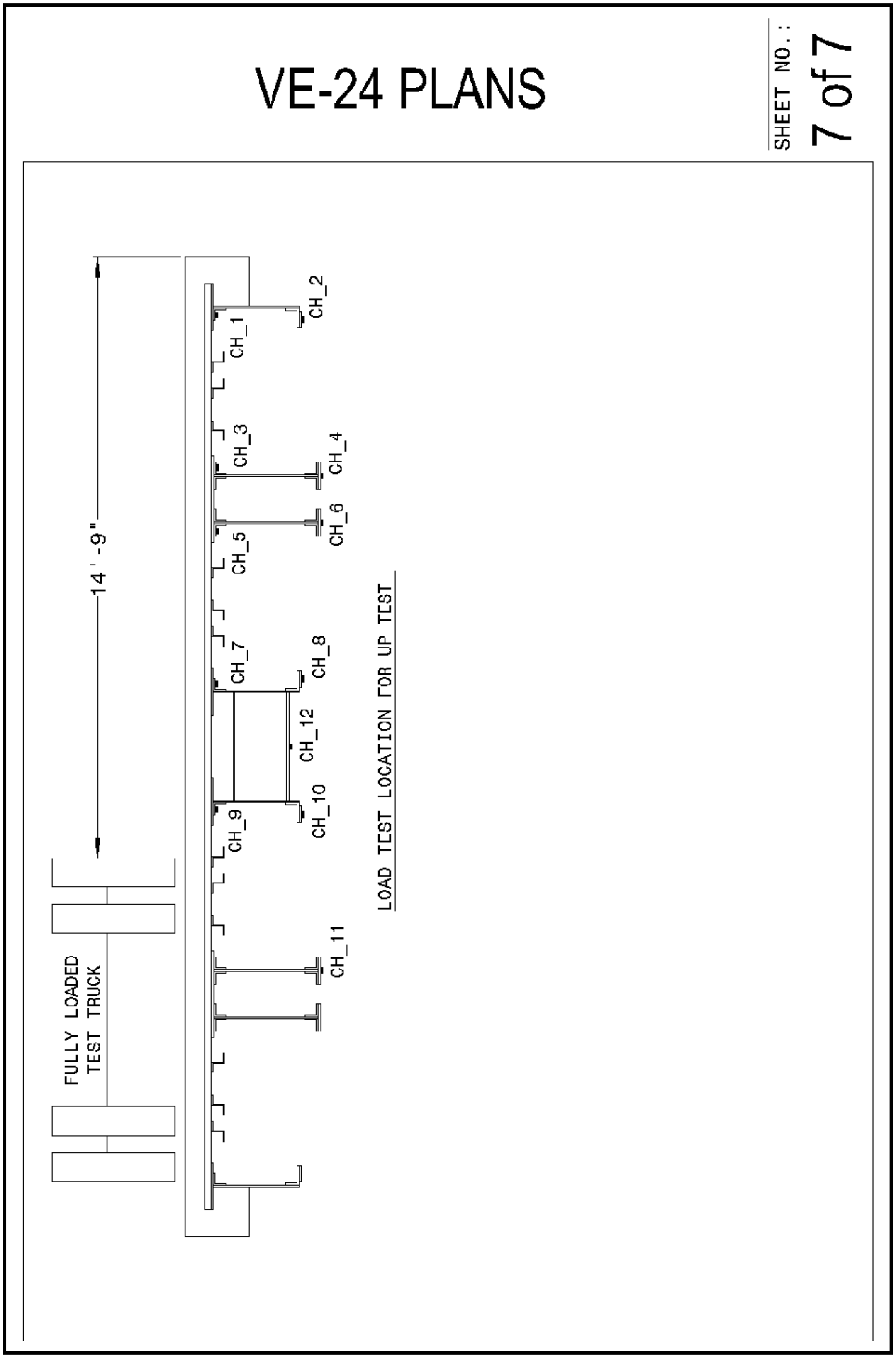



VE-24 PLANS
SHEET NO.:
5 of 7
N
25'-5½"
LOCATION OF STRAIN GAGES
CH_1
CH_2
CH_3
CH_4
CH_5
CH_6
CH_7
CH_8
CH_9
CH_10
CH_11
CH_12
GAGES LOCATED ON CENTER
LOCATION OF GAGES NEAR MIDSPAN

VE-24 PLANS

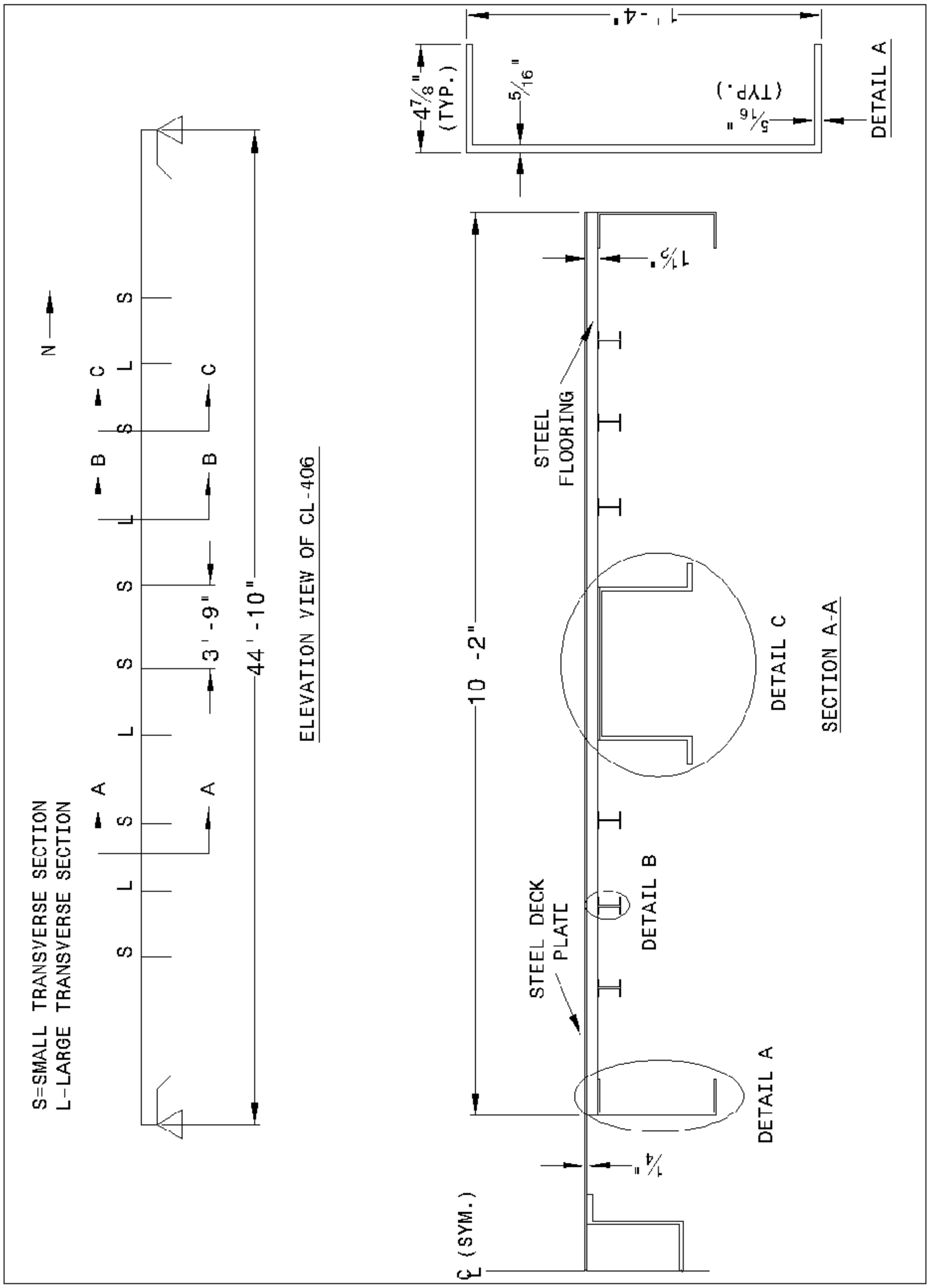
SHEET NO.:
6 of 7




VE-24 PLANS
SHEET NO.:
7 of 7
14 ' -9"
FULLY LOADED
TEST TRUCK
CH_1
CH_2
CH_3
CH_4
CH_5
CH_6
CH_7
CH_8
CH_9
CH_10
CH_11
CH_12
LOAD TEST LOCATION FOR UP TEST

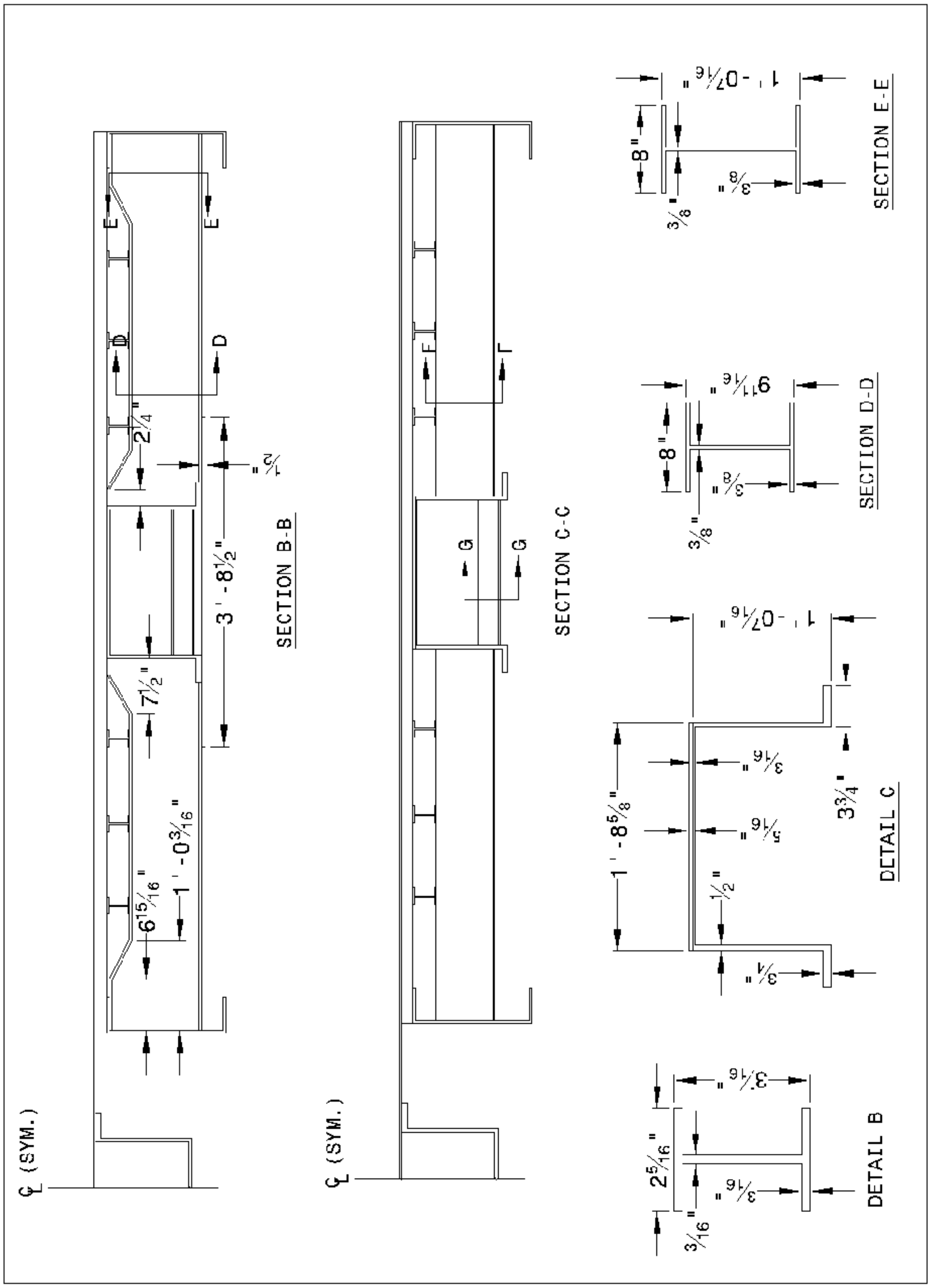
CL-406 PLANS

SHEET NO.: 1 of 5



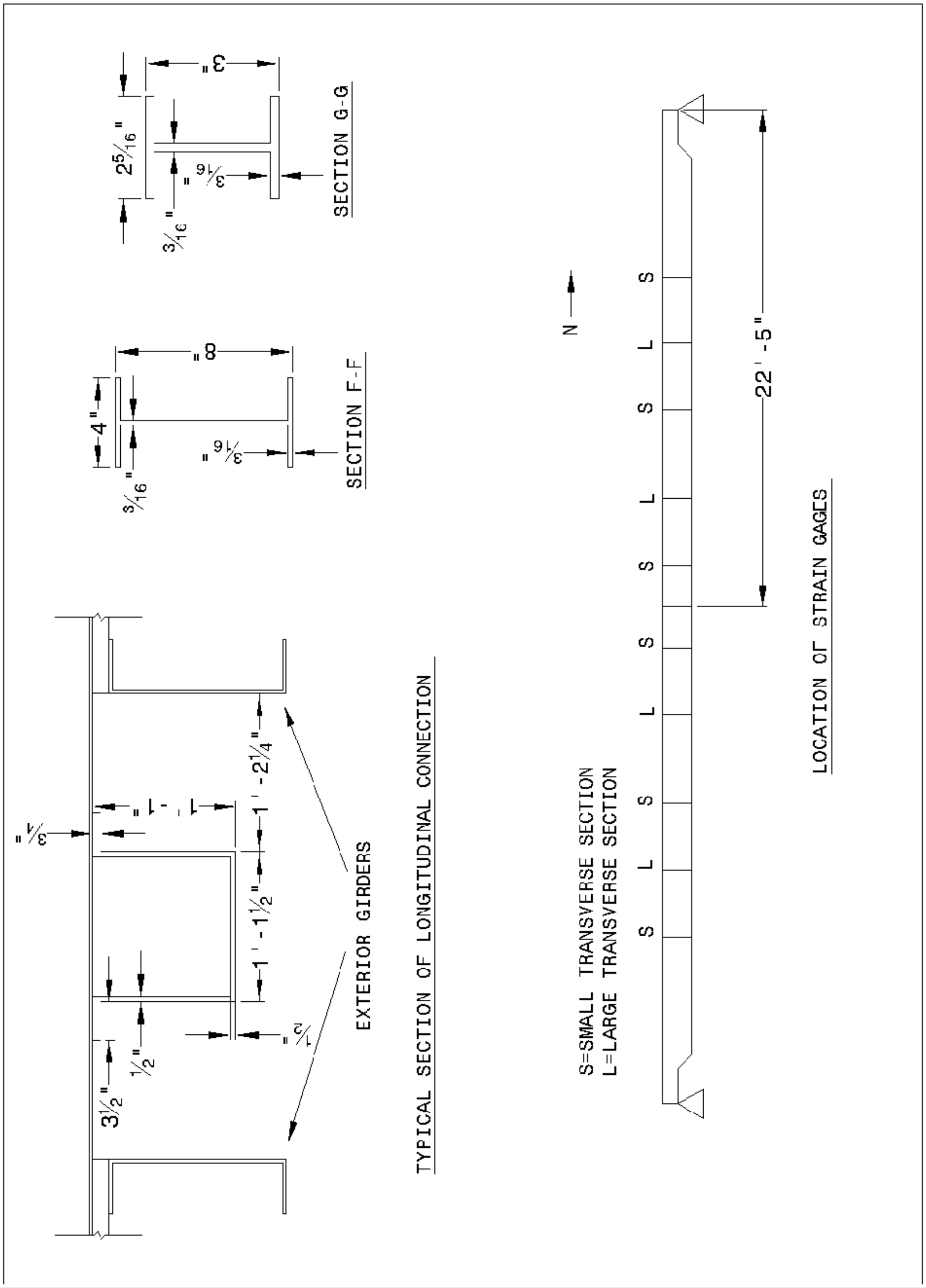
CL-406 PLANS

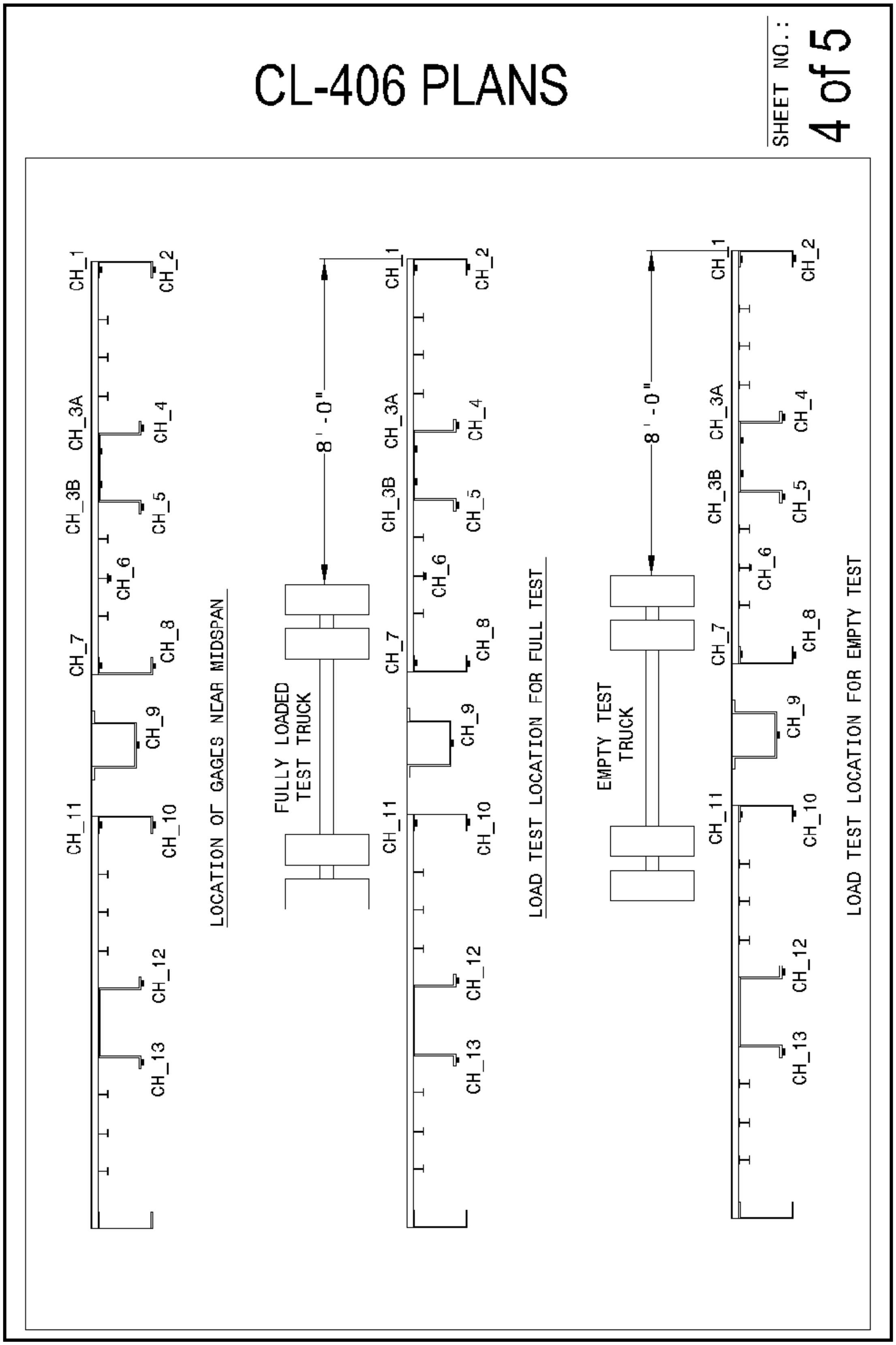
SHEET NO.:
2 of 5

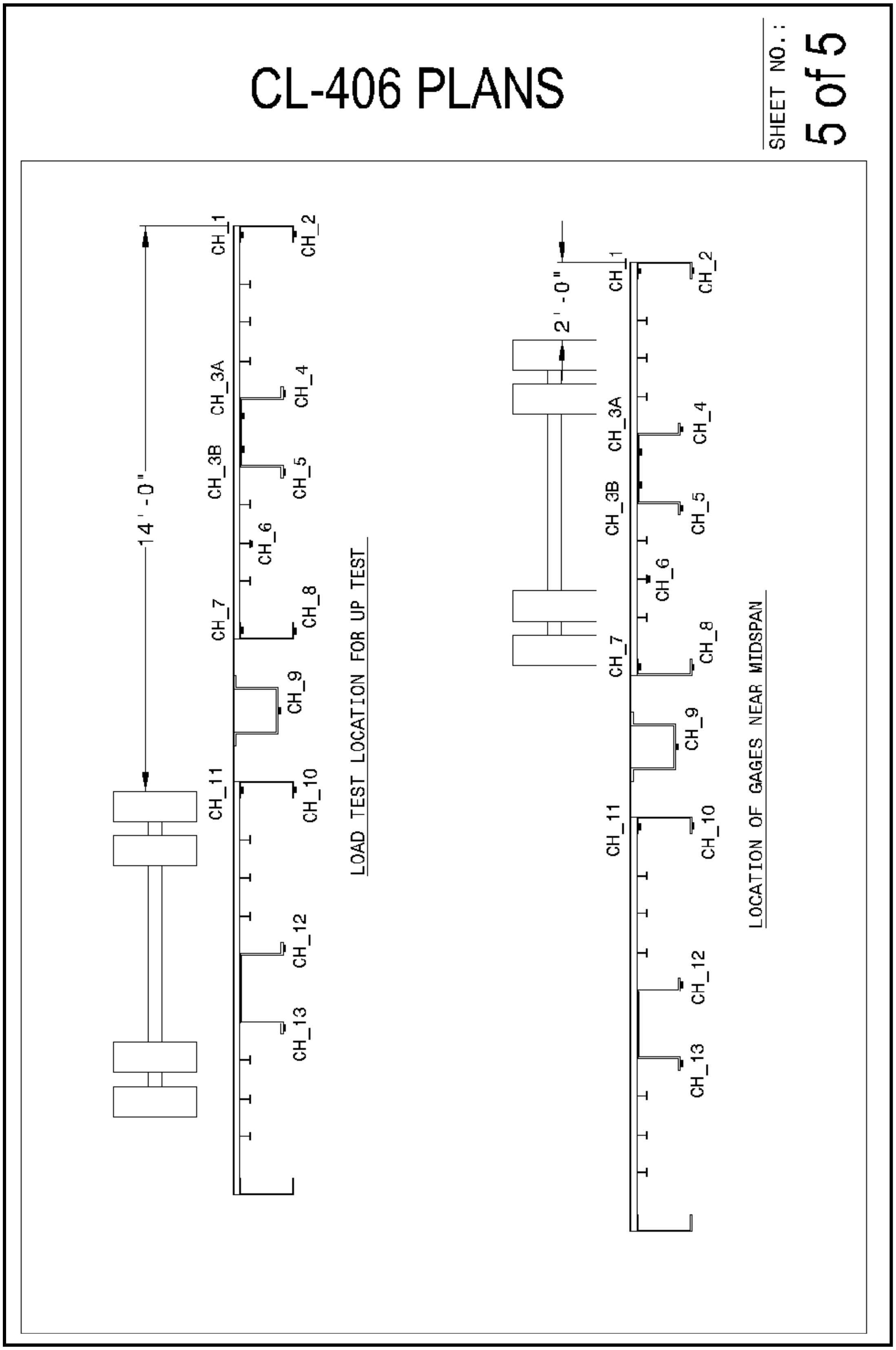


CL-406 PLANS

SHEET NO.: 3 of 5




CL-406 PLANS
SHEET NO.:
4 of 5
CH_1
CH_2
CH_3A
CH_4
CH_3B
CH_5
CH_6
CH_7
CH_8
CH_9
CH_11
CH_10
CH_12
CH_13
LOCATION OF GAGES NEAR MIDSPAN
8'-0"
FULLY LOADED TEST TRUCK
LOAD TEST LOCATION FOR FULL TEST
EMPTY TEST TRUCK
LOAD TEST LOCATION FOR EMPTY TEST


CL-406 PLANS
SHEET NO.:
5 of 5
14'-0"
CH_1
CH_2
CH_3A
CH_4
CH_3B
CH_5
CH_6
CH_7
CH_8
CH_9
CH_11
CH_10
CH_12
CH_13
LOAD TEST LOCATION FOR UP TEST
2'-0"
CH_1
CH_2
CH_3A
CH_4
CH_3B
CH_5
CH_6
CH_7
CH_8
CH_9
CH_11
CH_10
CH_12
CH_13
LOCATION OF GAGES NEAR MIDSPAN

Appendix C. Load Test Stress Results

MAIN GIRDER STRESS RESULTS

Bridge	FO-54
Load Test	CL, Crawl
Channel	Stress (ksi)
3	-3.0
4	8.5
5	-4.2
9	-3.3
10	9.7
16	-3.5

Bridge	FO-54
Load Test	UP, Crawl
Channel	Stress (ksi)
3	-2.2
4	4.5
5	-2.3
9	-4.5
10	13.8
16	-4.8

Bridge	FO-54
Load Test	UP, Crawl
Channel	Stress (ksi)
3	-4.6
4	12.0
5	-5.6
9	-2.0
10	5.6
16	-2.3

Bridge	CL-53
Load Test	UP, Crawl
Channel	Stress (ksi)
3	-0.9
4	3.3
5	-0.9
10	1.5
11	-0.6

Bridge	CL-53
Load Test	CL, Crawl
Channel	Stress (ksi)
3	-0.7
4	2.6
5	-0.7
10	2.7
11	-0.8

Bridge	CL-53
Load Test	DOWN, Crawl
Channel	Stress (ksi)
3	-0.5
4	1.5
5	-0.5
10	3.2
11	-0.9

Bridge	CL-179
Load Test	DOWN, Crawl
Channel	Stress (ksi)
4	-0.1
5	2.3
6	-0.3
10	0.5

Bridge	CL-179
Load Test	CL, Crawl
Channel	Stress (ksi)
4	-0.3
5	1.4
6	-0.3
10	1.9

Bridge	CL-179
Load Test	UP, Crawl
Channel	Stress (ksi)
4	-0.1
5	0.4
6	-0.1
10	2.9

Bridge	FO-256
Load Test	UP, Crawl
Channel	Stress (ksi)
3	-1.7
4	3.3
5	-2.2
12	1.7
13	-1

Bridge	FO-256
Load Test	CL_2, Crawl
Channel	Stress (ksi)
3	-3.6
4	7.0
5	-5.1
12	7.0
13	-4.7

Bridge	FO-256
Load Test	DECK, Crawl
Channel	Stress (ksi)
3	-1.7
4	3.3
5	-2.2
12	1.7
13	-1

Bridge	FO-256
Load Test	CL, Crawl
Channel	Stress (ksi)
3	BAD GAGE
4	2.8
5	-1.8
12	2.5
13	-1.6

Bridge	FO-25
Load Test	CL, Crawl
Channel	Stress (ksi)
3	-2.3
4	3.4
5	-2.4
11	BAD GAGE
12	2.9

Bridge	FO-25
Load Test	UP, Crawl
Channel	Stress (ksi)
3	-3.3
4	4.9
5	-3.5
11	BAD GAGE
12	1.1

Bridge	FO-25
Load Test	DOWN, Crawl
Channel	Stress (ksi)
3	-0.6
4	1.1
5	-0.6
11	BAD GAGE
12	5.6

Bridge	VE-24
Load Test	CL, Crawl
Channel	Stress (ksi)
3	-0.2
4	1.9
5	-0.2
6	2.0
11	1.7

Bridge	VE-24
Load Test	DOWN, Crawl
Channel	Stress (ksi)
3	-0.4
4	3.3
5	-0.3
6	3.0
11	0.7

Bridge	VE-24
Load Test	UP, Crawl
Channel	Stress (ksi)
3	-0.1
4	0.6
5	-0.1
6	0.6
11	2.6

Bridge	CL-406
Load Test	FULL, Crawl
Channel	Stress (ksi)
3A	-4.5
3B	-4.4
4	5.6
5	7.1
12	8.5
13	6.1

Bridge	CL-406
Load Test	DOWN, Crawl
Channel	Stress (ksi)
3A	-3.1
3B	-2.9
4	4.5
5	4.5
12	0.4
13	0.5

Bridge	CL-406
Load Test	CL, Crawl
Channel	Stress (ksi)
3A	-1.5
3B	-1.5
4	2.0
5	2.3
12	2.1
13	2.9

Bridge	CL-406
Load Test	**UP, Crawl**
Channel	**Stress (ksi)**
3A	-0.1
3B	-0.2
4	0.4
5	0.3
12	5.4
13	5.0

EXTERIOR GIRDER & STRINGER STRESS RESULTS

Bridge	FO-54	
Load Test	CL, Crawl	
	Δf (ksi)	
Channel	FA	STA
14	1.1	1.3
15	4.4	4.3
6	1.4	3.1
7	1.1	2.7

FA = front axle & STA = single tandem axle

Bridge	FO-54	
Load Test	DOWN, Crawl	
	Δf (ksi)	
Channel	FA	STA
2	0.8	1
11	0.6	1
12	1.7	2.4
13	2.7	2.3

Bridge	CL-53	
Load Test	DOWN, Crawl	
	Δf (ksi)	
Channel	FA	STA
6	1.1	2.7
8	2.6	4.1
9	4.2	4.5

Bridge	CL-53	
Load Test	DOWN, Crawl	
	Δf (ksi)	
Channel	FA	STA
2	0.7	1.0

Bridge	CL-179	
Load Test	DOWN, Crawl	
	Δf (ksi)	
Channel	FA	STA
1	-1	-1.3
2	BAD GAGE	
3	2.1	2.5
7	1.4	2.4
8	3.9	4.3

Bridge	CL-179	
Load Test	CL, Crawl	
	Δf (ksi)	
Channel	FA	STA
7	1.6	5.5
8	0.4	1.5
9	0.3	1.0

Bridge	FO-256	
Load Test	CL_2, Crawl	
	Δf (ksi)	
Channel	FA	STA
6	3.1	4.3
7	3.3	6.0

Bridge	FO-256	
Load Test	CL, Crawl	
	Δf (ksi)	
Channel	FA	STA
6	2.2	1.1
7	1.7	2

Bridge	FO-25	
Load Test	UP, Crawl	
	Δf (ksi)	
Channel	FA	STA
6	5.5	7.2
8	1.4	3.0

Bridge	FO-25	
Load Test	UP, Crawl	
	Δf (ksi)	
Channel	FA	STA
2	1.4	2.3

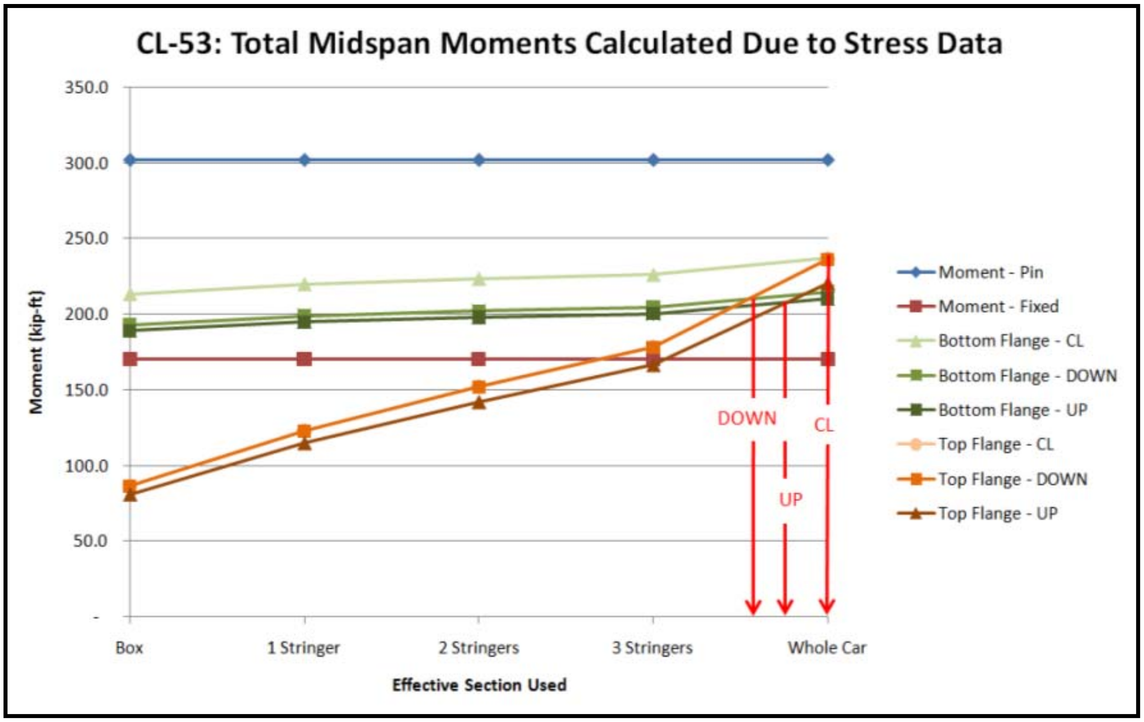
Bridge	FO-25	
Load Test	CL, Crawl	
	Δf (ksi)	
Channel	FA	STA
6	5.3	8.1
8	1.6	3.8

Bridge	CL-406	
Load Test	**DOWN, Crawl**	
Channel	**Stress (ksi)**	
2	5.0	
8	6.5	
	Δf (ksi)	
Channel	**FA**	**STA**
6	0.7	1.4

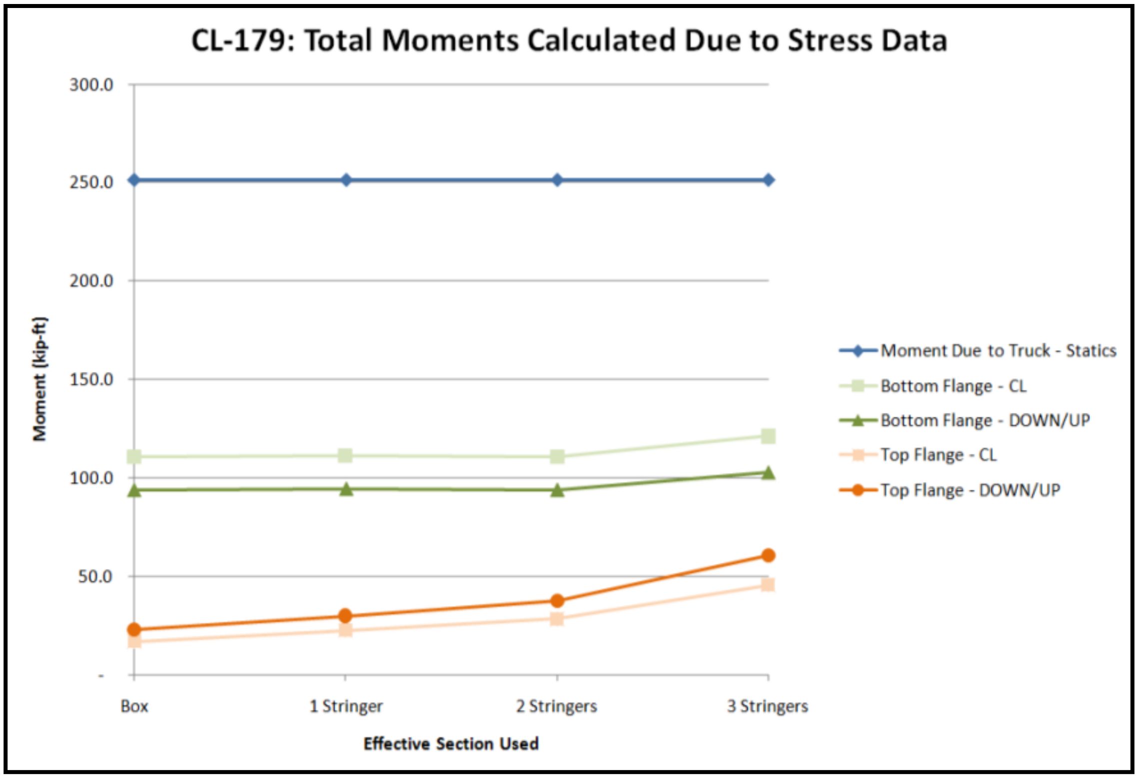
Bridge	CL-406	
Load Test	**FULL, Crawl**	
Channel	**Stress (ksi)**	
2	1.3	
8	17.4	
10	16.9	
	Δf (ksi)	
Channel	**FA**	**STA**
6	3.7	3.9

Bridge	CL-406	
Load Test	**CL, Crawl**	
Channel	**Stress (ksi)**	
2	0.4	
8	5.3	
10	4.9	
	Δf (ksi)	
Channel	**FA**	**STA**
6	2.0	1.2

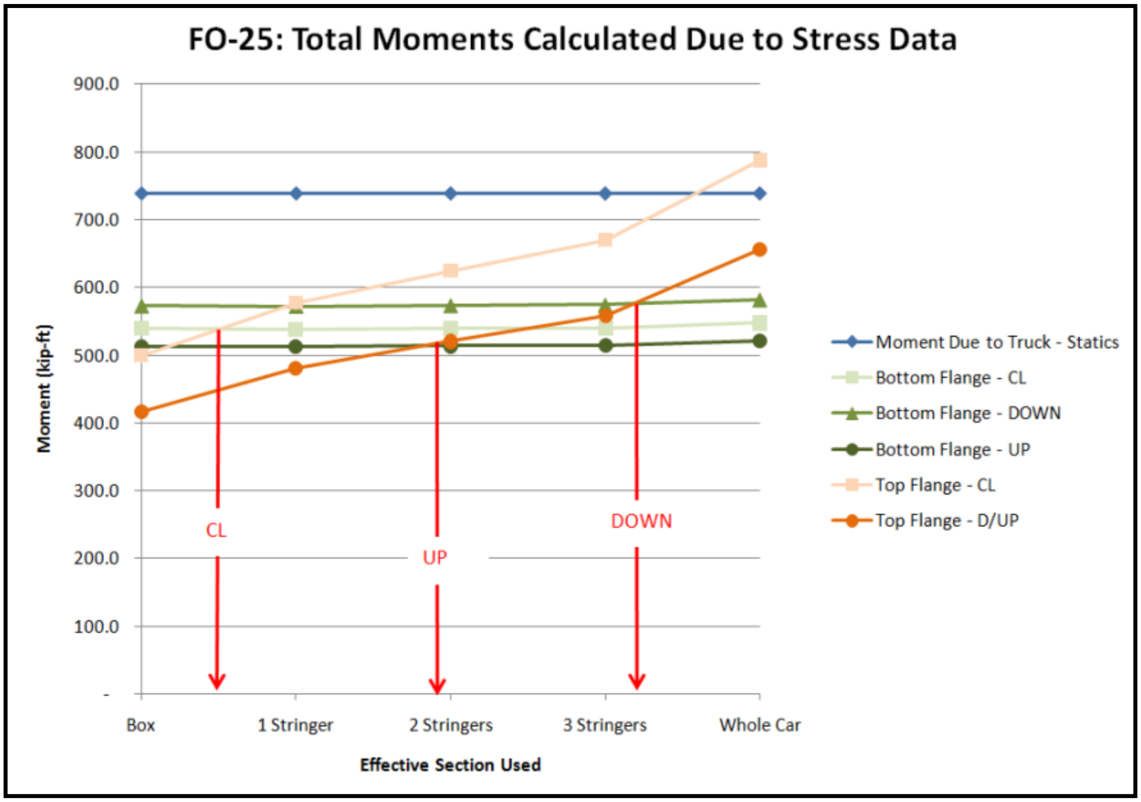
Appendix D. Effective Section Results



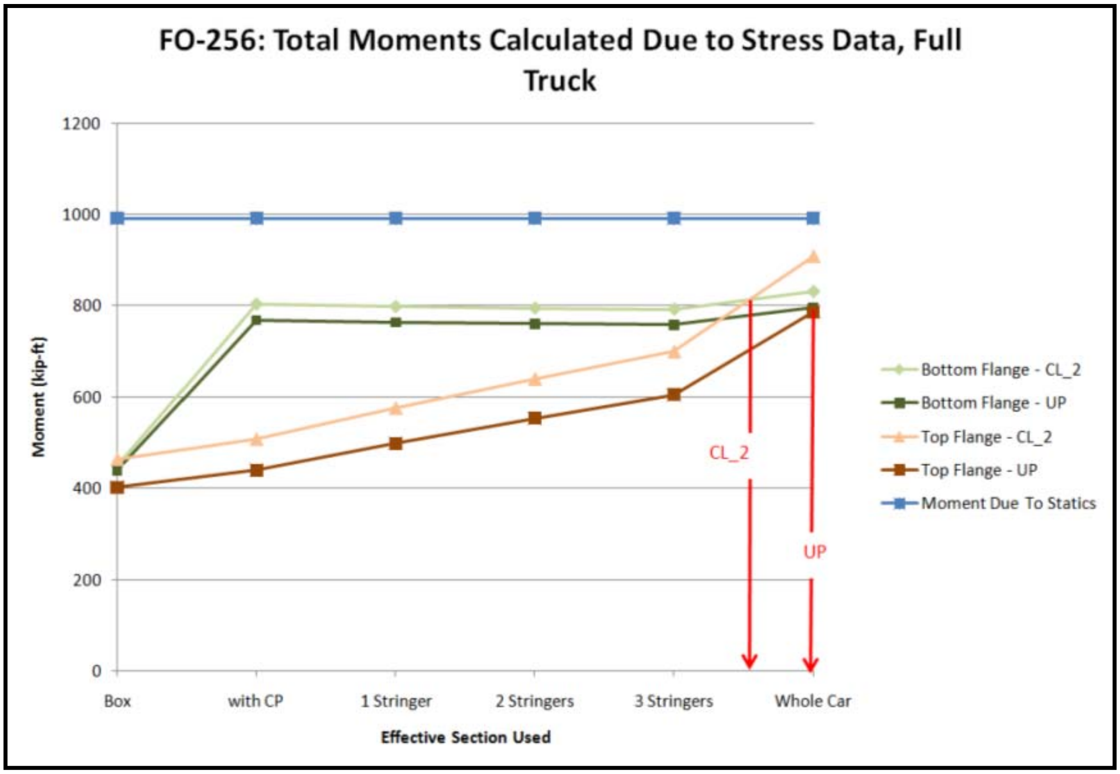
As seen in the figure for CL-53, two different types of end supports were considered for one of the abutments because this bridge utilized an integral abutment. Both the pinned and fixed conditions were used to calculate the maximum moment due to the test truck.



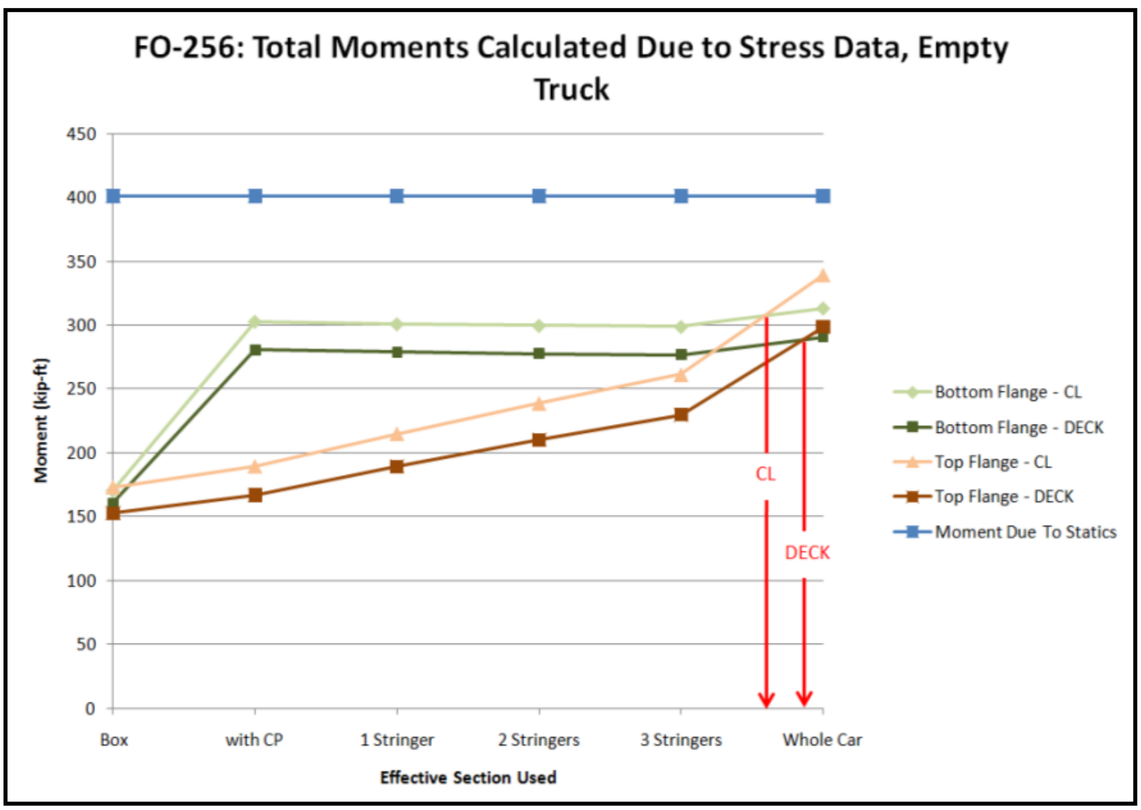
With regards to CL-179, an effective section of "2 stringers" was selected even though the moments due to the top and bottom stresses never matched. The "2 stringers" effective section was selected because it provided good agreement with other bridges and was still conservative with respect to CL-179.



As seen in the figure for FO-25, the CL test results suggest an effective section somewhere between using only the box girder and "1 stringer". The "2 stringer" section was selected, however, because it provided good agreement with the other bridges. As seen in the figure, the moment produced at the location where the top and bottom flange stresses provide agreement is still less than the maximum moment produced by the test truck. Part of this difference is accounted for with the stress modification factor, but the end result is still less than the maximum moment produced by the test truck. Thus the result is still conservative with the combination of the "2 stringers" effective section and the stress modification factor.



FO-256: Total Moments Calculated Due to Stress Data, Full Truck
1200
1000
800
600
400
200
0
Moment (kip-ft)
Box
with CP
1 Stringer
2 Stringers
3 Stringers
Whole Car
Effective Section Used
CL_2
UP
Bottom Flange - CL_2
Bottom Flange - UP
Top Flange - CL_2
Top Flange - UP
Moment Due To Statics



FO-256: Total Moments Calculated Due to Stress Data, Empty Truck
450
400
350
300
250
200
150
100
50
0
Moment (kip-ft)
Box
with CP
1 Stringer
2 Stringers
3 Stringers
Whole Car
Effective Section Used
CL
DECK
Bottom Flange - CL
Bottom Flange - DECK
Top Flange - CL
Top Flange - DECK
Moment Due To Statics

Appendix E. Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

1–INTRODUCTION	**C1**
1.1–General	**C1.1**
These guidelines describe a procedure for determining the maximum positive moment live load bending stress to be used when performing a load rating of the longitudinal flexural members of railroad flatcar (RRFC) bridges. The dead load bending stress may be calculated using traditional structural analysis techniques. Shear stresses to be used for rating may also be determined through the use of traditional structural analysis techniques.	Retired railroad flatcars are commonly used as bridges on low-volume roads in rural areas. Rating procedures and guidance for these structures are not readily available. The objective of these guidelines is to provide conservative but reasonable methods to rate these types of structures. The procedures are heavily based on data obtain from field instrumentation of several RRFC bridges.
1.2–Scope	**C1.2**
These guidelines are intended to be used for simply supported, single span RRFC bridges. Deck types which may be included consist of steel plate, timber, or concrete. The procedure described herein shall be used to determine the maximum live load bending stresses in primary and secondary longitudinal members. Primary members are defined as the main load carrying elements of a RRFC bridge. These consist of the main box girder(s) for a typical RRFC and the main girder and exterior girders for boxcars. Secondary members are defined as the structural elements which transfer load to the primary members of RRFC bridges. These consist of the exterior girders and stringers for typical RRFCs. The maximum positive live load bending stress for primary members shall be determined based on global bending of the structure. The maximum positive live load bending stress for secondary members shall be determined based on local bending of the element. The local bending stress shall then be added to the global stress to determine the total stress at a	Bridges in which the RRFC was cast in place with the abutment (i.e., integral abutments) can be considered simply supported for these guidelines.

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

particular location.

Typical RRFCs are defined as those constructed with either one or two main box girders, and generally contain one exterior girder on either side of the flatcar. There is typically a system of three longitudinal stringers located between the main girder and exterior girders, found on each side of the main girder.

Boxcars differ and hence distinguished from typical RRFCs. Instead of a box girder, the main longitudinal member typically consists of two Z-shapes facing opposite directions with their top flanges welded together.

These guidelines are intended to be applicable for all types of longitudinal connections. A longitudinal connection is defined as the connection between side by side RRFCs.

Figure 1 provides an example of railroad cars which are meant to be included within the scope of these guidelines. The figure also provides examples of which elements are defined as primary members or secondary members. Examples presented in the figure are not meant to be an all-inclusive list of railroad car types for which these guidelines are eligible, but are simply presented to provide engineers with additional guidance for load rating RRFC bridges.

The exterior girders of typical RRFCs are generally constructed with channels, while the stringers are generally constructed with inverted T-shapes or Z-shapes. Although these are typical features, the exterior girders and stringers may be constructed with different structural shapes.

Boxcars have been used as bridges after their sides and tops have been removed. These types of cars have been referred to as "car haulers." The two Z-shapes used to form the main girder generally contain a steel plate welded to the top flanges of each shape.

While it is not recommended boxcars be used as bridges, these guidelines do provide a procedure for load rating these structures.

Typically RRFC bridges are constructed by placing two (or more) RRFCs side by side. The exterior girders of adjacent RRFCs are commonly cut to form the longitudinal connection. This connection typically extends longitudinally along the length of the bridge.

Based on field studies of RRFC bridges (Provines 2011; Wipf et. al. 2007a; Wipf et. al. 2007b), there is a wide range of longitudinal connections used to connect adjacent flatcars. Particular longitudinal connection types were generally seen to be consistent within a particular area or county.

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

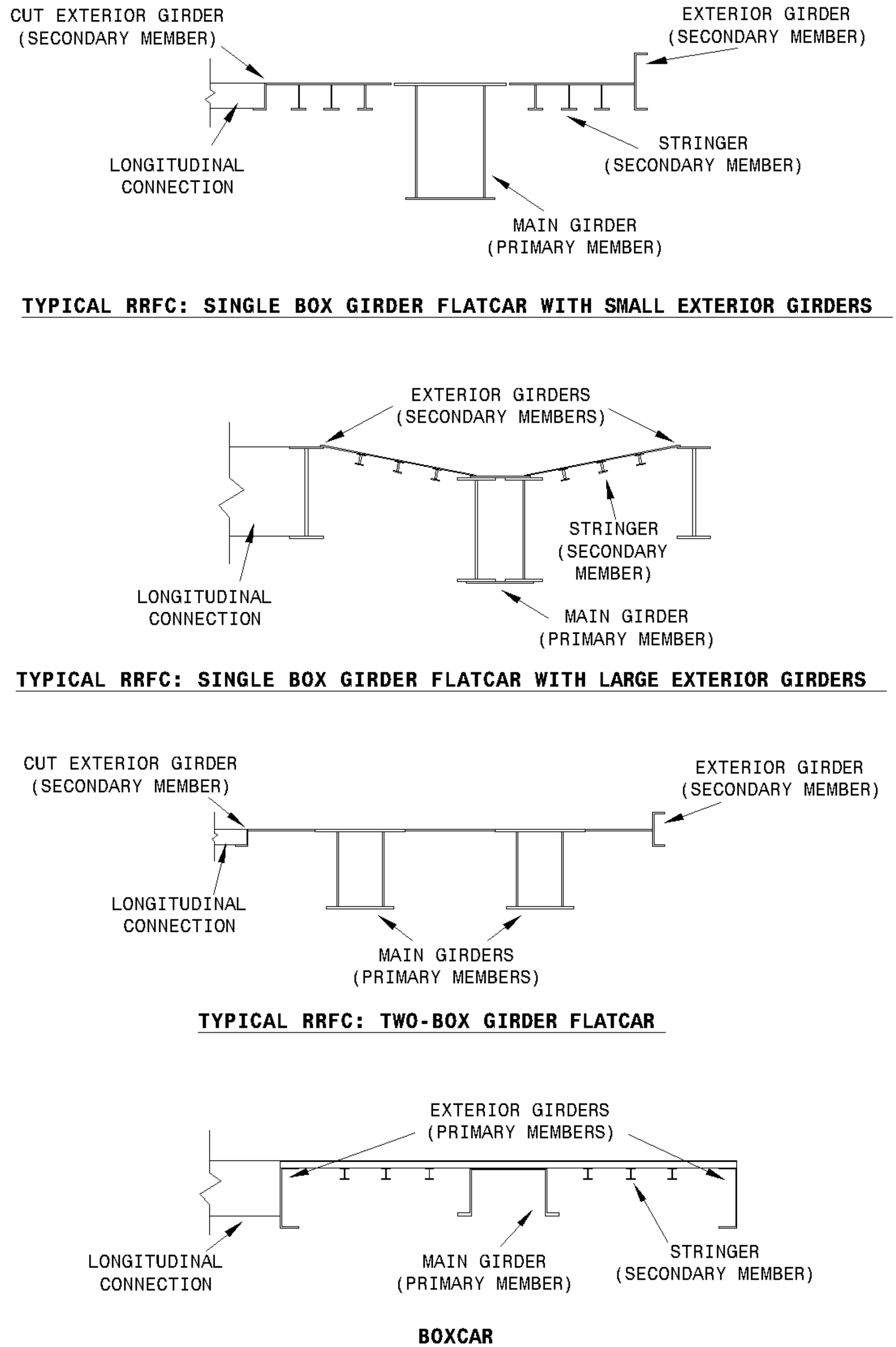


Figure 1: Examples of railroad cars included in scope & member definitions

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

1.2.1–Material Properties	**C1.2.1**
The elastic modulus of a steel RRFC may be assumed to be 29,000 ksi.	Based on coupon tests from multiple types of RRFCs (Wipf et. al. 2007a; Wipf et. al. 2007b), 29,000 ksi is an acceptable assumed elastic modulus value to be used when performing a load rating on a RRFC bridge.
The yield strength (F_y) of a steel RRFC shall be determined using one of the following methods: • Recorded from the structural plans of the RRFC • Material testing of sample taken from RRFC • An assumed value of 36 ksi	Based on discussions with several railroad companies and railroad car manufacturers (Provines 2011), the main structural elements of RRFCs have been constructed with high-strength low-alloy steels with yield strengths ranging from 50-70 ksi since the 1970's. However before the 1970's, RRFC were most likely constructed with steels with a yield strength of either 36 or 50 ksi. Therefore an assumption of a yield strength of 36 ksi is conservative. Coupon tests from multiple types of RRFCs (Wipf et. al. 2007a; Wipf et. al. 2007b), confirmed that 36 ksi is an acceptable assumed yield strength value.
The elastic modulus of concrete, if used as bridge deck, shall be determined based on *AASHTO The Manual for Bridge Evaluation.*	
1.2.2–Dynamic Load Allowance	**C1.2.2**
The static effects of the truck loads shall be increased by 33 percent to account for the dynamic effects due to moving vehicles.	Based on field instrumentation studies investigating the dynamic behavior of RRFC bridges (Wipf et. al. 2007a; Wipf et. al. 2007b), a 33 percent increase in the static bending stress provided conservative estimates for the dynamic bending stress. Although the measured dynamic impact factor varied between different RRFC bridges, a value of 33

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

	percent was chosen to be consistent with current load rating procedures in *AASHTO The Manual for Bridge Evaluation.*
1.2.3–Fatigue & Fracture Provisions The fatigue life of a RRFC bridge on a low-volume road may be considered sufficient if the truck traffic (or heavy vehicle traffic) remains low-volume over the life of the bridge. Sound engineering judgment shall be used when determining whether or not the RRFC bridge can be considered low-volume. If any fatigue cracks are found in the RRFC during its life, the fatigue life shall not be considered sufficient.	**C1.2.3** The stress ranges and number of cycles a RRFC experiences during its railroad service life are most likely much greater than those experienced during its life as a low volume road bridge. Flatcars are typically designed for heavy loads, sometimes up to 70-110 tons as discussed in Article C2.1.2, which are much greater than the majority of vehicles crossing a typical RRFC bridge. In a study investigating the use of RRFCs as low-volume road bridges (Wipf et. al. 1999), many agencies which use RRFC bridges were contacted and all of which verified that fatigue had not been an issue. If there are concerns regarding the susceptibility of fracture, Charpy V-Notch (CVN) tests may be performed on a material sample from the RRFC. The CVN results can be correlated to fracture toughness, which provides a measure of a material's resistance to fracture. However, in liu of a full fracture mechanics assessment, the CVN data may be compared to existing requirements for bridge steels.
1.3–Approach The maximum positive live load bending stress determined by these guidelines are intended to be used in conjunction with *AASHTO The Manual for Bridge Evaluation*. The guidelines are intended to be applicable for the allowable stress load rating procedure.	**C1.3** These guidelines are not applicable to the load and resistance factor rating (LRFR) or the load factor rating (LFR) because load and resistance factors were not developed. Further research is required if either of these two procedures is to be used.
2–BRIDGES CONSTRUCTED FROM TYPICAL RRFCS The following sections will describe the	**C2**

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

procedures to be used for determining the maximum positive live load bending stress in bridges constructed from typical RRFCs.	
2.1–Determination of Maximum Positive Live Load Bending Stress in Primary Members This section will describe the procedures which shall be used for determining the maximum positive live load bending stress in primary members.	**C2.1** As stated in Article 1.2, the primary members of typical RRFCs consist of the main box girder(s) located near the center of a flatcar.
2.1.1–General Equation The following general expression shall be used in determining the maximum positive live load bending stress: $\sigma_{LL} = (\alpha)\,(CDF)\,\frac{(DF)\,M_{LL}}{S_{eff}}$ (2.1.1-1) where: σ_{LL} = Maximum positive live load bending stress α = Stress modification factor as specified in Article 2.1.1.5 CDF = Car distribution factor as specified in Article 2.1.1.3 DF = Distribution factor as specified in Article 2.1.1.2 M_{LL} = Maximum positive live load moment as specified in Article 2.1.1.1 S_{eff} = Effective section modulus as specified in Article 2.1.1.4	**C2.1.1** The general equation for the determination of the maximum positive live load bending stress was developed through field instrumentation and controlled load testing of several RRFC bridges (Provines 2011).
2.1.1.1–Maximum Positive Live Load Moment	**C2.1.1.1**

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

The maximum positive live load moment (M_{LL}) shall be determined using procedures described in *AASHTO The Manual for Bridge Evaluation*.

2.1.1.2–Distribution Factor

The following expression shall be used in determining the distribution factor (DF):

$$DF = MP \leq 1.0 \qquad (2.1.1.2\text{-}1)$$

where:

DF = Distribution factor

PL = Moment proportion as specified in Article 2.1.1.2.1

C2.1.1.2

The distribution factor is intended to represent load distribution between flatcars. It is differentiated from the car distribution factor, which is intended to represent load distribution within a flatcar.

The distribution factor, as determined by Eq. 2.1.1.2-1, was developed based on field instrumentation results in which RRFC bridges were loaded with one tandem axle test truck (Provines 2011). Even if a bridge was loaded with two trucks, the data suggested that the moment proportion described in Article 2.1.1.2.1 would provide a conservative distribution factor.

2.1.1.2.1–Moment Proportion

The moment proportion (MP) shall be determined based on the lever rule, as described in the *AASHTO LRFD Bridge Design Specifications*. The lever rule shall be used to distribute the live load moment to each of the RRFCs. The reactions used when computing the lever rule shall be located at the centerline of each RRFC. The moment proportion shall be determined as follows:

C2.1.1.2.1

The load tests which resulted in the development of Eq. 2.1.1.2-1 were performed on bridges which were constructed of two RRFCs connected side-by-side (Provines 2011). It is reasonable to believe the lever rule provides conservative results for bridges with either less than two or more than two RRFCs in the cross section. For instance, if a bridge was constructed of a single RRFC, the lever rule result would be equal to 1.0. The lever rule should also be conservative if used on a bridge constructed with three RRFCs side-by-side. If a truck was located on one of the outside flatcars, according to the lever rule the flatcar on the opposite side would carry zero load provided the truck did not cross the centerline of the middle flatcar. The lever rule, and Eq. 2.1.1.2-1, were used to predict stresses

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

- If the longitudinal connection between RRFCs can be considered a rigid connection, then: MP = Result from lever rule - If the longitudinal connection between RRFCs cannot be considered a rigid connection or if there is no longitudinal connection, then: $MP = 1.0$	in multiple RRFC bridges in which field instrumentation was used (Wipf et. al. 2003; Wipf et. al. 2007a). Good correlation was found to exist between the calculated and field measured stresses. The lever rule is based on the assumption of a rigid deck. This assumption is violated if the longitudinal connection is not stiff enough in the transverse direction to be considered rigid, therefore no load can be transferred from one RRFC to the other. The evaluation of whether or not a longitudinal connection is stiff enough to transfer moment from one RRFC to another should be determined through the use of the bridge inspection report and engineering judgment.
2.1.1.3–Car Distribution Factor The car distribution factor (CDF) shall be determined as follows: - For RRFCs with one main box girder, then: $CDF = 1.0$ - For RRFCs with two main box girders, then: $CDF = \frac{3}{4}$	**C2.1.1.3** Based on field instrumentation results for RRFCs with only one main box girder, that main girder carries the entire global live load moment (Provines 2011). In other words, it is not distributed to any other members (i.e., the exterior girders) within the flatcar. No RRFCs with two main box girders were field tested in the study (Provines 2011). However, based on stress results from the single box girder RRFCs and boxcars, it seems reasonably conservative to assign a car distribution factor of 3/4 for RRFCs with two main box girders.
2.1.1.4–Effective Section The effective section modulus (S_{eff}) for bridges with RRFCs containing one main box girder shall be determined based on the	**C2.1.1.4**

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

following effective sections:

- For bridges which are constructed with RRFCs containing large exterior girders, the effective section shall consist of the entire RRFC, including the main girder, exterior girders, and any other structural longitudinal elements. Large exterior girders are defined as those which have a moment of inertia of at least 15% of the moment of inertia of the main girder.

Results from field instrumentation of RRFC bridges with large exterior girders (Provines 2011) showed it is conservative to assume the entire flatcar participates in global bending. Results from other field instrumentation studies confirmed this assumption to be reasonably conservative (Wipf et. al. 2003; Wipf et. al. 2007a).

- For bridges which are constructed with RRFCs containing small exterior girders, the effective section shall consist of the main box girder and two stringers on each side of the main girder. Small exterior girders are defined as those which have a moment of inertia of less than 15% of the moment of inertia of the main girder.

Results from a field instrumentation study showed (Provines 2011) it is conservative to assume only two stringers on either side of the main girder participate in global bending of RRFCs with smaller exterior girders.

The S_{eff} for bridges with RRFCs containing two main box girders shall be determined based on the shaded effective section shown in Figure 2. The effective section shall include any longitudinal structural elements within the section and shall have a minimum section of at least the box girder.

Although no RRFCs with two main box girders were tested, it is reasonable to believe the effective section for these types of cars is similar to RRFCs with one box girder. For RRFCs with one box girder, two stringers on each side represents roughly half the distance between the edge of the main girder and the edge of the flatcar. The effective section shown in the figure is based on the idea that half the distance between the main girder and the edge of the flatcar is participating in global bending.

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

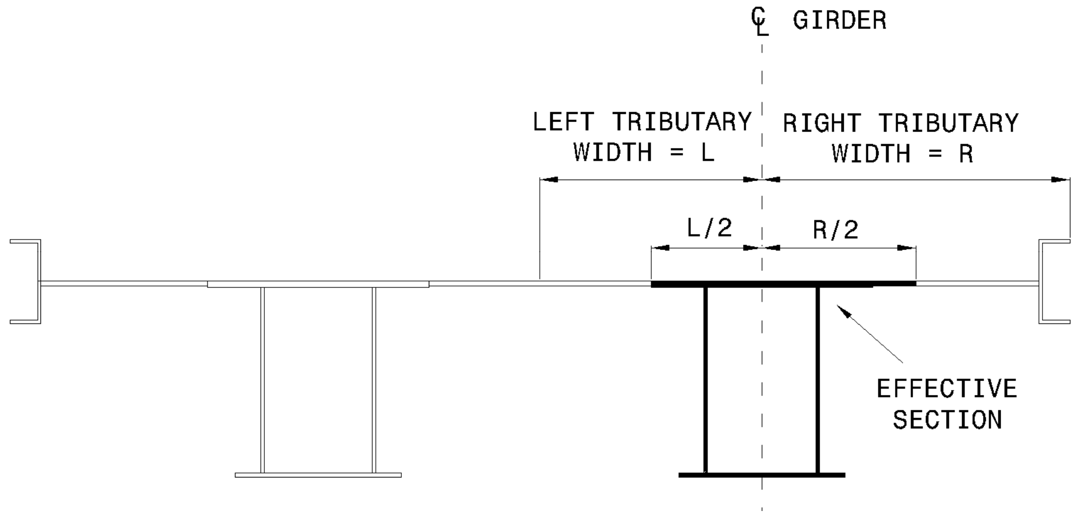


Figure 2: Effective section for typical 2-box girder RRFC

For bridges which are constructed with a composite concrete deck, the portion of the concrete deck contained in the effective section may be included when determining S_{eff}.

Composite action can be achieved through the use of shear studs, rivet heads extending from built-up members into the concrete deck, or other acceptable means of transferring load from the concrete deck to the RRFC.

Field instrumentation results from a bridge constructed of a flatcar with riveted built-up members showed composite action with its concrete deck (Provines 2011).

The dimensions used for determining the effective section shall be obtained through field measurements or as-built drawings. Any deterioration, such corrosion or cracks, in structural members shall be considered in these dimensions.

2.1.1.5–Stress Modification Factor

The stress modification factor (α) shall be taken equal to 0.75

C2.1.1.5

The stress modification factor was developed based on the field instrumentation test results to more accurately, but still conservatively, match stresses calculated using Eq.2.1.1-1 with those measured during field testing (Provines 2011). The stress modification factor of 0.75 was also verified through the results of previous field instrumentation studies of RRFC bridges

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

(Wipf et. al. 2003; Wipf et. al. 2007a). Although no bridges with RRFCs containing two box girders were tested in the field instrumentation study, it is reasonable to assume stress modification factor of 0.75 would be conservative for these types of structures.

2.1.2–Alternative Load Rating Procedure

An acceptable alternative approach to load rating the primary members of RRFC bridges is to ensure the maximum live load on the bridge is always less than the live load limit of the flatcar. For this to be an acceptable load rating approach, the RRFC shall be supported on its wheel trucks, which are defined as the locations where the wheels attach to the flatcar (shown in Figure 3). The RRFC shall be in good condition and the design live load limit shall be properly documented. The RRFC shall also have been designed after 1964.

C2.1.2

The design live load of a RRFC is called the live load limit. The live load limit is stenciled onto some RRFCs.

RRFCs are designed to be supported at the wheel trucks, thus their performance is better when they are supported at these locations. The specifications stated in Article 2.1.2 imply that flatcars which have been cut to fit a particular span length are ineligible for the alternative load rating procedure.

There was no standard loading for RRFCs prior to 1964, when the Association of American Railroads (AAR) Design Specifications were issued. Currently (AAR 2007) there are three major classifications of design live loads for RRFCs, which can be seen in Table C1.

Table C1: Design live loads for RRFCs

Live Load Limit kips (tons)	**Gross Rail Load** kips (tons)
140 (70)	220 (110)
200 (100)	263 (131.5)
220 (110)	286 (143)

In Table C1, the live load limit refers to the maximum live load that can be applied to the flatcar while the gross rail load refers to the maximum vertical load on the flatcar, including the live load plus the self weight of the flatcar.

The live load values presented in Table C1 can be applied to a RRFC in a number of different load cases, as per *AAR Manual of Standards and Recommended Practices Section C – Part II* (AAR 2007).

In a literature review performed regarding the use of RRFCs as low-volume road bridges (Provines 2011), it was not confirmed if the values in Table C1 date back to 1964 or if they were issued in a newer Specification; therefore the design loads for each particular RRFC must be known and documented when using the alternative load rating approach as specified in Article 2.1.2.

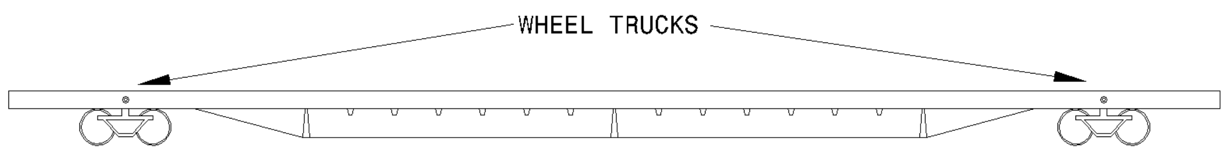


Figure 3: Location of wheel trucks on typical RRFC

2.2–Determination of Maximum Positive Live Load Bending Stress in Secondary Members

This section will describe the procedures which shall be used for determining the maximum positive live load bending stress in secondary members. The local bending stress shall then be added to the global stress to determine the total stress at a particular location.

C2.2

As stated in Article 1.2, the secondary members of typical RRFCs consist of the exterior girders and stringers.

2.2.1–Bridges With Concrete Decks

The local bending stresses in secondary members of RRFC bridges with concrete decks may be neglected.

C2.2.1

It has been shown through field testing (Provines 2011) that when a concrete deck is present, the local bending effects of secondary members are negligible.

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

2.2.2–RRFCs With Two Box Girders The following methods shall be acceptable for determining the maximum positive live load bending stress in secondary members of RRFCs with two box girders: • Orthotropic plate theory equations found in the *AASHTO LRFD Bridge Design Specifications* • Finite element analysis • Field instrumentation and testing • Any reasonable and accepted engineering method	**C2.2.2** No bridges constructed with RRFCs consisting of two box girders were tested through the use of field instrumentation (Provines 2011). Due to their large difference in geometry, it was not reasonable to presume the methods developed for RRFCs with one box girder would produce conservative stress results for RRFCs constructed with two box girders. Engineering judgment should be practiced when performing one of the four methods listed in Article 2.2.2.
2.2.3–General Equation For RRFCs With One Box Girder The following general expression shall be used in determining the maximum positive live load bending stress in secondary members of RRFCs with one box girder: $\sigma_{LL} = \frac{(DF)\,M_{LL}}{S_{eff}}$ (2.2.3-1) where: σ_{LL} = Maximum positive live load bending stress DF = Distribution factor as specified in Article 2.2.3.2 M_{LL} = Maximum positive live load moment as specified in Article 2.2.3.1 S_{eff} = Effective section modulus as specified	**C2.2.3** The general equation for the determination of the maximum positive live load bending stress in secondary members was developed through field instrumentation and controlled load testing of several RRFC bridges (Provines 2011).

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in Article 2.2.3.3.

2.2.3.1–Maximum Positive Live Load Moment

If the center-to-center span of the secondary member between adjacent transverse members is five feet or less, the following expression shall be used when determining the maximum positive live load moment:

$$M_{LL} = \frac{PL}{4} \quad (2.2.2\text{-}1)$$

where:

P = Weight of single rear axle wheel load

L = Center to center span of secondary member between adjacent transverse members

If the center-to-center span of the secondary member between adjacent transverse members is greater than five feet, the tandem and single axle wheel loads shall be positioned to establish the maximum positive live load moment. Moment equations for simply supported spans shall be used.

C2.2.3.1

Based on field measurements of RRFCs (Provines 2011), the simply supported moment equation yielded conservative, but reasonable stresses in secondary members.

The weight of a single rear axle wheel load can be determined by taking the weight of a rear axle of a design truck and dividing it by 4. The axle weight is divided by 2 because the rear axles (32 kip in HS-20 truck) in the AASHTO design trucks represent a pair of tandem axles. It has been shown through field testing that the presence of each individual axle causes local bending of secondary members. The single axle weight can then be divided by 2 again to represent the weight of each wheel load.

Although all of the RRFC bridges tested through the use of field instrumentation had secondary members with spans of less than five feet, it is reasonable to use the simply supported moment equations for determining moments on secondary members with greater span lengths.

Eq. 2.2.2-1 cannot be used for spans greater than five feet because the entire tandem can be located on the span.

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

2.2.3.2–Distribution Factor	C2.2.3.2
The distribution factor (DF) for secondary members shall be calculated as follows: • If $\frac{I_1}{I_2} \geq 3$, then: $DF = 1$ • If $3 > \frac{I_1}{I_2} \geq 2$, then: $DF = \frac{4}{5}$ • If $\frac{I_1}{I_2} < 2$, then: $DF = \frac{3}{5}$ where: I_1 = moment of inertia of secondary member being rated I_2 = largest moment of inertia of secondary member within group not being rated	Field instrumentation test results (Provines 2011) showed if one secondary member was at least three times as stiff any other secondary member in the group, it could attracted all of the live load moment. The results also showed that if the secondary members of a group were of relatively similar stiffness (e.g., less than two times as stiff), the maximum portion of the live load moment any stringer experienced was 3/5. A linear interpolation between these two results was reasonably done for secondary members with a relative stiffness between 2 and 3.
A group of secondary members shall be defined as those on one side of the main girder. The moment of inertia shall be determined based on the effective sections prescribed in Article 2.2.3.3.	A group of secondary members typically consists of one exterior girder, which may be cut if it is used to form the longitudinal connection, and three stringers.
2.2.3.3–Effective Section	**C2.2.3.3**
The effective section modulus (S_{eff}) shall be determined based on whether the secondary member has been cut and whether it is rigidly attached to a steel deck. A cut secondary member is defined as one which has had a	Many exterior girders which are located on the inside of the bridge, adjacent to another RRFC, are cut in the field in order to form a

portion of its structural shape removed. The effective section modulus shall be determined based on the following effective sections: • For exterior girders which are not cut and are rigidly attached to a steel deck, the effective section shall consist of the structural shape of the exterior girder. • For exterior girders which have been cut and are rigidly attached to a steel deck, the effective section shall consist of the remaining portion of the structural shape and a portion of the steel deck with a width equal to the width of the bottom flange of the structural shape of the exterior girder. • For exterior girders which are not rigidly attached to a steel deck, the effective section shall consist of the structural shape of the exterior girder. • For stringers which are rigidly attached to a steel deck, the effective section shall consist of the structural shape and a portion of the steel deck with a width equal to the width of the bottom flange of the structural shape of the stringer. • For stringers, which are not rigidly attached to a steel deck, the effective section shall consist of the structural shape of the stringer.	longitudinal connection between RRFCs. Field testing results (Provines 2011) showed portions of the steel deck participated in local bending if the secondary member was rigidly connected to the deck.
3–BRIDGES CONSTRUCTED FROM BOXCARS The following sections will describe the procedures which shall be used for determining the maximum positive live load bending stress in bridges constructed from boxcars.	**C3**

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

3.1–Determination of Maximum Positive Live Load Bending Stress in Primary Members	**C3.1**
This section will describe the procedures which shall be used for determining the maximum positive live load bending stress in primary members. The local bending stress shall then be added to the global stress to determine the total stress at a particular location.	As stated in Article 1.2, the primary members of boxcars consist of the main girder and the two exterior girders.
3.1.1–General Equation	**C3.1.1**
The following general expression shall be used in determining the maximum positive live load bending stress: $\sigma_{LL} = (\alpha)(CDF)\,\frac{(DF)\,M_{LL}}{S_{eff}}$ (3.1.1-1) where: σ_{LL} = Maximum positive live load bending stress α = Stress modification factor as specified in Article 3.1.1.5 CDF = Car distribution factor as specified in Article 3.1.1.3 DF = Distribution factor as specified in Article 3.1.1.2 M_{LL} = Maximum positive live load moment as specified in Article 3.1.1.1 S_{eff} = Effective section modulus as specified in Article 3.1.1.4	The general equation for the determination of the maximum positive live load bending stress was developed through field instrumentation and controlled load testing of a bridge constructed of boxcars (Provines 2011).

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

3.1.1.1–Maximum Positive Live Load Moment The maximum positive live load moment (M_{LL}) shall be determined using procedures described in *AASHTO The Manual for Bridge Evaluation*.	**C3.1.1.1**
3.1.1.2–Distribution Factor The following expression shall be used in determining the distribution factor (DF): $DF = MP \leq 1.0$ (3.1.1.2-1) where: DF = Distribution factor MP = Moment proportion as specified in Article 3.1.1.2.1	**C3.1.1.2** The distribution factor is intended to represent load distribution between boxcars. It is differentiated from the car distribution factor, which is intended to represent load distribution within a boxcar.
3.1.1.2.1–Moment Proportion The moment proportion (MP) shall be determined based on the lever rule, as described in the *AASHTO LRFD Bridge Design Specifications*. The lever rule shall be used to distribute the live load moment to each of the boxcars. The reactions used for when computing the lever rule shall be located at the centerline of each boxcar. The moment proportion shall be determined as follows:	**C3.1.1.2.1** The load tests which resulted in the development of Eq. 3.1.1.2-1 were performed on a bridge which was constructed of two boxcars connected side-by-side. It is reasonable to believe the lever rule provides conservative results for bridges using either less than two or more than two boxcars in the cross section. For instance, if a bridge was constructed of a single boxcar, the lever rule result would be equal to 1.0. The lever rule would be conservative if used on a bridge constructed with three boxcars side-by-side. If a truck was located on one of the outside boxcars, according to the lever rule the boxcar on the opposite side would carry zero load provided the truck did not cross the centerline of the middle boxcar. The lever rule is based on the assumption of

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

• If the longitudinal connection between boxcars can be considered a rigid connection, then: MP = Result from lever rule • If the longitudinal connection between boxcars cannot be considered a rigid connection, or if there is no longitudinal connection, then: $MP = 1.0$	a rigid deck. This assumption is violated if the longitudinal connection is not stiff enough in the transverse direction to be considered rigid, therefore no load can be transferred from one boxcar to the other. The evaluation of whether or not a longitudinal connection is stiff enough to transfer moment from one boxcar to another should be determined through the use of the bridge inspection report and engineering judgment.
3.1.1.3–Car Distribution Factor The car distribution factor (CDF) shall be determined as follows: • For main girders: $CDF = \frac{3}{4}$ • For exterior girders: $CDF = \frac{3}{5}$	**C3.1.1.3** The car distribution factors for each primary member of a boxcar were developed through field instrumentation results. The CDF values represent maximum distribution factors within a boxcar seen in the results.
3.1.1.4–Effective Section The effective section modulus (S_{eff}) shall be determined based on the following effective sections: • For main girders, the effective section shall consist of the structural shapes which make up the main girder. • For the exterior girders, the effective section shall consist of the structural shape of the exterior girder.	**C3.1.1.4** Based on the load testing and stress results (Provines 2011), the effective sections of the primary members of boxcar consist only of the structural shapes used to construct those members. Dissimilar to effective sections for typical RRFCs, the secondary members did not participate in global bending resistance.

Proposed Guidelines for Load Rating Bridges Constructed from Railroad Flatcars

3.1.1.5–Stress Modification Factor The stress modification factor (α) shall be taken equal to 0.75.	**C3.1.1.5** The stress modification factor was developed through field instrumentation test results to more accurately, but still conservatively, match stresses calculated using Eq.3.1.1-1 with those measured during field testing (Provines 2011).
3.2–Determination of Maximum Positive Live Load Bending Stress in Secondary Members The following methods shall be acceptable for determining the maximum positive live load bending stress in secondary members of boxcars: • Orthotropic plate theory equations found in the *AASHTO LRFD Bridge Design Specifications* • Finite element analysis • Field instrumentation and testing • Any reasonable and accepted engineering method	**C3.2** Based on the limited field testing data from a single boxcar bridge, no conclusive specific methods for determining bending stress in secondary members were developed.

Appendix F. Proposed Guidelines for Inspecting Bridges Constructed from Railroad Flatcars

Proposed Guidelines for Inspecting Bridges Constructed from Railroad Flatcars

1–INTRODUCTION	**C1**
1.1–General These guidelines describe a procedure for inspecting bridges constructed from railroad flatcars (RRFCs). The guidelines are intended to provide bridge inspectors with information about areas and details of RRFC bridges which require special attention.	**C1.1** Retired railroad flatcars are commonly used as bridges on low-volume roads in rural areas.
1.2–Scope These guidelines are intended to be used for all types of RRFCs and boxcars used as bridges on low-volume roads.	**C1.2**
1.3–Approach These guidelines are intended to be used in conjunction with *AASHTO The Manual for Bridge Evaluation.*	**C1.3**
2–SUPERSTRUCTURE	**C2**
2.1–Primary Members The primary members of RRFC bridges are those which carry the majority of the load. For typical RRFCs, the primary member is defined as the large box girder(s) located near the center of the car. For boxcars, the primary members are defined as the main girder located in the center of the car and the exterior girders on the sides of the car. These members shall be inspected carefully for damage, such as cracking, impact damage, and corrosion. Welds on the box girder shall be inspected extensively for cracking. Bottom flange cover plates shall be inspected to evaluate if they are properly connected to the primary member. Built-up members shall be evaluated for corrosion damage since they	**C2.1**

contain pockets for water and debris to accumulate.

2.2–Secondary Members — **C2.2**

Secondary members of RRFC bridges are those which transfer load to the primary members. Secondary members for RRFCs are defined as the exterior girders and stringers. For boxcars, the secondary members are defined as the stringers.

Secondary members of RRFCs are more likely to be damaged than the primary members due to their relative small size. Damage to the secondary members can consist of corroded, bent, fractured, or completely missing members. Although damage to a secondary member is not as critical as a primary member, it shall be reported and considered when performing a load rating.

3–SUBSTRUCTURE — **C3**

3.1–Location of Wheel Trucks — **C3.1**

RRFCs are designed to be supported at the wheel trucks, where the wheels connect to the flatcar. An example of this is shown in Figure 1. If the RRFC is being supported outside of the wheel truck locations, the ends of flatcar shall be inspected for damage such as bent or cracked members.

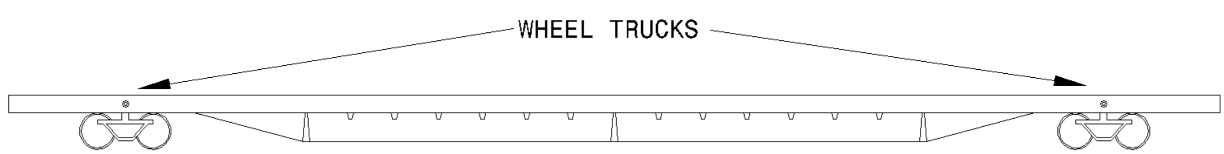


Figure 1: Location of wheel trucks on typical RRFC

Proposed Guidelines for Inspecting Bridges Constructed from Railroad Flatcars

3.2–Intermediate Supports Some RRFC bridges have intermediate supports located along the length of the bridge. These supports can be made of steel sections, concrete pedestals, etc. All intermediate supports shall be evaluated to ensure they are providing adequate support to the flatcar. The supports shall be inspected to evaluate if they are performing within reasonable limits of rotation and translation.	**C3.2**
3.3–Bearing Areas Many RRFC bridges are not supported on typical bridge bearings. Instead, shims, dirt, or other miscellaneous items are typically used to transfer load from the RRFC to the support. Since the bearings of most RRFC bridges are not standard, they shall be evaluated to ensure the bridge is safely supported.	**C3.3**
3.4–Integral Abutments Some RRFCs are cast integral with the end abutments. These types of abutments shall be evaluated for cracking in the abutment and corrosion of the RRFC.	**C3.4**
4–DECK The deck of RRFC bridges shall be evaluated to ensure it provides an adequate roadway surface. Many RRFC bridges contain thin steel plate decks, which are susceptible to cracking, local yielding, and complete fracture causing holes to form in the deck. Timber decks shall be inspected for section loss. If holes are found in the deck, the areas of the superstructure directly under the holes shall be inspected for corrosion damage.	**C4** Refer to *AASHTO The Manual for Bridge Evaluation* for additional guidance with regards to the inspection of bridge decks.

Proposed Guidelines for Inspecting Bridges Constructed from Railroad Flatcars

5–LONGITUDINAL CONNECTION		C5
Many RRFC bridges are formed by placing two (or more) flatcars side by side and connecting them with some form of longitudinal connection. These longitudinal connections can be simple, such as welding a steel plate to two RRFCs, or more complex, involving many steel sections (longitudinally and transversely) welded to each flatcar. No matter the longitudinal connection type, it shall be inspected to ensure it is a rigid connection between the two cars. Since many of these connections are not designed for a standard loading, they are especially susceptible to deterioration. Welded connections shall be inspected for cracks. Many connections contain pockets for debris and water to collect. These shall be inspected for corrosion damage. Connections which consist of a thin steel plate with little support shall be inspected extensively for cracking.		

Appendix G. Proposed Guidelines for Acquiring Railroad Flatcars to be Used as Low-Volume Road Bridges

Proposed Guidelines for Acquiring Railroad Flatcars to be Used as Low-Volume Road Bridges

BACKGROUND & SCOPE

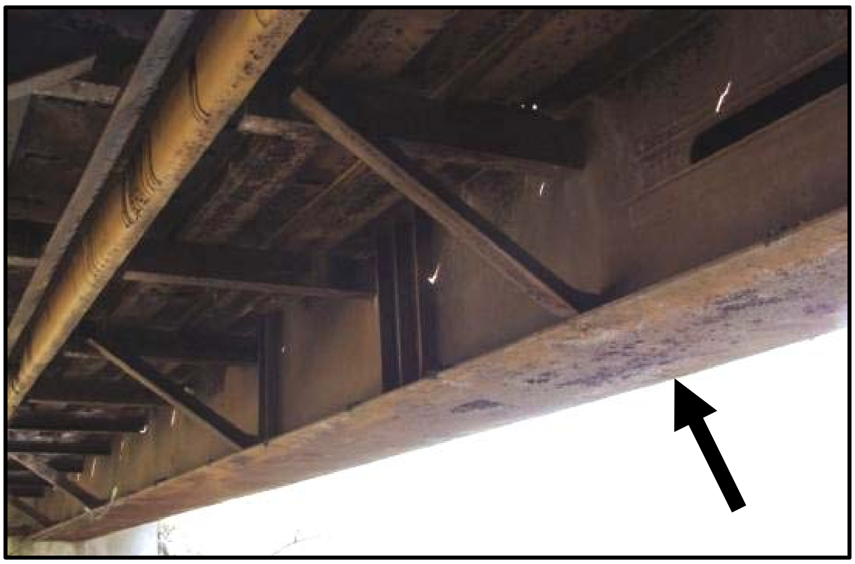
These guidelines are intended to provide personnel with guidance to assist them in making an informed decision regarding whether or not a given RRFC is suitable to be used as a low-volume road bridge. The guidelines were developed as part of an Indiana Local Technical Assistance Program (LTAP) research project. As part of the project, a Research Team from Purdue University reviewed past research and experiences other agencies have had with RRFC highway bridges, as well as having multiple discussions with personnel from railroad car manufacturers, railroad companies, and county officials who have a great deal of experience with these structures. The Research Team also visited 75 RRFC bridges and performed field instrumentation and controlled load testing on seven such bridges in Indiana. These guidelines reflect a culmination of these efforts.

These guidelines are intended to be used by county transportation officials and anyone else involved in the purchase or acquisition of RRFCs to be used as bridges. They are meant to be applicable to various types of RRFCs, as there are numerous types of railcars.

GEOMETRY

- The span length of the proposed bridge should be equal to or shorter than the distance between the centerline of the wheel truck supports. RRFCs are designed to be supported at these locations; therefore they perform better as bridges if they are supported at the wheel trucks. If the proposed span length is greater than the distance between the wheel trucks, intermediate supports (piers, etc.) and additional RRFC spans or alternative systems (i.e., steel rolled beams) should be considered.

- The main box girder of a RRFC should be large enough to support all traffic loads expected to use the proposed bridge. Since the main girder carries the majority of the load on the flatcar, its size has a great influence on how much load the bridge can carry and/or if the bridge will need to be load posted. The strength of the main girder can be verified through a basic engineering strength calculation. An example photograph of the main box girder is shown in the picture below.



Photograph of typical main box girder

- Since many RRFC bridges are formed by placing two (or more) flatcars side by side, the exterior girders of a RRFC should be suitable for forming the required longitudinal connection between the two cars. Depending on the type of longitudinal connection which will be used, considerations should be made for the size of the exterior girder and how easy it will be to form a connection between flatcars.

- The width of a RRFC should be considered in order to provide an adequate driving width. RRFCs which are narrow will require a wider longitudinal connection, which can be more problematic than narrower longitudinal connections.

Proposed Guidelines for Acquiring Railroad Flatcars to be Used as Low-Volume Road Bridges

- If two or more RRFCs are to be used side by side to form a bridge, they should have vertical similar cambers or longitudinal profiles. This will make constructing the longitudinal connection between the two cars easier, and will make for a smoother driving surface.

- Boxcars, or “car haulers”, are not recommended to be used as highway bridges. Experience has shown that traditional RRFCs have demonstrated much better performance as bridges than boxcars. A photograph of the underside of a typical boxcar is shown below. Note the shallow longitudinal load carrying elements.

Photograph of typical boxcar

Proposed Guidelines for Acquiring Railroad Flatcars to be Used as Low-Volume Road Bridges

CONDITION

- The main box girder of the RRFC should be visually inspected for signs of damage including corrosion and cracking. RRFCs with main girders which have significant cracking or corrosion should not be used as bridges unless the feasibility of repair is considered. Cracks, if present, can commonly be found extending out from welds.

- In addition to inspecting the main girder for damage, the overall RRFC should be visually inspected for damage. Damage could include bent members, cracks, corrosion, or members which are altogether missing. RRFCs with a significant amount of damage should not be used as bridges. Typically when some members are found to be damaged, others are damaged as well.

- The deck of the RRFC should be inspected to determine whether or not it is suitable to be used as a bridge deck. Steel plate decks should be inspected for holes or locations where the deck is bent. If the current deck is in poor condition, a new deck should be considered before the RRFC is put in place as a bridge.

- The paint/coating condition should be assessed to determine if a new coat of paint should be applied before placing the RRFC as a bridge. A satisfactory coat of paint can play an important role in protecting the RRFC from corrosion damage in the future life of the bridge.

ADDITIONAL CONSIDERATIONS

In addition to the previously stated guidelines, there are other items county officials (or others) may want to consider before purchasing a RRFC to be used as a low volume road bridge. These considerations could include the items listed below. It should be noted, these items are not meant to be standard specifications, but are simply listed as items to consider.

- How the RRFC will be transported to the bridge site
- How the RRFC will be picked (i.e., using a crane, excavator, etc.)
- Type of longitudinal connection which will be used (if any)

Appendix H. Load Rating Examples

TYPICAL RRFC BRIDGE LOAD RATING EXAMPLE

A sample bending stress load rating, using the allowable stress method as described in *AASHTO The Manual for Bridge Evaluation*, will be performed on a typical RRFC bridge. The load rating will be for one lane loaded and will be an Inventory level rating. References made to *AASHTO The Manual for Bridge Evaluation* will be shown in parenthesis.

The following general expression, as stated in *AASHTO The Manual for Bridge Evaluation* (Eq. 6B.5.1-1), will be used in determining the rating factor of the structure:

$$RF = \frac{C - A_1 D}{A_2 L\,(1 + I)}$$

where:

RF = Rating factor for the live load carrying capacity

C = Bending stress capacity of the member

D = Dead load bending stress on the member

L = Live load bending stress on the member

I = Impact factor to be used with live load effect = 0.33

A_1 = Factor for dead loads = 1.0

A_2 = Factor for live loads = 1.0

The following general expression, as stated in *AASHTO The Manual for Bridge Evaluation* (Eq. 6B.5.1-2), will be used in determining the bending stress load rating of the bridge:

$$RT = (RF)W$$

where:

RT = Bridge member rating (tons)

W = Weight of nominal truck used in determining the live load bending stress, L (tons)

For the allowable stress method, the load factors are as follows (6B.5.2):

$$A_1 = 1.0 \;\; \& \;\; A_2 = 1.0$$

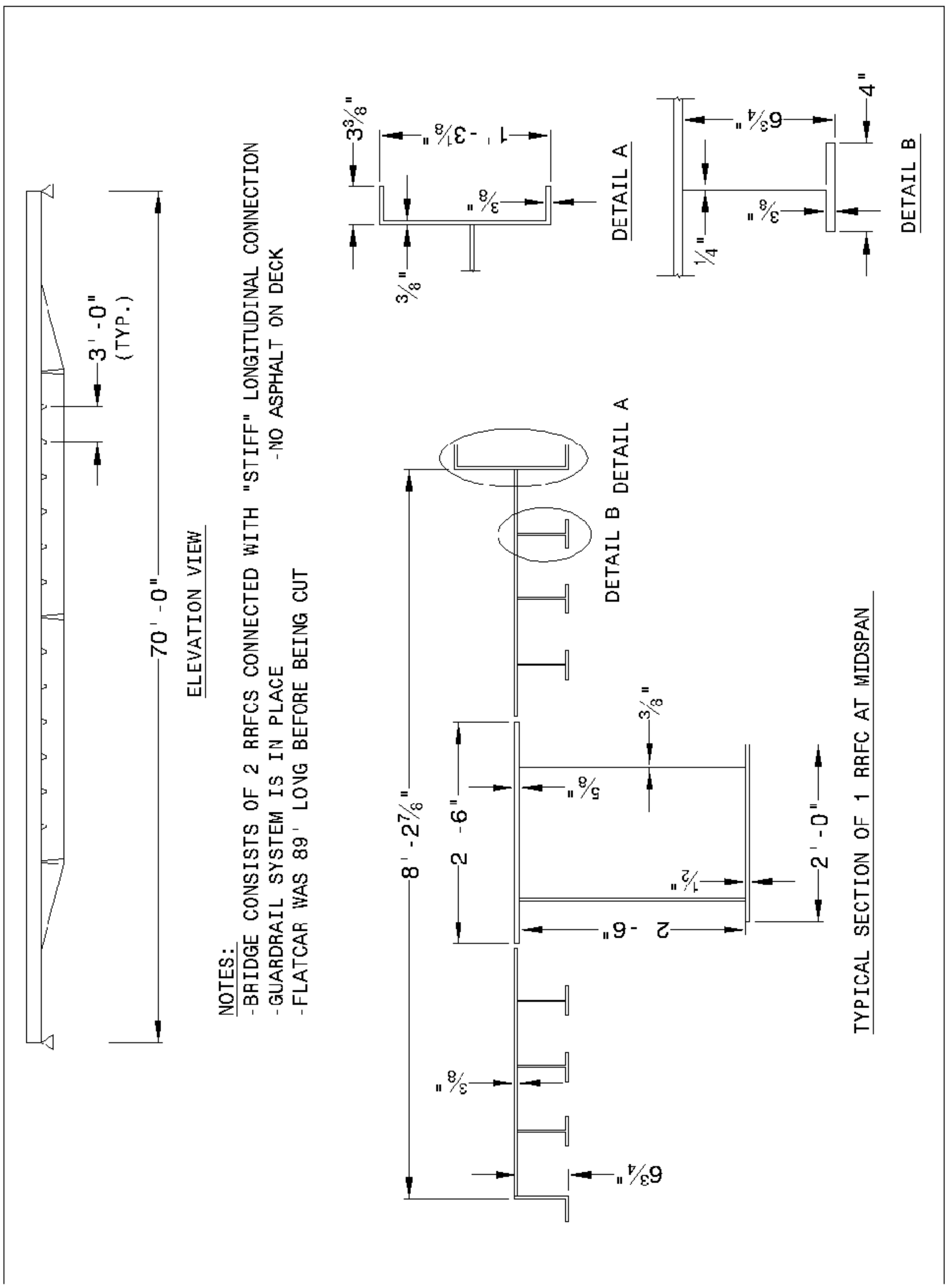
The steel used in constructing the bridge will be assumed to have a yield strength (F_y) of 36 ksi. Therefore, the capacity (C) of the members is as follows (Table 6B.6.2.1-1):

$$C = 0.55\, F_y = (0.55)(36\; ksi) = 19.8\; ksi$$

Drawings and necessary information of the example bridge to be rated are shown on the following page.

TYPICAL RRFC BRIDGE

SHEET NO.:
1 of 1



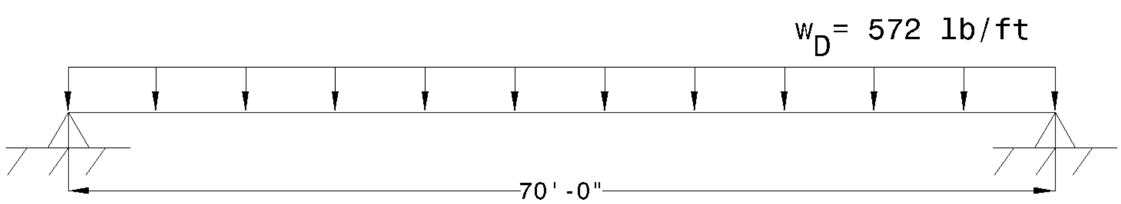
Rating of Primary Members

First, the inventory rating will be determined based on global bending in the primary members. The sample rating will be performed on the bottom flange of the main girder.

The dead load bending stress will be determined for the primary members (i.e., main girders). Since the example bridge only has one primary member per flatcar, all of the dead load per flatcar will be assumed to be carried by a single box girder. A beam line model of a single box girder will be used to determine the dead load bending stress based on the following information:

Span length = 70'-0"
Weight of single 89' RRFC = 42,000 lb (Wipf et. al. 2007a)
Guardrail system = 100 lb/ft (assumed)
Additional deck surface = 0 lb/ft (no additional deck other than RRFC steel plate)

$$w_D = \left(\frac{42{,}000\ lb}{89\ ft}\right) + \left(100\ \frac{lb}{ft}\right) + \left(0\frac{lb}{ft}\right) = 572\ \frac{lb}{ft}$$



The maximum dead load moment is calculated as follows:

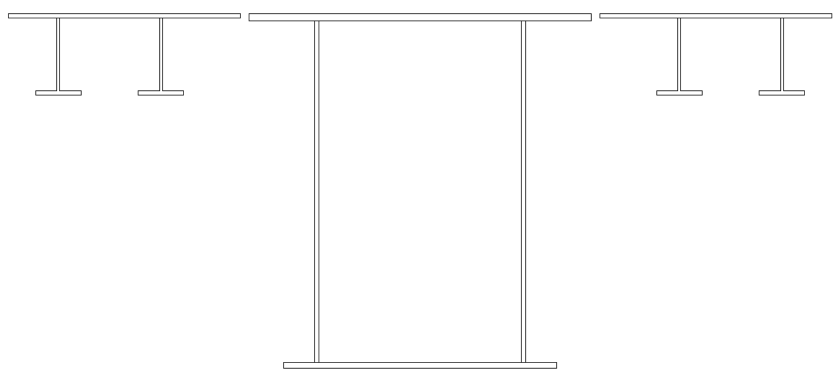
$$M_D = \frac{W_D\ l^2}{8} = \frac{\left(572\ \frac{lb}{ft}\right)\ (70\ ft)^2}{8} = 350\ kip\text{-}ft$$

In order to determine the effective section used to resist dead load, the moments of inertia of the exterior girder and main girder will be calculated. These moments of inertia are calculated about each girder's respective centroid, and are shown below:

$I_{main} = 8,672\ in^4$

$I_{ext} = 231\ in^4$

When comparing the moments of inertia, I_{ext} is approximately 2.7% of I_{main}. Therefore, the effective section is taken as the main girder plus two stringers on each side of the main girder. The "2 stringer" effective section used is shown below:



The effective section modulus for the bottom flange of the "2 stringer" section will now be determined:

$I = 10,892\ in^4$

$y_{bot} = 20.9\ in$

$$S_{eff,bot} = \frac{I}{y_{bot}} = \frac{10,892\ in^4}{20.9\ in} = 521\ in^3$$

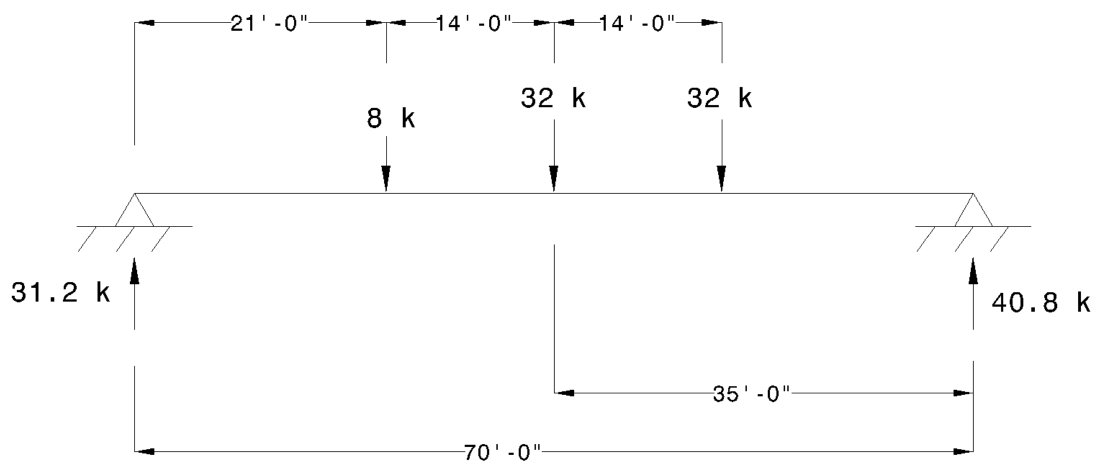
The dead load bending stress will now be calculated for a single primary member:

$$D = \sigma_D = \frac{M_D}{S_{eff,bot}} = \frac{(350\ kip - ft)\left(12\ \frac{in}{ft}\right)}{521\ in^3} = 8.1\ ksi$$

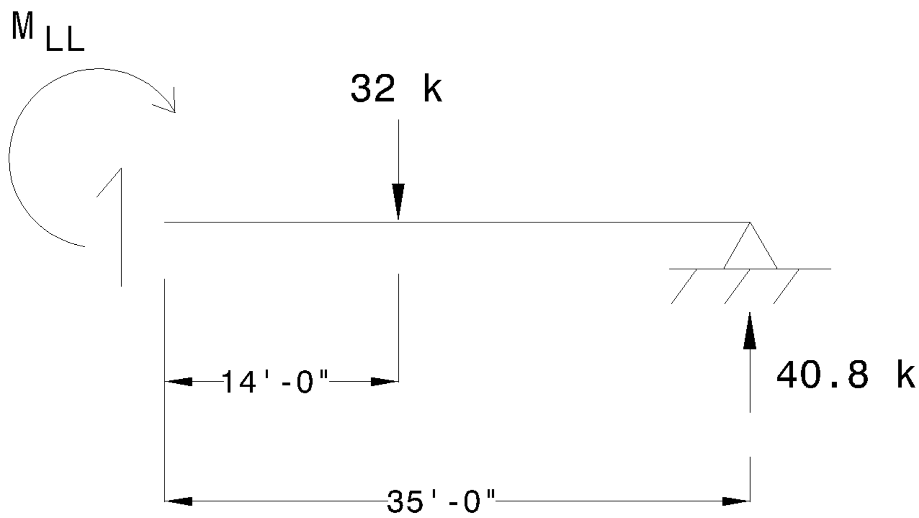
The live load bending stress will be calculated using the following general expression presented in the proposed load rating guidelines found in Appendix E:

$$\sigma_{LL} = (\alpha)\,(CDF)\,\frac{(DF)\,M_{LL}}{S_{eff}}$$

The maximum positive live load moment will be determined from positioning an HS-20 truck in a location to maximize the live load moment. A beam line model of this position is shown below.

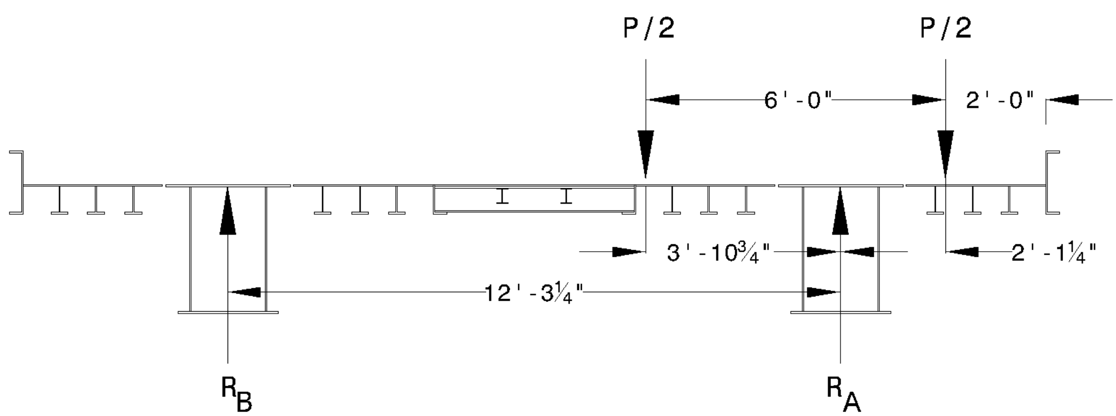


The maximum positive live load moment will be determined at midspan. The free body diagram used to determine this value is shown.



$$M_{LL} = (40.8\,k)\,(35\,ft) - (32\,k)\,(14\,ft) = 980\,kip\text{-}ft$$

To determine the distribution factor, an HS-20 truck will be placed two feet from the outside edge of the driving surface, as shown in the diagram.



The distribution factor (DF) for the loaded girder (R_A) will be determined using the lever rule. Assuming the longitudinal connection can be considered a rigid connection, the distribution factor is computed as follows:

$$DF = R_A = \frac{\left(\frac{P}{2}\right)\left(14' - 4\,{}^{1}\!/_{2}\,"\right)}{\left(12' - 3\,{}^{1}\!/_{4}\,"\right)} + \frac{\left(\frac{P}{2}\right)\left(8' - 4\,{}^{1}\!/_{2}\,"\right)}{\left(12' - 3\,{}^{1}\!/_{4}\,"\right)} = 0.93\,P$$

Therefore, using the lever rule yields a distribution factor of 0.93. Using Eq. 2.1.1-1 from the proposed load rating guidelines (Appendix E), the live load bending stress can be calculated.

$$\sigma_{LL} = (\alpha)\,(CDF)\,\frac{(DF)\,M_{LL}}{S_{eff}}$$

where:

α = 0.75 as stated in Article 2.1.1.5

CDF = 1.0 for typical RRFCs as stated in Article 2.1.1.3

DF = 0.93 as determined by the lever rule according to Article 2.1.1.2

M_{LL} = 980 kip-ft as determined by HS20 truck according to Article 2.1.1.1

S_{eff} = 521 in^3 as determined using “2 stringer” section according to Article 2.1.1.4

Therefore the maximum live load bending stress is computed as follows:

$$L = \sigma_{LL} = (0.75)\,(1.0)\,\frac{(0.93)\,(980\ kip\text{-}ft)\left(12\,{}^{in}\!/_{ft}\right)}{(521\ in^3)} = 15.7\ ksi$$

Using the general rating factor equation as specified in *AASHTO The Manual for Bridge Evaluation,* the rating factor can be determined as follows:

$$RF = \frac{C - A_1 D}{A_2 L\,(1 + I)} = \frac{19.8\ ksi\ -\ (1.0)(8.1\ ksi)}{(1.0)(15.7\ ksi)(1 + 0.33)} = 0.56$$

The rating in tons (RT) can then be determined as follows, knowing the total weight of an HS20 truck is 36 tons:

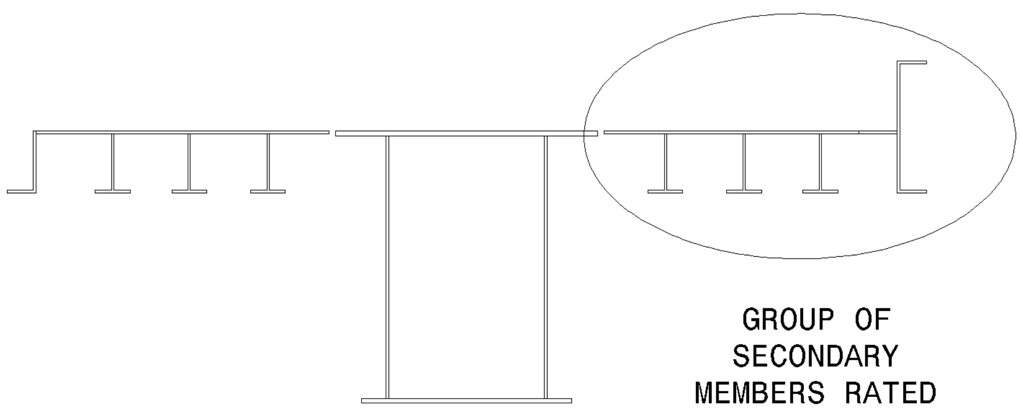
$$RT = (RF)\,W = \,(0.56)\,(36\,tons) = 20.2\,tons$$

Therefore the bridge has an inventory load rating of 20 tons when considering global bending effects on the primary members.

Rating of Secondary Members

Next, the inventory rating will be determined based on local bending in the secondary members. The sample rating will be performed on the bottom flange of a secondary member.

When performing a rating of the secondary members, all types of secondary members must be considered. This sample rating will be performed on the group of secondary members on the outside of one of the RRFCs, as shown in the figure below.



Since the rating of the secondary members for the bridge can be controlled by an exterior girder or a stringer, the dead load bending stress will need to be determined for both members. Both types of secondary members will be assumed to carry the local bending of their self weight plus the tributary width of the steel deck plate between members.

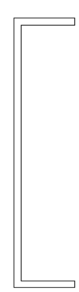
Using the dimensions shown in drawings for this example bridge, the exterior girder has an area of 7.92 in^2 and the steel deck plate has an area of 1.62 in^2. Using an assumed steel weight of 490 lb/ft^3, the dead load on the exterior girder can be calculated as follows:

$$w_{D,ext} = (490\ lb/ft^3)\left(\frac{1}{12^2}\frac{ft^2}{in^2}\right)(7.92\ in^2 + 1.62\ in^2) = 32.5\ lb/ft$$

The dead load moment, using the simple span moment equation, as specified in the proposed load rating guidelines, can be calculated. The span length of the secondary members, as shown in the drawings, is 3’-0”.

$$M_{D,ext} = \frac{w_{D,ext}\ l^2}{8} = \frac{(32.5\ lb/ft)\ (3'\text{-}0)^2}{8} = 0.04\ kip\text{-}ft$$

The section modulus of the exterior girder will be calculated using the following effective section as specified in the proposed load rating guidelines. Since the exterior girder has not been cut, the section will consist of only the structural shape used as the exterior girder.



Using the dimensions shown in the drawings, the effective section modulus for the bottom flange of the exterior girder will be determined.

I = 230.5 in^4

y_{bot} = 7.56 in

$$S_{eff,bot} = \frac{I}{y_{bot}} = \frac{230.5\ in^4}{7.56\ in} = 30.5\ in^3$$

The dead load bending stress will now be calculated for the exterior girder:

$$D_{ext} = \sigma_D = \frac{M_{D,ext}}{S_{eff,bot}} = \frac{(0.04\ kip\text{-}ft)\ \left(12\ {}^{in}/_{ft}\right)}{30.5\ in^3} = {\sim}0\ ksi$$

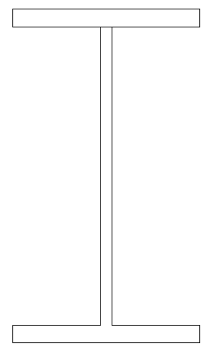
Using a similar procedure, the dead load stress in the stringer will now be computed. The stringer which will carry the most dead load will be the stringer located closest to the main girder since it carries a greater portion of the steel deck. Using the dimensions shown in the drawings, the stringer has an area of 3.09 in^2 and the steel deck plate has an area of 4.31 in^2. Again, using an assumed steel weight of 490 lb/ft^3, the dead load on the stringer can be calculated as follows:

$$w_{D,str} = (490\ lb/ft^3)\left(\frac{1}{12^2}\ \frac{ft^2}{in^2}\right)(3.09\ in^2 + 4.31\ in^2) = 25.2\ lb/ft$$

The dead load moment acting on the stringer will now be computed using the same span length as the exterior girder.

$$M_{D,str} = \frac{w_{D,str}\ l^2}{8} = \frac{(25.2\ lb/ft)\ (3' - 0")^2}{8} = 0.03\ kip\text{-}ft$$

The effective section of the stringer will now be determined according to the proposed load rating guidelines. Since the stringer is rigidly attached to the steel deck, the effective section of the stringer will consist of the structural shape and a portion of the steel deck with a width equal to the width of the bottom flange of the stringer. Thus, the effective section which will be used is shown:



Using the dimensions shown, the effective section modulus for the bottom flange of the stringer will be determined.

$I = 39.6\ in^4$

$y_{bot} = 3.56\ in$

$$S_{eff,bot} = \frac{I}{y_{bot}} = \frac{39.6\ in^4}{3.56\ in} = 11.1\ in^3$$

The dead load bending stress will now be calculated for the stringer:

$$D_{str} = \sigma_D = \frac{M_{D,str}}{S_{eff,bot}} = \frac{(0.03\ kip\text{-}ft)\left(12\ {}^{in}/_{ft}\right)}{11.1\ in^3} = \sim 0\ ksi$$

Now, the live load bending stresses will be calculated for both the exterior girder and the stringer according to the proposed load rating guidelines. The general expression for calculating live load bending stress is as follows:

$$\sigma_{LL} = \frac{(DF)\,(M_{LL})}{S_{eff}}$$

The maximum positive live load moment will be determined using a single rear axle wheel load from an HS20 truck axle. An HS20 tandem axle has a weight of 32 kips. This will be divided by 4 to obtain the single axle wheel load. It is divided by 2 once to split the tandem axle into two separate axles. It is divided by 2 again to separate the axle weight into two wheel loads. Therefore, the live load moment produced from a single rear axle wheel load can be calculated as follows:

$$M_{LL} = \frac{P\,L}{4} = \frac{\left(\frac{32\ kips}{4}\right)\,(3'\text{-}0")}{4} = 6.0\ kip\text{-}ft$$

Next, the distribution factor for each secondary member will be determined. The moments of inertia, determined previously, of the two different types of members are as shown:

$I_{ext} = 230.5\ in^4$

$I_{str} = 39.6\ in^4$

The moment of inertia of the exterior girder is clearly more than 3 times greater than the moment of inertia of the stringer. Therefore the distribution factor for the exterior girder is 1.0 and the distribution factor for stringer is 3/5. Using these distribution factors, the live load moment, and the section properties previously determined, the live load bending stresses in both secondary members can now be calculated.

$$L_{ext} = \sigma_{LL,ext} = \frac{(DF)\,(M_{LL})}{S_{eff}} = \frac{(1.0)\,(6\ kip\text{-}ft)\,\left(12\ {}^{in}/_{ft}\right)}{30.5\ in^3} = 2.4\ ksi$$

$$L_{str} = \sigma_{LL,str} = \frac{(DF)\,(M_{LL})}{S_{eff}} = \frac{\left(\frac{3}{5}\right)\,(6.0\;kip\text{-}ft)\,\left(12\;{}^{in}/_{ft}\right)}{11.1\;in^3} = 3.9\;ksi$$

Using the general rating factor equation as specified in *AASHTO The Manual for Bridge Evaluation*, the rating factor for each secondary member can be determined as follows:

$$RF_{ext} = \frac{C - A_1 D}{A_2 L\,(1 + I)} = \frac{19.8\;ksi\; - \;(1.0)(0\;ksi)}{(1.0)(2.4\;ksi)(1 + 0.33)} = 6.2$$

$$RF_{str} = \frac{C - A_1 D}{A_2 L\,(1 + I)} = \frac{19.8\;ksi\; - \;(1.0)(0\;ksi)}{(1.0)(3.9\;ksi)(1 + 0.33)} = 3.8$$

The rating in tons (RT) can then be determined for each member as follows, knowing the total weight of a single rear axle wheel load of an HS20 truck is 9 tons:

$$RT_{ext} = (RF_{ext})\,W = \;(6.2)\;(9\;tons) = 55.8\;tons$$

$$RT_{str} = (RF_{str})\,W = \;(3.8)\;(9\;tons) = 34.2\;tons$$

As shown, the inventory rating for the stringer will control, so for the secondary members, RT = 34.2 tons.

Therefore the maximum inventory rating for a single wheel load on a stringer when considering local bending effects is 34 tons.

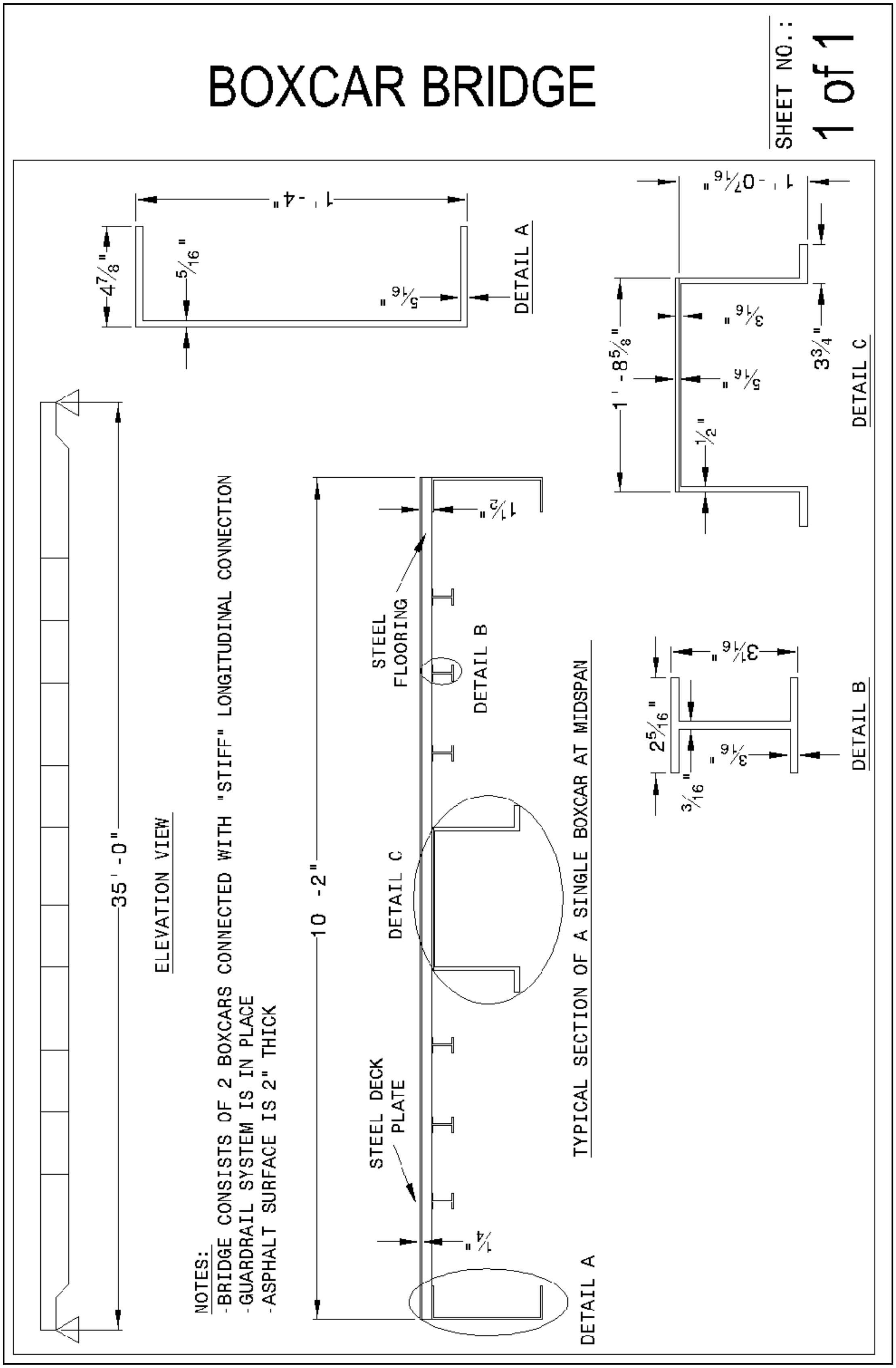
TYPICAL BOXCAR BRIDGE LOAD RATING EXAMPLE

A sample bending stress load rating, using the allowable stress method as described in *AASHTO The Manual for Bridge Evaluation* will be performed on a bridge constructed with a boxcar. The load rating will be for one lane loaded and will be an inventory level rating. The process will be similar to that found in the previous example in which a load rating was performed on a typical RRFC bridge.

The steel used in the boxcar will be assumed to have a yield strength of 36 ksi. Therefore according to *AASHTO The Manual for Bridge Evaluation* (Table 6B.6.1-1), the capacity (C) of the members are calculated as shown:

$$C = 0.55\,F_y = \ (0.55)\,(36\,ksi) = 19.8\,ksi$$

Drawings and necessary information of the example boxcar bridge to be rated are shown on the following page.



BOXCAR BRIDGE
SHEET NO.:
1 of 1
ELEVATION VIEW
35'-0"
NOTES:
-BRIDGE CONSISTS OF 2 BOXCARS CONNECTED WITH "STIFF" LONGITUDINAL CONNECTION
-GUARDRAIL SYSTEM IS IN PLACE
-ASPHALT SURFACE IS 2" THICK
10'-2"
STEEL DECK PLATE
STEEL FLOORING
1/4"
1 1/2"
DETAIL A
DETAIL B
DETAIL C
TYPICAL SECTION OF A SINGLE BOXCAR AT MIDSPAN
1'-4"
4 7/8"
5/16"
5/16"
DETAIL A
1'-0 7/16"
1'-8 5/8"
3/16"
5/16"
1/2"
3 3/4"
DETAIL C
3 1/16"
2 5/16"
3/16"
3/16"
DETAIL B

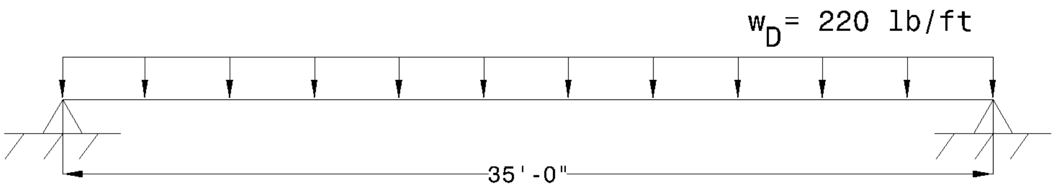
Rating of Primary Members

First, the inventory rating will be determined based on global bending of the primary members. The sample rating will be performed on the bottom flange of an exterior girder. (In practice, a load rating would need to be performed on the main girder as well.)

The dead load bending stress will be determined for the exterior girder. It is assumed that tributary widths can be used to distribute dead load to each of the primary members (one main girder and two exterior girders). As can be determined using the drawings of this bridge, the tributary width for the exterior girder is 2'-6". A beam line model of the exterior girder will be used to determine the dead load bending stress based on the following information:

Span length = 35'-0"
Guardrail system = 100 lb/ft (assumed)
Weight of guardrail applied to exterior girder = (100 lb/ft) (2'-6") / (10'-2")
= 24.6 lb/ft
Weight of boxcar = 25,000 lbs (assumed, including steel deck and flooring)
Weigh of boxcar applied to exterior girder = (25,000 lbs) (2'-6") / (10'-2")
= 6,150 lb
Weight of asphalt = 45 lb/ft^3 (assumed)
Weight of asphalt applied to exterior girder = (45 lb/ft^3) (2 in) (1 ft/12 in) (2'-6")
= 18.8 lb/ft

$$w_D = \left(\frac{6{,}150\ lb}{35\ ft}\right) + \left(24.6\ \frac{lb}{ft}\right) + \left(18.8\ \frac{lb}{ft}\right) = 220\ \frac{lb}{ft}$$



The maximum dead load moment is calculated as shown:

$$M_D = \frac{w_D\, l^2}{8} = \frac{\left(220\ \frac{lb}{ft}\right)(35\ ft)^2}{8} = 34\ kip\text{-}ft$$

The effective section used to resist dead load consists of the structural shape used as the exterior girder, according to the proposed guidelines. Section properties of the exterior girder are calculated and the effective section modulus is determined.

$I = 282\ in^4$

$y_{bot} = 8\ in$

$$S_{eff,bot} = \frac{I}{y_{bot}} = \frac{282\ in^4}{8\ in} = 35.3\ in^3$$

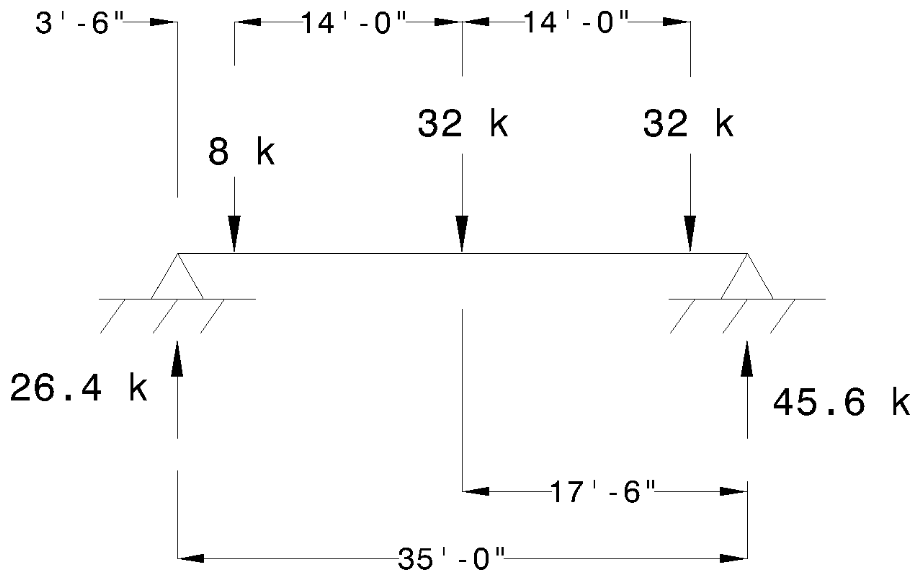
The dead load bending stress will now be calculated for the exterior girder:

$$D = \sigma_D = \frac{M_D}{S_{eff,bot}} = \frac{(34\ kip - ft)\left(12\ \frac{in}{ft}\right)}{35.3\ in^3} = 11.6\ ksi$$

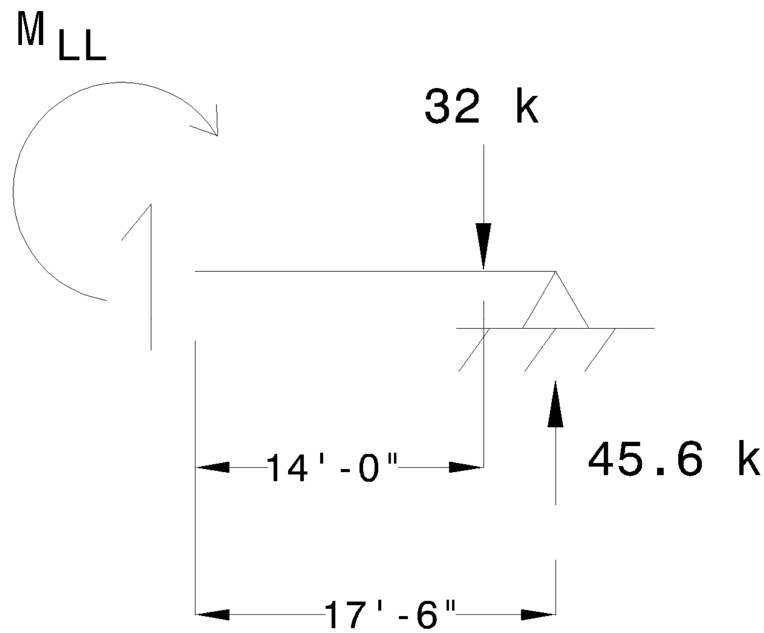
The live load bending stress will be calculated using the following general expression presented in the proposed load rating guidelines found in Appendix E:

$$\sigma_{LL} = (\alpha)\,(CDF)\,\frac{(DF)\,M_{LL}}{S_{eff}}$$

The maximum positive live load moment will be determined from positioning an HS-20 truck in a location to maximize the live load moment. A beam line model of this position is shown below.

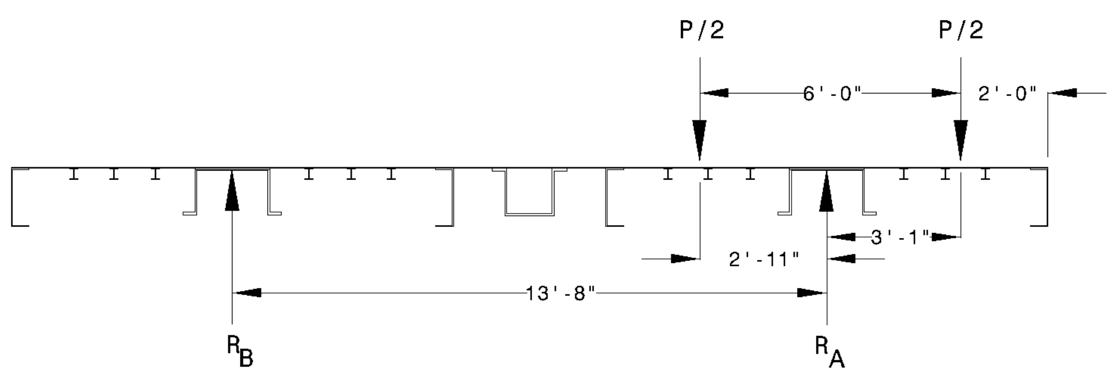


The maximum positive live load moment will be determined at midspan. The free body diagram used to determine the moment is shown.



$$M_{LL} = (45.6\ k)\,(17.5\ ft) - (32\ k)\,(14\ ft) = 350\ kip\text{-}ft$$

To determine the distribution factor, an HS-20 truck will be positioned two feet from the outside edge of the driving surface, as shown in the diagram.



The distribution factor (DF) for the loaded girder (R_A) will be calculated using the lever rule. Assuming the longitudinal connection can be considered a rigid connection, the distribution factor is computed as follows:

$$DF = R_A = \frac{\left(\frac{P}{2}\right)(16' - 9")}{(13' - 8")} + \frac{\left(\frac{P}{2}\right)(10' - 9")}{(13' - 8")} = 1.0\,P$$

Therefore, using the lever rule yields a distribution factor of 1.0. Using Eq. 2.1.1-1 from the proposed load rating guidelines (Appendix E), the live load bending stress can be calculated.

$$\sigma_{LL} = (\alpha)\,(CDF)\,\frac{(DF)\,M_{LL}}{S_{eff}}$$

where:

α = 0.75 as stated in Article 3.1.1.5

CDF = 3/5 for an exterior girder of a boxcar as stated in Article 3.1.1.3

DF = 1.0 as determined by the lever rule according to Article 3.1.1.2

M_{LL} = 350 kip-ft as determined by HS20 truck according to Article 3.1.1.1

S_{eff} = 35.3 in^3 as the effective section according to Article 3.1.1.4

Therefore the maximum live load bending stress is computed as follows:

$$L = \sigma_{LL} = (0.75)\left(\frac{3}{5}\right)\frac{(1.0)\,(350\,kip\text{-}ft)\left(12\,{}^{in}/_{ft}\right)}{(35.3\,in^3)} = 53.5\,ksi$$

Using the general rating factor equation as specified in *AASHTO The Manual for Bridge Evaluation,* the rating factor can be determined as follows:

$$RF = \frac{C - A_1 D}{A_2 L\,(1 + I)} = \frac{19.8\,ksi\; - \;(1.0)(11.6\,ksi)}{(1.0)(53.5\,ksi)(1 + 0.33)} = 0.12$$

The rating in tons (RT) can then be determined as follows, knowing the total weight of an HS20 truck is 36 tons:

$$RT = (RF)\,W = \ (0.12)\,(36\ tons) = 4.3\ tons$$

Therefore the bridge has an inventory load rating of 4 tons when considering global bending effects on the primary members. Since limited data were collected from this type of RRFC during the research, the provisions for rating were purposely calibrated to yield conservative. Hence, overall, this rating is believed to be conservative.

Appendix I. Comparison of Live Load Stresses Using Proposed Guidelines to Field Measurements for Iowa State University RRFC Bridges

This appendix presents the results of calculating the live load bending stresses for the typical RRFC bridges tested by Iowa State University. Included are six of the single span bridges tested by Iowa State University. Key values used in calculating the stresses are presented in the following tables. The resulting stresses are presented and are compared to the stresses measured by Iowa State University.

The stresses for critical load cases are highlighted in the tables. As seen, there is good correlation between the calculated and measured stresses. For load cases in which the truck(s) were in one of the outer edges (i.e., load cases which should produce the highest stresses), the proposed guidelines provided reasonably conservative stress results.

TYPICAL RRFC BRIDGES WITH SMALL EXTERIOR GIRDERS

The following tables are for bridges which contained small exterior girders. For these bridges, according to the proposed rating guidelines, the effective sections consisted of the main girder plus two stringers on each side of the main girder.

Bridge: DCB (Wipf et. al. 2007a)

Span Length: 67'-6"

Test	Bottom Flange of Member Rated	Total Moment (kip-in)	Lever Rule	I_x (in^4)	y_{bot} (in)	S_{bot} (in^3)	CDF	α	Bottom Flange Stress	
									σ_{calc} (ksi)	σ_{meas} (ksi)
Lane 1	S main girder	8,103	0.92	11618	22.1	525	1	0.75	10.6	6.7
Lane 1	N main girder	8,103	0.08	11618	22.1	525	1	0.75	0.9	5.5
Lane 2	S main girder	8,103	0.5	11618	22.1	525	1	0.75	5.8	5.4
Lane 2	N main girder	8,103	0.5	11618	22.1	525	1	0.75	5.8	6.7
Lane 3	S main girder	8,103	0.08	11618	22.1	525	1	0.75	0.9	4.1
Lane 3	N main girder	8,103	0.92	11618	22.1	525	1	0.75	10.6	8.7

Bridge: BCB3 (Wipf et. al. 2007a)

Span Length: 66'-2"

Test	Bottom Flange of Member Rated	Total Moment (kip-in)	Lever Rule	I_x (in^4)	y_{bot} (in)	S_{bot} (in^3)	CDF	α	Bottom Flange Stress	
									σ_{calc} (ksi)	σ_{meas} (ksi)
Lane 1	N main girder	7,960	0.16	11618	22.1	525	1	0.75	1.8	4.1
Lane 1	center main girder	7,960	0.68	11618	22.1	525	1	0.75	7.7	5.2
Lane 1	S main girder	7,960	0.16	11618	22.1	525	1	0.75	1.8	4.1
Lane 2	N main girder	7,960	0	11618	22.1	525	1	0.75	0.0	1.2
Lane 2	center main girder	7,960	0.08	11618	22.1	525	1	0.75	0.9	2.6
Lane 2	S main girder	7,960	0.92	11618	22.1	525	1	0.75	10.5	8.7
Lane 3	N main girder	7,960	0.92	11618	22.1	525	1	0.75	10.5	9
Lane 3	center main girder	7,960	0.08	11618	22.1	525	1	0.75	0.9	3.8
Lane 3	S main girder	7,960	0	11618	22.1	525	1	0.75	0.0	1.7

Bridge: WCB2 (Wipf et. al. 2007a)

Span Length: 66'-4"

Test	Bottom Flange of Member Rated	Total Moment (kip-in)	Lever Rule	I_x (in^4)	y_{bot} (in)	S_{bot} (in^3)	CDF	α	Bottom Flange Stress	
									σ_{calc} (ksi)	σ_{meas} (ksi)
Lane 1	N main girder	8,200	0.16	11618	22.1	525	1	0.75	1.9	5.8
Lane 1	center main girder	8,200	0.68	11618	22.1	525	1	0.75	8.0	5.8
Lane 1	S main girder	8,200	0.16	11618	22.1	525	1	0.75	1.9	4.6
Lane 2	N main girder	8,200	0	11618	22.1	525	1	0.75	0.0	2.2
Lane 2	center main girder	8,200	0.08	11618	22.1	525	1	0.75	0.9	4.8
Lane 2	S main girder	8,200	0.92	11618	22.1	525	1	0.75	10.8	8
Lane 3	N main girder	8,200	0.92	11618	22.1	525	1	0.75	10.8	10
Lane 3	center main girder	8,200	0.08	11618	22.1	525	1	0.75	0.9	5.2
Lane 3	S main girder	8,200	0	11618	22.1	525	1	0.75	0.0	1.9
Lane 4	N main girder	8,200	0	11618	22.1	525	1	0.75	0.0	4.1
Lane 4	center main girder	8,200	0.67	11618	22.1	525	1	0.75	7.8	5.2
Lane 4	S main girder	8,200	0.33	11618	22.1	525	1	0.75	3.9	6.1

TYPICAL RRFC BRIDGES WITH LARGE EXTERIOR GIRDERS

The following tables are for bridges which contained large exterior girders. For these bridges, as according to the proposed rating guidelines, the effective sections consisted of the entire flatcar.

Bridge: BCB1 (Wipf et. al. 2003)

Span Length: 51'-9"

Test	Bottom Flange of Member Rated	Total Moment (kip-in)	Lever Rule	I_x (in^4)	y_{bot} (in)	S_{bot} (in^3)	CDF	α	Bottom Flange Stress	
									σ_{calc} (ksi)	σ_{meas} (ksi)
LT2, Test 1	S main girder	5,892	0.15	20,936	21.4	981	1	0.75	0.7	1.3
LT2, Test 1	center main girder	5,892	0.7	20,936	21.4	981	1	0.75	3.2	2.3
LT2, Test 1	N main girder	5,892	0.15	20,936	21.4	981	1	0.75	0.7	0.8
LT2, Test 2	S main girder	5,892	0	20,936	21.4	981	1	0.75	0.0	0
LT2, Test 2	center main girder	5,892	0.03	20,936	21.4	981	1	0.75	0.1	1
LT2, Test 2	N main girder	5,892	0.97	20,936	21.4	981	1	0.75	4.4	3.4
LT2, Test 3	S main girder	5,892	0.97	20,936	21.4	981	1	0.75	4.4	4.2
LT2, Test 3	center main girder	5,892	0.03	20,936	21.4	981	1	0.75	0.1	1
LT2, Test 3	N main girder	5,892	0	20,936	21.4	981	1	0.75	0.0	0
LT2, Test 4	S main girder	5,892	0	20,936	21.4	981	1	0.75	0.0	0.7
LT2, Test 4	center main girder	5,892	0.5	20,936	21.4	981	1	0.75	2.3	2.2
LT2, Test 4	N main girder	5,892	0.5	20,936	21.4	981	1	0.75	2.3	1.7
LT2, Test 5	S main girder	5,892	0.97	20,936	21.4	981	1	0.75	4.4	4.4
LT2, Test 5	center main girder	5,892	0.06	20,936	21.4	981	1	0.75	0.3	2.3
LT2, Test 5	N main girder	5,892	0.97	20,936	21.4	981	1	0.75	4.4	3.6
LT2, Test 6	S main girder	5,892	0.49	20,936	21.4	981	1	0.75	2.2	3.4
LT2, Test 6	center main girder	5,892	1.02	20,936	21.4	981	1	0.75	4.6	4.1
LT2, Test 6	N main girder	5,892	0.49	20,936	21.4	981	1	0.75	2.2	2.5

Bridge: BCB2 (Wipf et. al. 2007a)

Span Length: 54'-0"

Test	Bottom Flange of Member Rated	Total Moment (kip-in)	Lever Rule	I_x (in^4)	y_{bot} (in)	S_{bot} (in^3)	CDF	α	Bottom Flange Stress	
									σ_{calc} (ksi)	σ_{meas} (ksi)
Lane 1	S main girder	6,094	0.97	21865	21.9	998	1	0.75	4.4	3.4
Lane 1	N main girder	6,094	0.03	21865	21.9	998	1	0.75	0.1	1.2
Lane 2	S main girder	6,094	0.5	21865	21.9	998	1	0.75	2.3	2.6
Lane 2	N main girder	6,094	0.5	21865	21.9	998	1	0.75	2.3	2.3
Lane 3	S main girder	6,094	0.03	21865	21.9	998	1	0.75	0.1	1.7
Lane 3	N main girder	6,094	0.97	21865	21.9	998	1	0.75	4.4	3.5

Bridge: TCB (Wipf et. al. 1999)

Span Length: 42'-0"

Test	Bottom Flange of Member Rated	Total Moment (kip-in)	Lever Rule	I_x (in^4)	y_{bot} (in)	S_{bot} (in^3)	CDF	α	Bottom Flange Stress	
									σ_{calc} (ksi)	σ_{meas} (ksi)
B1	E main girder	4,141	0.85	12828	16.0	800	1	0.75	3.3	3
B1	W main girder	4,141	0.15	12828	16.0	800	1	0.75	0.6	1.4
B2	E main girder	4,141	0.5	12828	16.0	800	1	0.75	1.9	2.5
B2	W main girder	4,141	0.5	12828	16.0	800	1	0.75	1.9	1.6
B3	E main girder	4,141	0.15	12828	16.0	800	1	0.75	0.6	1.7
B3	W main girder	4,141	0.85	12828	16.0	800	1	0.75	3.3	2.3