STRUCTURAL CHECK OF A STEEL THROUGH TRUSS BRIDGE

by

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Abstract

The Abilene & Smoky Valley Railroad Association offers train excursions to sightseers wishing to see historic Abilene and natural Kansas scenery. Currently, a diesel powered ALCO locomotive is used to pull the passenger cars. They wish to use a 1919 Baldwin steam locomotive in the future. Part of the excursion includes a slow crossing of the Smoky Hill River over a two-span steel truss bridge. The company approached the Kansas State University Civil Engineering Department with the task of performing a structural check of the bridge. By using the Baldwin locomotive, the bridge is required to support much larger loads than when the diesel engine is used.

First, a basic visual inspection and site visit of the bridge was performed. The inspection was not thorough, but was used to familiarize the team with the bridge and its components. Using the inspection and data supplied, a structural analysis was performed using the software, RISA. After completion of the analysis for both loading situations, the resulting stress increases were calculated. Other calculations performed include buckling loads of the compression members, deflections of the bottom chord and stresses in some of the connections.

After completion of the analysis and calculations, large increases in member stress were found. For most of the members, the increase of live loads stress was between 80% and 100%. The largest stress found due to the dead and live load, which was under 15 ksi, occurred in the bottom chord for the steam locomotive loading situation. Some truss members experienced stress reversal, but relatively low values were noted. Deflection calculations for the two loading situations yielded similar results to the stress calculations. Again, an increase in deflection between 80% and 100% was found for the joints located on the bottom chord.

It is recommended that a more detailed inspection and a more thorough analysis of the connections, supports, piers, and foundations be performed before the heavier locomotive is used.

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CHAPTER 1 - Introduction

1.1 Background

The Abilene & Smoky Valley Railroad Association (ASVRA) was formed in 1993 as a 501(c)(3) not-for-profit corporation, dedicated to restoring and operating historic railroad equipment and passing on the history of the rails. The railroad is operated by volunteers, including the engineers, crew, car attendants that narrate every trip, and depot workers, as well as mechanical and track maintenance workers. The excursion is a tourist attraction in Abilene, Kansas. The company transports sightseers around historic Abilene, pointing out landmarks and native Kansas wildlife.

Currently, a 1945 ALCO S-1 locomotive, with a 660-hp diesel-electric engine originally designed for WWII submarines, is used to pull the excursion train. The train includes a 1902 wood KATY passenger car converted to a dining car, two open-air gondola cars with canopy tops, and a caboose. The company is planning to use a 1919 Baldwin 4-6-2 “Pacific” Santa Fe #3415 steam locomotive (currently being restored) to pull the train in the near future. This steam locomotive is larger than the diesel locomotive in operation at the present time.

One of the highlights offered by the Abilene & Smoky Valley Railroad excursion is the slow-speed crossing of the Smoky Hill River near Enterprise, Kansas, over a two-span steel truss bridge. The company has approached the Department of Civil Engineering at Kansas State University with the task of performing a structural analysis of the aforementioned steel truss bridge. The main objective is to know if the bridge and its structural members will safely withstand the increased loads due to use of the steam locomotive, and if resulting stresses are higher, how much is the percentage of that increase.

The bridge is 218 ft long consisting of twin 109-ft long, simply supported, steel through-trusses. All four trusses consist of built up members made of multiple angles and plates riveted together. The compression members, top chord excluded, are angles laced together with strap steel lattice plates. Tension members are made of the same angles but include no lacing. The first and last panels are 12.5 ft long while the other seven panels are 12 ft long. At each panel, floor beams that carry the train and track transfer all loads to the joints. The truss is simply supported, bearing on steel plates on the top of masonry piers. Figure 1.1 shows the bridge crossing the Smoky River.

Figure 1.1 Smoky River Bridge

1.2 Objective and Scope of Work

The objective of this study is to investigate the effect of changing the moving load crossing a typical single-track, through-truss existing steel railroad bridge. The case study used is the Abilene and Smoky Valley Railroad bridge subject to the two locomotive engines mentioned in the previous section.

The structural analysis was completed using the computer software package RISA 3-D, which is used to determine the forces carried by each member due to the train passage. This software was also used to plot the influence lines of the main members under the moving axle loads. Stresses in each member are then calculated and checked for safety. After checking stresses and stress increases against safe design limits, suggestions were provided for possible remedial actions and/or structural modifications. The main connections in the structure were also checked for capacity and adequacy.

The project does not include detailed examination of every connection in the structure, nor the inspection or analysis of the piers and foundations. Only basic visual structural inspections were performed, and no instrumented or non-destructive evaluations were within the scope of this work that required certification and special technologies. Also, it must be emphasized that this is not intended to be a substitute for consulting and engineering services but is for a Master's project.

During this project, a literature review was performed on case studies similar to this project, and the articles were summarized. The work reviewed for this report includes several projects involving aging steel bridges, their analysis, and the rehabilitation work performed. The articles reviewed include different analysis methods including field testing, as well as laboratory testing. Also, many of the articles discuss different methods that were used to rehabilitate the bridges so that they could be used safely.

1.3 Report Organization and Overview

This report summarizes the process followed over the course of the project. The report covers in detail all of the steps used and completed. It includes a literature review, a summary of the analyses completed, a thorough explanation of the findings, and a list of recommendations for future work.

The literature review is included in Chapter 2. The literature review focused on projects that dealt with the analysis and rehabilitation of aging steel bridges. The summaries include how the bridges were originally inspected and analyzed. Many of the summaries also include how the bridge was rehabilitated or retrofitted in order for the structure to be used safely. The literature review was used to provide a process that could be followed during this project as well as give the owner a guideline and ideas on how to approach further analysis and rehabilitation for the bridge in this study.

The steps used to analyze the bridge and the results from the analysis are explained in detail in Chapter 3. A simple truss analysis was performed using RISA-3D. In this chapter all assumptions and techniques used are explained. In the last half of the chapter, results from the analysis are discussed, compared, and tabulated.

Chapter 4 includes the final conclusions and several recommendations for future work. The first portion of Chapter 4 summarizes the findings from the project. The second part of the chapter lists the recommendations that were reached. Using the literature review and the results from the analysis, ASVRA should investigate and inspect the bridge thoroughly before allowing the heavier steam locomotive to cross the bridge. It is also possible that the bridge will need some form of rehabilitation or retrofit before it is able to carry the heavier loads.

CHAPTER 2 - Literature Review

Over the past few decades, several projects throughout the United States have been undertaken to rehabilitate aging steel bridges. As the design lives of these steel bridges have passed by, the strong desire to keep them in service has lead to large scale projects involving inspection, analysis and rehabilitation.

As these bridges age, a host of problems arise that need to be addressed. The first and most obvious problem are the materials used to construct the bridge have been subjected to weathering for decades and are deteriorating at a fast rate. The second challenge that exists is the need for the bridges to carry more and heavier loads than when designed. Thirdly, as some of these bridges were designed at the turn of the last century, engineers often do not have records of the properties of the metal used to construct the bridges during this time. Combining these factors with the desire of owners to keep the bridges because of historical importance, rehabilitation projects are a common occurrence. For the purpose of this project, focus was placed on analyses and work performed on steel trusses.

2.1 Field Testing and Analysis

2.1.1 Instrumentation of Two 1890s Truss Bridges

Pullaro (2001) investigated the analysis of two bridges originally constructed in the 1890s. The bridges in this study are the Walnut Street Bridge in Chattanooga, Tennessee and the Strawberry Mansion Bridge in Philadelphia, Pennsylvania. The two cities that own the bridges wished to continue using them because of the historical importance. However, both bridges were in need of inspection and rehabilitation.

For the load test of Walnut Street Bridge, a pre-weighed truck drove across the bridge. Instrumentation on the members gathered strain data to find any overstressing occurrences. Gathered strain data averaged 20 to 30% less than the computed strains. Usually, truss performance is better in actuality than in analysis due to member end constraints and overall composite action. During analyses, trusses are assumed to be perfectly pinned and this is never the case. Ultra-sonic tests were performed to determine if discontinuities existed in steel members. High frequency sound waves can be used to detect fractures and other defects. To determine what type of metal was used in the construction of this bridge, coupons were removed and tested for chemical composition and strength. A similar inspection was completed on the Strawberry Mansion Bridge to determine load capacity. Members were instrumented with strain gauges as known loads drove across the wearing surface. Material testing determined the chemical composition and strength of the materials used during construction. Lastly, an underwater inspection was performed to check piers.

2.1.2 Determining Stress Distribution of Truss Bridge

Delgrego et al. (2008) carried out a study on a century old steel through-truss bridge in Connecticut. The goal was to evaluate how aging had affected the structural behavior of the truss. The structural behavior in question was the live load distribution and how it affected the performance of the bridge as a whole. The truss consisted of several built-up members, eyebars in tension and truly pinned connections.

To determine the forces being carried by critical members, a system of 372 weldable strain gages, a 96-channel data logger and a laptop were used. During testing, 16 different trains of known weights drove across the bridge trusses. The trains used consisted of a mix of several different weights and uses. This group of test trains also included the heaviest type expected to cross it. Specific tasks included analyzing stress distributions in indeterminate panels, shared stresses in tension members made up of multiple eyebars, and out-of-plane bending due to floor beam-to-truss member connections.

After gathering data, analysis led to many conclusions. Strain data taken indicated that in members made up of multiple eyebars, an unequal stress distribution existed. Several factors are thought to contribute to this. When manufactured, eyebars were not made to exactly the same lengths nor were holes placed at exactly the same spot. Quality control today is superior to that of the time period when this bridge was constructed. Second, the floor-beam to truss connections were rigid. As the axles pass over the floor-beams, rotation is caused in the truss member closest to the floor-beam. This is shown in the data by the outer-most and inner-most eyebars having the highest concentration of stress.

2.1.3 Inspecting and Rating Older Bridges

Patel (1981) shows that the importance of regular bridge inspections is evident in the case studies of the Ellamore and Harmon truss bridges in West Virginia. (Due to the lack of proper inspections and to the poor maintenance, many bridges have structural deficiencies.) These deficiencies cause the structure to be unsafe.

Close inspection of the superstructure yielded several problems. Some stringers on both bridges had lost up to 50% of the section due to corrosion. Section loss also occurred in the floor beams due to exposure to de-icing chemicals. To some degree, every truss member had lost some cross section. Steel losses at several connections were up to 80%. Many other areas of the bridges had damages due to several different reasons including section-loss and vehicle collision.

After inspections are completed, bridges undergo a rating that describes the overall performance capabilities. For this study two classifications describe different structural components of the bridges. Operating rating refers to the absolute maximum load that the structure can support. An inventory rating refers to a load that the structure can support for an infinite amount of time.

For the analysis of the two bridges, which were constructed sometime between 1905 and 1936, an allowable operating stress of 22.5 ksi and an allowable inventory stress of 16 ksi were used.

2.1.4 Determining Steel Properties of Historic Bridges

Jelinek and Bartlett (2001) investigated the material properties, yield strength and ultimate strength of a bridge built in 1922. The investigation sought to determine the common properties of steel used to construct steel structures at the turn of the century. The East Brough's Bridge in London, Ontario was scheduled for replacement and this offered an excellent opportunity to test the material used for the construction of the bridge.

The East Brough's Bridge is typical of other bridges built during the earlier part of the 20th century. Several truss members were built-up sections of angles and plates, compression members were laced and joints were riveted. A mixture of Carnegie, Bethlehem and some generic steel stamped "Canada" was used to fabricate different members of the bridge.

The test consisted of cutting sound, uncorroded material from truss members and floor beams. 93 specimens were removed from several parts of the bridge. While testing the coupons, ASTM A370 was followed. Yielding strength was determined using the 0.2% offset method. Ultimate strength was determined by dividing the ultimate load by the original area of the coupon. After testing all coupons, a yield strength of 36.7 ksi and an ultimate strength of 58.6 ksi was determined.

2.1.5 Analysis of Several Historical Truss Bridges

Hatfield (2001) reviewed the rehabilitation of several historic bridges for the current use as pedestrian bridges. Several bridges were removed from the original place of service and placed in the Calhoun County Historic Bridge Park in Battle Creek, Michigan. The bridges consist of several Pratt through and pony trusses approximately 100 years old.

The main reason behind the removal and rehabilitation of this study's bridges is because of the increased loads of today's traffic. Although structural steel manufacturing and its quality have been perfected the past century, low carbon steels do not exhibit significantly lower strength than that of the currently used higher carbon steels. All of the bridges in this study were analyzed with present day specified loading to estimate the stresses induced in the bridge members. If the stresses were greater than the allowable stress, the necessary actions were taken.

For analysis, the bridges were treated as pin-connected trusses. For some of the bridges, this was not a perfect assumption due to connections not being truly pinned. Most truss bridges perform to this assumption approximately. As the design is closer to ideal conditions, forces can be computed with sufficient accuracy. For these assumptions, all forces present are axial loads with no bending. Lastly, fatigue tests were performed on multiple eyebars to predict the lifespan of the members.

2.1.6 Coping with Aging Railroad Structures

Uppal (2005) discusses aging railroad bridges and the problems and dangers that are inherent with these structures. At the time of designing these bridges, trains carried much smaller loads. As time passes, load requirements keep increasing to account for higher demands. In addition to the change in loads carried by the railroads, aging of the bridges bring about diminished load carrying capacities. Due to these increased loads and diminished capacities, bridges classify as one of three types: sufficient, marginal and deficient.

Railroad engineers use a loading system originally created by Theodore Cooper in 1880. The first system of loads used was the Cooper E-30. This means that each driving axle has a load of 30,000 pounds. As time went by, the Cooper E series increased to today where some railroad bridges now carry a Cooper E-100 load. Another important design concept is that of hammer blow which is present in steam locomotives.

When classifying a bridge's capacity, the material with which the bridge was constructed is considered. Cast iron's strength depends on the amount of carbon found in the material. Wrought iron has a minimum yield stress of 25 ksi as recommended by AREMA. The railroad generally used three types of structural steel with ultimate strengths ranging from 40 ksi all the way to 80 ksi, but a minimum yield stress of 30 ksi as recommended by AREMA.

Inspections performed on bridges on a regular basis ensure the quality and safety of the structures. Careful and close inspections allow accurate analysis and classification according to load carrying capacities. Different types of bridges require different inspections. Steel is inspected for fatigue cracking, yielding, loss of material and out-of-plane bending. All members require careful inspection for the previously discussed problems.

Assigning ratings and classifications occurs after carefully analyzing the structures. After the rating and classification of the bridge, utilization of the structure changes or rehabilitation begins. Another possibility is total replacement of the structure if the load rating is too low. Changes in loading or retrofitting a structure help in the carrying of trains.

2.1.7 Condition Assessment of Steel Truss Bridge

Spyrakos et al. (2004) presented a process of analysis and retrofit for a railroad truss bridge still in use. The Greek Railway Organization owns the bridge being analyzed which is located in southern Greece. Being a historical bridge, the owners wanted an estimate of the remaining life for the bridge including the factors of heavier train loads and aging material. The bridge was originally constructed during the 1890s and was one of a group of bridges strengthened in 1963.

To test the truss performance, the truss was instrumented with strain gages. As a known 800 kN engine drove across the bridge, strain readings were taken and normal stresses were recorded. Also, strains were recorded for typical trains that crossed the bridge. Calculations were completed for trains traveling in both directions. Part of the study included removing coupons from the bridge components for both chemical and strength testing purposes. Chemical analysis allowed for the type of material to be determined. During strength tests yield and ultimate strengths were determined and fatigue tests allowed for calculation of the remaining fatigue life of the material.

Finite element analysis was used to calculate the response of the structure to loading. During static analysis, the connections were modeled as rigid with three-dimensional beam elements. Because of the physical connections, some moments are carried through the top and bottom beams of the truss, which was reflected in the model. It was assumed that pinned connections existed at the vertical supports. For dynamic analysis, lumped mass formulation was used.

After completion of the study, it was decided that some members were in need of strengthening in order to carry heavier train loads. The strengthening method used was to add plates to the bottom flange of the truss. Spyrakos et al. also came to the conclusion that systematic and regular inspections and maintenance have allowed the life of the bridge to be extended.

2.2 Rehabilitation

2.2.1 Rehabilitation of Two 1890s Truss Bridges

Pullaro (2001) reported on the rehabilitation of two 1890s steel truss bridges. With the analysis phase of the Walnut Street Bridge completed, rehabilitation of the bridge took place. The analysis found that tension in the eyebars due to the live loads was too great. To reduce tension stress on the eyebars caused by dead loads, high strength tendons were stressed parallel to them. This removed some of the tension in the members from dead loads and allowed for a greater live load capacity. This was accomplished using 0.6 inch diameter, grade 270 coated prestressing strands. To rehabilitate the Strawberry Mansion Bridge, the deck was replaced with a lighter weight system to reduce dead load to the structure. To strengthen the truss itself, steel sections were added while being able to keep the historical appearances intact.

2.2.2 Innovative Rehabilitation Techniques

Sabnis et al. (1981) discuss multiple methods to rehabilitate older steel truss bridges. The bridges being investigated were bridges that have been in use for several decades. With the aging of these bridges comes a decrease in load carrying rating. This is due to deterioration in the bridges themselves and the increase in loads and traffic volumes. If a bridge has been rated low because of these reasons, rehabilitation can often be an economical solution when compared to total replacement. Before beginning a rehabilitation project, factors such as urgency, costs, material accessibility, and physical condition of present structure should be considered.

The first method to improve structural capabilities of the bridge is that of stiffening the compression members. Not all compression members will need to be stiffened however. A structural analysis and inspection will aid in the identification of critical members. Stiffening can be achieved by reducing the length of the member or by increasing the section size. By doing this the slenderness ratio is decreased.

The second method is by prestressing the structure. The portion of the bridge discussed here is the bottom (tension) chord. By prestressing the truss, more load is required to reach a controlling stress. To do this truss prestressing, cables are mounted parallel to the tension member and secured. Prestressing can be achieved by jacks or turnbuckles. This allows for the truss to carry additional live load.

The third and fourth methods discussed involve truss supports and the reduction of span that follows. Shifting the end supports that the truss is resting on reduces the span length which reduces stress in members. However, when shifting the supports, small cantilevers will occur at the ends of the truss, and these should be checked. Lastly, a support can be added at mid-span of the truss. This will cut the span in half and make the span continuous, which greatly reduces stress in the truss members.

2.2.3 Removal and Rehabilitation of Truss Bridge

McKeel et al. (2007) investigates the processes involved in rehabilitating and moving historical truss bridges. The particular bridge in this study is the Goshen Bridge in Rockbridge County in Virginia owned by the Virginia Department of Transportation. The Goshen Bridge consists of two steel through trusses and crosses the Calfpasture River. It was disassembled, rehabilitated and put back into service. The paper outlines the general process that should be followed when restoring historic bridges in order for them to carry present day loading situations.

The Goshen Bridge was plagued with widespread corrosion and section loss in several structural members. The result was that the bridge could not safely carry present day loads.

Although it would have been much easier and much less expensive to replace the bridge, for historical purposes it was restored. After inspection and structural analysis, rehabilitation began. It was concluded that proper maintenance and lack of regular inspections led to the current state of the bridge.

Rehabilitation began with the complete disassembly of the bridge. During this stage, great care was taken to mark the members so that identification would be easy when it was time to reassemble the bridge. After disassembly, members with drastic section loss were replaced with new, yet historically appropriate, steel sections. All other members were cleaned and painted. For testing purposes, coupons were removed and tested for strength properties. Lastly, the fracture critical members of the lower chord were tested for cracks and other defects by use of magnetic particle tests.

After testing was completed, new members were fabricated. During this stage, careful and very accurate measurements were taken from the old pieces. This allows for ease of construction and ensures mis-fitting members induce no extra stresses. In order to resist the increased loading, the entire lower chord was replaced. Counters replaced diagonal members. Counters are members that are adjustable and can be adjusted to add rigidity to the entire structure and prevent stress reversal. The counters were the only new members that did not resemble the old members. To complete all construction stages, a system of piers and falsework was used.

2.2.4 Retrofit of Historic Nashville Bridge

Wasserman and Pullaro (2004) investigated the rehabilition of the Shelby Street Bridge in Nashville, Tennessee. The Shelby Street Bridge construction began in 1907 and was completed in 1909. The bridge spans the Cumberland River and consists of three through high steel trusses, an inverted truss span and six reinforced concrete trusses. The steel trusses are a variation of a Pratt truss. Due to its historical importance and other factors, officials decided to rehabilitate the bridge as opposed to replacing it. In 1994, the bridge was posted for loads of 14 tons, though mainly because of the concrete components. Finally, in 1997 the decision to rehabilitate the bridge and convert it to a pedestrian bridge was reached.

Minor repairs were required for the steel through trusses. With overall good shape, few strengthening techniques were performed. Strengthening was required in the isolated main and bracing members of the bottom connections as well as the eyebars in the bottom chord. The most extensive strengthening that took place was the addition of two high strength threadbars placed along the bottom chord and clamped to the pins. The deck truss also required very little strengthening. It was removed and placed on the ground in order for work to be performed on it. During the time that the deck truss was on the ground, expansion bearing devices were replaced. Then, the deck truss was relocated in position on the piers.

Most of the work completed on the Shelby Street Bridge was for the concrete piers and trusses. The river piers were originally constructed of unreinforced concrete and the surfaces of the columns and cap beams had deteriorated. They were retrofitted by being encased in fiber reinforced plastic jackets. Also, the concrete arches were in dire need of rehabilitation. Post-tensioned members were added to the bottom chords of the trusses while keeping the original geometry to maintain the bridge's historical appearance.

CHAPTER 3 - Analysis and Results

As discussed earlier, the objective of this study is to investigate the effect of changing the moving load crossing a typical single-track, through-truss existing steel railroad bridge. Currently, ASVRA is using a 1945 ALCO S-1 locomotive, with a 660 hp diesel-electric engine originally designed for WWII submarines. The train includes a 1902 wood KATY passenger car converted to a dining car, two open-air gondola cars with canopy tops, and a caboose. The company is planning to use a 1919 Baldwin 4-6-2 “Pacific,” Santa Fe #3415 steam locomotive (currently being restored) to pull the train in the near future. This steam locomotive is larger than the diesel locomotive in operation at the present time.

The analysis phase of the project consists of determining the loads to be carried by the through truss bridge in question, calculating the forces induced in all truss members, determining the resulting stresses and comparing the stresses induced by both engines. The truss analysis was completed using RISA software. An analysis was performed for the ALCO diesel engine followed by an analysis for the Baldwin steam locomotive. Lastly, the increase in live load stresses are calculated and reported. In addition to change in member stresses, deflections, pier reactions, and influence lines of diagonal members were investigated.

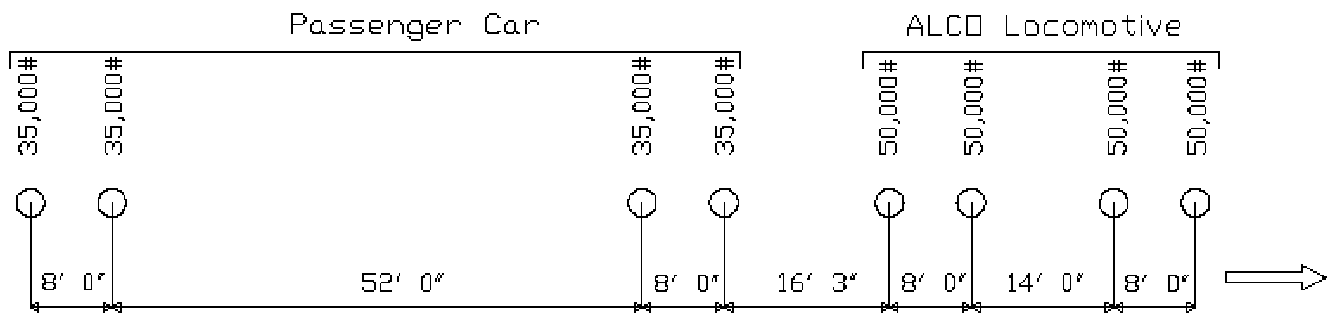
3.1 Analysis

To begin the analysis phase of the project, the owners of the bridge supplied the main information necessary to complete the analysis. The information supplied included bridge geometry, dimensions, and loading schemes. Using the information given, the most critical loading cases were determined and used for the analysis. Since both spans are identical, only one span was analyzed. The loading configurations were chosen such that the entire span of the bridge is fully loaded.

3.1.1 Loading Configurations

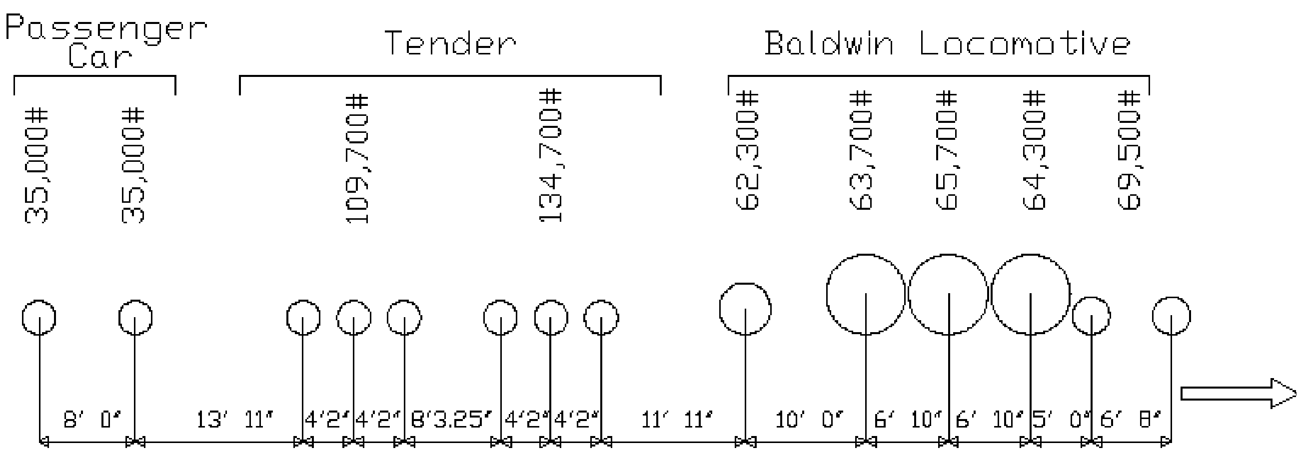
The current engine being used is a 100 ton 1945 ALCO S-1 diesel locomotive. It is the lone engine being used to pull the passenger and dinner cars. It is supported by four axles carrying equal loads of 25 tons or 50 kips. The spacing of the axles from front to back is as follows: 8 ft, 14 ft, and 8 ft. The total length of the diesel engine is 44.5 ft. The four axles are comprised of two trucks that equally distribute the load to the rails. The axle loading and spacing are shown in Figure 3.1.

Figure 3.1 The ALCO Diesel Locomotive Axle Loads and Spacing



The engine that the Abilene & Smoky Valley Railroad Association (ASVRA) wishes to use is a 285 ton (569.9 kips) assembly consisting of a 1919 Baldwin 4-6-2 “Pacific,” Santa Fe #3415 steam locomotive and a tender. The steam locomotive and tender combination has a total length of 82.7 ft and has 12 axles that distribute the weight to the rails. The tender “tends” the locomotive and is where the water and fuel is stored. For the analysis, the tender is assumed to be fully loaded and weighs 244.4 kips. However, it must be stated that the tender will most likely never be fully loaded due to the 10 mile roundtrip excursions offered by the Abilene & Smoky Valley Railroad Association. Figure 3.2 illustrates the axle loads and spacing for the 1919 Baldwin steam locomotive.

Figure 3.2 The 1919 Baldwin Steam Locomotive Axle Loads and Spacing



As discussed above, in order to have a better comparison of induced stress levels in the truss members, a fully loaded span was considered. In other words, a conservative combination of engines and cars that resulted in the heaviest loads and highest number of axles on the bridge at one time was considered for both cases. This would allow for the most conservative analysis of the bridge and ensure that the 109 ft span of the bridge is fully loaded. Comparing just the diesel engine loading to the steam locomotive and tender, which is about 40 ft longer than the diesel engine, would not yield representative results. In order to fully engage the bridge, the heaviest passenger car was added to the loading combination of axles. The heaviest passenger car weighs 70 tons, has a length of 86 ft, and has four axles that equally distribute the weight of the car and passengers. Figures 3.1 and 3.2 (above) include additional axles from the passenger car. Table 3.1 compares the total maximum weight present on a single span at any time.

Table 3.1 Comparison of Total Load on Single Span

	ALCO Locomotive (kips)	**Baldwin Locomotive (kips)**
Locomotive	200	326
Tender	-	244
Passenger Car Axles	140	70
Total Weight	340	640

3.1.2 Structural Model

For analysis purposes, all joints were assumed to be pinned. This assumption is conservative in this case and is most likely how the bridge was originally designed. During the literature review portion of the project, multiple articles discussed that several factors lead to this assumption being conservative. Composite action of the truss and connections being somewhat rigid leads to lowered levels of axial stresses. Adding to the rigidity of the connections is the fact that the bottom and top chords are made of long continuous sections that are not cut at the joints. Additionally, the rails will help distribute the axle loads to other joints.

In order to ensure that the load was only applied to the joints of the truss, an extra "auxiliary structure" was added to the model. This secondary structure consists of nine simply supported beams that are located directly underneath each of the truss panels. Vertical hangers, whose function is to transfer the reactions to the joint of the bridge truss, support each beam. This system was used in accordance to the assumption that the bridge acts as a true truss.

For modeling purposes, each of the vertical hangers in the secondary structure was taken as being two feet long. The length of these hangers does not effect the analysis because the members are there only to transfer loads. The results (stresses, deflections, etc.) for this auxiliary structure were ignored due to the fact that the hangers are short links. The secondary structure was modeled in RISA using an arbitrary section with a negligible area and section properties so as not to add any dead load to the analysis. This system of beams and hangers represents how the bridge's longitudinal girders and floor-beams transfer the train loads to the joint of the structure. Figures 3.3 and 3.4 depict the model used in RISA. The dead loads shown in Figure 3.4 (-6k) represent the dead loads due to the longitudinal girders, floor-beams, tracks, ties, etc. (The negative signs in RISA indicate that the loads are pointing downward). The moving (live) loads are half the axle loads of the Baldwin locomotive, tender, and two axles of the passenger car (Figure 3.2).

Figure 3.3 The RISA Model with "secondary structure"

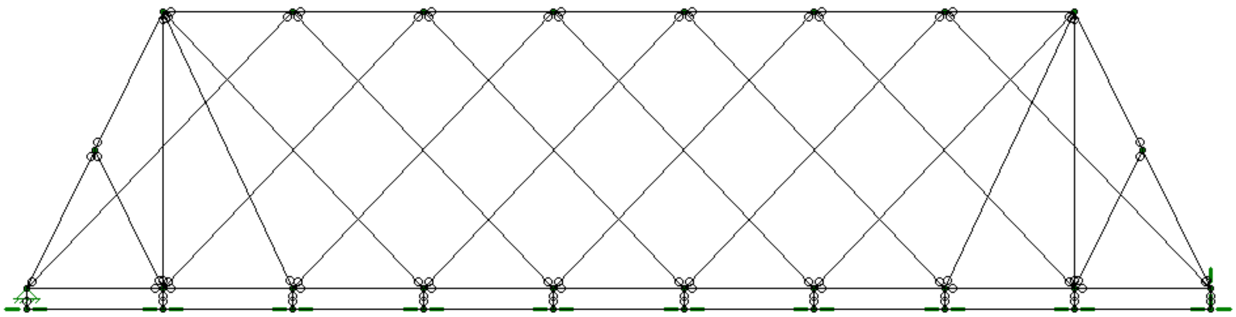
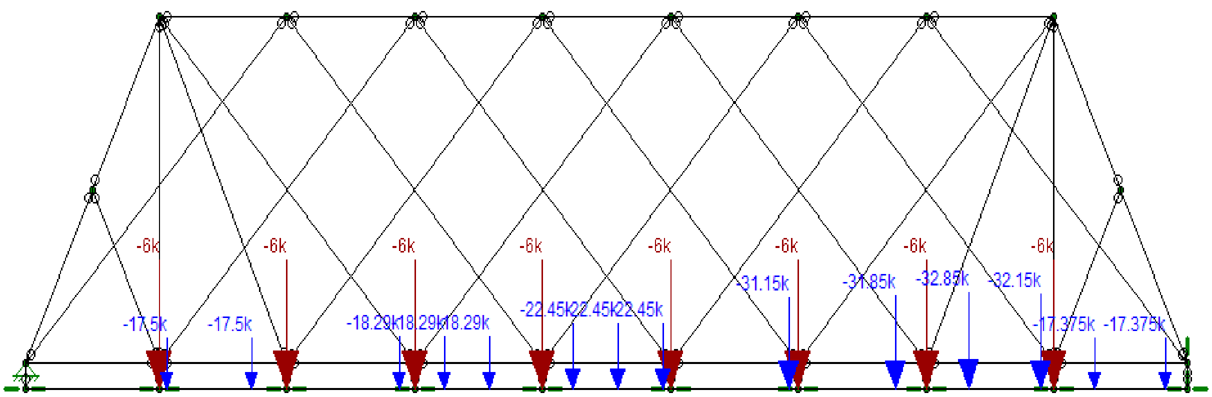


Figure 3.4 Dead Loads and Moving Load on "secondary structure"



A letter and number identify the members and their locations. The letters indicate the type of member: the letter B identifies a member belonging to the bottom chord, V is for the vertical members, T denotes that the member is in the top chord, P is for the end-portals, and L and R identify diagonal members. The numbering starts from the left and increases to the right. There are nine different types of sections used in the construction of the truss members. The sections are built up by riveting plates and angles together in multiple combinations. Figures 3.5 and Figure 3.6 show the member and joint labels as used in the RISA model.

Figure 3.5 Member Labels in the RISA Model.

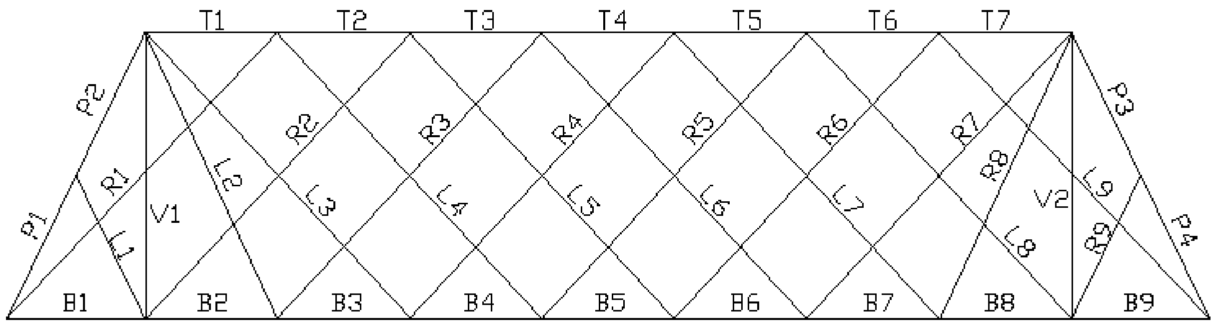
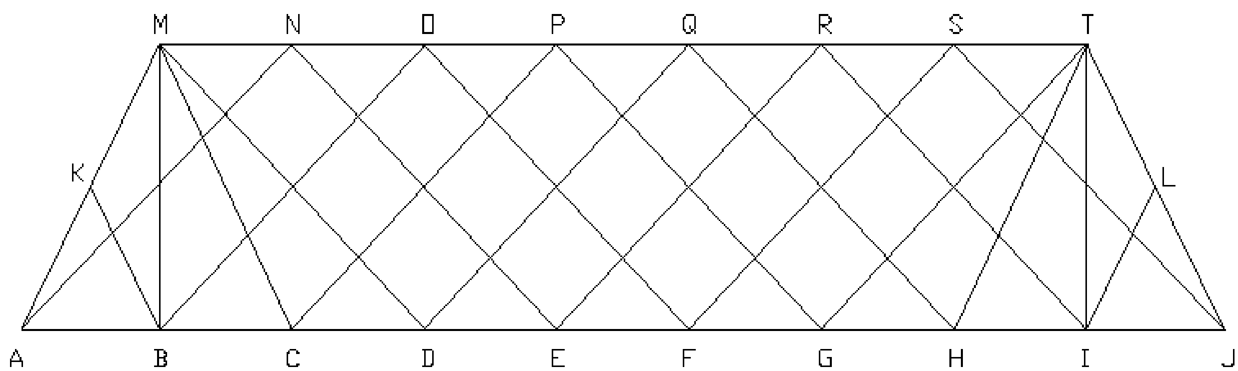


Figure 3.6 Joint Labels in the RISA Model.



3.1.3 Cross-Sectional Properties

Using information and drawings provided by Mr. Joe Minick of ASVRA, the section properties of each member were calculated and input into RISA. The areas and moments of inertia were calculated. This would allow for the calculation of axial stresses as well as buckling loads for the compression members. After the stress calculations were completed, the results were compared to find the percent increase that occurred because of the heavier steam locomotive. Figure 3.7 shows the different sections used for the truss members. Appendix A includes detailed drawings of these sections. Table 3.2 lists the section properties including area and moments of inertia.

Figure 3.7 Truss Member Section Labels

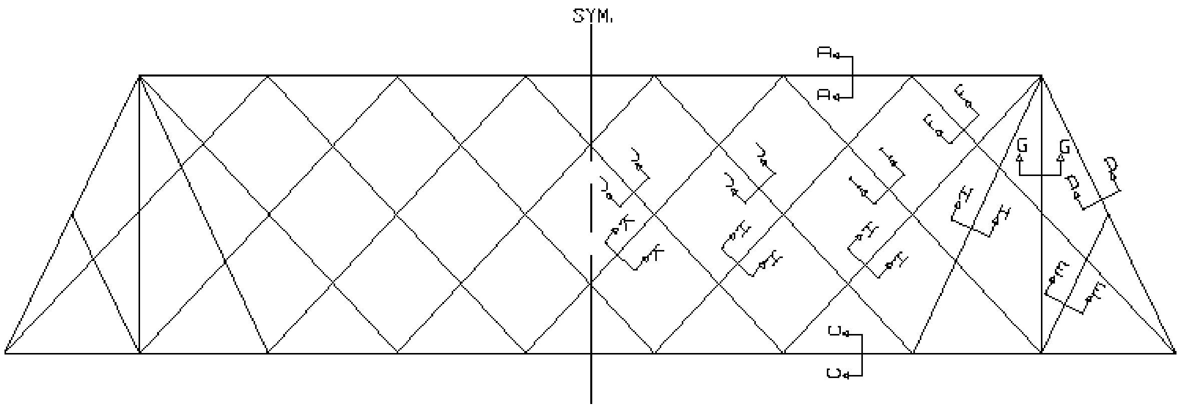


Table 3.2 Section Properties of Members

Section Properties			
	Area (in^2)	**Strong Axis Moment of Inertia (in^4)**	**Weak Axis Moment of Inertia (in^4)**
C	18.93	297.05	11.03
D	20.8	545.95	488.12
E	4.22	217.19	3.5
F	7.05	133.6	16.82
G	7.05	235.41	16.82
H	6.18	117.23	9.52
I	5.74	82.94	9.04
J	4.87	66.7	3.96
K	4.43	147.49	3.74

3.1.4 Stress Analysis

Checking stresses was comprised of two steps. First, only self weight and dead load stresses were calculated for the members. The dead load is comprised of the member self weight, rails, ties and floor beams. The dead weight was added to the previously discussed secondary structure by adding point loads to the sub-structure at the joints (-6k in Figure 3.4). Commonly used design weights for these materials were taken from the 1937 AREMA design code. The dead weight calculations are based on the following: rails- 200 plf, timber- 60 pcf and steel- 490 pcf.

Second, the moving live loads were introduced to the structure. After calculating the stresses induced due to the introduction of the live load plus dead load and self weight, the stress results from the dead load and self weight only scenario were subtracted. The increase of stress due to only live loading was calculated. Then, the increase in stress was found for the live load plus dead load and self weight. By comparing these results, a better and more informed decision can be made regarding the use of the bridge.

To check allowable stresses, the Railroad Bridge code was used (AREMA, 1937). Section 15, "Iron and Steel Structures" listed the requirements for structural members. The following checks are taken directly from this section.

Allowable Axial Tension Stress

$$\leq 18{,}000\,psi \tag{1}$$

Allowable Compressive Stress for l/r ≤140

$$15{,}000 - \frac{1}{4}\frac{l^2}{r^2} \tag{2}$$

Allowable Compressive Stress for l/r >140

$$p = \frac{\frac{y}{f}}{1 + 0.25 Sec \frac{0.75l}{2r}\sqrt{\frac{fp}{E}}} \tag{3}$$

where:

p= allowable compressive unit stress

l= length of member, in inches

r= least radius of gyration of member, in inches

y= yield point in tension, 33,000 for structural steel

f= factor of safety, 1.76 for structural steel

Stresses in the diagonal-to-bottom chord connections were also checked for increases due to the heavier loads. The connections were checked for shearing stress in the rivets as well as bearing stresses due to the rivets on the plate material surrounding the rivets' shanks.

3.2 Results

As previously discussed, RISA was used to compare results of both the ALCO diesel engine and the Baldwin steam locomotive. The results compared from this analysis include increased stress, deflection and reactions. When comparing the results of the stress increase, an analysis was first performed only considering the dead loads. Then, analyses were performed using both dead load and live load. The stress results from the dead load case were subtracted from the dead plus live load results. Then, stress increases were found for the live load only, as well as for the total load.

After finding the increases in stress, several other calculations were performed. Influence lines were plotted for diagonal truss members R4, R5, and L8 to help understand where stress reversals occur in these members. Bottom-chord connections were checked for adequacy. Along with a comparison of stress levels, a comparison of deflections for the two locomotives was completed.

3.2.1 Comparison of Stresses

The first stress comparison performed during the project was to determine the increase in stress in the truss members due to dead and live loading. Secondly, a comparison of stress due to live loading only was completed. To do this, the results from the dead load only scenario were subtracted from the dead load plus live load scenario. This was necessary because the self-weight is always included in the analysis performed by the RISA program, regardless of the load cases. In order to isolate the self weight of the members, the analyses were performed using both the dead loads and live loads.

The first analysis performed to find stress increase in members included dead loads. RISA calculates the weights of members based on the cross-sectional area used and automatically includes them in the analysis performed. To include other dead weight, point loads were added to the auxiliary structure joints. The dead weight calculated included the weight of rails, cross beams and ties. The calculated dead load to be applied to each joint was approximately 5.5 kips. To be conservative, the loads were rounded up to 6 kips to account for incidental dead weight including fasteners and connections (See Figure 3.4.). Obviously, after including stresses due to dead loads, the relative increase in stress due to the larger locomotive were smaller than when taking into account only the live load. The relative increase in stress from the overall load was generally lower by 20% to 30% for most of the members when compared to the results of the live load only analysis.

For the ALCO analysis including dead loads the highest tensile stress was found in the diagonal member R7 (or L3) and had a magnitude of 8.23 ksi. The highest compressive stress was found in the top chord member T4 and was 6.81 ksi. The highest tensile stress in the bottom chord was 5.99 ksi and was found in B5.

The highest tensile stress for the Baldwin loading including dead weight was found to occur in R7 and had a magnitude of 12.84 ksi. The largest compressive stress was again found in T4 and was 11.57 ksi. The largest tensile stress found in the diagonal members was in R8 and was 12.39 ksi. The largest tensile force in the bottom chord was 10.81 ksi and was found in B5. The highest compressive force in the diagonal truss members was in L9 and was 9.18 ksi. Table 3.4 lists some of the results from this portion of the analysis. Results for all members of the truss are included in Appendix B.

Table 3.3 Comparison of Stresses in Critical Members Due to Total Load

	Member Stresses and Increases		
	ALCO + DL (ksi)	**Baldwin + DL (ksi)**	**Increase (%)**
B5	5.99 (T)	10.81 (T)	80.47
T4	6.81 (C)	11.57 (C)	69.90
R7	8.23 (T)	12.84 (C)	56.01
R8	7.98 (T)	12.39 (T)	55.26

After separating the ALCO live loading results for the bridge, a maximum stress of 7.32 ksi was found in the vertical member V2 (and V1 by symmetry). The highest tensile stress was found in the diagonal truss member R8 (and L2 from symmetry) and had a magnitude of 6.78

ksi. The highest compressive stress was found in the top chord member T4 (at midspan) and was 5.35 ksi. The highest tensile stress in the bottom chord (also at midspan) was 4.59 ksi and was found in B5.

The highest tensile stress found for the Baldwin loading was found to occur in R8 (or L2) and had a magnitude of 11.19 ksi. The largest compressive stress was again found in T4 and was 10.11 ksi. The largest tensile stress found in the bottom chord members was in B5 and was 9.41 ksi. The highest compressive stress in the diagonal truss members was in L9 (or R1) and was 9.18 ksi.

Then, the two sets of results were compared to find the increase in stress in all of the truss members. Due to the large increase in weight due to the heavier locomotive, the increase in stress in the members was also very large. All members of the bottom chord experienced increases in tensile stress of at least 91.82% (B3) and as much as 106.71% (B8). The increases in compressive stress for the top chord range from 80.62% (T6) to 97.48% (T1). Table 3.3 shows the increases in stress for some critical members.

Table 3.4 Comparison of Stresses in Critical Members Due to Live Load

	Member Stresses and Increases		
	ALCO (ksi)	**Baldwin (ksi)**	**LL Increase (%)**
B5	4.59 (T)	9.41 (T)	105.01
T4	5.35 (C)	10.11 (C)	88.97
R8	6.78 (T)	11.19 (T)	65.04

3.2.2 Comparison of Deflections

During analysis, the deflections for the bottom chord joints were recorded. First, with only the dead load applied, deflections were recorded. Finding these dead load-only deflections allows for a better comparison for increase of deflection for the two locomotives. After this analysis, the truss was analyzed for the two locomotives and the increase in deflection was calculated. For all eight bottom chord joints, increases between 80% to 100% were observed. Table 3.5 lists the deflection results of the bottom chord joints for all load cases.

Table 3.5 Comparison of Bottom Chord Joint Deflections

	Joint Deflections			
	Dead Load Only (inches)	ALCO + DL (inches)	Baldwin + DL (inches)	LL Increase (%)
B	0.049	0.227	0.39	91.57
C	0.073	0.33	0.587	100.00
D	0.094	0.429	0.755	97.31
E	0.103	0.518	0.859	82.17
F	0.103	0.523	0.876	84.05
G	0.094	0.46	0.791	90.44
H	0.073	0.35	0.618	96.75
I	0.049	0.248	0.42	86.43

3.2.3 Influence Lines

As stated in the scope of work, influence lines were plotted in order to find when reversals of stress occur in the truss members. To do this, unit loads of one kip were placed at each of the bottom chord joints and the forces in the truss members were recorded. Then, the Baldwin axle load was moved across the truss in 10 ft increments and the loads placed on each of the bottom chord joints was recorded. Using the method of superposition, the member force results from the unit loads were multiplied by the load in each of the joints to find the overall force in each truss member. This exercise was performed for three members: R4, R5, and L8. Figures 3.8, 3.9, and 3.10 show the stress variations for these three members, respectively, as well as the locations of the front axle where stress reversals take place and how many stress reversals occur for each single passage of the train. For axle locations of over 109 ft, the front axle is no longer on the truss, but several axles are still loading the truss.

It should be noted that these diagrams are drawn for the loads shown in Figure 3.2. Additional stress reversals will take place due to the other axles from the rest of the passenger/dinner cars, although the stress magnitudes will be significantly lower.

Figure 3.8 Stress Variation for Truss Member R4

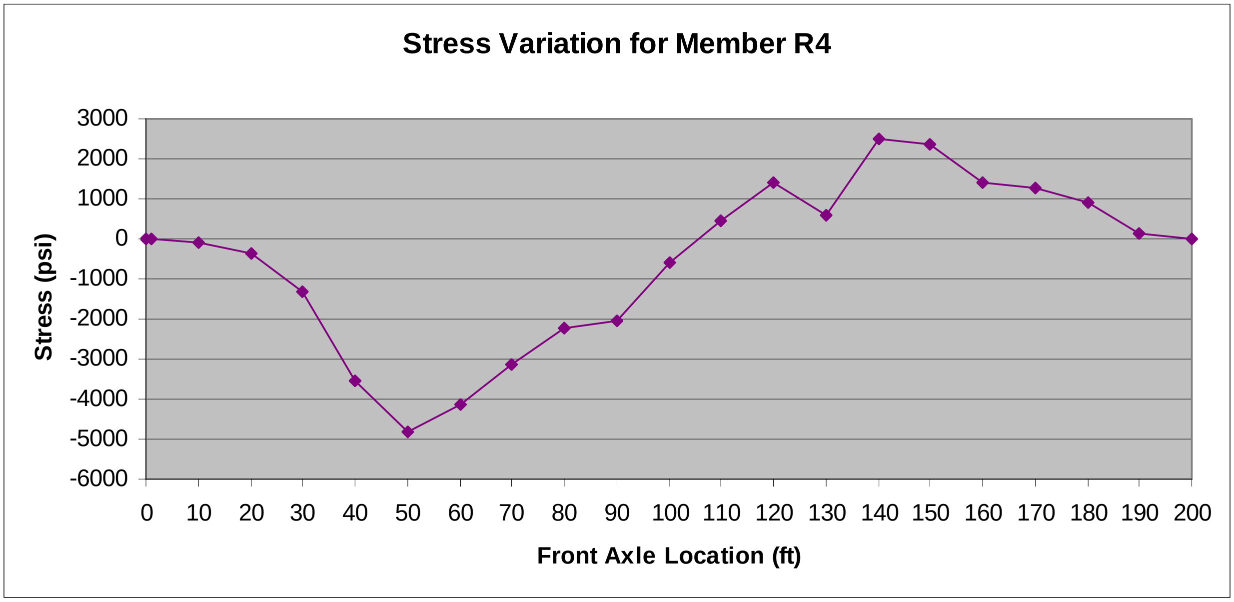


Figure 3.9 Stress Variation for Truss Member R5

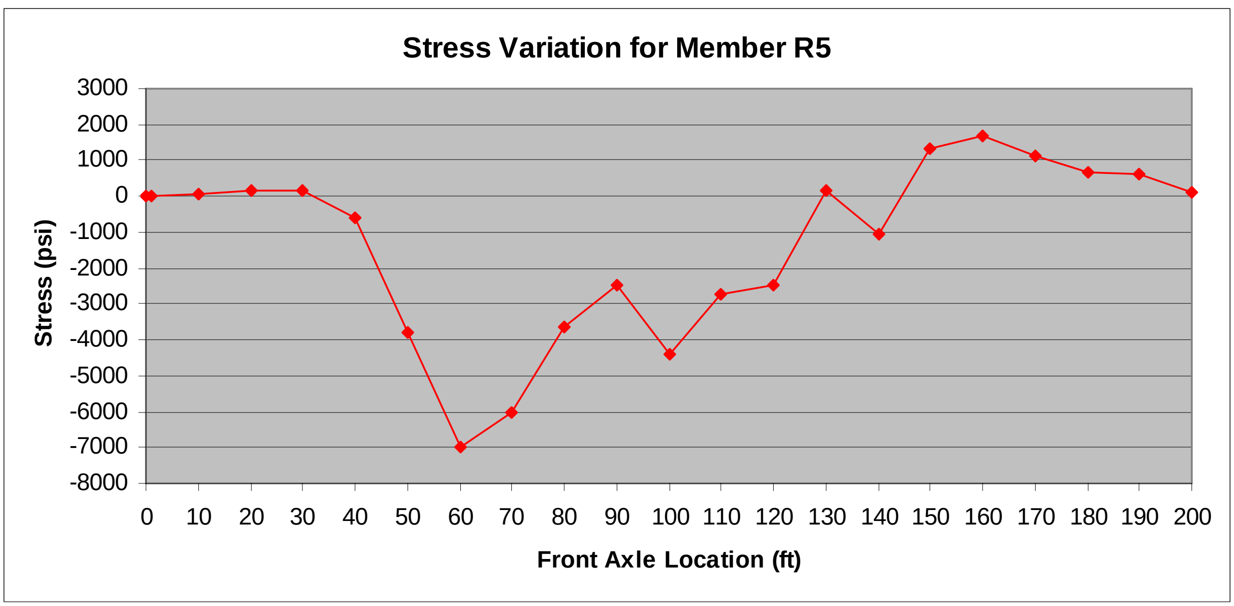
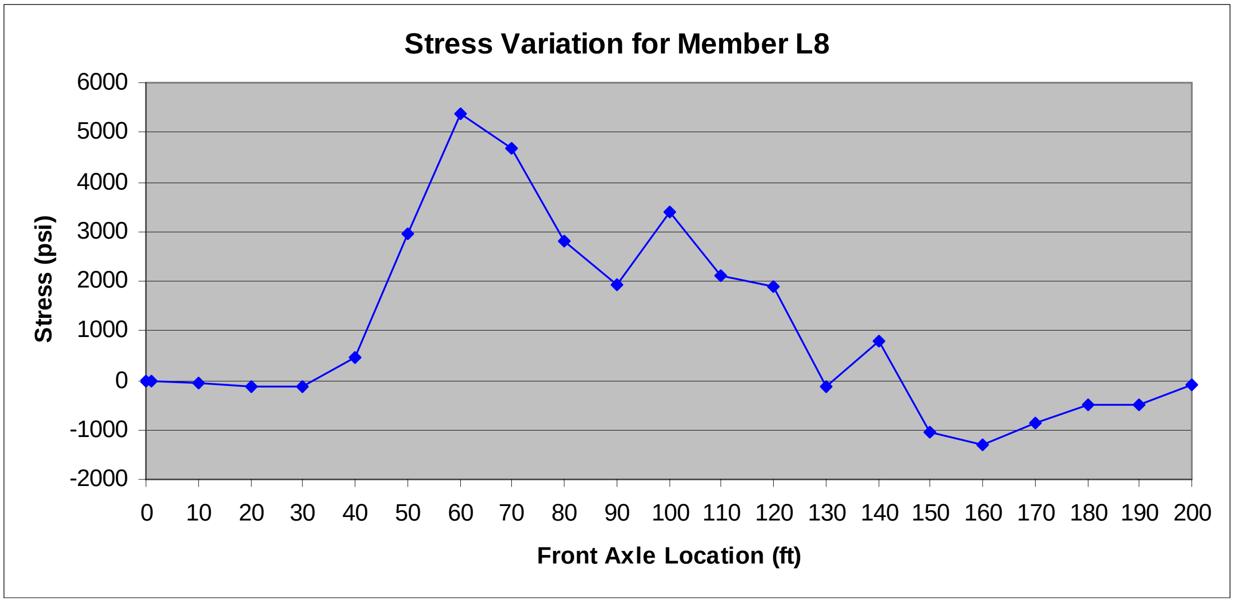


Figure 3.10 Stress Variation for Truss Member L8



Using the stress variation diagrams, the number of cycles a member endures during each passage of the train can be determined. Each time a member undergoes a stress reversal, or cycle, the remaining fatigue life of that member is diminished. These diagrams can be used to determined how many cycles the members are expected to support within the next several years. All members have a finite fatigue life and can fail due to too many fatigue cycles.

3.2.4 Middle Pier Reaction

For reference, the increase in reactions at the piers was also noted. The increase in reaction at the left pier at Joint A (for the RISA model, the train travels from left to right) was 94 kips or a 97 % increase. The increase in reaction for the right pier at Joint J was about 91 kips or an 87% increase. These changes in reaction are only when the train is traveling on one truss. The maximum reaction that the middle pier will support occurs when the train has axles being supported by both trusses. For the ALCO diesel locomotive loading, the maximum reaction at the middle pier occurs when the front axle is 30 ft past the pier and onto the next truss. At this location, the fourth locomotive axle is directly above the pier and all four passenger car axles are carried by first truss. The reaction that results from this loading is 248 kips. The maximum reaction occurring at the middle pier due to the Baldwin steam locomotive loading scenario is 502 kips. It occurs when the front axle of the locomotive is 35 ft past the pier and onto the second truss which places the sixth axle within a foot of the pier. This is a 102% increase in loading on the middle pier due to the most conservative placement of loads across the pier. This information can be used for possible rehabilitation of the piers.

3.2.5 Connection Stresses

Lastly, stresses in the connections were checked. The diagonals of the truss are connected directly to the plates that make up the bottom chord. The steel angle is riveted to the bottom chord by one leg. The second leg of the diagonal angle is riveted to a second shorter piece of angle, which in turn is riveted to the bottom chord plate. All connections are completed using between eight to twenty ¾ inch rivets. A typical connection is illustrated in Figure 3.11.

Figure 3.11 Typical Diagonal-to-Bottom Chord Connection

The bottom chord connections were checked for rivet shear and bearing stresses. When checking for rivet shear stresses, the rivets were assumed to be hand-driven. This was the most conservative assumption since the method of installation is unknown. For hand-driven rivets, the maximum allowable stress is 11,000 psi as opposed to machine driven rivets which have a maximum allowable stress of 13,500 psi (AREMA, 1937, 15-9). The most conservative maximum bearing stress allowed by this manual is 20,000 psi. Again, this is based on the assumption that the rivets used on the bridge were hand-driven. The allowable stresses were used in rivet single shear and rivet bearing stress equations, and a maximum force was calculated. The maximum allowable forces were checked against the actual member forces resulting from the Baldwin loading case.

After calculation of the connection stresses, one connection did not pass specifications. The diagonal to bottom chord connection of R7 failed to meet the maximum rivet shear stress. The connection consists of 16 rivets and has a maximum allowable force of 77.44 kips. The maximum force induced by the Baldwin loading sequence is 79.35 kips. All other bottom chord connections met the maximum stress requirements.

It is recommended that the connections on the top chord also be checked, especially those at joints M and T be checked carefully since they are the points where the highest loaded diagonal members are connected.

CHAPTER 4 - Conclusions and Recommendations

4.1 Summary

The first step of the project was to conduct a literature review of several articles that investigated the process of analyzing and rehabilitating older steel bridges. By completing this review, an outline for this project was developed and followed. Also, the literature review yielded ideas on how to rehabilitate aging truss bridges.

For the analysis stage of this project, RISA 3-D was used to determine all forces in the truss members. Comparing forces induced by both locomotives, stress increase was calculated for each of the truss members. In addition to stress increase, comparisons of deflections and pier reactions were completed. Also, stress variation diagrams were plotted for the diagonal members subjected to the most cyclic loads to illustrate when stress reversals occur.

Combining the results from the literature review and the analysis, conclusions were reached and recommendations were made. The conclusions and recommendations are listed and discussed below.

4.2 Conclusions

Throughout the study, stress levels and increases have been calculated using a conservative analysis model. It was determined that several members experienced a near 100% increase of live load stress. This includes most of the members undergoing at least an 80% stress increase due to live load changes. When comparing the stress increases due to live load and dead load combined, most members undergo a stress increase of 55% to 80%. When checked against the allowable stress of 18 ksi, per 1937 AREMA code, all tension members pass safely. When checked against the allowable stress of $15{,}000 - 0.25*(l^2/r^2)$ (psi), all compression members passed.

The main objective for the project was to find the increase in member stresses due to the change in live loading. After that objective was completed, several additional comparisons were made including deflection increases, pier reactions, stresses in connections and when stress reversals occur. The increase in deflection for the bottom chord joints due to increase in the live loads range from 82% to 100%. The reaction to the middle pier increases from 248 kips to 502 kips, which is an increase of 102%. Eight diagonal truss members undergo stress reversals, of which three influence lines have been plotted. The influence lines are included in the previous chapter.

Lastly, connection stresses were checked for capacity by calculating rivet shear and bearing stresses for the bottom chord connections. After completion of theses calculations and checks, it was found that one bottom chord connection did not pass the maximum rivet shear stress requirements. The maximum allowable force in the connection is 77.44 kips while the force present in the member is 79.35 kips. It is recommended that the connections on the top chord also be checked, especially those at joints M and T be checked carefully since they are the points where the highest loaded diagonal members are connected.

4.3 Recommendations

For this analysis, it was assumed that the truss performed as though it had perfectly pinned joints. The actual truss does not have true-pinned connections and also has long sections that are not cut at the joints. To gain a better understanding of exactly how the structure acts and how forces are distributed throughout the truss, instrumenting of the bridge by use of strain gages should be completed. After instrumentation is completed, strain data can be recorded for a known load passing across the bridge.

During the initial stages of the project, a brief visual inspection of the bridge was performed. Before using the larger steam locomotive, a more thorough inspection should be performed for all bridge components. All members and connections should be checked for cracks and any other defects. This can be accomplished using a magnetic particle testing apparatus. In addition to the truss, the piers and the foundation should also be checked. Careful inspections and maintenance should be carried out on a regular basis to ensure that the bridge's condition does not deteriorate to a dangerous level.

It would also be advantageous to understand exactly what material was used to construct the trusses. Engineers do not always have records of the materials used to construct bridges at the beginning of the 20th Century. In order to determine the properties of the steel used to build this bridge, coupons should be cut out from non-critical locations. The coupons should then be tested for chemical composition, mechanical properties, and the yield strength and ultimate strength should be determined.

In addition to testing coupons for physical properties, tests should be performed on additional coupons to determine the remaining fatigue life of the material. Although very few loading cycles are passing across the bridge now, there is no way to determine the number of cycles that have passed across to date. As Figures 3.8, 3.9, and 3.10 illustrate, multiple stress reversals occur with each passing of the train which diminishes the fatigue life further. Also, a fatigue analysis should be performed for all of the connections on the bridge.

The easiest way to ensure lower stresses in the truss is to control the loading that passes across the bridge. First, since the excursions are relatively short (10 miles roundtrip), the tender should be filled with the minimum amount of fuel and water. By filling the tender with half of the maximum fuel, a weight savings of approximately 60 kips occurs. Second, follow the tender with the lightest of the passenger or dinner cars. This should also be done when using the diesel engine. Any time that stress levels can be reduced on aging structures is beneficial.

For retrofitting members that carry heavier stresses, adding material to the member will ensure lower stresses and deflections. While doing this, great care must be taken to avoid damaging surrounding members. Also, the new sections must be connected in a way that ensures that stresses in existing connections are also lowered. Adding material only to the members does not increase the capacity of the connections. For retrofitting techniques, the papers discussed in the literature review portion of this project (Chapter 2) will offer some additional aid.

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Appendix A - Member Section Drawings

Figure A.1 Truss Member Section Label

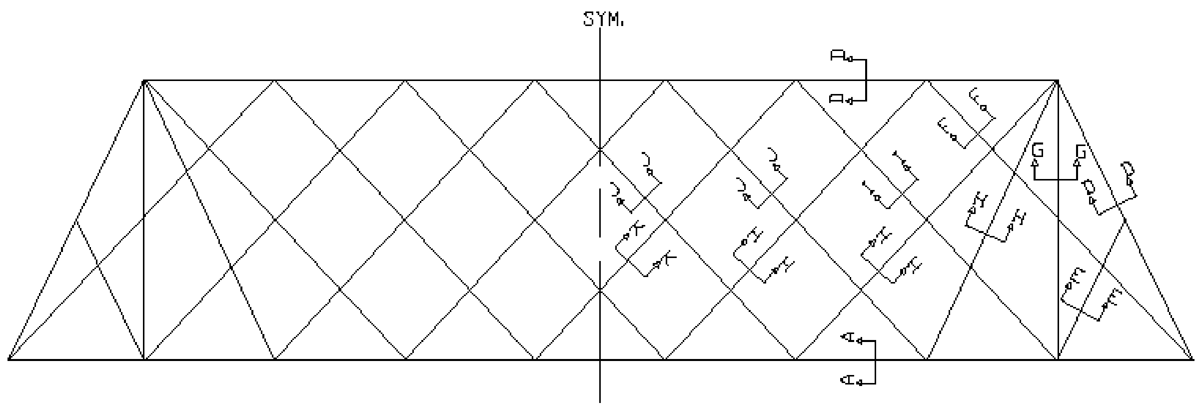


Figure A.2 Section A Through Both Lower Chords

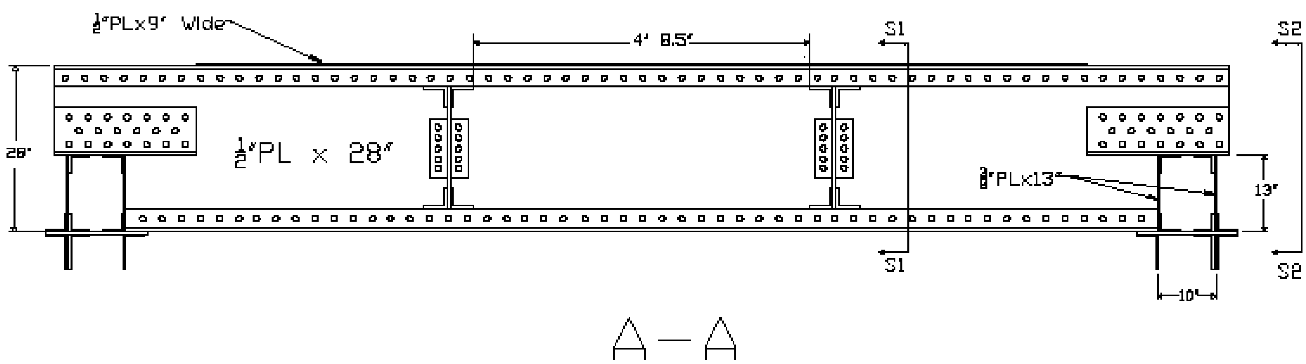


Figure A.3 Sectional View S1 of Figure A.2

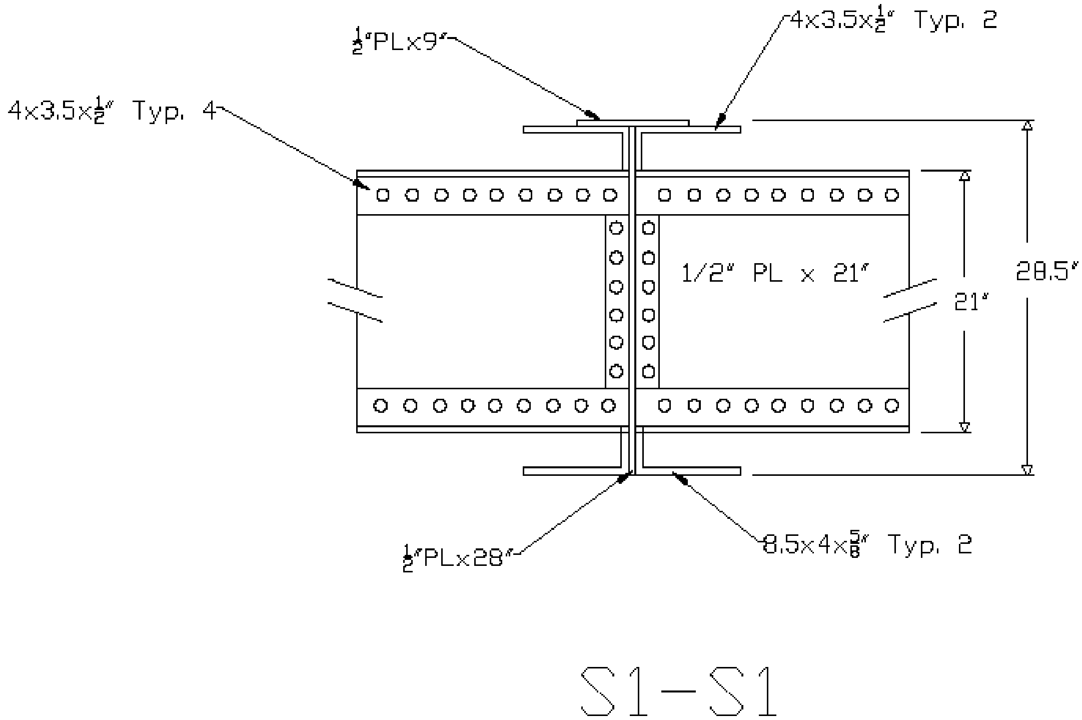


Figure A.4 Sectional View S2 of Figure A.2

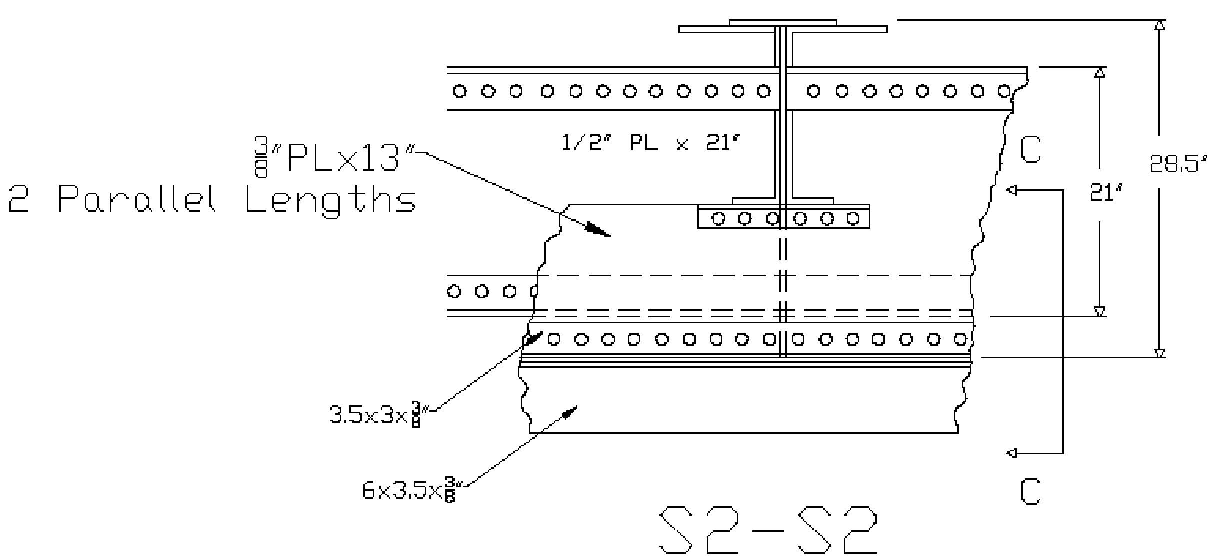


Figure A.5 Section C

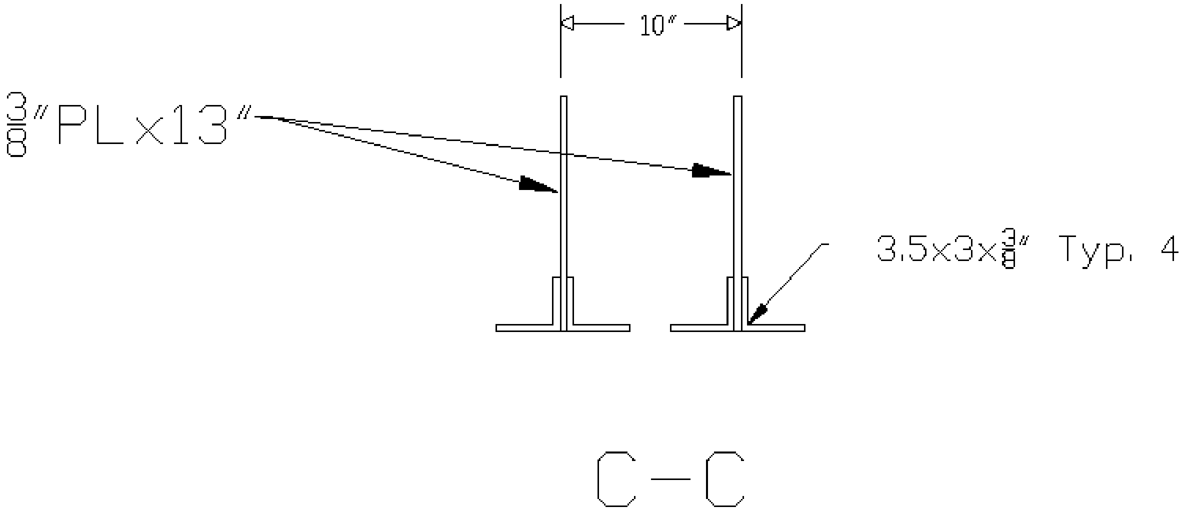


Figure A.6 Section D

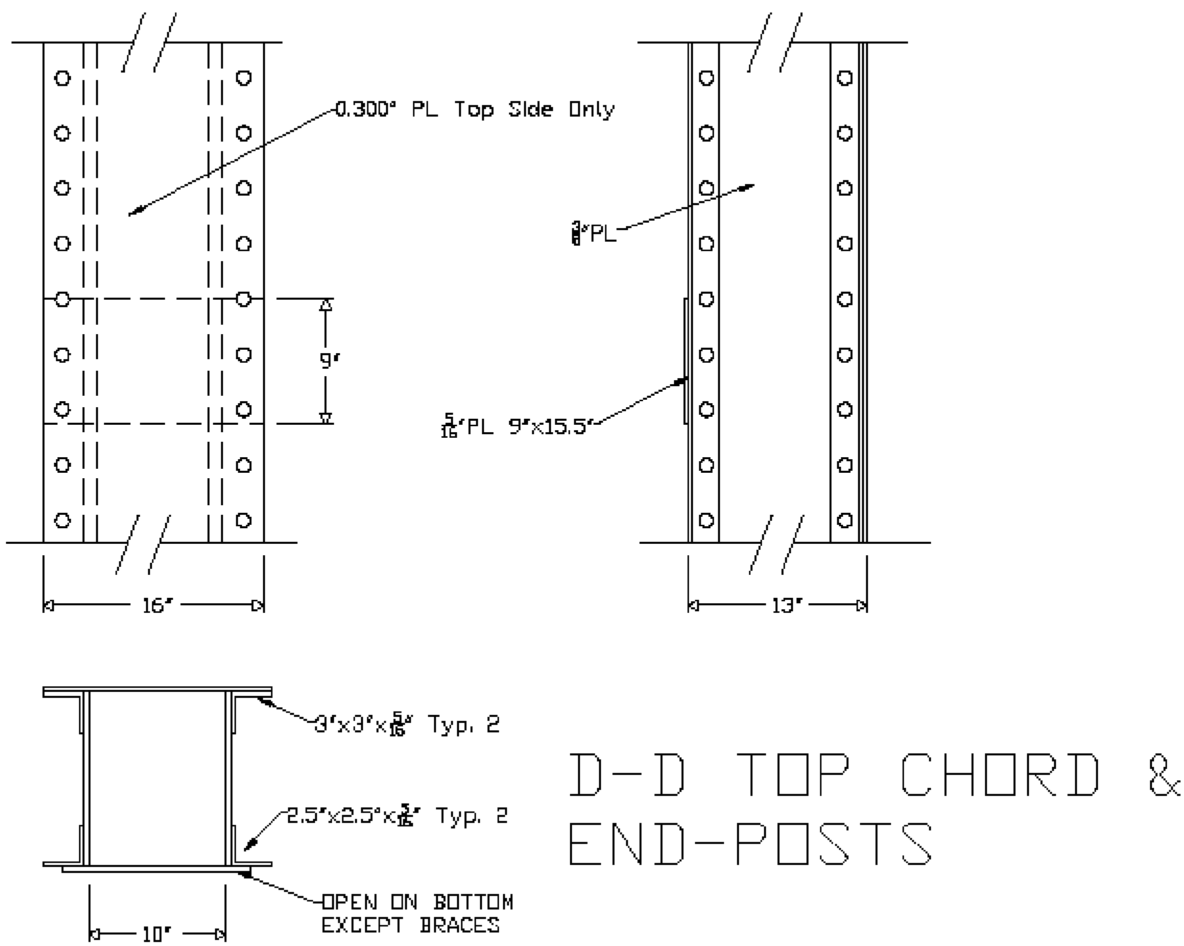


Figure A.7 Section E

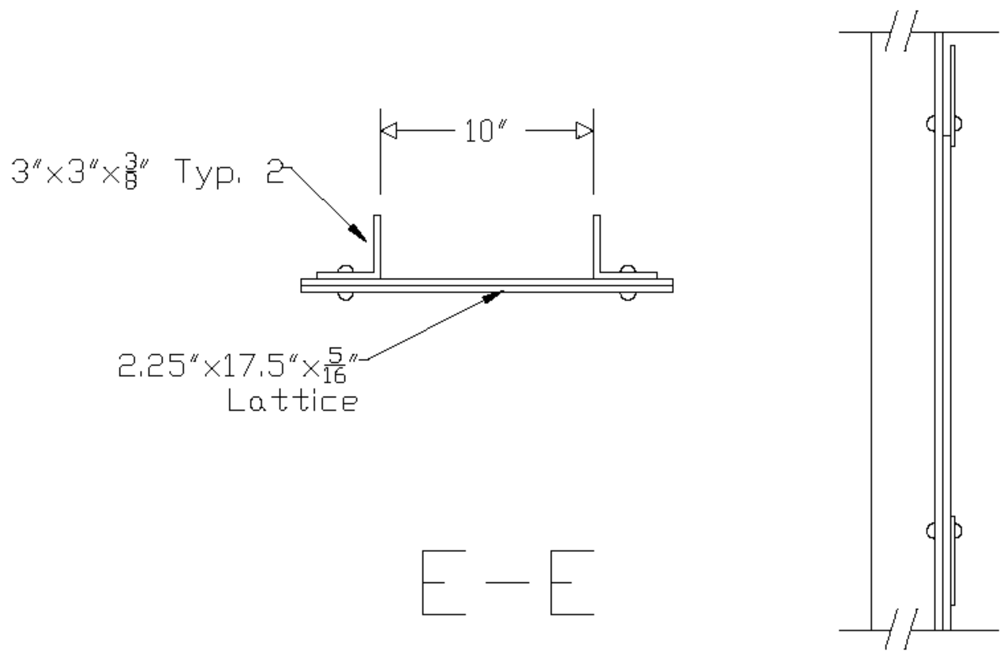


Figure A.8 Section F

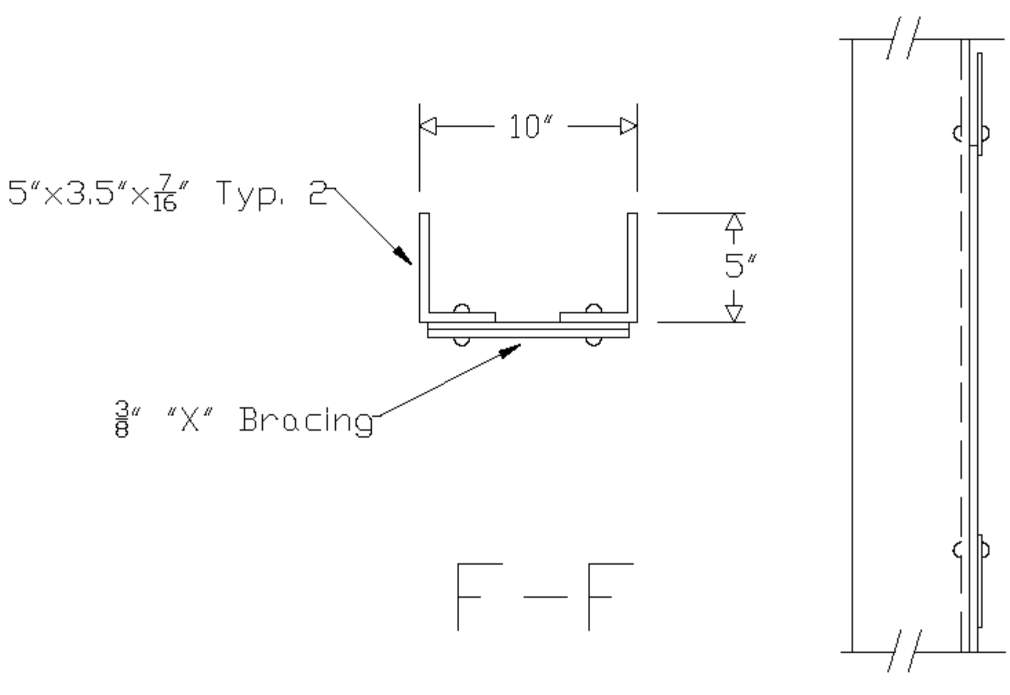


Figure A.9 Section G

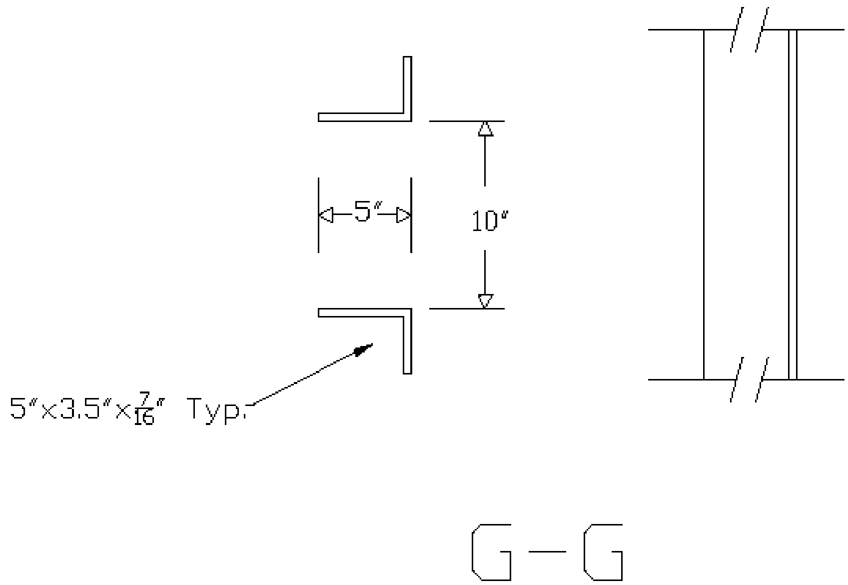


Figure A.10 Section H

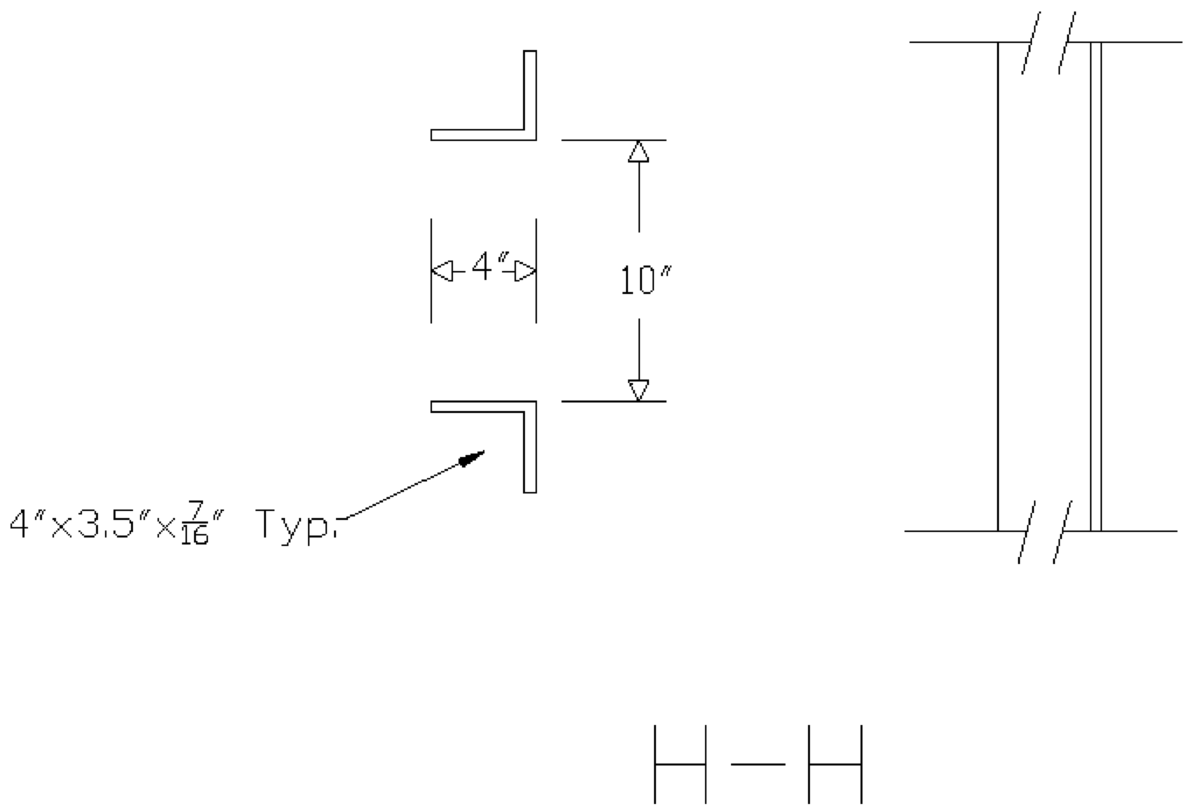


Figure A.11 Section I

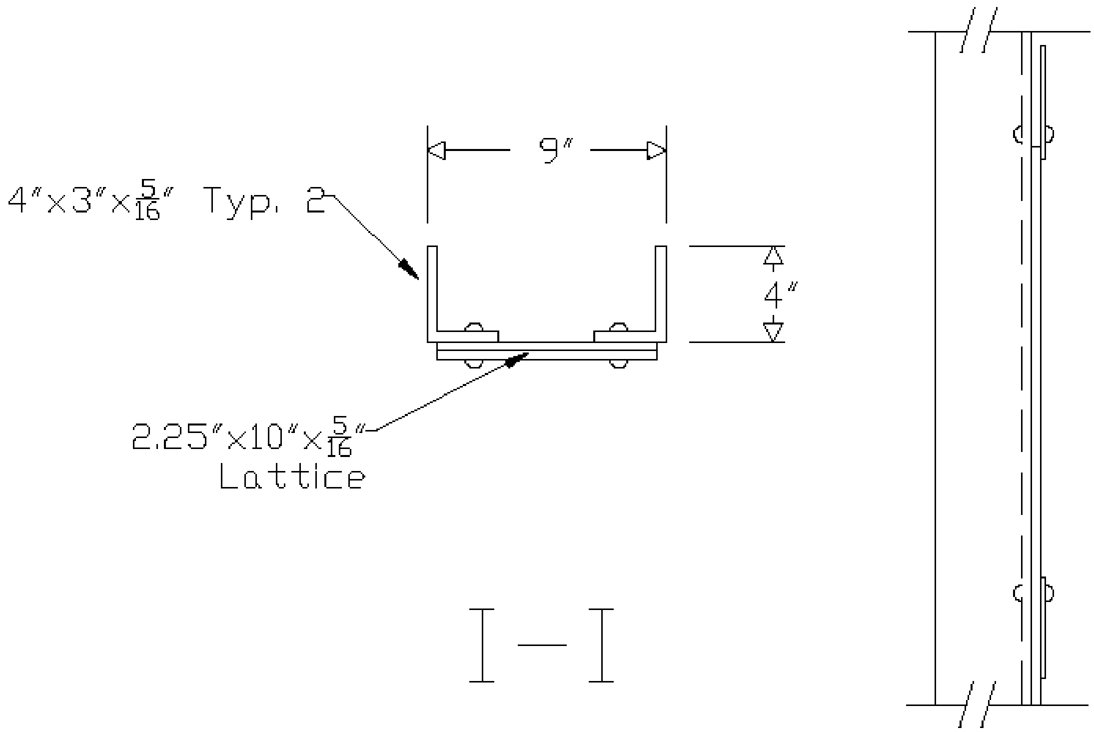


Figure A.12 Section J

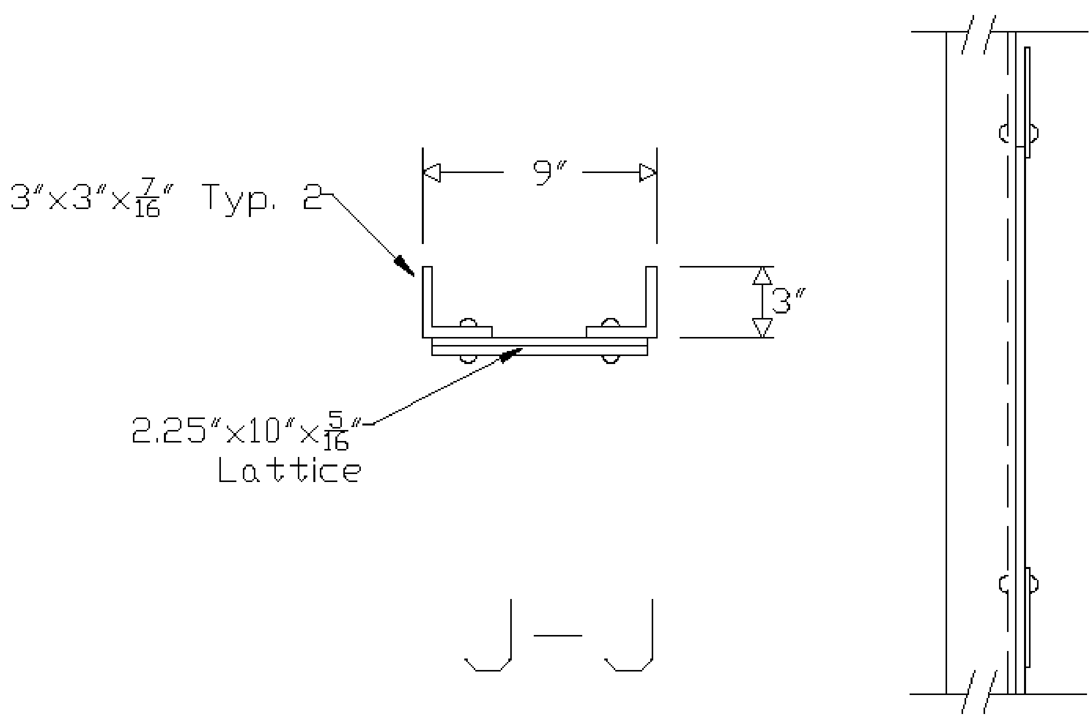
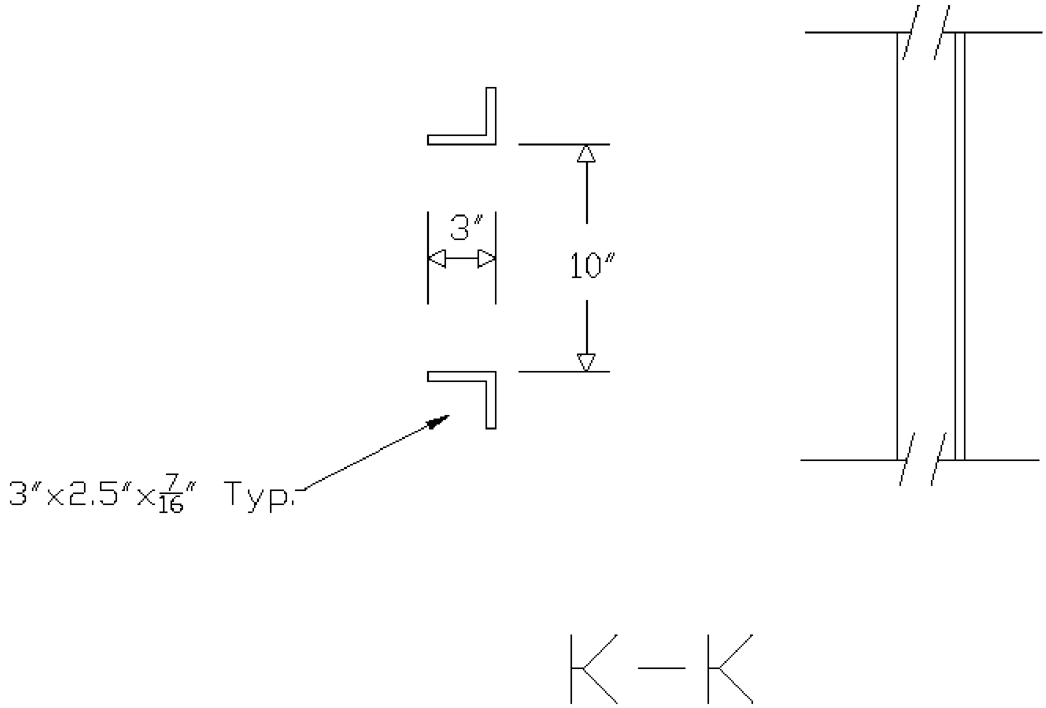


Figure A.13 Section K



Appendix B - Stress Results for All Members

Table B.1 Stress Results for All Members

Member Maximum Stresses, Percent Increases, and Allowable Stresses						
	DL + SW Only (ksi)	**ALCO + DL+ SW (ksi)**	**Baldwin + DL + SW (ksi)**	**Allowable Stress (ksi)**	**LL Increase (%)**	**Total Increase (%)**
B1	0.74 (T)	3.24 (T)	5.64 (T)	18.00 (T)	96.00	73.07
B2	0.80 (T)	3.81 (T)	6.60 (T)	18.00 (T)	92.69	73.23
B3	1.01 (T)	4.80 (T)	8.28 (T)	18.00 (T)	91.82	72.50
B4	1.24 (T)	5.44 (T)	9.87 (T)	18.00 (T)	105.48	81.43
B5	1.40 (T)	5.99 (T)	10.81 (T)	18.00 (T)	105.01	80.47
B6	1.24 (T)	5.47 (T)	9.89 (T)	18.00 (T)	104.49	80.80
B7	1.01 (T)	4.65 (T)	8.33 (T)	18.00 (T)	101.10	79.14
B8	0.80 (T)	3.63 (T)	6.65 (T)	18.00 (T)	106.71	83.20
B9	0.74 (T)	3.38 (T)	6.05 (T)	18.00 (T)	101.14	79.00
T1	0.85 (C)	4.02 (C)	7.11 (C)	14.78 (C)	97.48	76.87
T2	1.27 (C)	5.73 (C)	9.82 (C)	14.78 (C)	91.70	71.38
T3	1.40 (C)	6.59 (C)	10.91 (C)	14.78 (C)	83.24	65.55
T4	1.46 (C)	6.81 (C)	11.57 (C)	14.78 (C)	88.97	69.90
T5	1.40 (C)	6.63 (C)	11.14 (C)	14.78 (C)	86.23	68.02
T6	1.27 (C)	6.12 (C)	10.03 (C)	14.78 (C)	80.62	63.89
T7	0.85 (C)	4.33 (C)	7.47 (C)	14.78 (C)	90.23	72.52
P1	1.04 (C)	4.75 (C)	8.45 (C)	14.78 (C)	99.73	77.89
P2	1.04 (C)	4.75 (C)	8.46 (C)	14.78 (C)	100.00	78.11
P3	1.04 (C)	5.55 (C)	9.14 (C)	14.78 (C)	79.60	64.68
P4	1.04 (C)	5.55 (C)	9.12 (C)	14.78 (C)	79.16	64.32
V1	1.05 (T)	5.72 (T)	9.37 (T)	18.00 (T)	78.16	63.81

V2	1.03 (T)	7.32 (T)	11.9 (T)	18.00 (T)	72.81	62.57
R1	0.90 (C)	5.41 (C)	8.42 (C)	13.15 (C)	66.74	55.64
R2	0.33 (C)	3.65 (C)	4.67 (C)	12.20 (C)	30.72	27.95
		1.06 (T)	1.46 (T)	18.00 (T)	28.78	37.74
R3	0.20 (C)	3.34 (C)	4.43 (C)	9.59 (C)	34.71	32.63
		1.85 (T)	3.27 (T)	18.00 (T)	69.27	76.76
R4	0.19 (T)	1.75 (C)	2.58 (C)	9.59 (C)	42.78	47.43
		3.40 (T)	5.44 (T)	18.00 (T)	63.55	63.55
R5	0.43 (T)	1.35 (C)	1.91 (C)	9.82 (C)	31.46	41.48
		5.20 (T)	7.89 (T)	18.00 (T)	56.39	51.73
R6	1.03 (T)	6.75 (T)	10.59 (T)	18.00 (T)	67.13	56.89
R7	1.18 (T)	8.23 (T)	12.84 (T)	18.00 (T)	65.39	56.01
R8	1.20 (T)	7.98 (T)	12.39 (T)	18.00 (T)	65.04	55.26
R9	0.03 (T)	0.16 (T)	0.26 (T)	18.00 (T)	76.92	62.50
L1	0	0.01 (C)	0.01 (C)	18.00 (T)	0	0
L2	1.20 (T)	7.32 (T)	11.56 (T)	18.00 (T)	69.28	57.92
L3	1.18 (T)	6.97 (T)	11.01 (T)	18.00 (T)	69.78	57.96
L4	1.02 (T)	6.11 (T)	9.51 (T)	18.00 (T)	66.80	55.65
L5	0.43 (T)	1.38 (C)	1.89 (C)	9.82 (C)	28.18	36.96
		4.73 (T)	6.05 (T)	18.00 (T)	30.70	27.91
L6	0.20 (T)	1.85 (C)	3.27 (C)	9.59 (C)	69.27	76.76
		3.34 (T)	4.43(T)	18.00 (T)	34.71	32.63
L7	0.19 (C)	3.40 (C)	5.44 (C)	9.59 (C)	63.55	60.00
		1.75 (T)	2.58 (T)	18.00 (T)	42.78	47.43
L8	0.34 (C)	4.01 (C)	6.09 (C)	12.20 (C)	56.68	51.87
		1.04 (T)	1.47 (T)	18.00 (T)	31.16	41.35
L9	0.89 (C)	5.85 (C)	9.18 (C)	13.15 (C)	67.14	56.92

DL = Dead Load

SW = Self Weight

LL = Live Load (axles)

Appendix C - Photographs of the Smoky River Bridge

Figure C.1 The Smoky River Bridge (Looking West)

Figure C.2 End View of The Smoky River Bridge (Looking West)

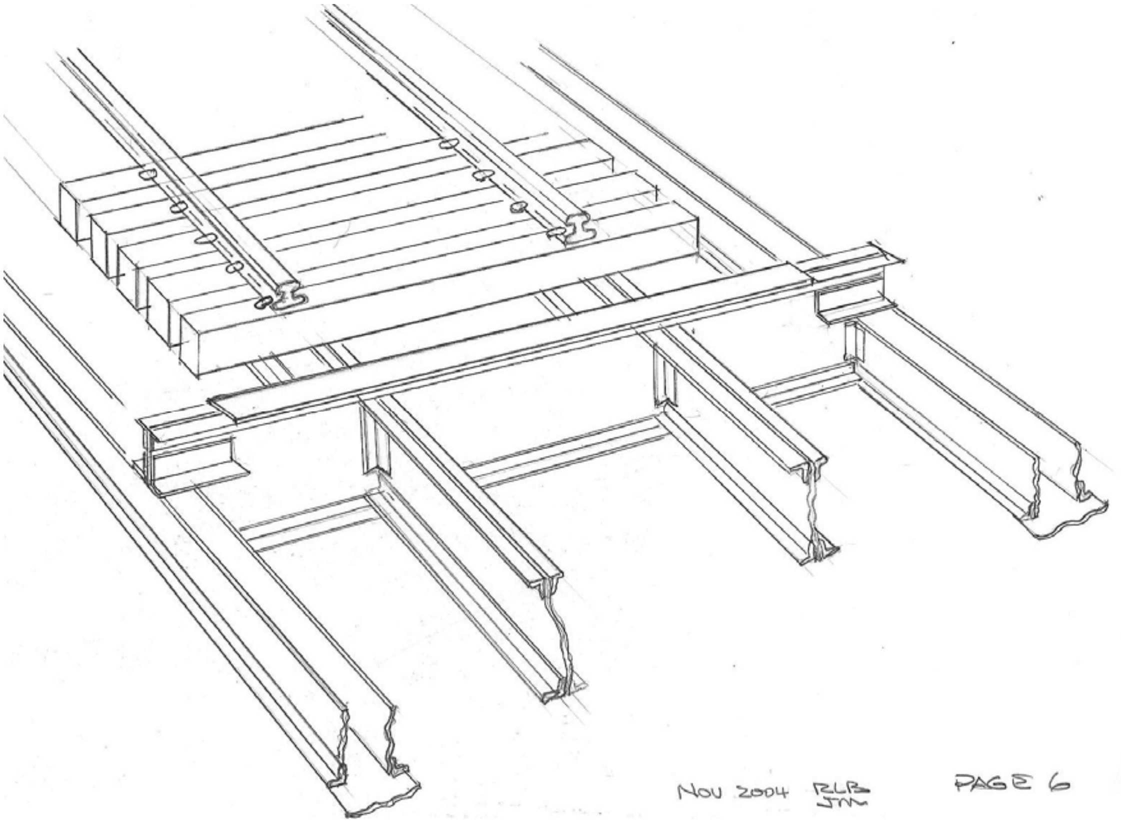
Figure C.3 Typical Floor-Beam to Truss Connection

Figure C.4 Trusses Bearing on Middle Pier

Figure C.5 Underside of Smoky River Bridge and Floor-Beams

Figure C.6 West Pier

Figure C.7 Sketch of Floor-Beam System and Railway



NOTE: All pictures and sketches in Appendix C were provided by Joe Minnick.