



Impact of Bridge Ratings on the Timber Transportation Industry

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16. Abstract The logging industry plays an important role in the State of Wisconsin's economy. Additionally, the condition of some of Wisconsin's bridges, as well as the state of our nation's bridges, is deteriorated due to lack of funding for replacement. As a result of this, bridges are being load posted which prohibits travel over them by many commercial vehicles. One industry in particular that is of interest to this project is the forest products/timber industry, but Wisconsin's dairy industry is also extensively affected. As a result of the load posting, hauling routes are longer due to detours and it is costing the timber industry more money to haul raw timber. The purpose of this project was to investigate the current load ratings of bridges and look at the effects that logging vehicles have on single span bridges. Following these analyses, solutions to help alleviate some of the challenges the timber industry is experiencing due to load posted bridges were examined. Currently, bridges in Wisconsin are load rated based on specified unique design vehicles such as the HS20. This project looked into how the moment and shear effects of logging trucks compared to the effects of common design and State vehicles. Thirty-one logging vehicles with varying configurations and gross weights were measured and used in this comparison analysis. In addition to the logging truck analysis and comparison, several bridges that are of major concern to a prominent timber association in Wisconsin, the Great Lake Timber Professionals Association, were investigated and load rated using the two currently available methods, the Load Factor Rating method and the Load and Resistance Factor Rating method. This was done to better understand the methods used for load rating and to compare the load ratings from this project with current load postings. Lastly, possible solutions to the current issues the Great Lake's Timber Professionals Association and the timber industry are experiencing were investigated. The first solution was looking at optimization of the current logging vehicles including optimizing the distribution of weight on the axles as well as optimizing the axle configuration on the truck to decrease the effects of the trucks on bridges. Additionally, the amount of gross weight reduction for timber trucks that would be necessary to reduce effects to a level equivalent to the design vehicles was calculated. The final solution that was investigated was looking into potential economical bridge strengthening options. The main option that was considered was the use of Mechanically Fastened Fiber Reinforced Polymer Strips as a strengthening option for reinforced concrete bridges. This strengthening technique has been previously utilized successfully in Missouri. Wisconsin bridge B380513, which is load posted and a large concern to the logging industry has been chosen and examined as a candidate for strengthening using this technique.		14. Sponsoring Agency Code	
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**IMPACT OF BRIDGE RATINGS ON THE TIMBER TRANSPORTATION
INDUSTRY**

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CFIRE 03-05: Evaluation of Wisconsin Bridges for Truck Loads

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2009

Abstract

The logging industry plays an important role in the State of Wisconsin's economy. Additionally, the condition of some of Wisconsin's bridges, as well as the state of our nation's bridges, is deteriorated due to lack of funding for replacement. As a result of this, bridges are being load posted which prohibits travel over them by many commercial vehicles. One industry in particular that is of interest to this project is the forest products/timber industry, but Wisconsin's dairy industry is also extensively affected. As a result of the load posting, hauling routes are longer due to detours and it is costing the timber industry more money to haul raw timber. The purpose of this project was to investigate the current load ratings of bridges and look at the effects that logging vehicles have on single span bridges. Following these analyses, solutions to help alleviate some of the challenges the timber industry is experiencing due to load posted bridges were examined.

Currently, bridges in Wisconsin are load rated based on specified unique design vehicles such as the HS20. This project looked into how the moment and shear effects of logging trucks compared to the effects of common design and State vehicles. Thirty-one logging vehicles with varying configurations and gross weights were measured and used in this comparison analysis.

In addition to the logging truck analysis and comparison, several bridges that are of major concern to a prominent timber association in Wisconsin, the Great Lake Timber Professionals Association, were investigated and load rated using the two currently available methods, the Load Factor Rating method and the Load and Resistance Factor Rating method.

This was done to better understand the methods used for load rating and to compare the load ratings from this project with current load postings.

Lastly, possible solutions to the current issues the Great Lake's Timber Professionals Association and the timber industry are experiencing were investigated. The first solution was looking at optimization of the current logging vehicles including optimizing the distribution of weight on the axles as well as optimizing the axle configuration on the truck to decrease the effects of the trucks on bridges. Additionally, the amount of gross weight reduction for timber trucks that would be necessary to reduce effects to a level equivalent to the design vehicles was calculated.

The final solution that was investigated was looking into potential economical bridge strengthening options. The main option that was considered was the use of Mechanically Fastened Fiber Reinforced Polymer Strips as a strengthening option for reinforced concrete bridges. This strengthening technique has been previously utilized successfully in Missouri. Wisconsin bridge B380513, which is load posted and a large concern to the logging industry has been chosen and examined as a candidate for strengthening using this technique.

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1 Introduction

The transportation industry is constantly evolving. New methods for load rating bridges are being developed as part of this evolution. Additionally, the vehicles used to haul materials have greatly increased in both size and weight over the past century. Because of this, the vehicles used to design bridges are being updated and changed so that they match the effects of the actual vehicles on the roads. Simultaneously, the condition of our Nation's infrastructure is deteriorating and this is resulting in load posted bridges. As a result of these load posted bridges, commerce vehicles are being forced to take longer alternate routes which cost the industries both time and money. All of these issues have led to many questions and concerns over accessibility of certain bridges to use by timber hauling vehicles. This research focused on the concerns of the forest products and process industry, an industry which relies on roadways and bridges to transport its raw materials from the forest to the timber mills.

1.1 Problem Statement

As a result of the economic impacts being felt by the timber industry, several questions surfaced. First, what are the actual effects of timber trucks on bridges and how do they compare to the effects of certain specified "design vehicles" which are used to rate bridges. Additionally, how can the effects of load posted bridges in causing detours be reduced for the timber hauling industry and can these posted bridges be strengthened? And do all the posted bridges actually require posting? The purpose of this project was to investigate these questions and to look to improve the situation the logging industry is currently faced with.

Logging trucks in use today are all unique; there is no standard configuration. As a result of this, bridge designers and owners don't know how the effects of logging trucks compare to the effects of "design vehicles". One focus of this project was to determine how the different vehicles compare to one another. Additionally, the timber industry is currently facing an increase in hauling costs due to the load posted bridges in the State of Wisconsin. This project also investigated ways to alleviate the economic strain currently on the timber industry as a result of load posted bridges in Wisconsin. Lastly, there are several bridges in Wisconsin which were never designed to carry the loads of current logging and other commerce vehicles. Due to this issue, this project also investigated a cost effective bridge strengthening technique which has the potential to increase the load carrying capacity of some bridges.

1.2 Project Objectives

In solving the problems defined above, three primary paths were used in this project. The first was to look at the impact current logging trucks have on single span bridges of various lengths and to compare these effects to the effects of nominal design vehicles that are used to rate the capacity of bridges. Design vehicles are the vehicles used in design of a new structure and are not real vehicles but rather vehicles created to represent the loading effects of real vehicles. In order to complete this comparison analysis, logging vehicles used in Wisconsin needed to be weighed and measured, then the effects of the vehicles on bridges would be calculated.

The second major objective was to compare results from the two bridge load rating methods currently being used. Templates that could be used to determine load ratings of

bridges for both methods would be created using MathCAD. These templates could then be used to load rate seven bridges in Wisconsin that are of primary concern to the Great Lakes Timber Professionals Association. The templates may be used in the future to provide bridge owners an alternative to load rate their bridges on their own.

The final major objective of this project was to investigate solutions to improve the current situation the logging industry is facing as a result of load posted bridges that do not have sufficient capacity to carry timber trucks. This will be solved through two main avenues. First, the current logging vehicles will be examined and optimized based on axle configuration and weight distribution. Secondly, options for strengthening load posted or deteriorating bridges will be researched.

2 Statement of Scope

The scope of this research was defined relative to the three objectives. The first is the analysis of logging truck effects on bridges. Though a series of logging trucks were actually measured, the analysis of effects on bridges will be constrained to three vehicles which represent the average of the measured trucks. These vehicles were an average of the weight distribution and the axle configuration of the trucks measured in the field. A limited analysis was also done using each individual measured logging truck. Only single spans were considered for these analyses. As part of a recent Wisconsin Truck and Weight Study other configurations similar to actual vehicles were also created (Cambridge Systematics, Inc., 2009). The effects of those vehicles were also analyzed. Two of these vehicles had similar configurations to those of the logging vehicles measured. A comparison using these vehicles was completed to check how well the proposed configurations demonstrated the effects of the logging vehicles. The logging truck analysis and comparison can be found in Chapter 6.

The second focus of this research was on the current load rating of bridges as well as the methods used in load rating. Rather than examining bridges throughout the State, the bridges in just two counties, which have a prevalent timber industry, were investigated to determine the percentage of bridges which have a load rating less than the current design level of HS20. The scope of this investigation was limited to two counties; Lincoln and Marathon. Additionally, only seven bridges that are of concern to the logging industry were load rated using both the Load Factor Rating method and the Load and Resistance Factor Rating. Six of these bridges were steel rolled W-shapes bridges and one of the bridges was a reinforced concrete T-beam bridge. The two load rating methods are both currently allowed for determining the strength and service capacity of bridge structures. Information on the

seven bridges chosen for load rating can be found in Chapter 8. All information regarding load rating methods and results can be found in Chapter 10.

The final objective was to look at solutions that could improve bridge capacity and reduce the logging industry's hauling routes. In this study, two solutions were studied. One option included optimizing current logging vehicles which could reduce the effects of the trucks, possibly allowing them to travel over bridges that the current configurations are restricted from. A second option was cost effective strengthening of load posted bridges which could potentially remove postings, again allowing for truck traffic to travel over bridges that are currently not allowed. For the logging truck optimization, only the average five-axle vehicle was examined. The logging truck optimization includes axle configuration and weight distribution analyses and focuses on reducing the effects the vehicles have on single span bridges. The research and results of logging vehicle optimization can be found in Chapter 11. Finally, bridge strengthening options were investigated. The use of Mechanically Fastened Fiber Reinforced Polymer (MF-FRP) strips on concrete girders is the main method examined. Information regarding this strengthening process can be found in Chapter 12.

3 Project Background

In 2007, the forest products and processing industry employed 84,818 people in the State of Wisconsin and had an industry output of 26.2 billion dollars. This industry output, which is the gross income for the timber industry, is equal to 5.4% of Wisconsin's statewide industrial output (Wisconsin Department of Natural Resources; Division of Forestry Staff, 2009). Based on these figures, it is clear that the forest products industry is a very important contributor to the State of Wisconsin's economy. Currently, an issue this industry has been faced with is that haul routes to transport timber have been eliminated or lengthened due to load posting of bridges. This has hindered the industry because longer haul routes mean an increase in both haul time and cost to deliver timber. Estimated costs incurred as a result of these posting will be discussed later.

3.1 Load Limits: Federal Bridge Gross Weight Formula

In the 1950's and 1960's the overall gross weight of vehicles and how the weight was distributed began to vary and it was decided limits were needed to protect bridges from overload. To do this, the Federal Bridge Gross Weight Formula was enacted by congress in 1975 (Federal Highway Administration, 2006) and limits the gross truck weight allowed based on the number and spacing of axles. One of the primary concerns and focus of this equation is axle spacing. The figure below shows how the axle spacing of a truck can play a large role in the effects on a bridge. Clearly a longer truck, or wider spacing of axle, is likely to place less load or have less of an effect on a bridge.

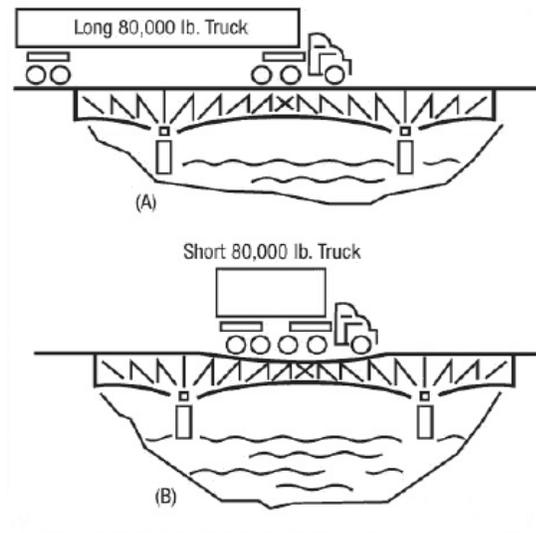


Figure 3-1: Illustrated comparison of bridge effects based on truck axle spacing

In addition to the gross weight formula, trucks must comply with the requirements which limit one axle to 20,000 lbs, a tandem axle (i.e.- set of two axles closely spaced) to 34,000 lbs. and the overall gross weight of the vehicle to 80,000 lbs.

The formula for the maximum allowable weight is based on:

L=the spacing in feet between the outer axles of any two or more consecutive axles

N= number of axle being considered

The federal bridge gross weight formula can be found below:

$$W = 500 * \left(\frac{L * N}{N - 1} \right) + 12 * N + 36 \quad (1)$$

(Federal Highway Administration, 2006)

This equation is used to determine or check whether a truck configuration is acceptable. In addition to the formula, tables have also been prepared that allow for users to find allowable weight based on L and N. One thing to note about this formula is that it has to be checked for all possible configurations on the vehicle. For example, a simple five-axle tractor trailer vehicle with a front steering axle and two tandem axles would require three

checks. The first check would use an L equal to the distance between the steering axle and the second wheel of the first tandem axle. The second check would be for the overall length of the vehicle. The final check would be from the second axle to the fifth axle. In summary, the first check would be for three axles, the second check for five-axles, and the final check would be for four axles.

Figure 3-2 shows the lengths, L and axles that are used for each of the three checks.

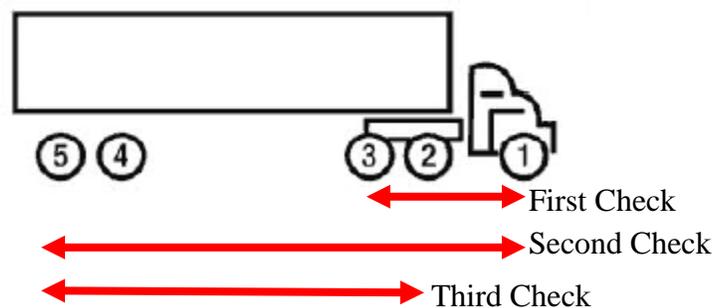


Figure 3-2: Federal Bridge Gross Weight Formula Example Vehicle

All trucks on U.S. highway bridges must comply with the formula with exception of a few states which have grandfathered laws written previously to the 1975 law.

3.2 Minnesota Truck Size and Weight Study Project

In addition to the Federal Limits on truck configurations, individual States also create limits. Wisconsin and Minnesota have both recently completed a truck size and weight study. These studies look at the current truck size and weight laws and then look at and assess changes that might be beneficial to all parties involved. In addition to creating new vehicle configurations which might be added as acceptable configurations set by the state, both studies also looked at how states within the Midwest differ and compare in terms of the truck size and weight laws enforced. These studies helped see where the future of the truck

size and weight laws may be going and where the logging configurations may fit into this future.

The Minnesota Truck and Weight study was completed in 2006. (Cambridge Systematics, Inc., 2006) The purpose of the study was to investigate Minnesota's current truck size and weight (TS&W) laws and to look into changes to the laws which would be beneficial to the State's economy as well as keeping safety and the roadway infrastructure in mind.

One of the first steps in this study was to hold public meetings to identify issues and concerns about their current TS&W laws. A main issue for Minnesota was that surrounding states and Canada allow for heavier vehicles in many situations. This has a negative impact on Minnesota hauling because the cost to haul is higher for the amount of goods due to weight restrictions. This issue is prevalent near the border of Minnesota and the Dakotas where there is a large sugar beet industry. North and South Dakota have maximum allowed gross vehicle weights of 105,500 lbs. and 129,000 lbs respectively compared to Minnesota's 80,000 lbs. Another issue was that the current laws are very complex. This can make them difficult for drivers to comply with due to a lack of understanding. In addition to issues with the complex current law system, there were also many concerns with changes being made from the bridge owner point of view. The main concern was the potential impacts on infrastructure if weight increases were allowed. The need for more investment into infrastructure was also voiced.

The three main considerations in this study were the pavement, bridges, and highway safety. One of the key aspects for bridges included the additional cost for bridge owners to inspect, load rate and possibly load post bridges if an increase in gross weight was

implemented. Additionally it was noted that bridges are heavily dependent not just on the gross weight of a vehicle but the axle spacing and the weight distribution.

In addition to considerations regarding the highways, bridges, and safety of those on the roadways, the regulations set by the Federal Highway Administration (FHWA) also required compliance. These limits for length, width and weight were defined in the 1982 Surface Transportation Assistance Act (STAA). Table 3-1 and below summarizes the requirements. Figure 3-3 and Figure 3-4 also show the trailer length requirements.

Table 3-1: Federal Limits defined in the 1982 Surface Transportation Assistance Act (STAA)

Length	48ft minimum trailers for tractor trailer combinations
	28ft minimum for any additional trailer in either tractor trailer or trailer-trailer combination
Width	Maximum width of 102in.
Weight	80,000 lbs. maximum gross vehicle weight (GVW)*
	20,000 lbs. for single axle
	34,000 lbs. for tandem axle
*Applicable to the Interstate System	

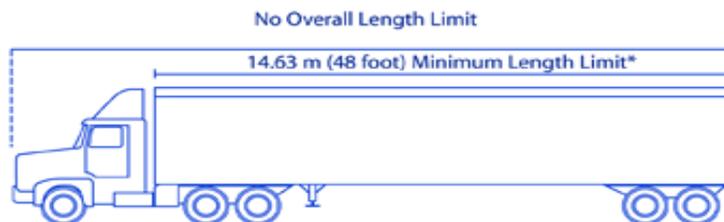


Figure 3-3: Minimum Length Requirements for Trailer in Tractor-Trailer Combinations(Federal Highway Administration, 2004)

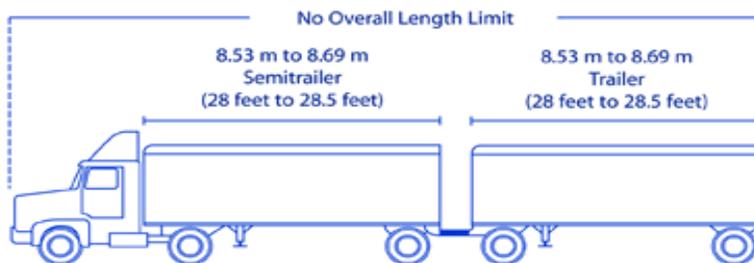


Figure 3-4: Minimum Length Requirements for additional Trailers in Tractor-Trailer Combinations (Federal Highway Administration, 2004)

Before the Minnesota Truck Size and Weight study was completed, the Minnesota weight laws were based on tonnage networks. A tonnage network sets limitations for maximum gross vehicle weight (GVW) as well as maximum single axle weight for a given roadway. Designated highway limits are based on the 10-Ton network, which allows 80,000 lbs. for GVW with five-axles or more and a single axle maximum weight of 20,000 lbs. There are similar 9-Ton, 7-Ton and 5-Ton networks for non-designated highways.

Several comparisons with surrounding states and provinces in Canada were done within the study. The comparison of primary interest to this project was the maximum allowable gross weight comparisons among entities.

Table 3-2: Summary of Truck Weights in Minnesota and Neighboring States and Provinces (Cambridge Systematics, Inc., 2006)

Weights		Minnesota	Iowa	Michigan	North Dakota	South Dakota	Wisconsin	Federal	Manitoba ^a	Ontario
GVW Interstate	5-Axle Vehicle	80,000	80,000	80,000	80,000	80,000	80,000	80,000	87,083	97,224 ^b
	6-Axle Vehicle	80,000	80,000	101,400 ^c	100,000	88,000	80,000		102,515	111,333 ^b
Other State Highways	5-Axle Vehicle	80,000 ^d	80,000 ^e	87,400	88,000	81,200	80,000	80,000	82,673	97,224 ^b
	6-Axle Vehicle	80,000 ^d	80,000 ^f	101,400 ^c	105,500	88,700 ^g	80,000		98,106	111,333 ^b
Axle Weights	Single-Axle Weight	20,000	20,000	20,000	20,000	20,000	20,000	20,000	20,062	22,000
	Tandem (2-Axle) Weight	34,000	34,000	34,000	34,000	34,000	34,000	34,000	37,478	39,700
	Tridem (3-Axle) Weight	42,000	42,000	42,000	48,000	42,000	42,000	See ^h	52,911	57,320 ⁱ
Routine Maximum Permit ^j	Gross Vehicle Weight	92k/144k	100k/160k	80k/164k	103k/136k	116k ^k	110k/191k	N/A	137,788 ^l	139,994 ^l
	Single Axle	20,000	20,000	13,000	20,000	31,000	20,000		20,062 ^l	20,062 ^l
	Double Axle	40,000	40,000	26,000	45,000	52,000	60,000		37,478 ^l	37,478 ^l
Seasonal Limits	Spring Load Restrictions	Yes	Yes ^m	Yes ^m	Yes ^m	Yes	Yes	Yes	Yes	Yes
	Winter Weight Increase	Yes 88,000	No	No	Yes 10 percent	No	No	N/A	Yes	No

- Notes:
- ^a Manitoba weight limits depend on the highway classification, axle spacing, and configuration type; limits here are shown for five- and six-axle tractor and semitrailer combinations on the Roads and Transportation Association of Canada (RTAC) System (roughly Interstate equivalent) and A-1 system (roughly equivalent to "Other State Highways"). Lower limits exist for the B-1 Provincial system. Maximum axle weights shown are for the RTAC System.
 - ^b GVW limits in Ontario depend on axle spacings; the values in the table reflect the generally applicable limits. The maximum limits are 105,600 pounds GVW for five axles and 123,860 pounds GVW for six axles.
 - ^c Based on axle spacing and tire size. Vehicles with 11 axles and proper axle spacings of 164,000 pounds GVW maximum.
 - ^d On the 9-ton system, five-axle combinations may operate with a maximum of 73,280 pounds GVW and six axles up to 80,000 pounds GVW. Lower limits apply to the 7- and 5-ton systems.
 - ^e On non-Interstates, five-axle livestock trucks with spread-axle trailers are allowed 86,000 pounds GVW.
 - ^f Construction and livestock vehicles up to 96,000 pounds.
 - ^g Maximum GVW is controlled by Federal Bridge Formula (maximum practical GVW is 129,000 pounds).
 - ^h There is no set maximum specified in Federal regulations for tridem axles, but the Federal Bridge Formula allows for tridem axle weights between 34,000 and 60,000 pounds depending on wheel spacing of the axle group.
 - ⁱ Effective January 2006.
 - ^j Permits issued regularly without special conditions and include widely used configurations for specific industries.
 - ^k Five-axle routine permit value is estimated assuming two, 52,000-pound tandem groups, and 12,000-pound steer axle. Ad hoc determination based on 600 pounds per inch width of tire and proper axle spacings.
 - ^l Determination on a case-by-case basis.
 - ^m In Iowa, Michigan, North Dakota, and South Dakota few state roads are posted with Spring Load Restrictions.

Table 3-2 shows the comparisons of allowed weights for six states, two Canadian provinces as well as the federal weight limits. One thing to note is that this study was completed before a State resolution to allow six-axle 98,000-lb vehicles in Wisconsin was passed. Comparing Wisconsin to its neighboring states of Minnesota, Michigan, and Iowa, it is evident that Michigan allows the most weight for the different axle configurations. Wisconsin, Iowa, and Minnesota all have very similar regulations except for gross vehicle permit weight. Wisconsin has the highest allowable weights for the permit weights at 110k/191k followed by Michigan, Iowa and Minnesota, which has the lowest routine permit weights.

With this background information known, the next step in the study was to research and propose changes to the current TS&W laws including new vehicle configurations. One of the considerations was to see how many additional bridges in the state would be load posted as a result of each proposed vehicle configuration. This means how many bridges would have to be posted because they would be unable to carry this load without risking permanent damage to the bridge. A unit cost associated with the cost to load rate, post and maintain the bridge for these new loads was determined. Additionally, costs were linked to the savings the trucks would gain from hauling a larger load. Other factors also had costs associated with them and in the end, a cost analysis was done to see if the proposed new vehicle limit would have an overall positive or negative benefits to the state.

As a result of the Minnesota study, four proposed vehicle configurations were created. These vehicles were found to be feasible and to have overall statewide benefits. They include the following configurations: six-axle 90,000-lb semi, seven-axle 97,000-lb

semi, eight-axle twin 108,000-lb and a Single Unit (SU) up to 80,000 lbs. Table 3-3 summarizes each vehicle that was proposed as a result of the Minnesota Truck Size and Weight Study.

Table 3-3: Minnesota Truck Size and Weight Study Proposed Vehicle Configurations (Cambridge Systematics, Inc., 2006)

Proposed Vehicle Configurations^a	
6-Axle 90,000 lb. GVW on Non-Interstate 10-ton Network	7-Axle 97,000 lb. GVW on Non-Interstate 10-ton Network
<ul style="list-style-type: none"> • Must meet bridge formula, axle, and tire weight limits • 53 ft. maximum trailer length (no change) • 99,000 lb. GVW winter and seasonal increases; no further tolerances or exemptions • Allowed on 10,000-mile 10-ton Network (not on Interstates) • Requirements: permits with fees; axles to be added by certified remanufacturer; brakes required on every wheel 	<ul style="list-style-type: none"> • Must meet bridge formula, axle, and tire weight limits • 53 ft. maximum trailer length (no change) • 99,000 lb. winter and seasonal increases; no further tolerances or exemptions • Allowed on 10,000-mile 10-ton Network (not on Interstates) • Requirements: permits with fees; axles to be added by certified remanufacturer; brakes required on every wheel
8-Axle 108,000 lb. Twin Trailer on Non-Interstate MN Twin Trailer Network and National Truck Network	80,000 lb. GVW Single Unit (SU) Truck on 10-ton Network (including Interstate)
<ul style="list-style-type: none"> • Must meet bridge formula, axle, and tire weight limits • 28.5 ft. each maximum trailer length (no change) • Allowed on pre-approved state trunk highway routes only (approximately 6,700 miles) • No harvest or winter increases; no tolerances or exemptions • Requirements: permits with fees; B-train coupling; axles to be added by certified remanufacturer; brakes required on every wheel; driver CDL endorsement required for double trailer operation 	<ul style="list-style-type: none"> • Must meet bridge formula, axle, and tire weight limits • Vehicle length increase up to 45 ft. max (from current 40 ft.) • Lift axles must be down with loads • Axles in excess of 4 must be self-steering castering wheels • Requirements: permits with fees; axles to be added by certified remanufacturer; breaks required on every wheel

3.3 Wisconsin Legislature: 2007-2008 Resolution to increase gross weight limit of Six-axle vehicles to 98,000 lbs.

A Wisconsin statute was passed in 2007/2008 which allowed six-axle vehicles carrying “raw forest products” to have a maximum gross weight which does not exceed the gross weight limitations by more than 18,000 lbs. (Wisconsin State Legislature , 2007). The gross weight limitation without a permit for a six-axle vehicle is 80,000lb so this statute allows six-axle timber vehicles to carry a maximum load of 98,000 lbs. with a permit. In addition to the gross weight limitations, the statutes also state that a single axle can’t exceed a weight of 18,000 lbs. and every axle must carry at least eight percent of the gross vehicle weight.

The main flaw in this statute was that there is nothing included in regards to axle configuration or minimum truck length. This means that any six-axle vehicle, regardless of whether the axles are spaced 4ft on center or 15ft on center can carry the 98,000 lbs. It is clear that a truck with axles closely spaced will have a much larger impact on a bridge. As a result, the Great Lakes Timber Professionals Association noted that many more bridges were load posted after the resolution went into effect. Some bridges may have been posted due to a concern that trucks with close axle spacings would induce damage and others because funds were not available to conduct an accurate load rating study to check the bridge capacity.

3.4 Wisconsin Truck Size and Weight Study

The overall goals of the recent Wisconsin Truck Size and weight (TSW) study were the same as Minnesota’s TSW study: to assess potential changes in Wisconsin’s TSW laws that would benefit the Wisconsin economy while protecting the roadway and bridge infrastructure and maintaining safety (Cambridge Systematics, Inc., 2009). According to a

survey in 2002, 74 percent of total Wisconsin freight is carried by truck, which reinforces how important the trucking industry is to Wisconsin.

The three considerations for this study were pavement, bridges, and safety. One of the main concerns about bridges is the capacity and the permanent effects that a bridge can experience as a result of increased loads. As a rule of thumb, most bridges designed after the late 1970's were designed using the AASHTO Load Factor Design (LFD) standards and should be able to withstand the loads of the proposed trucks within this study. The most recent standards are the AASHTO Load Resistance Factored Design (LRFD) and bridges designed to these standards should also be able to carry the new proposed loads. The main conflicting issue with these is that a vast majority of bridges were designed and built before this time frame which could propose challenges. The estimated costs to load rate and post bridges if necessary as a result of proposed loads was included in the estimate of economic effects within this study. Lastly, the gross weight, axle spacing, and number of axles are all important parameters when looking at stresses caused on bridges by vehicles. Therefore, the Wisconsin study also emphasized the importance of vehicles adhering to the Federal Bridge Gross Weight formula.

The study was divided into three main activities: outreach, research and analysis. First, outreach was done through workshops and other avenues to find out what types of changes the private sector and public agencies who work with the truck size and weight laws were looking for. The results were similar to those of the Minnesota TSW study. The public sector was concerned with evaluating the safety of bridges and the enforcement of current laws. From the private sector the concerns were about having consistency of allowable loads

from state to state as well as looking at finding more productive vehicle configurations which can accommodate heavier loads.

The research phase of the project focused on the current TSW laws and the TSW laws in surrounding states. It also looked at other things which might affect future laws such as the trends of truck technology.

Lastly, the outreach and research led to options of new truck configurations and policy changes. These changes were analyzed to see both the benefits and drawbacks of each configuration.

As a result of this study six candidate vehicles were created and a cost analysis was completed. Four of these truck configurations were acceptable based on the Federal Gross Weight Bridge Formula and two of the vehicles were not. Below are figures showing the configurations for each of the vehicles.

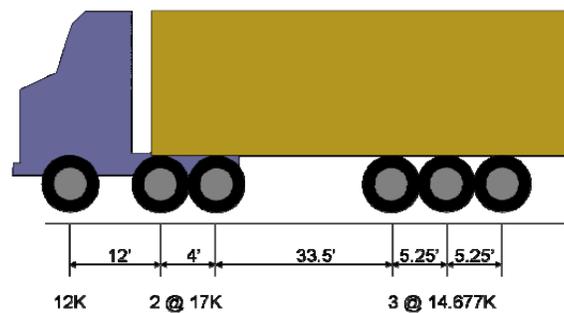


Figure 3-5: Six-axle 90,000lb Tractor-semitrailer (6aTST 90)(Cambridge Systematics, Inc., 2009)

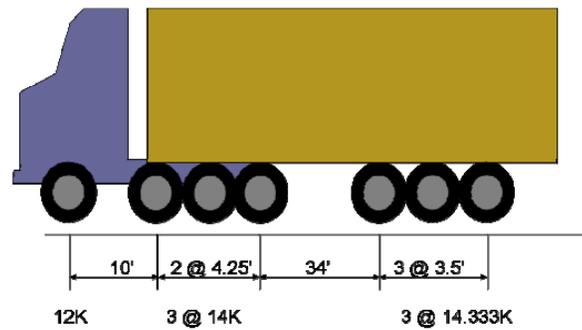


Figure 3-6: Seven Axle 97,000lb. Tractor Semi-trailer (7a TST 97)(Cambridge Systematics, Inc., 2009)

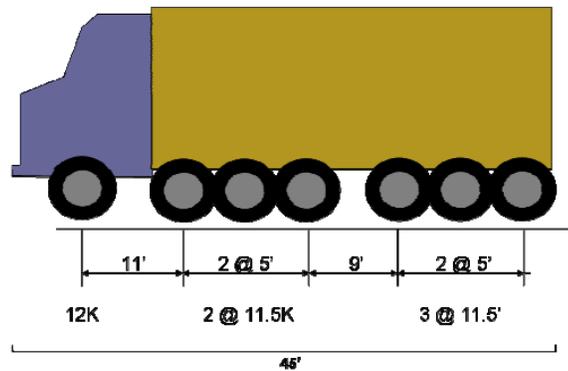


Figure 3-7: Seven Axle 80,000lb Single Unit Truck (7a SU 80)(Cambridge Systematics, Inc., 2009)

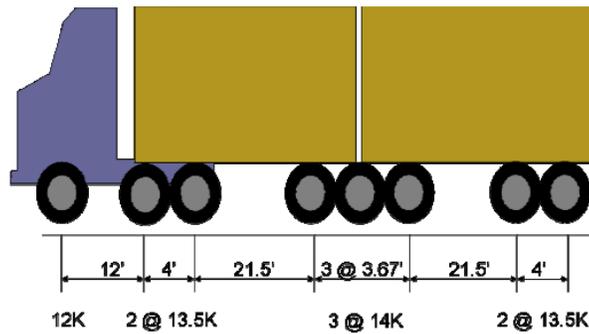


Figure 3-8: Eight-Axle 108,000lb Double Truck (8a D 108)(Cambridge Systematics, Inc., 2009)

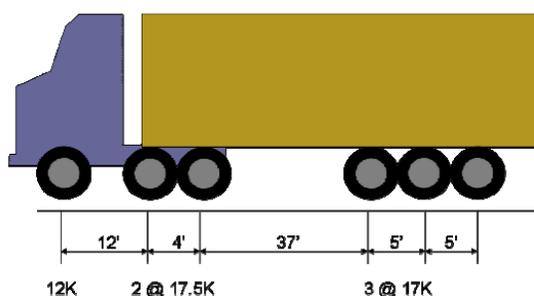


Figure 3-9: Six-axle 98,000lb Tractor Semi-trailer (6a TST 98)(Cambridge Systematics, Inc., 2009)

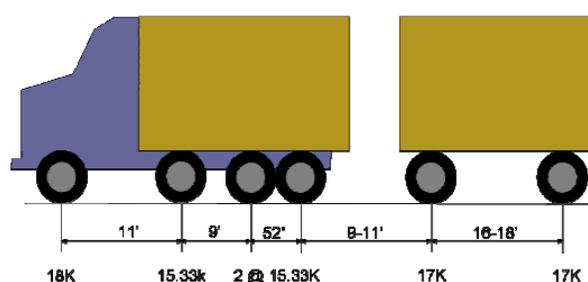


Figure 3-10: Six-Axle 98,000lb Straight Truck plus Pup Trailer (6a STT 98)(Cambridge Systematics, Inc., 2009)

Five of the six candidate vehicles yielded positive net benefits based on the cost analysis for non-highway operation. These are roads not classified as highways or interstates. The cost analysis was divided up into five sections: transport costs savings, safety costs savings, congestion cost savings, pavement cost savings, and bridge costs. Within bridge costs there were two sections. One section was for the cost associated with bridge repair and replacement as a result of the TSW study proposed configurations. The second costs reflected the existing bridge needs in the State. This cost is estimated at \$55.5 million per year. These costs are all based on average bridge types and actual costs could be much higher. Table 3-4 shows the cost analysis for each vehicle for operation on non-highway roads.

**Table 3-4: Annual Costs and Benefits for Candidate Configurations for non-highway operation
(Cambridge Systematics, Inc., 2009)**

Fed Bridge Formula	Configuration	System User Benefits			Public Agency Benefits and Impacts			Net Benefits	
		Transport Savings	Safety	Congestion	Pavement	Bridge Costs for TSW Configs	Baseline Bridge Costs	With TSW Bridge Costs Only	With All Bridge Costs
Y	Base Case	0.00	0.00	0.00	0.00	0.00	(55.50)	0.00	(55.50)
Y	6a TST 90	5.50	0.46	0.92	2.57	(2.18)	(55.50)	7.26	(48.24)
Y	7a TST 97	6.27	0.70	0.85	3.87	(3.08)	(55.50)	8.62	(46.88)
Y	7a SU 80	2.46	0.11	0.08	0.40	(2.26)	(55.50)	0.78	(54.72)
Y	8a D 108	3.42	0.46	0.49	3.34	(6.02)	(55.50)	1.69	(53.81)
N	6a TST 98	19.19	1.52	1.89	1.10	(8.48)	(55.50)	15.23	(40.27)
N	6a STT 98	2.19	0.09	0.06	0.03	(4.22)	(55.50)	(1.85)	(57.35)

Note: All values in millions (assumes non-Interstate highway operation only).

The study also looked at the use of the candidate vehicles on interstates or highway operations. These benefits were much higher when the candidate vehicles were allowed on highways however the costs associated with the proposed vehicles were also higher. Based on the cost analysis, three of the candidate vehicles were found to have positive net gain. These vehicles were the six-axle 90,000-lb tractor semi-trailer, the seven-axle 97,000-lb tractor semi-trailer, and the six-axle 98,000-lb tractor semi-trailer. Table 3-5 shows the benefits and cost analysis for the candidate vehicles if allowed on the interstate.

Table 3-5: Annual Cost and Benefits for Candidate Configurations including highway operation

Fed Bridge Formula	Configuration	System User Benefits			Public Agency Benefits and Impacts			Net Benefits	
		Transport Savings	Safety	Congestion	Pavement	Bridge Costs for TSW Configs	Baseline Bridge Costs	With TSW Bridge Costs Only	With All Bridge Costs
Y	Base Case	0	0	0	0	0.00	(55.50)	0.00	(55.50)
Y	6a TST 90	36.64	3.48	3.44	14.65	(2.18)	(55.50)	56.03	0.53
Y	7a TST 97	41.83	4.43	4.08	19.91	(3.08)	(55.50)	67.18	11.68
Y	7a SU 80	9.83	0.53	0.09	1.53	(2.26)	(55.50)	9.73	(45.77)
Y	8a D 108	22.77	2.90	1.65	16.76	(6.02)	(55.50)	38.06	(17.44)
N	6a TST 98	127.94	9.40	11.03	10.19	(8.48)	(55.50)	150.09	94.59
N	6a STT 98	14.61	0.68	0.26	0.32	(4.22)	(55.50)	11.65	(43.85)

Note: All values in millions (assumes Interstate highway and non-Interstate highway operation).

Some notable final results are that all the proposed vehicles have net benefits if the existing bridge costs are not considered and if all the costs including bridge costs are taken into account and the vehicles are allowed to travel on the interstate, the most beneficial vehicle is the six-axle 98,000-lb tractor semi-trailer. The only downside to this vehicle is that it does not meet the federal bridge gross weight formula, although this is a commonly used vehicle and is currently allowed under an exception in the Wisconsin law. Several other analyses were completed and conclusions were made in terms of policy of the TSW laws, however, they were not applicable to this project.

The two six-axle 98,000-lb candidate vehicles provide the closest match with the logging vehicles that were measured as a part of this research project. These two candidate vehicles are compared to the logging truck configurations they most closely resemble in Section 6.7.

3.5 Economic Impact on Timber Industry in Wisconsin

As stated previously, load posted bridges are currently affecting the forest products and processing industry. The posted bridges have led to longer haul routes for operators, which leads to higher costs to deliver timber. Two estimates were provided as to the extent of the damage load posted bridges are having on the forest products and processing industry.

The first estimate was provided by the PCA Mill in Tomahawk, WI. More than three hundred producers deliver approximately 700,000 tons of raw timber to the mill annually. The PCA mill estimated that about 75% of the timber coming into the mill is affected by load posted bridges and that on average the additional haul distance incurred is ten miles. Based on this data an additional haul cost is estimated at \$451,000 dollars. This is based on a haul rate of \$0.086/mile/ton.

The second estimate provided was for the cost incurred state wide due to load posted bridges in the State of Wisconsin. This estimate was provided by the Great Lakes Timber Professionals Association. State wide, 1.6 million cords are produced and transported annually, which is equal to 3,840,000 tons. It was conservatively estimated that approximately 30% of all timber transported across the state is affected by bridge postings. The average additional haul distance due to the bridge posting was estimated at 10 miles. The average haul rate for the state was estimated at \$0.095/mile/ton. This equates to a total of \$1,094,400 additional hauling costs. Table 3-6 summarizes both of the estimates provided.

Table 3-6: Two Estimates of Economic Impact of Load Posted Bridges on the Forest Products and Processing Industry

	PCA Mill: Tomahawk, WI	State Wide
Raw Timber Produced	700000 ton	1.6 million cords
		3,840,000 tons
Extra Haul Route due to Bridge Posting	10 miles	10-15 miles
Amount of Timber Effected by Postings	75%	30% (conservative)
Haul Rate	\$.086/mile/ton	\$0.095/mile/ton
Total Cost (10 mile detour)	\$451,500	\$1,094,400.00

3.6 History of Bridge Design Vehicles

Many bridges still in use today were designed before the current standard AASHTO design vehicle was created. The State of Wisconsin has approximately 13,600 bridges currently in service, some of which were originally built as early as 1880 (Wisconsin Department of Transportation, 2003). Bridges are classified as structures with a span of 20ft or larger. Looking at the country as a whole, there are just under 600,000 bridges currently in the National Bridge Inventory database. Understanding the history of the design vehicles used in bridge design provided a better idea as to what loads some of the older bridges in Wisconsin, and the rest of the country, were originally designed to withstand.

The earliest recorded mention of bridge design live loads was found in the American Society of Civil Engineer's (ASCE) Proceeding Volume L, printed in 1924 (American Society of Civil Engineers, 1924). Included in the proceedings was a section titled "Specifications for Steel Highway Bridge Superstructure". This section classified bridges based on traffic loads and assigned design live loads to each type of classification. The table

below describes each type of bridge classification and also the truck live load associated with it. The class D classification was created for bridges carrying electric traffic in addition to highway traffic. It is presumed that the class D electric traffic loads were referring to electric rail cars or trolley lines.

Table 3-7: Classification of Bridges per ASCE 1924 Proceedings

Classification	Live Load	Description
Class A	H-20	City Bridges or other bridges carrying a highway traffic of exceptionally heavy load units
Class B	H-15	Bridges on Primary Roads
Class C	H-13	Bridges on Secondary Roads
Class D		Bridges carrying electric traffic in addition to highway traffic

Three vehicles were specified in the 1924 ASCE Proceedings. These are listed in under the “live loads” category. The H designates that is it a typical truck loading and the number following the H is the total weight of the vehicle in tons. Each of these vehicles had two loadings; one for the floor systems and one for girders or trusses. The floor system loading was to design the deck and floor beams of the bridge and was the two axle truck associated with the bridge classification with the loads distributed as shown in the figure below.

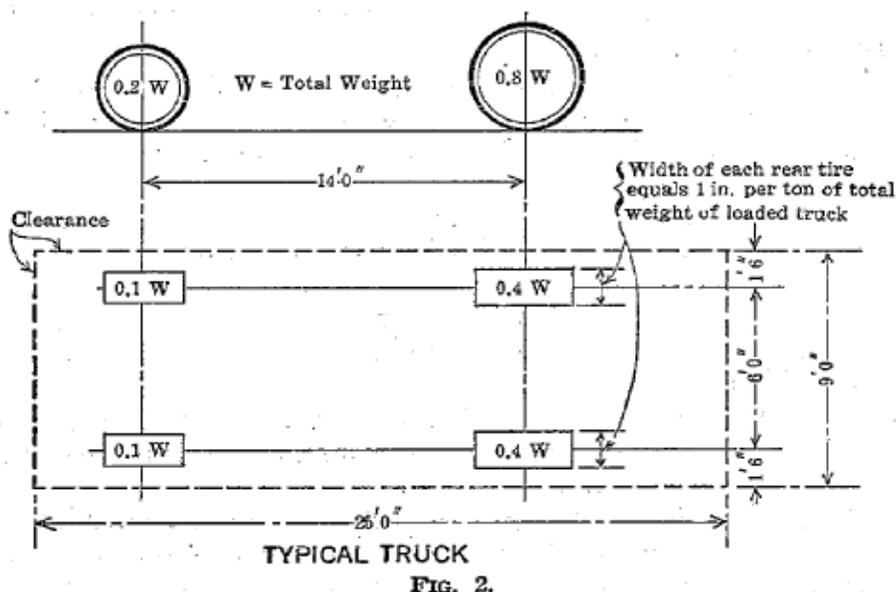


Figure 3-0-11: Typical Truck Distribution from ASCE 1924 Proceedings

The design loading for girders and trusses consisted of a unique uniform load and a concentrated load for each type of vehicle. The table below summarizes the three vehicles and their designated loads for girders or trusses. Design of floor systems was done using the distribution in Figure 3-11 with the weight in tons equal to the value following the H designation.

Table 3-8: Summary of 1924 ASCE Proceedings Design Vehicles

Design Vehicle	Gross Weight	Girders/Trusses	
		Uniform Load	Concentrated Load
H-20	20 ton	600lb/ft	28,000lb
H-15	15 ton	450lb/ft	21,000lb
H-13	13 ton	390lb/ft	18,200lb

Following ASCE's 1924 proceedings, the first edition of American Association of State Highway Officials (AASHO) bridge design specifications, which today is known as AASHTO (American Association of State Highway and Transportation officials), was published in 1931 (Kulicki & Mertz, 2006). Basic design trucks were incorporated in this

document, which included the H-20, a 20ton (40kip) single unit vehicle. Other lighter vehicles, such as the H-15 were also included. The H-15 had a gross weight equal to seventy five percent of the H-20. During that time period, these single unit trucks were combined together as a 'train truck' on the bridge. These train truck loadings were officially added to the 1935 AASHTO standard and were known as the H-20-35 and the H-15-35. The 35 designates the year, 1935 and the number following the H represents the heaviest truck used in the train truck combination. For example, the heaviest truck used in the H-20-35 train loading would be the 20 ton H-20 vehicle (Tonias, 1995). Figure 3-2 shows the official AASHTO 1935 train truck loadings.

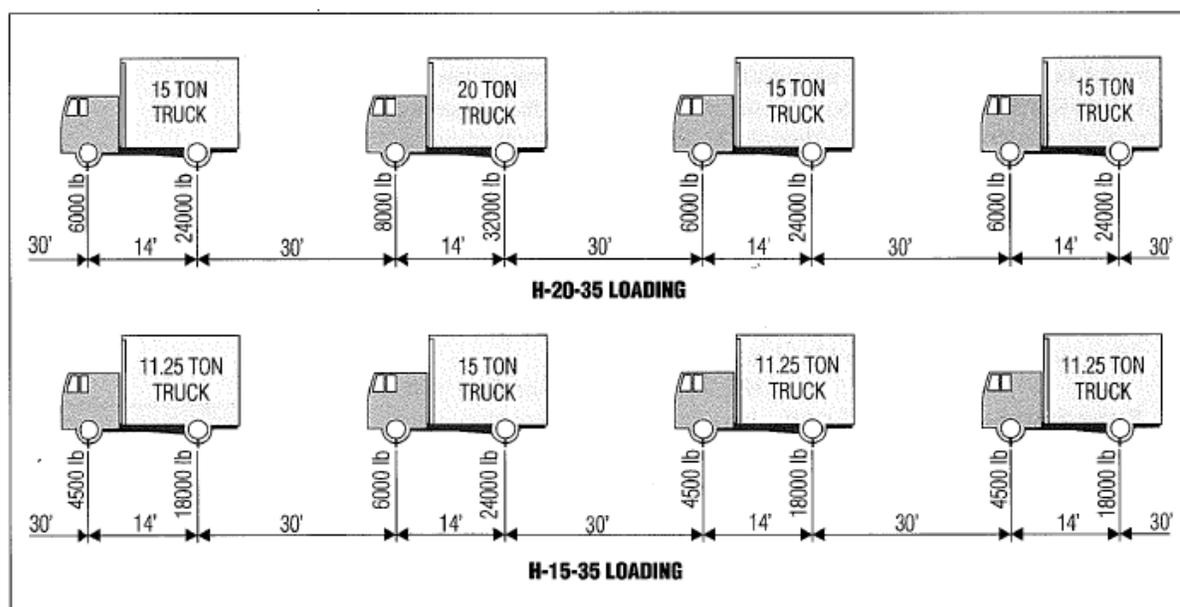


Figure 3-2: AASHTO 1935 Train Truck Loadings (Tonias, 1995)

Up until 1944, all design trucks were single unit, two axle vehicles. At that time gross weights of truck traffic were increasing, which in turn meant larger loads on bridges. In order to meet these demands, tractor trailer design vehicles were developed. Two new

trucks were created in 1944. Table 3-9 lists the five design vehicles included in the 1944 AASHTO specifications (Tonias, 1995).

Table 3-9: Design Vehicles included in the 1944 AASHTO Bridge Specifications (Tonias, 1995)

Vehicle	Gross Weight
H10-44	20,000lb
H15-44	30,000lb
H20-44	40,000lb
HS15-44	54,000lb
HS20-44	72,000lb

The naming system for the two new trucks is similar to the trucks added in the 1935 specifications. The 44 represents the year the trucks were added to the specification. The S designates that the vehicle is a semi tractor trailer and has more than 2 axles. The number following the H or HS designation is the weight in tons on the front two axles of the truck.

In 1956, a new design vehicle was added by the Federal Highway Administration (FHWA) to represent heavy military vehicles traveling on U.S. Interstates highways. This vehicle, known today as the tandem vehicle, consists of two axles, each with a load of 25 kips, spaced four feet on center. The vehicle that was then used in design of common bridges was either the HS-20-44 or the tandem vehicle, whichever caused the larger shear and moment effects. This vehicle combination was used for much of the construction of the interstate system. The tandem vehicle is still in use today for bridge design.

In the 1970's a new design method, Load Resistance and Factor Design (LRFD), was developed (Kulicki & Mertz, 2006). During this time, design vehicles were also investigated. Current vehicles at that time were again much different in terms of gross weight and axle configuration from those that the 1944 vehicles were based on. For that reason, new representative live load vehicle models needed to be developed. Five options

were created and compared to a group of vehicles representing all the legal truck limits. The option found to best fit these vehicles was the HL-93 design load. This design load was adopted by AASHTO in 1993 and became part of the new 1994 AASHTO LRFD Specifications (American Association of State Highway and Transportation Officials, 1994), which is still in use today. It consists of either a truck identical to the HS-20 or the tandem vehicle plus an additional uniform live load of 0.64kip per foot of land length.

The figure below is a timeline that summarizes the history of the design vehicles from 1924 to date.

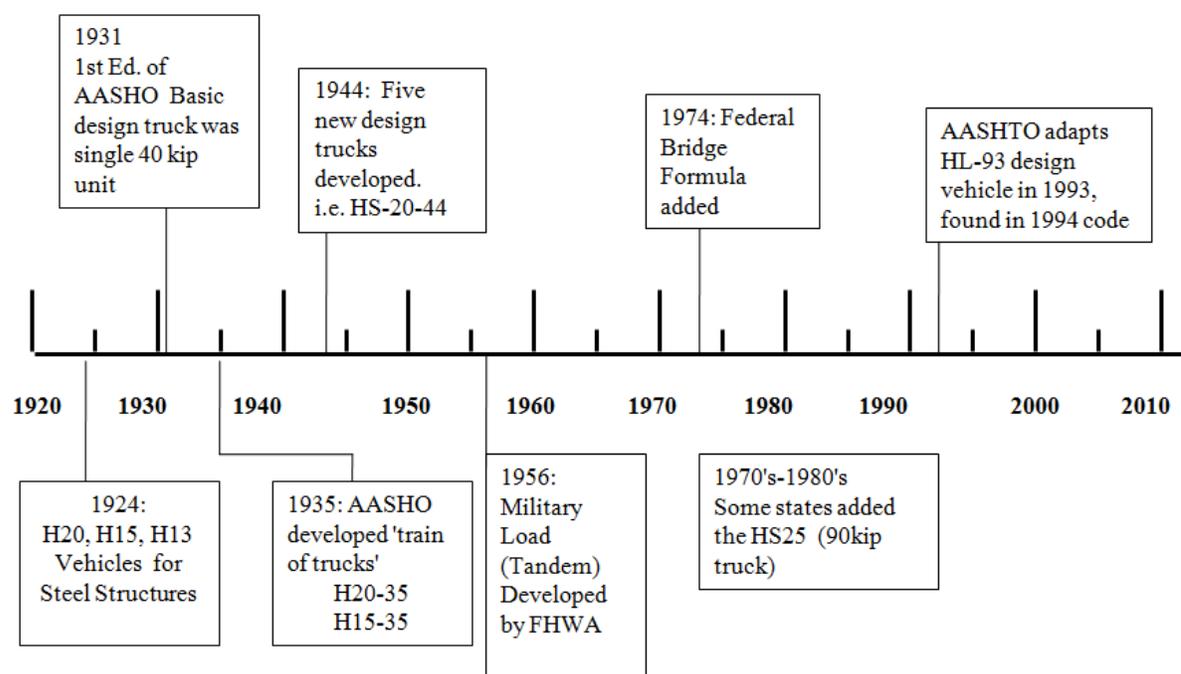


Figure 3-3: Timeline of the History of Bridge Design Vehicles

4 Wisconsin Bridge Load Posting Procedures

In Wisconsin, a bridge is load posted based on the lowest restricted weight of the standard posting vehicles. In Load and Resistance Factor Rating (LRFR), a restricted load would be indicated by a rating of less than 1.0 for the operating rating. There are nine standard vehicles used for load posting. Three of the vehicles are the commercial AASHTO vehicles which include the Type 3, Type 3-3 and Type 3S2. Four AASHTO Specialized Hauling Vehicles are also used in determining whether load posting is necessary. These four vehicles are all single-unit trucks with four, five, six and seven axles. The final two vehicles used in load posting are Wisconsin DOT Specialized Annual Permit Vehicles. One of these vehicles is a six-axle truck and pup configuration and the other is a six-axle tractor trailer combination. Both vehicles have a gross weight of 98 kip. Exact weight distribution and axle configuration for all of these vehicles can be found in Chapter 45 of the Wisconsin Bridge Manual (Wisconsin Department of Transportation, 2008).

5 Mechanically Fastened-Fiber Reinforced Polymer Strips for Bridge Strengthening

One option for the logging industry in avoiding the use of detours and load posted bridge is to look at strengthening essential bridges in Wisconsin. As part of this project, research was done on what different options there are for economic bridge strengthening. During this investigation, it was decided to narrow the scope and look at one particular type of strengthening, the use of Mechanically-Fastened Fiber Reinforced Polymer strips to the bottom side of concrete girders and concrete decks. Research on this application has been done at the University of Wisconsin-Madison and the University of Missouri-Rolla. Additionally, the State of Missouri has used this application on three bridges and has plans to continue to use this application as a cost effective way of strengthening concrete bridges. The following sections look at the process of using the MF-FRP strips as a means of strengthening and also describe four bridges which were strengthened using this method.

5.1 Mechanically Fastened Fiber Reinforced Polymer (MF-FRP) Strips

The use of MF-FRP strips consists of two major components, the FRP strips and the fasteners used to attach the strips externally to the concrete. The FRP strips are made through a pultrusion process and are typically composed of glass, carbon or aramid fibers and vinyl, ester, polyester or epoxy resins. The FRP strips are typically categorized based on tensile strength or bearing strength among other characteristics. The strips typically used for MF-FRP flexure strengthening are unidirectional fiber strips oriented in the longitudinal direction.

The other main component of this strengthening system is the fasteners which attach the strips to the concrete surface. One available system that can be used to attach the

fasteners is through a powder actuated fastener system (PAFS), which uses a gunpowder gun to drill the pin fasteners into the concrete. In addition to several types of guns which can be used, there are also different options available for fasteners depending on the type of material the fastener is being drilled into (Hilti). Another alternative is using threaded screw fasteners.

There are many benefits to using this strengthening method on reinforced concrete structures. One of the large benefits to using this method is that the concrete surface requires very little preparation before installation. Additionally, this system is cost effective in comparison to replacing the structure as is demonstrated in the four case studies that follow. Lastly, installation of this system does not require skilled labor and a strengthened bridge is open to traffic immediately following the installation. (Bank, Nanni, & Arora, 2004).

5.2 Edgerton Bridge: University of Wisconsin-Madison

Wisconsin bridge P-53-702 was located in the city of Edgerton, WI and was strengthened using MF-FRP Strips in August of 2002. This bridge was built in 1930 and served as a representative of structurally deficient short span bridges in Wisconsin. A four to five foot clearance under the bridge with a shallow stream made it easy for workers to get under the bridge which made it a strong candidate for this strengthening. The bridge had an inventory load rating of HS17.6 before strengthening. P-53-702 was a twenty three foot long slab bridge which when originally constructed, had a twenty inch reinforced concrete deck. Asphalt overlays had been added to the bridge deck over its 70 year life. No bridge plans were available for this bridge; however plans for similar slab bridges built in the time period were used to estimate some of the bridge properties. Additionally, this bridge was an

excellent candidate for this project because it was scheduled to be replaced. This allowed for destructive load testing to be completed.

The goal of the strengthening project was to increase the LFR inventory rating to an HS25. This bridge strengthening technique had a total material cost equal to \$7572 and was completed over the course of three days. The design included FRP strips fastened 12" on center using forty fasteners spaced three inches on center. Additionally, an anchor bolt was placed on each end of the strip. Figure 5-1 shows the MF-FRP strengthening design created for bridge P-53-702.

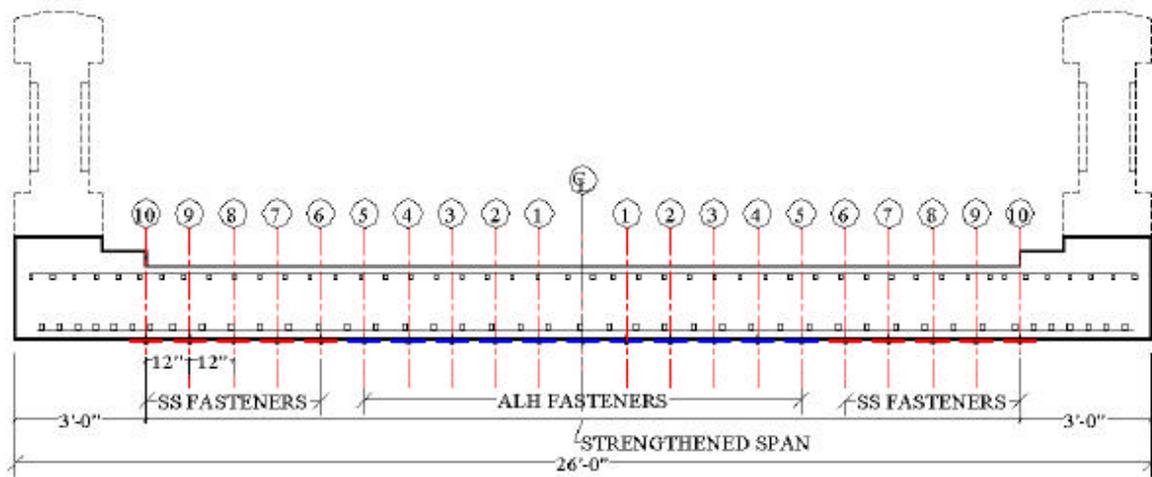


Figure 5-1:MF-FRP Strengthening Design for Bridge P-53-702 (Arora, 2003)

Ten months after being strengthened, the Edgerton Bridge was tested to ultimate failure in two locations. These locations were labeled the east side and the west side. These portions of the deck were isolated from the rest of the bridge by cutting the deck in three places the length of the bridge. Due to the large depth of the deck, which was as deep as twenty-six inches, the contractor was unable to cut through the entire deck and a 1.5 cover at the bottom of the deck connected the sections in some areas. These connections were later broken in the early stages of the testing so that each tested section was isolated. After the

cuts were made, the test apparatus was installed and the top layers of asphalt were removed so that compression failures in the concrete could be identified.

The west side of the deck was tested first followed by the east side. The failure mode of the west side was concrete crushing in the compression zone of the deck. After this failure, more load was applied to try to force failure in the FRP strips. This failure did not occur until about eight inches of deflection and was largely due to spalling of the concrete deck on the underside of the bridge leaving the strips with no concrete to fasten to.

Before testing the east side section, two additional strips were added to test the strength gained from adding additional strips. The failure mode of the east section was similar to the west section with concrete crushing occurring in the compression zone. Again, loading was continually applied after the failure to try to achieve failure in the FRP strips. After the section reached a deflection of seven inches, no failure was observed in the strips and tests were stopped for safety reasons. The load that was applied did produce noticeable damage to both the fasteners and the FRP strips.

Based on the test results, it was evident that the MF-FRP strips did add strength to the deck. An unstrengthened section was not tested during the load testing, however a smaller scale beam was tested in the laboratory and this was used as a control section. The beam tested in the laboratory had materials which didn't exactly match the actual bridge materials, but still provided a good indicator of the capacity of an unstrengthened section. Compared to the base beam tested in the laboratory, the west section had an increased moment capacity of 11.5% and the east side had an increased capacity of 30.8%. The west side was strengthened with three strips spaced at 12 inches on center and the east side was reinforced with five strips spaced six inches on center.

During the ten month duration after strengthening and before destructive testing, the bridge endured several types of weather which provided information as to how the FRP strips held up to the outdoor elements. Before the ultimate load test, the MF-FRP system were looked over and no damage was identified on the strips themselves. In locations where the FRP strips were missing a fastener some deterioration was identified. The only damage that was observed was some rusting and corrosion in the washers on the fasteners and in the fasteners which were not properly imbedded which was probably due to the moisture from frost in the spring. No corrosion was observed in the stainless steel X-CR fasteners which were imbedded properly. Overall, the MF-FRP system was in good condition as no significant damage or deterioration was observed.

5.3 University of Missouri-Rolla Research: Strengthening of Rural Bridges using Rapid Installation FRP Technology

Three bridges in Missouri were strengthened using the MF-FRP technology. All three bridges are located in Phelps County in Missouri. In Phelps County and the surrounding rural counties of Crawford and Dent, one third of the bridges are considered structurally deficient based on Missouri Department of Transportation (MoDOT) standards. The state of Missouri is focusing on rural bridges because the replacement or even conventional strengthening and rehabilitation methods are too expensive. Rural bridges struggle to obtain funding for such projects because of the low annual traffic flows the bridges experience compared to urban bridges.

5.3.1 Missouri Bridge No. 1330005

Bridge No. 1330005 is a bridge which before strengthening had a load posting of 10tons. This bridge is a 26ft long single span simply supported bridge which consists of four

reinforced concrete girders that were cast monolithically with the deck. Bridge plans were unavailable for this bridge, so visual inspection and non-destructive load testing was used to determine the properties of the bridge. The visual inspection showed cracking at midspan of the girders as well as a longitudinal crack in the deck. Additionally, some corroded rebar was identified in the girders and in the deck of the bridge. Concrete coring was completed to determine the concrete properties as well as the locations of the reinforcement in the bridge. A rebar detector was also used in confirming the locations of reinforcement. The analysis of the original structure was completed using the HS15-44 design vehicle. In addition to analyzing the structure in the longitudinal direction, the deck was also analyzed in the transverse direction. The deck was analyzed as a beam supported by two girders.

MF-FRP strips were used for both the flexural strengthening of the bridge girders and the bridge deck. In total, 45 strips were used to strengthen the bridge deck. The strips were spaced 18 inches on center on the underside of the deck. Five strips were used to strengthen the girders, three on the underside of the girder and one on each side of the girder. Figure 5-2 shows the strengthening design for the girders.

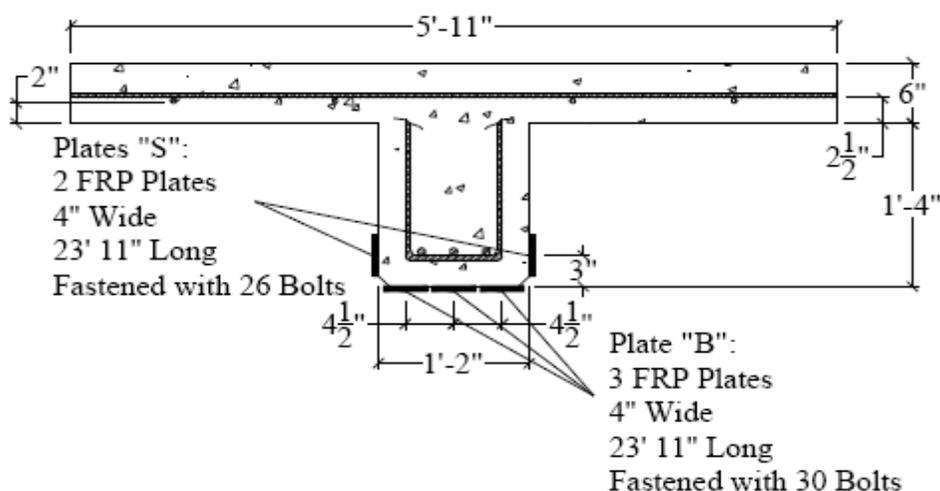


Figure 5-2: Girder Strengthening Design for Missouri Bridge No. 1330005 (Rizzo, Galati, & Nanni, 2007)

In the longitudinal direction, the use of the MF-FRP strips increased the nominal capacity of the bridge by 184.5 kip*ft. Additionally, in the transverse direction, the capacity in the deck increased for 2.5 kip*ft to 12.8 kip*ft. The transverse capacities are based on an eighteen in wide strip of the deck.

The bridge was load rated using the LFR method following the strengthening using MF-FRP strips. Flexural load rating controlled the load rating for the bridge. In Missouri, the load posting of bridges is based on the H20 vehicle and the 3S2 AASHTO legal vehicle. Based on the H20, which controls, this bridge would need to be posted at 24.1 ton. However, since the State of Missouri already limits single unit vehicles to a gross weight of 23ton the bridge posting can be removed. Additionally, the load rating allowable weight for the 3S2 was 80.2ton and the allowable tractor trailer weight in Missouri is 40 ton. The original load posting on the bridge was 10 ton so it can be concluded that the MF-FRP strengthening technique was successful in strengthening the bridge (Rizzo, Galati, & Nanni, 2007). This strengthening project had a total cost of \$16,502 which includes materials and labor (Bank, Nanni, & Arora, 2004).

5.3.2 Missouri Bridge No. 3855006

Bridge No. 3855006 is located on Route 3855 in Phelps County, Missouri. This bridge is a two-span reinforced concrete bridge with three beams. The total length of the bridge is 25ft 10in. The bridge was strengthened in 2004. No bridge plans were available for this bridge therefore site investigation was done to try to determine what the bridge was composed of. This investigation included taking concrete core, cutting areas of the beams to determine the reinforcement and using a rebar locator to try to determine the location of the

steel reinforcement. Through the inspection, it was determined that the bridge didn't have flexural rebar in the proper place and had no discovered shear reinforcement. Additionally, reinforcement over the middle support in the negative moment region was not found. Because of these findings, the bridge was strengthened as if it were two simply supported spans and the girders were ignored so the bridge was treated as a slab bridge.

The strengthening of Bridge No. 3855006 consisted of placing MF-FRP strips on the underside of the deck. Strips were not placed on the bottom of the girders as the girders were ignored in the strengthening design. Approximately two months after the strengthening work was completed, in June of 2004, load tests were completed on the bridge. An H15 vehicle was used for both static and dynamic load tests. The bridge was also set up with instrumentation. LVDT's measured displacements while strain gages were placed on the FRP strips to measure stresses in the strengthening material. The LVDT's showed that the displacements were less than accepted by the AASHTO code which allowed 0.187in. The strain gages showed that not all the FRP strips were engaged when the load was placed on the bridge. This was expected because the strips were non-bonded to the concrete and they were critical strips which will only engage under larger deformations than what were measured. Lastly, upon strengthening the bridge, load rating was completed using the load factor method and the results yielded that the previous posting could be removed. The operating load rating was 1.293 which is equivalent to 42.2 tons and the inventory rating was 0.775. (Rizzo, Galati, & Jones, Design and In-situ Load Testing of Bridge No. 3855006 Route 3855-Phelps County, MO, 2005)

This bridge was strengthening using 203 meters (666 ft) of FRP strip and cost of total installation was \$13,115 (Bank, Nanni, & Arora, 2004).

5.3.3 Missouri Bridge No. B2210010

The final Missouri bridge that was strengthened in Phelps County, Missouri, was bridge number B2210010. This bridge is a three span bridge which has a total length of 32ft. The bridge is a slab bridge with a nine inch thick slab. Two of the spans are continuous and the third span is simply supported. For design, all three spans were taken as simply supported slabs. No bridge plans were available for this bridge so visual and non destructive testing was used to determine the properties of the bridge. The inspection included but was not limited to taking concrete cores and using a rebar locator. A visual inspection showed that there was exposed and eroded rebar as well as cracks in the slabs which ran parallel and perpendicular to the direction of traffic. The bridge was strengthened with the use of MF-FRP strips in the spring of 2004. In addition to attaching MF-FRP strips to the bottom of the slabs of the bridges, vertical strips were also attached to one of the abutments because there were visible cracks due to active earth pressures.

In addition to information about the strengthening design of this bridge, the report also focused on the connection used for fastening the FRP strip to the concrete (Rizzo A. , Galati, Nanni, & Jones, 2005). This is very important as the max strength of the MF-FRP is directly dependent on the capacity of the connection used to fasten the FRP to the concrete. Additionally, the capacity of the connection is based on the number of fasteners used. Determining the number of fasteners needed for the bridge was based on the following equation.

$$n_{b,min} = \frac{F_{FRP}}{R_b} \quad (2)$$

(Rizzo A. , Galati, Nanni, & Jones, 2005)

F_{FRP} is the maximum load that the FRP experiences at ultimate conditions and R_b is the capacity of a single fastener. The capacity of a single fastener was based off of test results with a safety factor of 1.25. Lastly $n_{b,min}$ equals the number of fasteners needed to fasten each individual strip which was 10ft 4in long for bridge number B2210010. $n_{b,min}$ was found to be 26 for this bridge. Fewer fastener bolts would result in a failure at the connection. This equation was found applicable because tests at the University of Missouri-Rolla yielded results that the applied load is uniformly distributed between all the fasteners on an FRP strip.

The fastener used for this bridge consisted of a 2 1/4" concrete wedge anchor, a 5/16" x 9/16" nut and a steel washer. The figure below shows the details of the connection used.

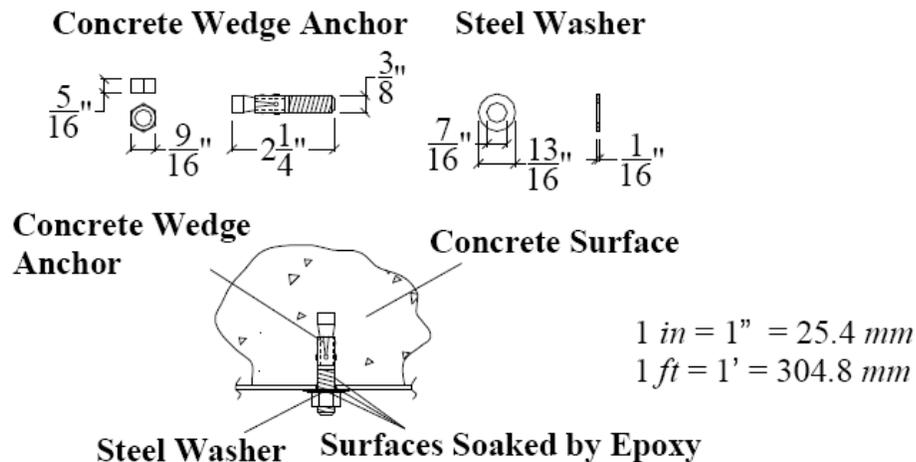


Figure 5-3: Details of a Connection between the Concrete and the FRP Strip (Rizzo A. , Galati, Nanni, & Jones, 2005)

After strengthening, this bridge was load rated using the LFR method. The controlling load effect was shear in the deck and it was determined that the maximum allowable load was 45 ton. This load exceeds the allowable loads for all vehicles in Missouri, so the load posting could be removed (Rizzo A. , Galati, Nanni, & Jones, 2005). The total

cost for the bridge strengthening of Missouri Bridge No. 2210010 was \$11,200 for both labor and materials and a total of 153m (502 ft) of FRP strips was used.

All three of the bridges strengthened using MF-FRP strips showed increases in allowable capacity and point to the fact that this is a successful strengthening process. The future for this strengthening technique in Missouri is strong. As of January of 2007, there was a plan to choose up to 31 bridges in a four county region to be strengthened using MF-FRP strips.

6 Truck Analysis

Today, bridges are typically designed and analyzed based on design vehicle live loads provided in the American Association of State Highway Transportation Officials (AASHTO) Code. Design vehicles are not real trucks. They're vehicles created to represent the highest stresses expected from a variety of real vehicles. Over time, the vehicles used for design have been updated to match effects of current vehicles in actual use. One of the main goals of this project was to compare the effects of the logging trucks used in Wisconsin to those of the bridge design vehicles.

6.1 Data Collection

Information regarding the logging trucks was desired in order to analyze Wisconsin logging trucks and compare them to the design vehicles. Logging trucks are not mass manufactured vehicles and as a result, each logging truck's axle configuration differs. In addition to axle spacing, the distribution of weight on each axle also varies with each individual logging truck and can vary in day-to-day operation as well. Each operator has different methods for determining the distribution of weight on axles. Typically, operators use the tire pressure gages as an indicator of distribution; some methods are more accurate than others. With this in mind, a wide variety of trucks were measured in the field in order to collect an adequate representation of logging trucks used in Wisconsin.

Measurements of the trucks were taken at three different locations in Northern Wisconsin using weigh scales at each site. The goal of the measurements was to obtain size and weight information for a wide range of logging trucks. Only raw timber trucks were

measured. Information recorded for each truck included gross weight, axle spacing and weight distribution of individual or tandem axles.

The first measurements were taken on August 21, 2008 at Biewer Lumber in Prentice, WI. Biewer Lumber is a company that manufactures a large range of building materials. Nine logging trucks hauling softwoods were measured at the Biewer Lumber Location. The scale was above ground requiring the truck to use a ramp for access. A gross weight of the vehicle was obtained first, and then the truck would slowly ‘axle off’ – pulling one axle off and measuring the remaining load- so that the weight of individual axles could be determined. From these measurements the difference could be calculated to determine the weight each axle was bearing.

On August 22, 2008, measurements were taken at the New Page transfer yard in Fifield, WI. This location is not a mill, but served as a raw timber transfer station. Raw timber was brought there from the forests and subsequently hauled out to a lumber mill in Wisconsin Rapids, Wisconsin. The station was also located on a rail line, so timber could be transported to its next destination via rail. Twelve trucks were measured at this location, some arriving and some leaving. All outbound trucks are measured by the lumberyard as well. This ensures the trucks are not overloaded and more timber was added if they were not yet at the maximum allowed gross capacity for the truck configuration. The scale at this location was flush with the ground. Weights were taken in the same manner as at the previous site, measuring gross weight first and then the weights of the individual axles.

The last of the truck measurements was completed on August 28, 2008 at a Louisiana Pacific mill in Tomahawk, WI. Louisiana Pacific is a building material manufacturer and

supplier. Ten arriving trucks were measured at this location. The scale at this location was flush with the ground.

In total, data on weight and axle spacing was collected for thirty-one trucks. In terms of axle spacing, four general types of configurations were measured: five-axle trucks, six-axle trucks, five-axle truck and pup, and six-axle truck and pup. The straight five and six-axle trucks are standard tractor trailer trucks. The figures below show photos of the two types of tractor trailers that were measured.



Figure 6-1: Standard Five-axle Tractor Trailer (Great Lakes Timber Professionals Association, 2009)



Figure 6-2: Standard Six-axle Tractor Trailer(Great Lakes Timber Professionals Association, 2009)

The truck-and-pup configurations have a hinged back trailer known as a pup. The back two axles of the truck are on the pup portion of the vehicle. Figure 6-3 and Figure 6-4 show examples of a five and six-axle truck and pup vehicle configuration.



Figure 6-3: Standard Five-axle Truck and Pup(Great Lakes Timber Professionals Association, 2009)



Figure 6-4: Standard Six-axle Truck and Pup(Great Lakes Timber Professionals Association, 2009)

Measure data for each of the trucks can be found in Appendix A1. The most prevalent type of truck measured was the five-axle tractor trailer configuration. The table below shows the breakdown of the type of trucks measured.

Table 6-1: Type and Quantity of Logging Vehicles Measured

Type of Vehicle	Number Measured
5 Axle Truck	12
6 Axle Truck	9
5 Axle Truck and Pup	7
6 Axle Truck and Pup	3

The six-axle truck and pup configuration was not used often in subsequent analytical comparisons for two reasons. First, because only three trucks of this type were measured it was assumed that this is not the most prevalent type of truck being used to carry raw timber. Secondly, it was determined that the number of trucks of this type measured was not enough to accurately represent the six-axle logging truck and pup configuration.

6.2 Average Trucks

The initial objective was aimed at determining whether the logging trucks would have significant effects on bridges. To avoid analyzing the effects of each individual truck on various bridges, representative average trucks were created based on the information obtained from measuring logging trucks in the field. Three average vehicles were created, a five-axle, six-axle and five-axle truck and pup. Not enough six-axle truck and pup vehicles were measured to use as a base to create an average six-axle truck and pup vehicle.

For the average six-axle vehicle the maximum allowable gross weight of 98,000 lbs. according to Wisconsin Department of Transportation Statutes, was used. For the average five-axle vehicle a gross weight of 94,000 lbs. was used. The statutes state that five-axle vehicles carrying raw forest products are allowed a gross weight of 90,000 lbs., however the majority of the five-axle vehicles measured had a gross weight around 94,000lbs. This was chosen because it would be a better representation of the logging trucks measured. Table 6-2 shows the maximum allowable gross weights for given configurations for Raw Forest Products. These weights are based on the vehicles obtaining permits. The maximum allowable gross weight without a permit is 80,000lbs.

Table 6-2: Maximum Allowable Gross Weights for Raw Forest Products Transportation

Number of Axles	Maximum Allowable Permit Gross Weight
5 Axle	90,000lb
6 Axle	98,000lb

The axle locations of the average vehicles were taken as the average of axle dimensions measured in the thirty-one trucks. The total weight of the truck was distributed across the axles in the same proportions as the average results from the measured trucks.

Thus, the “average trucks” represent the average of the measured trucks in axle locations and weight distribution, but not in total weight since total weight was set at either 94,000 lbs or 98,000 lbs. Figure 6-5 shows the axle configuration and weight distribution for the average five-axle vehicle.

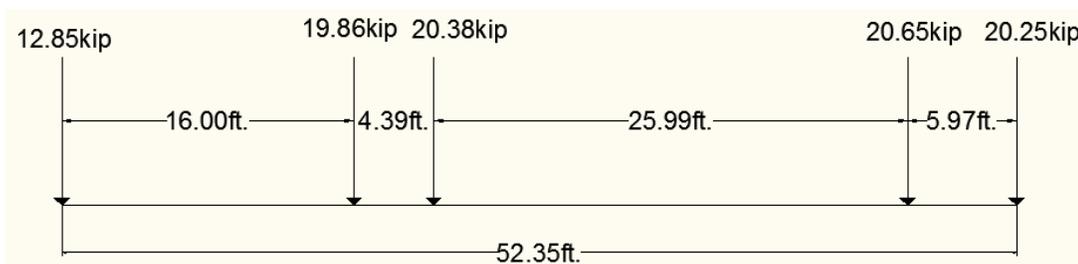


Figure 6-5: Average Five-axle 94k Logging Vehicle

The five-axle logging vehicle was the most prevalent truck measured. The measured gross weights of this type of truck ranged from 84,040 lbs. to 96,660 lbs. As stated above, the gross weight used for the average five-axle truck was 94,000lb which is 4,000 lbs. greater than the allowable 90,000 lbs. Only three of the measured five-axle vehicles had gross weights which exceeded 94,000 lbs., however, eleven of the five-axle vehicles had gross weights exceeding 90,000lbs. The overall length of the five-axle vehicles ranged from 27ft 8in to 59ft 3in. The shortest truck at 27ft 8in was much shorter compared to the rest of the five-axle tractor trailers measured. This vehicle was still included in the average vehicle calculations because it was conservative to do so. The next shortest five-axle truck was 51ft in overall length. The average truck, shown in Figure 6-5 has an overall length of 52ft 4in.

The nine six-axle vehicles that were measured over the three days had gross weights ranging from 89,080 lbs. to 98,280 lbs. The maximum allowed gross weight for a six-axle raw timber vehicle is 98,000lb. The lengths ranged from 51ft 7in to 58ft 11in. The average

vehicle created has a gross weight of 98,000 lbs. and a length of 55ft 6.7in as shown in Figure 6-6.

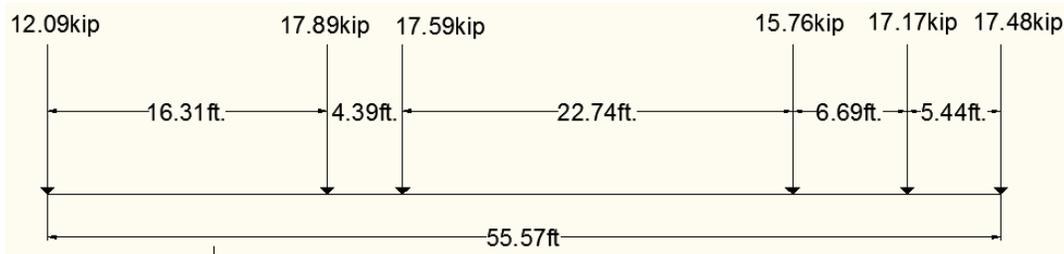


Figure 6-6: Average Six-axle 98k Logging Vehicle

The final average vehicle that was created was the five-axle truck and pup. Seven vehicles with this configuration were measured. The truck and pup configuration is a favorite in the timber industry because of its maneuverability in the woods. Where the pup is connected to the truck serves as a pivoting point to help the truck maneuver in tight locations. The pup can also be removed so the truck can be repositioned if needed. The gross weights varied from 89,500 lbs. to 94,620 lbs. The overall length ranged from 51ft to 57ft 1in. The average five-axle truck and pup has a total length of 54ft 2.64in and is shown below in Figure 6-7. The gross weight of this vehicle is 94,000 lbs.

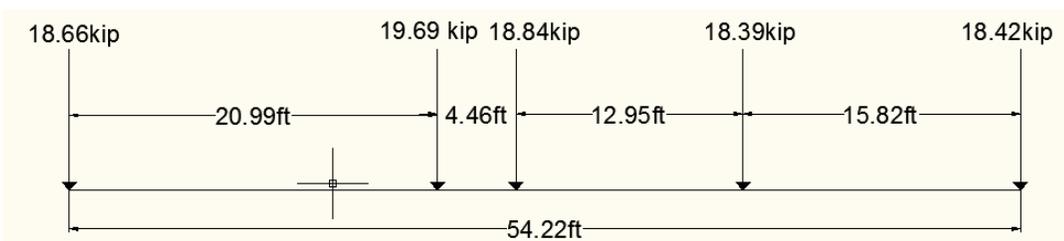


Figure 6-7: Average Five-axle 94k Truck and Pup Vehicle

6.3 Design Comparison Vehicles

Effects of the current standard bridge design vehicles on bridges of various length were then compared to the effects of average logging vehicles. The first of these was the

HS20-44 design vehicle, which has a gross weight of 36 tons. The HS20-44 vehicle is part of the HL-93 design loading which is used in current bridge design. Figure 6-8 shows the configuration for the HS-20 design vehicle.

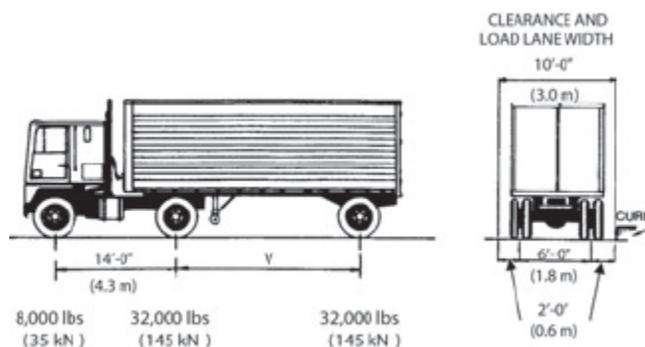


Figure 6-8: HS-20 Design Truck Configuration (American Association of State Highway and Transportation Officials, 2004)

The HS-15-44 was also used in the comparison and has the same axle configuration as the HS-20 as seen in Figure 6-8. The gross weight of the HS-15 is 75% of the gross weight of the HS20-44, equal to 27 tons. The weight distribution of the HS-15 is 6,000 lbs. on the first axle and 24,000 lbs. on each of the back two axles. Many old, deteriorating bridges are currently load rated at an HS-15 so this vehicle is a good representation of the loads these bridges can currently withstand or for older bridges, it may be what they were designed to originally withstand.

The next design vehicle used in the comparison was the tandem vehicle, which was created in 1956 to represent military vehicles. This vehicle consists of just two axles closely spaced at 4ft on center and each axle has a concentrated weight of 25kip as shown in Figure 6-9.

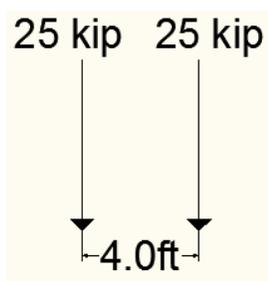


Figure 6-9: Tandem Design Vehicle Configuration

The Wisconsin DOT statute vehicle was the next vehicle that was used in the comparisons. The statute vehicle has the same axle configuration as the HS20 except that it has a gross weight of 57,100 lbs. It has more weight distributed to the front axle compared to the HS20 even though the overall gross weight is 15,000 lbs. less. After preliminary investigation, it was determined that the Wisconsin statute vehicle would always apply stresses to the bridge that were less than the logging vehicles and the HS20-44. With this established, the Wisconsin Statute vehicle was not used in additional comparisons. Figure 6-10 shows the configuration and axle weight distribution for the Wisconsin Statute Vehicle (B. Maximum Weight by Statute).

The next comparison vehicle used was the Wisconsin permit vehicle. The permit vehicle has the same axle configuration as the HS20-44 with a gross weight three tons heavier. This is the heaviest design vehicle used in the truck comparisons. The axle weight distribution is shown in Figure 6-10 (C: Maximum Weight by Annual Permit).

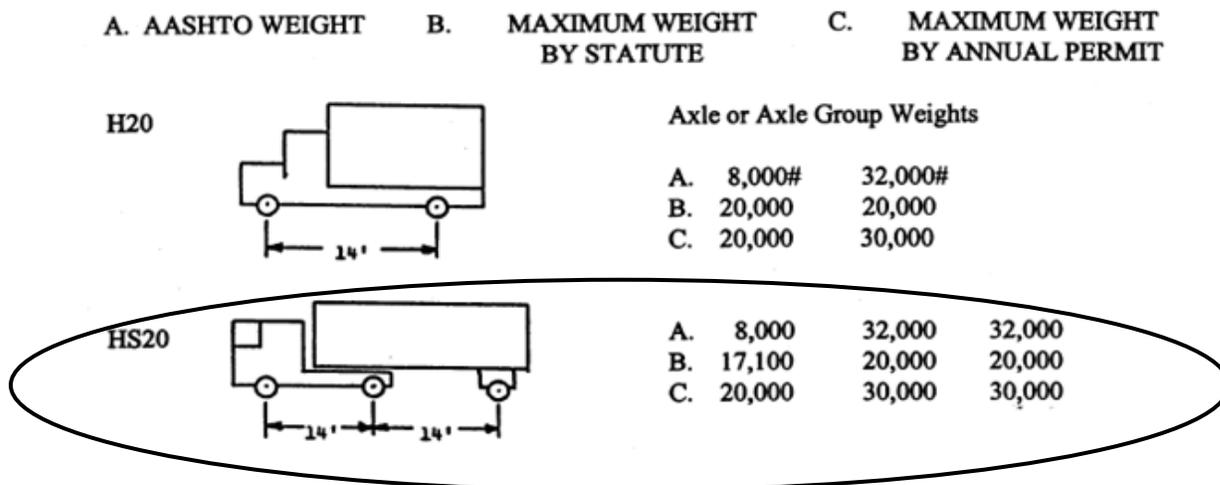


Figure 6-10: Wisconsin Department of Transportation Statute and Permit Vehicle Configurations

The final truck used in the comparisons was the HL-93. The HL-93 is the current design vehicle used in LRFD bridge design. This design truck consists of a uniform lane load of 0.64kip per linear foot along the length of the bridge simultaneous with the larger of the maximum effects created by either the HS20 or Tandem Vehicle. For simple spans less than 40ft, the tandem vehicle will control for moment. For spans less than 26ft, the tandem vehicle will control for shear. For spans larger than those stated above the HS20 design vehicle controls.

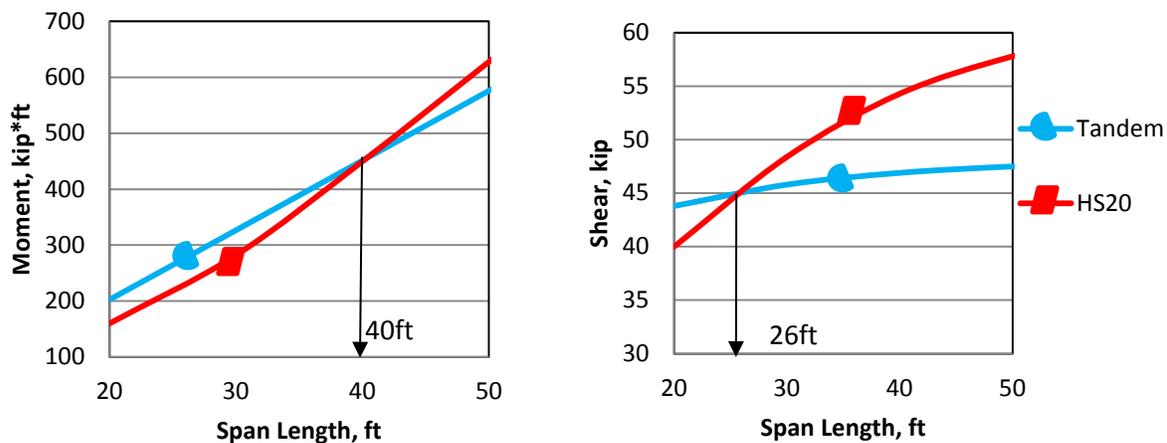


Figure 6-11: Comparison of Tandem Vehicle and HS20

6.4 Analysis Methods

The goal of the analytical work was to compare the effects the different logging vehicles would have on typical highway bridges with the effects of standard design vehicles. In order to narrow the scope of the analysis, only single span bridges ranging from 20ft to 170ft in length were investigated. Both moment and shear effects on bridges were considered. To determine the moment and shear envelopes, which are the plotted maximum possible effects at each point along the length of the beam, the program PCBRIDGE was used (Murphy, 1990). PCBRIDGE “steps” the concentrated loads simulating the vehicle crossing the bridge and determines the maximum moment and shear for each half-foot increment along the length of a bridge for the given span. A deflection envelope is also included in the PCBRIDGE output but was not utilized in the comparisons as the focus of this analysis was on strength rather than service effects.

Using the information provided by PCBRIDGE moment and shear envelopes were plotted for each vehicle at each span length. Figure 6-12 and Figure 6-13 below show examples of these and were very useful in comparing effects of different vehicles for a given span length. The five-axle and six-axle trucks and the five-axle truck and pup are the “average Wisconsin logging trucks” that were defined earlier.

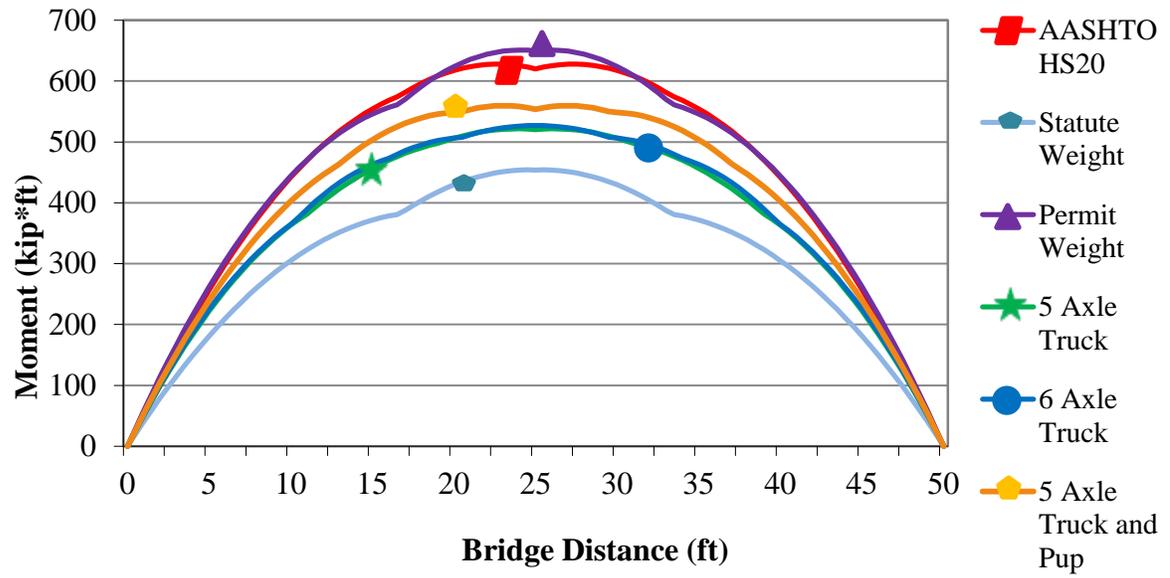


Figure 6-12: Example of Moment Envelope for a variety of trucks based on PCBRIDGE results for 50ft Span

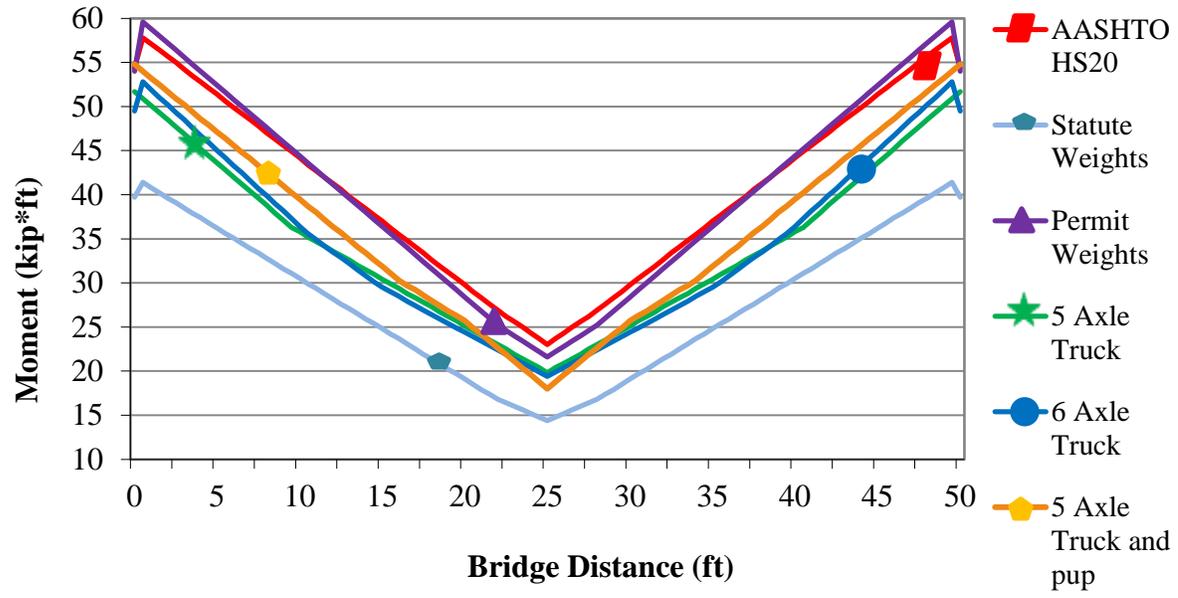


Figure 6-13: Example of Shear Envelope for a variety of vehicles based on PCBRIDGE results for 50ft Span

Figure 6-12 and Figure 6-13 show the envelopes for a 50ft. span. Plots similar to these were made for the moment and shear envelopes for spans ranging from 20ft to 140ft at 10ft intervals. All moment and shear envelope plots can be found in Appendix A2.

6.5 Comparison of Effects on Bridges from Average Trucks

Based on the moment and shear envelope plots it became evident that, for some span lengths, the average logging vehicles had less of an effect than the design vehicles. For example, Figure 6-12 shows the moment envelopes for the average logging trucks, the HS20 design vehicle, and the Wisconsin statute and permit vehicles. At this span length, the logging trucks have moments greater than the Wisconsin statute vehicle but less than both the HS20 and the Wisconsin permit vehicle. This implies that any 50ft bridge designed for the current design trucks could easily carry the Wisconsin logging vehicles. Older bridges, however, might have been designed for lower loads and not be able to carry the logging trucks.

The next step in the comparison analysis was to determine in what specific span ranges the average logging vehicles produced smaller or larger moments than the design vehicles. Those ranges would indicate the lengths of spans where logging trucks would cause overload and travel on the bridge should logically be limited. In order to determine this, the maximum moment and shear for each vehicle was tabulated at 10ft increments. For moment, the spans ranged from 20ft to 170ft and for shear the spans ranged from 50ft to 140ft. The maximum values for a given span range were determined using the moment and shear envelopes described previously.

Using these values, a trend line could be plotted for each individual vehicle showing the maximum effects (moment or shear) for any span in the ranges stated above. By combining each individual vehicles trend lines, the effects of all the vehicles can be evaluated against one another. Figure 6-14 plots the maximum positive moment vs. span length for all the vehicles included in the comparison analysis. No load factors were used in this analysis; all factors were equal to one. In addition to the figures throughout this chapter, tables tabulating the maximum moments and shear values for each of the vehicles used in the comparison analysis can be found in Appendix A3.

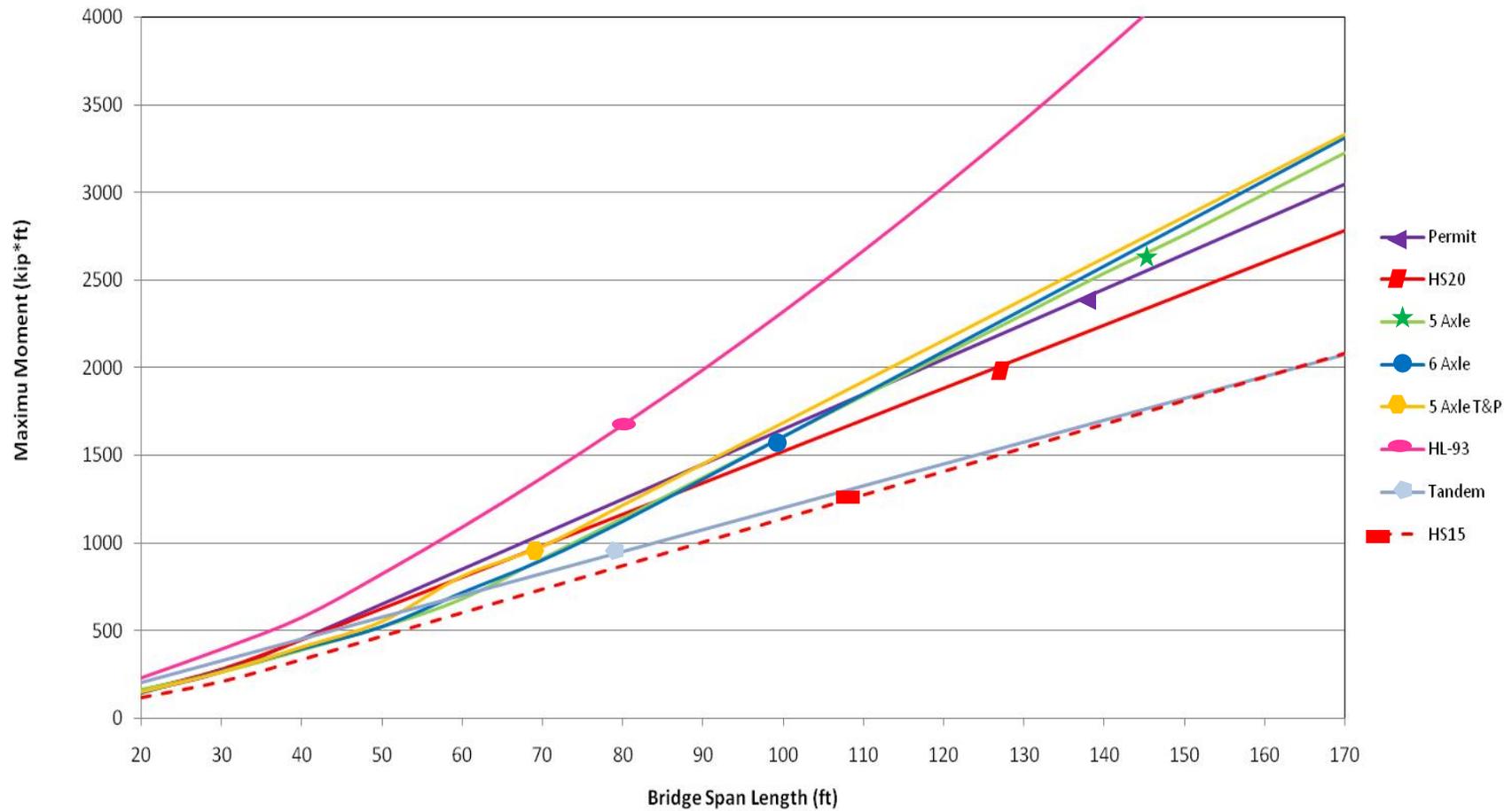


Figure 6-14: Maximum Positive Moment vs. Span Length for all Comparison Vehicles

Although difficult to distinguish in the crowded figure above, the logging vehicles fluctuate between being higher and lower than the HS-20 design vehicle's moments. There is not one clear span length for which each logging vehicle's moment effects start to exceed a particular design vehicle. For example, for small span ranges, the average five-axle truck exceeds the HS20 moments and then at a point drops below the HS20 maximum moment curve only to exceed it again at a longer span length. The span ranges for which the logging vehicle are considered acceptable compared to the design vehicles will be discussed in future sections.

Similar comparisons were done for the maximum shear. Figure 6-15 shows the maximum shear for a given span length for all the vehicles in the comparison analysis.

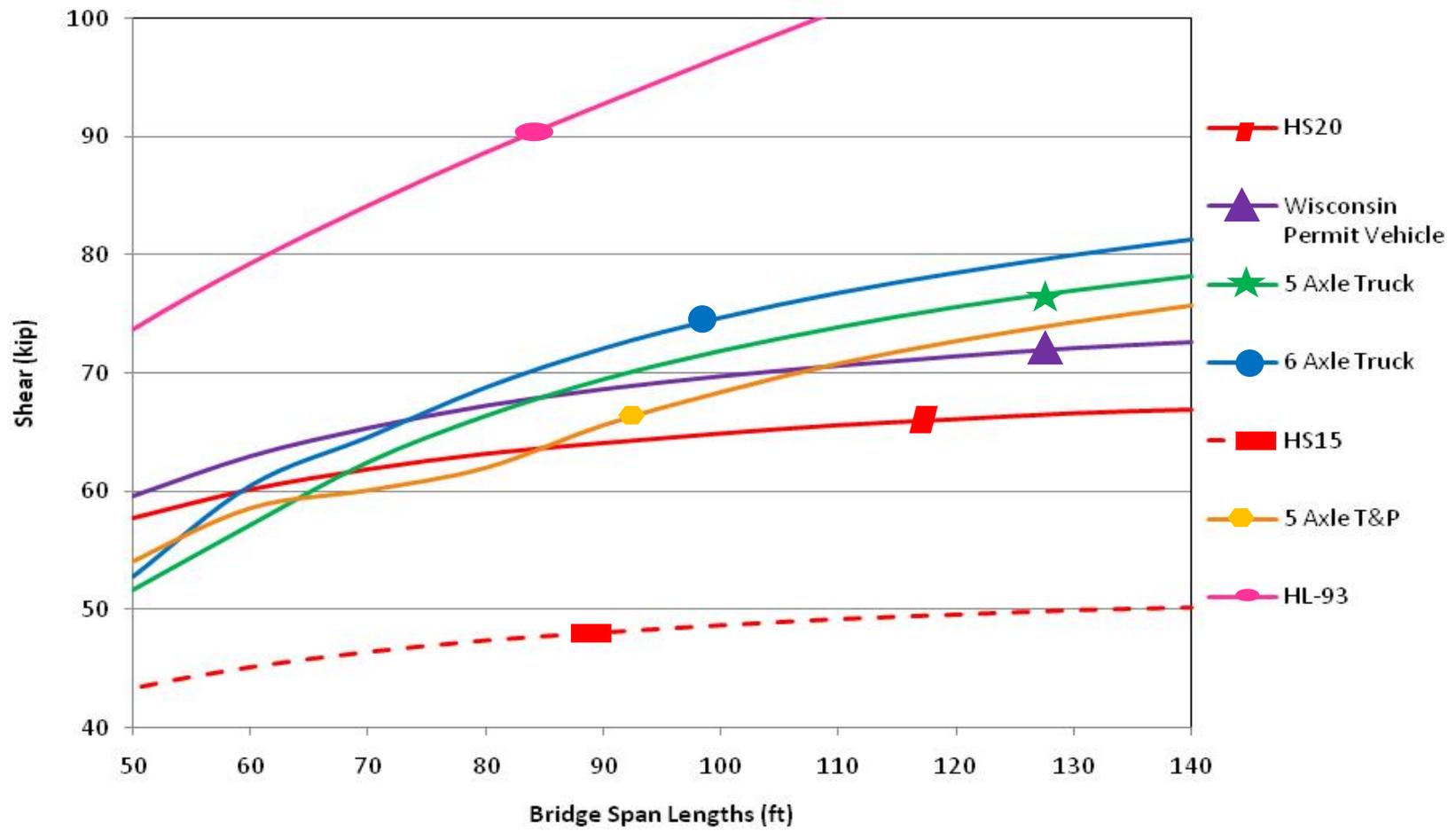


Figure 6-15: Maximum Shear vs. Span Length of all Comparison Vehicles

In Figure 6-15 it is easier to visualize how the vehicles compare to one another in creating shear force in a bridge. One major difference between the shear and moment plots is that with shear effects, there are no lower bound limits. The three average logging vehicles have shear effects less than the design vehicles up to a certain span length and then exceed the design vehicles effects from that point on. This is because shear is dependent on the largest concentrated weight, which in the case of the logging vehicles is typically a single or tandem axle with the highest weight. Regardless of the length of the bridge, the maximum shear force will generally be created by having this single or double axle near the support, independent of bridge span length. With moment calculations, the span length can be a factor of getting the entire truck loading on the bridge if the spans are very short.

6.5.1 Comparison of Design Vehicles

Before looking more in depth at the average logging vehicles, the design and State vehicles were analyzed. Figure 6-16 compares the maximum positive moment for the HS20, HS15, Wisconsin permit vehicle, Wisconsin statute vehicle, tandem design vehicle and the HL-93. The HL-93 vehicle is equal to the vehicle with the largest effects between the tandem and the HS20 plus an additional uniform live load of 0.64kip/ft. None of the other vehicles include a uniform live load. Lastly, the loads applied by all these vehicles are unfactored loads.

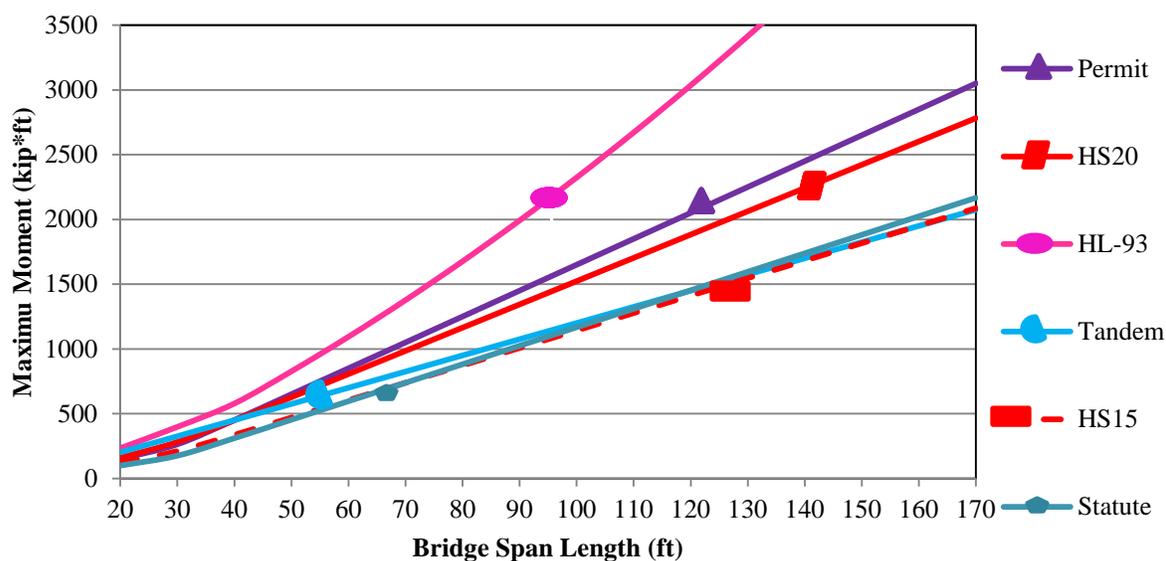


Figure 6-16: Maximum Moment Diagram Comparison for Design Vehicles

Figure 6-16 includes a span range from 20ft to 170ft. The following conjectures were made based on this span range. First, looking at the HL-93, the current design vehicle, it is evident that the unfactored moment effects on a bridge due to this truck are greater than all other design vehicles for all span lengths examined.

On the opposite side of the spectrum, the HS15 and statute vehicles produce a moment on the bridge that is less than all the other design vehicles for all span lengths examined. With this in mind, the Wisconsin statute vehicle is not included in future comparisons as it is known that its moments and shears are less than other vehicles with the exception of the HS15 for long span lengths. The HS15 will be included in a future comparison as an indication of what types of loadings bridges designed or load rated at a HS15 rating can withstand.

The tandem vehicle creates higher moments than all vehicles except the HL-93 for short spans up to 42ft. At higher spans, the tandem vehicle creates moment effects less than

all vehicles with the exception of the HS15. The HS20 and Wisconsin permit vehicle are the most similar to one another in terms of moment effects. For spans up to approximately 41ft, the HS20 produces a higher moment and for spans greater than 41ft the Wisconsin permit vehicle controls. One final thing to note is that the curves representing the moment of the HS15 and HS20 are parallel to another. This is because the two vehicles have the same axle configuration and the weight distribution on each axle. The only difference is that the HS15 is 75% of the gross weight of the HS20 so the load on each axle is reduced by 25%.

In addition to the moment effects from the design vehicles, the shear effects were also compared. Figure 6-17 shows the shear effect each design vehicle imposes on bridges with spans of 20ft to 140ft.

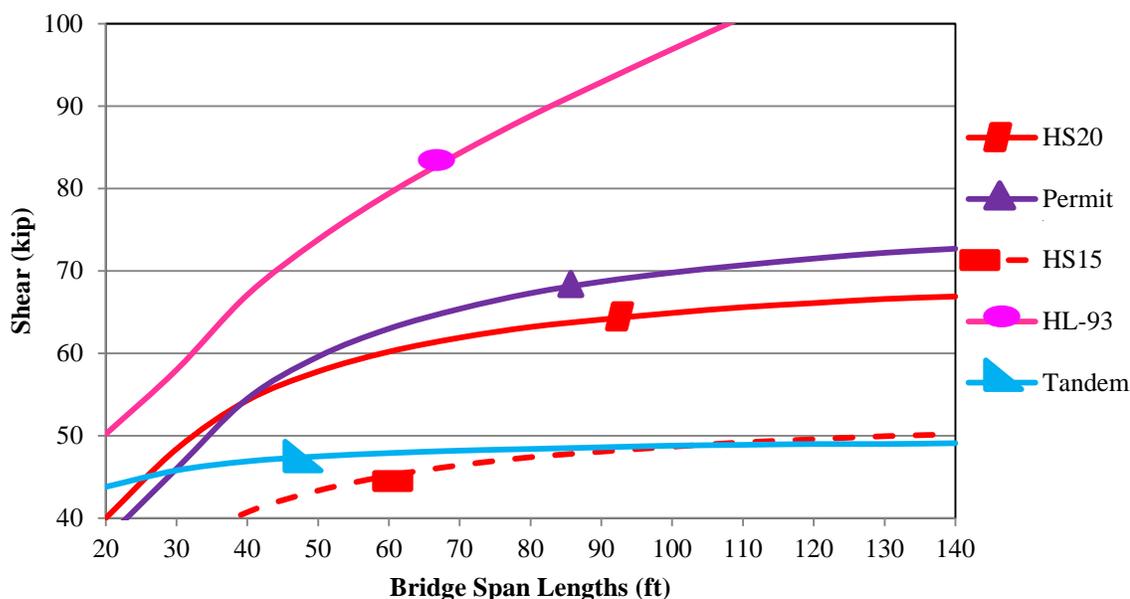


Figure 6-17: Maximum Shear Comparison of Design Vehicles

As with the moment comparison results, the HL-93 vehicle has shear effects that surpass all the other design vehicles. The tandem vehicle controls over the Wisconsin permit vehicle and HS20 for very small span lengths. For span lengths up to approximately 100ft,

the HS15 creates the smallest amount of shear effects. As with the moment comparison, the HS20 and the Wisconsin permit vehicle have similar effects with the HS20 controlling for small spans and the Wisconsin permit vehicle controlling for longer span lengths. Additionally, as with the moment comparisons, the HS15 and HS20 curves run parallel to one another.

The effects of the current design loading standard, the HL-93, surpasses the effects of all previous design vehicles used. This means that new bridges are being designed to carry higher loads to ideally encompass the larger vehicles using the roadways today. Additionally, it should be noted that the HS15 vehicle produces relatively small moment and shear effects. Many bridges today were originally designed for or are still load rated at an HS15 and the trend lines for the HS15 represent the maximum moment and shear effects those bridges are capable of withstanding. That large difference between the HL-93 and the HS15 represents how variable the loading capacities of Wisconsin bridges can be. Similar results are expected if an examination of bridges across the nation was made.

Based on the analysis of all the design vehicles, four vehicles will be used in a more detailed comparison of each individual average logging vehicle. The HS15 load will be used in some comparisons to demonstrate the limits of bridges load rated at HS15. The HL-93 will also be used in some figures to demonstrate the strength limits for new bridges. Additionally, the HS20 truck and Wisconsin permit vehicle will be included in all comparisons.

6.5.2 Average 5 Axle Truck

The first of the logging vehicles to be compared to the design vehicles was the average five-axle tractor trailer vehicle. The moment and shear effects were compared. Upper and lower bound limits were established for span ranges over which the five-axle logging vehicle created smaller moments and shears than the design vehicles. First, moment comparisons were used. Figure 6-18 displays results for the average five-axle logging truck compared to the HS20 and the permit vehicles on bridges with span lengths less than 30 ft. The span length at which the five-axle truck moment begins to fall below the design vehicle moment will be described as the “lower bound limit”.

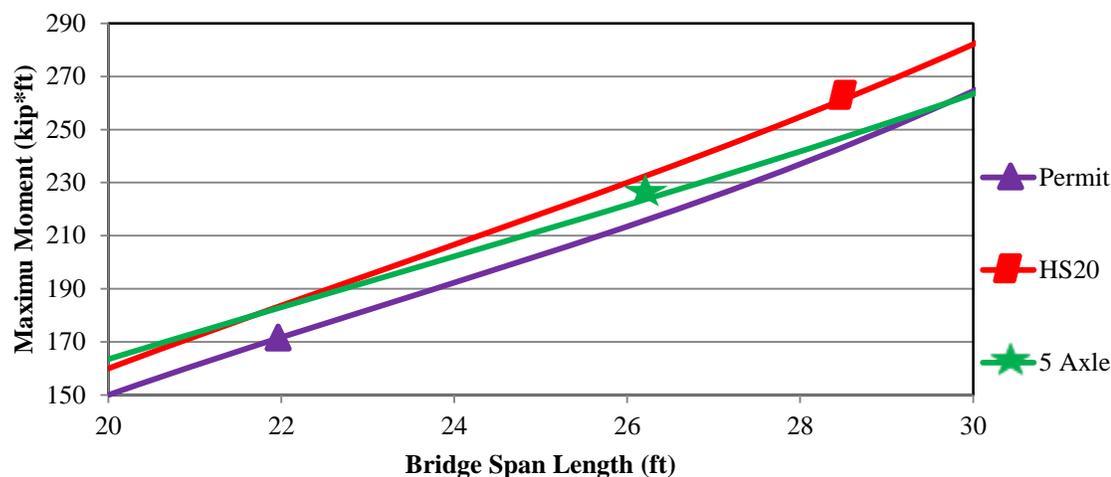


Figure 6-18: Average Five-axle Lower Bound Limits

Based on Figure 6-18, the moment produced by the five-axle truck exceeds the HS20 for spans up to 22ft and exceeds the permit vehicle for spans up to 29ft. The lower bound limits are not of primary concern because the majority of the bridges with low load ratings used by the logging industry are longer than 30ft. The “upper bound limits,” which are spans at which the logging truck moments begin to exceed the design truck moments, are more significant than the lower bound limits. Figure 6-19 shows the moments created by the five-

axle logging truck compared to the HS20 and permit vehicles moments for longer span bridges.

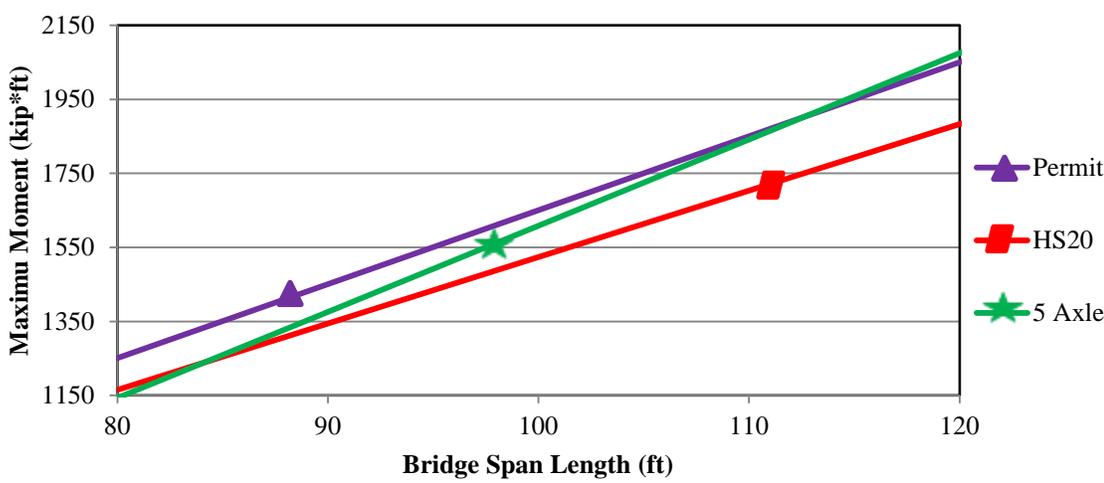


Figure 6-19: Average Five-axle Upper Bound Limits

The average five-axle logging vehicle’s moment exceeds the HS20 design vehicles at a span of 84ft and exceeds the permit vehicle at a span of 113ft as shown in Figure 6-19. This implies that in terms of moment effects, logging trucks should be permitted on bridges with a load rating of HS20 or higher for spans between 22ft and 84ft.

In addition to the moment comparisons, shear effects were also compared. When looking at these effects, there are not lower bound limits when comparing any two vehicles. The figure below compares the shear effects of the average five-axle truck to the HS20, permit vehicle, HS15 and HL-93.

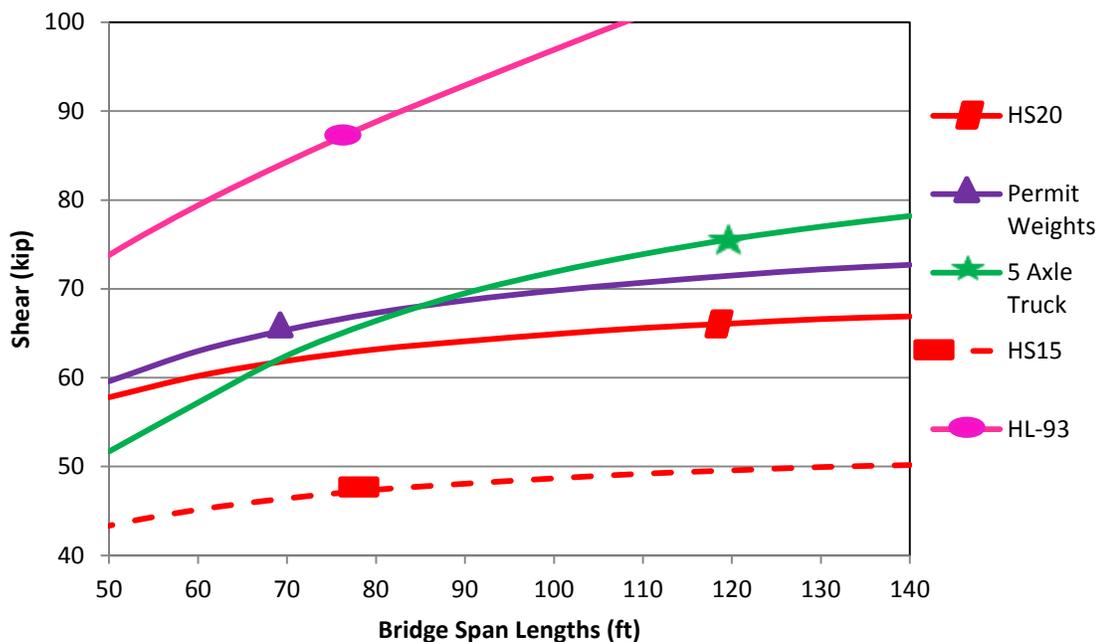


Figure 6-20: Average Five-axle Truck Shear Comparisons

Using Figure 6-19, it is evident that the shear effects caused by the average five-axle logging vehicle greatly exceed the effects caused by the HS15 for all spans ranging from 50ft to 140ft. Additionally, based on that trend, it is expected that the average five-axle logging truck's shear effects will exceed the HS15 for any given span length. Conversely, the shear effects of the average five-axle logging truck are less than the HL-93 design vehicle for all spans 50ft to 140ft. Again, based on that trend, it can be expected that the average five-axle logging truck will have shear effects less than the HL-93 for all larger span lengths.

The shear effects of the HS20 design vehicle are larger than the average five-axle vehicle for span lengths up to 68ft. This means that for bridges with a span less than or equal to 68ft, the average five-axle logging vehicle will have smaller shear effects than those imposed on the bridge by the HS20. The last vehicle that the average five-axle logging truck was compared to was the Wisconsin permit vehicle. The permit vehicle imposed larger shear

effects on the bridge than the HS20, and the shear effects caused by the average five-axle vehicle begin to exceed those of the permit vehicle at a span length of 85ft. This signifies that the average five-axle vehicle will have shear effects less than the Wisconsin permit vehicle for span lengths less than 85ft.

In summary, the average five-axle logging vehicle effects will exceed the effects of the HS15 design vehicle for both moment and shear for all the spans analyzed. Additionally, the average logging vehicle will produce moments and shears which are less than the current design vehicle, the HL-93. When compared to the HS20 and Wisconsin permit vehicle there are span ranges in which the average five-axle logging vehicle produces smaller effects than the design vehicles. The upper and lower bounds of these span ranges are shown in Table 6-3 below.

Table 6-3: Comparison results for the Average Five-axle Logging Vehicle

	Truck Weight (lbs)	Moment				Shear	
		HS20		Permit		HS20	Permit
		Lower bound	Upper Bound	Lower Bound	Upper Bound		
5 Axle Truck	94000	21.75	84.09ft	29	112.6ft	68ft	85ft

(The average five-axle logging vehicle has less effect between the lower and upper bound span ranges, if only one bound is given then the vehicle is acceptable up to that span.)

6.5.3 Average 6 Axle Truck

The next truck that was compared to the HS20 and permit design vehicles was the average six-axle tractor trailer vehicle. Figure 6-21 shows the “lower bound limits,” where for spans less than the limit, the moment effects of the six-axle vehicle are larger than the HS20 vehicle and the permit vehicle on bridges with short simply supported spans.

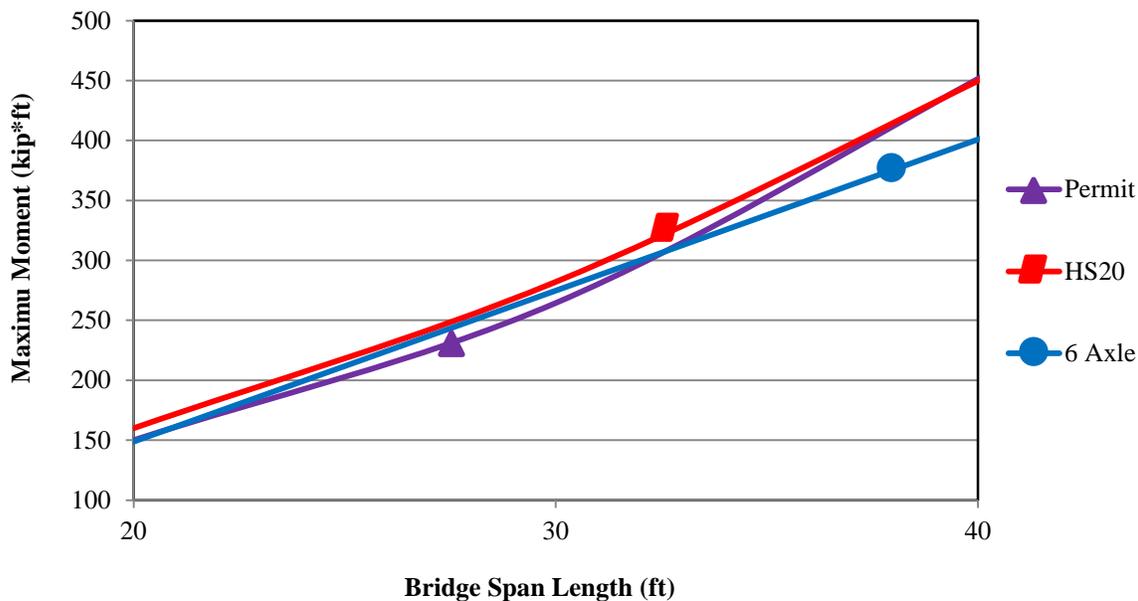


Figure 6-21: Average Six-axle Lower Bound Limits

As seen in the plot above, the average six-axle logging vehicle has no “lower bound” when compared to the HS20. The average six-axle vehicle is acceptable for all span ranges on bridges designed to carry HS20 trucks, up to the upper bound which is discussed below. When compared to the permit vehicle different results were discovered. First, for spans less than 21ft, the permit vehicle produces a higher moment than the average six-axle vehicle. Next, for spans between 21ft and 32ft, the average six-axle vehicle produces slightly higher moments than the permit vehicle. At 32ft, the permit vehicle again produces the higher maximum moment.

Next, the upper bound limits of the average six-axle vehicle were determined. Figure 6-22 displays the moments created by the average six-axle vehicle compared to the HS20 and the permit vehicle on longer span bridges.

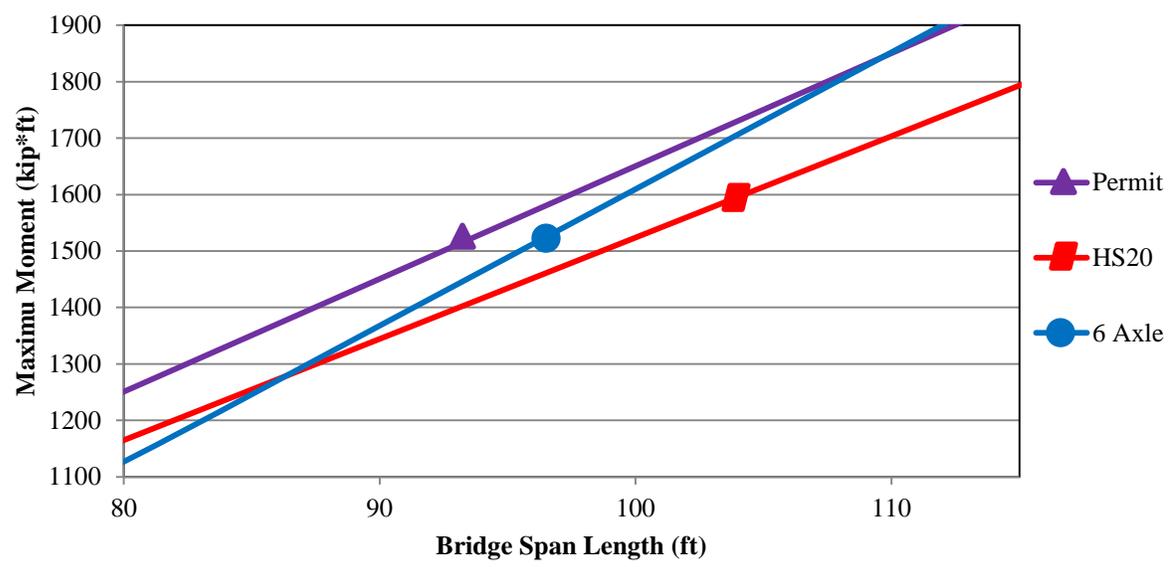


Figure 6-22: Average Six-axle Vehicle Upper Bound Limits

The average six-axle vehicle’s moments begin to exceed the HS20 at a span length of 86ft and it begins to exceed the Wisconsin permit vehicle at 110ft. These results show that the average six-axle vehicle will not damage a bridge load rated at HS20 for span ranges between 32ft and 86ft. In addition to the moment comparisons, shear comparisons were also completed for the average six axle logging vehicle.

Figure 6-23 shows the shear effects of the average six-axle vehicle as well as the HS20, Wisconsin permit vehicle, HS15, and HL-93 design vehicles for spans 50ft to 140ft.

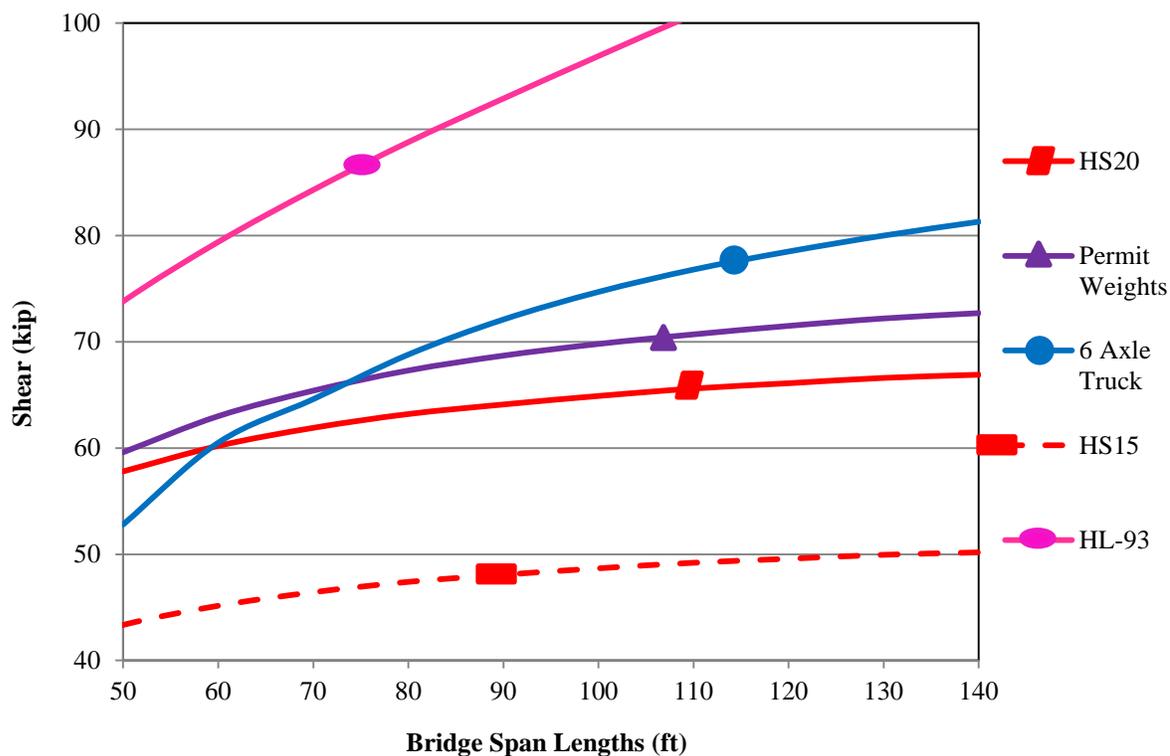


Figure 6-23: Shear Comparisons for the Average Six-axle Logging Vehicle

The results shown above are similar to the results seen with the average five-axle vehicle. The HS-15 shear effects are less than those of the average six-axle logging vehicle for all spans examined. The HL-93 vehicle has shear effects that are larger than the average six-axle vehicle for spans from 50ft to 140ft. Based on the trend seen in Figure 6-23 it can be inferred that the HS-15 will produce shear effects less than and that the HL-93 will always produce greater shear effects than the average six-axle logging vehicle on simple span bridges in the length range of interest.

The average six-axle logging vehicle was also compared to the HS20 design vehicle and the Wisconsin permit vehicle. The average six-axle logging vehicle has shear effects that are less than the HS20 for span lengths less than 59ft. This means that in terms of shear,

for bridges with span less than 59ft, the average six-axle truck will have smaller effects than the HS20 and shouldn't damage a bridge load rated at an HS20 or above.

Lastly, the average six-axle logging vehicle was compared to the Wisconsin permit vehicle in terms of shear. For bridge spans up to 74ft, the average six-axle truck has less of a shear effect than the permit vehicle. One the span exceeds 74ft the average logging truck produces larger shear effects than those produced by the permit vehicle.

In summary, the average six-axle logging vehicle will likely always produce a higher shear effect than the HS15 and it will likely always produce a lower shear effect than the current HL-93 design vehicle. When comparing to the HS20 and the Wisconsin permit vehicles on smaller spans the average six-axle logging truck will produce smaller shear effects. In comparing moment effects, the average six-axle truck has both upper and lower bound limits. The minimum and maximum span lengths for which these statements are valid can be found in the table below.

Table 6-4: Comparison Results for the Average Six-axle Logging Vehicle

	Truck Weight (lbs)	Moment				Shear	
		HS20		Permit		HS20	Permit
		Lower bound	Upper Bound	Lower Bound	Upper Bound		
6 Axle Truck	98000	none	86.14ft	20.75ft 32.5ft	32.5ft 109.6ft	59ft	74ft

(The average six-axle logging vehicle has less effect between the lower and upper bound span ranges, if only one bound is given then the vehicle is acceptable up to that span.)

6.5.4 Average 5 Axle Truck and Pup

The final average logging truck that was created based on measured data was a five-axle truck and pup vehicle. The initial comparison completed was looking at the moment of

this vehicle compared to those of the HS20 and Wisconsin permit vehicle. The average five-axle truck and pup has both upper and lower bounds when compared to the Wisconsin permit vehicle, however, it does not have a lower bound limit when compared to the HS20. Figure 6-24 shows the lower bound limit when compared to the Wisconsin permit vehicle.



Figure 6-24: Average Five-axle Truck and Pup Lower Bound Limits

The average five-axle truck and pup vehicle has a greater moment effect than the permit vehicle on bridge spans up to 30.75ft. Once span lengths exceed 30.75ft the average five-axle truck and pup has a lower moment than the Wisconsin permit vehicle to an upper bound which will be discussed next.

With the lower bound limit established, the upper bound limits for the five-axle truck and pup vehicle were determined next. The average five-axle truck and pup has upper limits for both the HS20 and the Wisconsin permit vehicle. Figure 6-25 displays these upper bound limits.

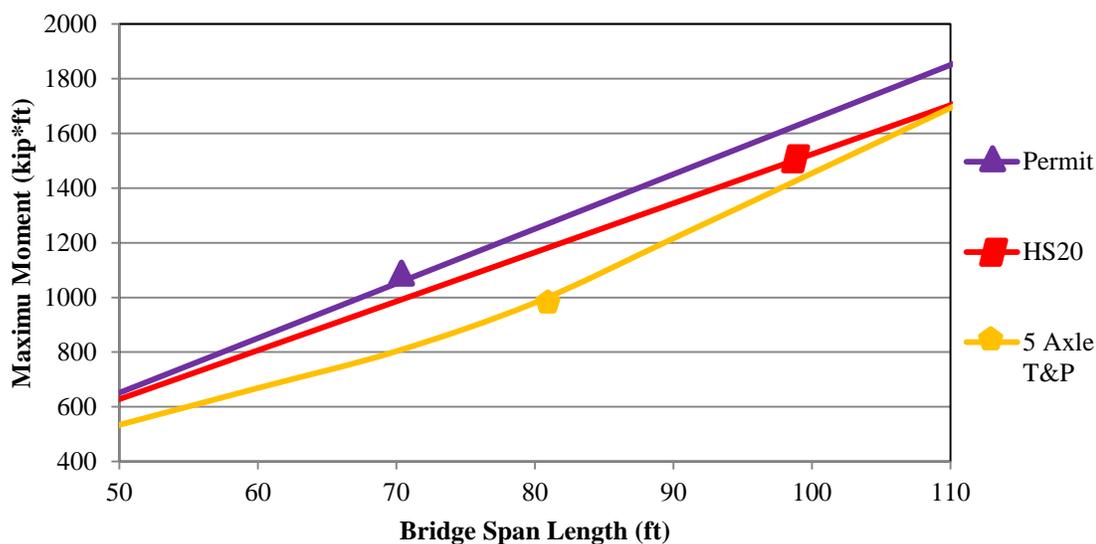


Figure 6-25: Five-axle Truck and Pup Upper Bound Limits

The upper bound limit for the permit vehicle is reached at a span length of 89ft. For spans greater than 89ft, the average five-axle truck and pup will exceed the moment effects of the Wisconsin permit vehicle. The upper bound comparison between the average five-axle truck and pup and the HS20 is not as clear. The average vehicle fluctuates back and forth around the HS20 trend line. Initially, for spans up to 59ft, the HS20 produces the larger moment. Next, from spans 59ft to 66ft, the average five-axle truck and pup produces just slightly larger moments. At a span length of 66ft, the HS20 again produces the higher moment up to a span length of 71ft. Above 71ft, the average five-axle truck and pup produces the largest moment. For simplicity, it was decided that an upper bound limit of 71ft would be used when comparing the HS20 and average five-axle truck and pup.

In addition to the moment comparisons, shear effect comparisons were also completed for the average five-axle truck and pup vehicle. Figure 6-26 shows the shear comparisons for the average truck and pup.

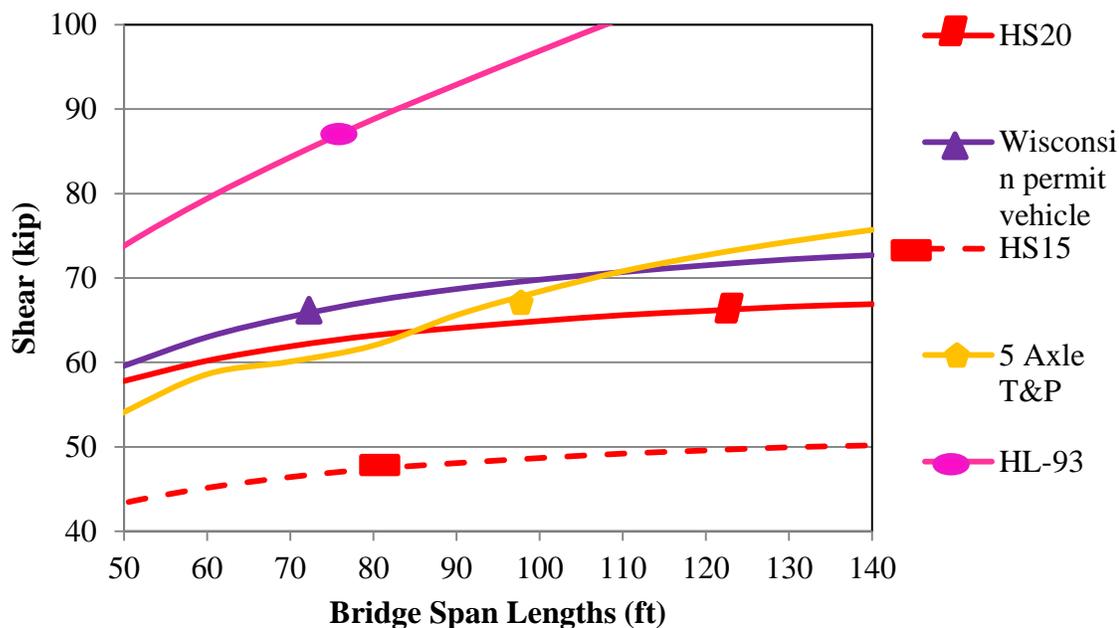


Figure 6-26: Shear Comparison for Average Five-axle Truck and Pup

In the plot above, the HL-93 and the HS15 are included as visual aids to show the current design vehicle (HL-93) and also the maximum load many deteriorated bridges can withstand (HS15). The average five-axle truck and pup shear effects fall in between the effects of these two vehicles.

The average five-axle truck and pup creates lower shear effects than the HS20 for spans less than 84ft. When compared to the Wisconsin permit vehicle, the average five-axle truck and pup creates smaller shear effects for spans up to 108ft. Once the spans are larger than 108ft, the average truck and pup will produce shears greater than the Wisconsin permit vehicle.

All of the upper and lower bound results including both moment and shear for the average five-axle truck and pup are summarized in Table 6-5.

Table 6-5: Summary of Average Five-axle Truck and Pup Limits

	Truck Weight (lbs)	Moment				Shear	
		HS20		Permit		HS20	Permit
		Lower bound	Upper Bound	Lower Bound	Upper Bound		
Five-axle Truck and Pup	94000	----- 66ft	59ft 71ft	30.75ft	89ft	84ft	108ft

(The average five-axle truck and pup logging vehicle has less effect between the lower and upper bound span ranges, if only one bound is given then the vehicle is acceptable up to that span.)

When comparing these results to the results from the other average logging vehicles, the upper bound limits for shear are much higher than those determined for the average five and six-axle logging vehicles. This signifies that the average five-axle truck and pup creates smaller shear effects than the other two average trucks. Additionally, the average five-axle truck and pup moment upper bounds were lower than the other two average vehicles meaning the average five-axle truck and pup creates higher moments at lower span ranges. These conclusions mean that the average five-axle truck and pup is less severe than the other two logging vehicles in terms of shear but worse in terms of moment effects.

6.6 Comparison of Individual logging vehicles

Up to this point, all the logging truck analyses were based on averaged vehicles. In addition to those analyses, each of the individual logging trucks measured were also investigated, thirty-one cases in total. This section focuses on the five and six-axle tractor trailer configurations only as they were the most prevalent configurations measured. The individual truck analyses were completed for several reasons. First, the effects of the average trucks, could be compared to the individual trucks to detect the amount of variation from average. This showed if the average trucks were conservative in comparison to all the trucks

measured. Additionally, the individual trucks could be compared to the design vehicles to see if some of the logging vehicles were configured more optimally than others.

The first step in comparing individual logging trucks was to analyze each individual truck using PCBRIDGE. This was done in the same manner as the average logging vehicles. Based on the PCBRIDGE results, trend lines for each vehicle were plotted for both moment and shear. Next, all the maximum moment curves for a given configuration were plotted together. Figure 6-27 shows the maximum and minimum moments for the measured five-axle tractor trailers and compares those values to the average five-axle logging vehicle moment curve. The bold lines represent the maximum and minimum moment effects created by any measured five-axle tractor trailer vehicle.

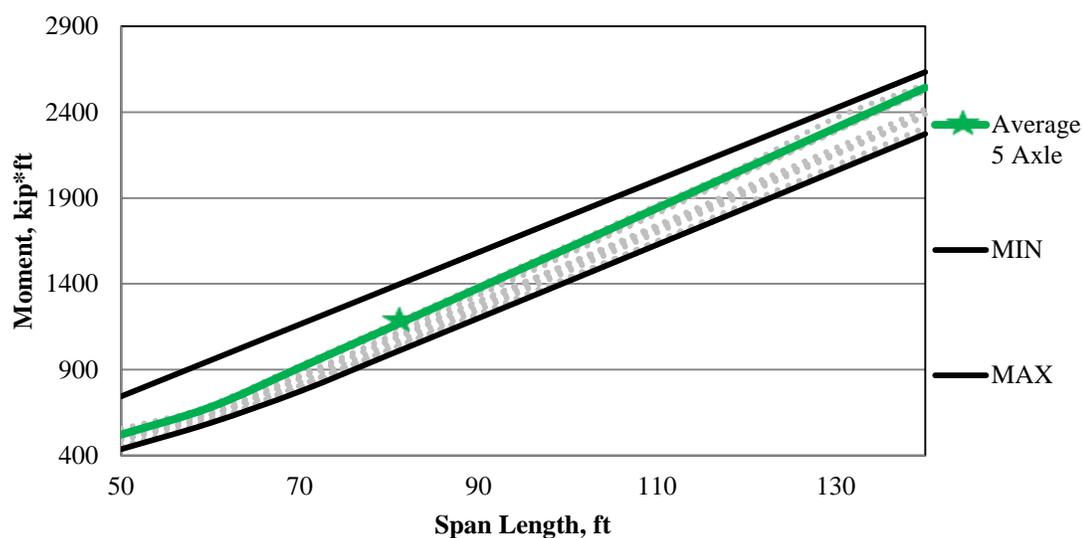


Figure 6-27: Moment Envelope for five-axle trucks compared to the average five-axle vehicle

The grey dotted lines represent all the individual five-axle truck moment curves, the heavy black lines being the minimum and maximum effects from any given vehicle. The plot shows that the average five-axle vehicle is relatively conservative. There are only two

measured logging trucks which had greater moments than the average five-axle vehicle at any given span from 50ft to 140ft.

The average five-axle vehicle was also compared to all the individual five-axle tractor trailer vehicles in terms of shear effects as shown in Figure 6-28.

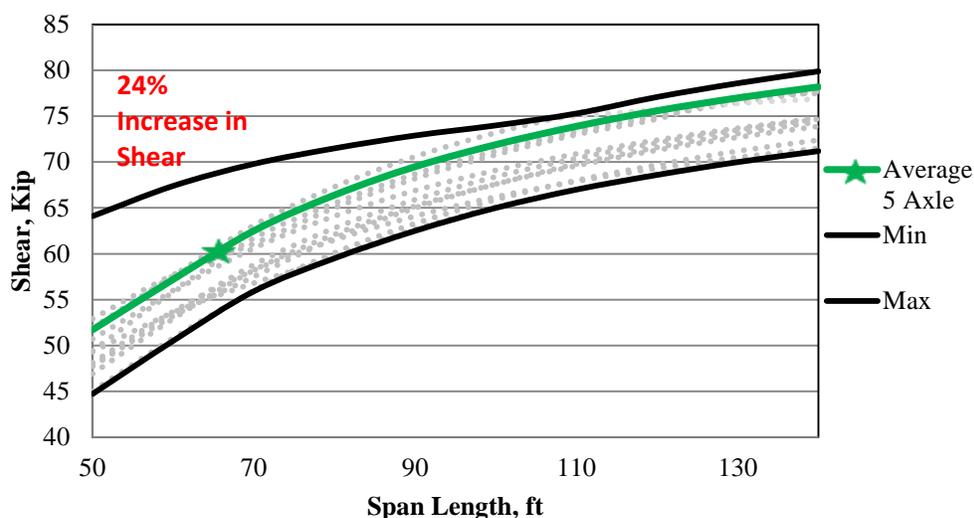


Figure 6-28: Shear Envelope for five-axle trucks compared to the average five-axle vehicle

The average five-axle vehicle is a good representation of the logging vehicles measured in terms of shear with the exception of one vehicle. This vehicle had a much larger shear effect than the average vehicle for small span ranges. At a span of 50ft, it had a shear force that is 24% larger than the shear force from the average five-axle vehicle. This outlier vehicle had a gross weight of 96,660 lbs. which exceeds the allowable permit weight (and the weight of the average five-axle truck) by more than 2000 lbs. Additionally, individual axles on this vehicle weighed in excess of 21,000 lbs. which is the maximum weight allowed on a given axle by permit. Based on this information the average five-axle vehicle is a good, fairly conservative, representation of the legal five-axle trucks measured in the field.

In addition to comparing the five-axle tractor trailer vehicle curves to the average vehicle, they were compared to the HS20 design vehicle and the Wisconsin permit vehicle to see how the moment and shear effects compared. Figure 6-29 shows the moment comparison between the measured five-axle tractor trailer vehicles and the HS20 design vehicle.

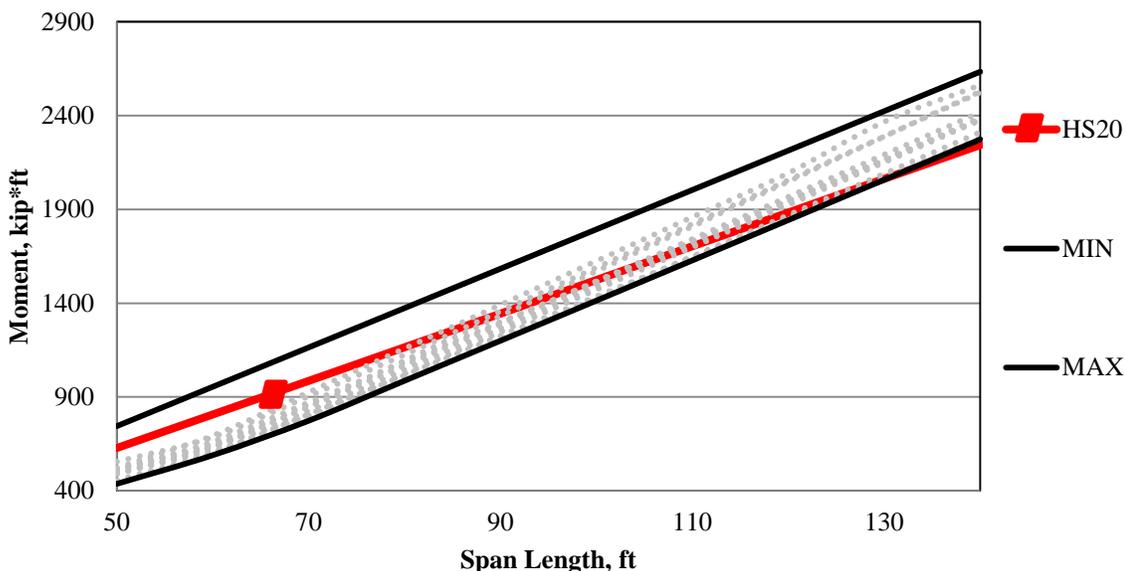


Figure 6-29: Moment Envelope Comparison of five-axle tractor trailer vehicles compared to the HS20

The first thing to note from Figure 6-29 is that for long spans exceeding 140ft all the vehicles measured will exceed the moment effects of the HS20. Secondly, the trends found previously from comparing the average five-axle vehicle to the HS20 are fairly similar to the comparison of the results seen here. Most of the logging vehicles measured have a moment effect that is less than the HS20 up to spans around 80ft. This is excluding the outlier vehicle, which was discussed earlier. Similar results were done to compare the shear effects of the HS20 and the five-axle tractor trailer vehicles. Figure 6-30 shows this comparison.

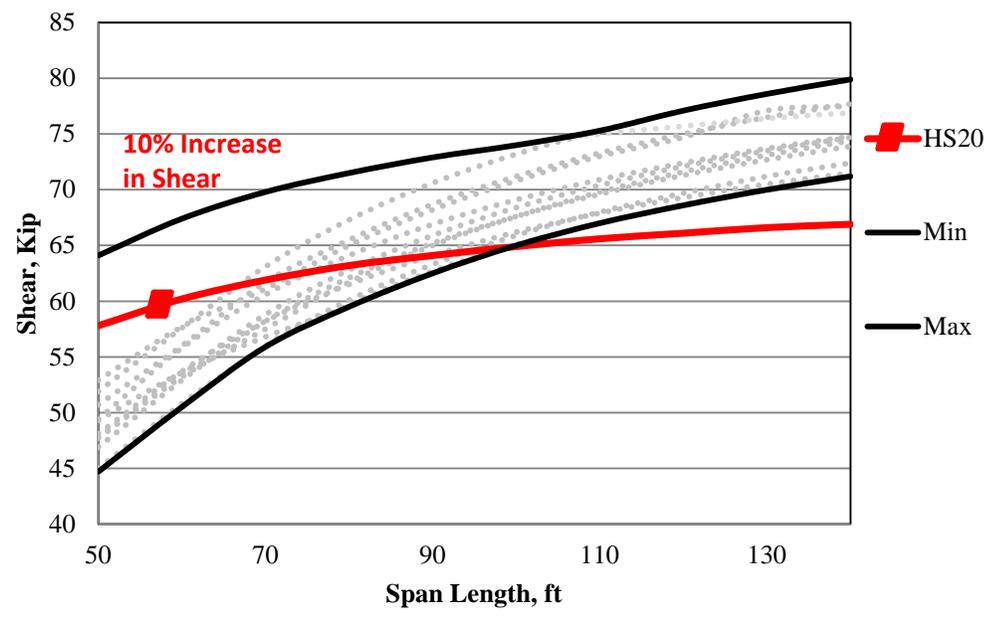


Figure 6-30: Shear envelope comparison of the five-axle vehicles and the HS20

The results from Figure 6-30 are similar to those seen with the average five-axle vehicle. For most of the vehicles measured the shear effects start to surpass the HS20 at spans from 60ft to 90ft. For long spans exceeding 100ft all the trucks measured have shear effects greater than the HS20. At small span ranges, up to 60ft all the trucks have shear effects less than the HS20 with the exception of the overloaded vehicle which has a shear effect 10% higher than the HS20. This overloaded vehicle is a good representation of how exceeding the legal weight limits can have a large effect on bridges.

The final comparison, shown in Figure 6-31 and Figure 6-32 was a comparison with the Wisconsin permit vehicle.

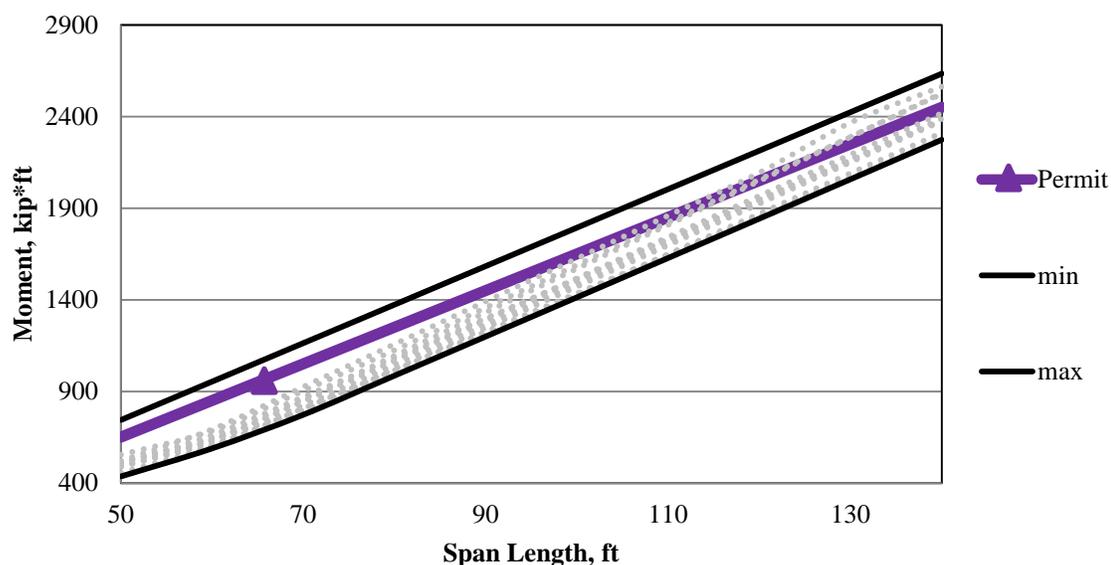


Figure 6-31: Moment Envelope of five-axle vehicles compared to the Wisconsin permit vehicle

For short span ranges the permit vehicle controls in terms of moment effects, except for the one overloaded logging vehicle. The five-axle vehicle's moments begins to exceed the effects of the Wisconsin permit vehicle at spans around 100ft to 110ft. This is similar to the results that were found when the permit vehicle was compared to the average five-axle vehicle which again confirms the average five-axle vehicle was a good representation of the actual measured vehicles.

The shear effects of the individual five-axle vehicles and the Wisconsin permit vehicle were compared as shown in Figure 6-32.

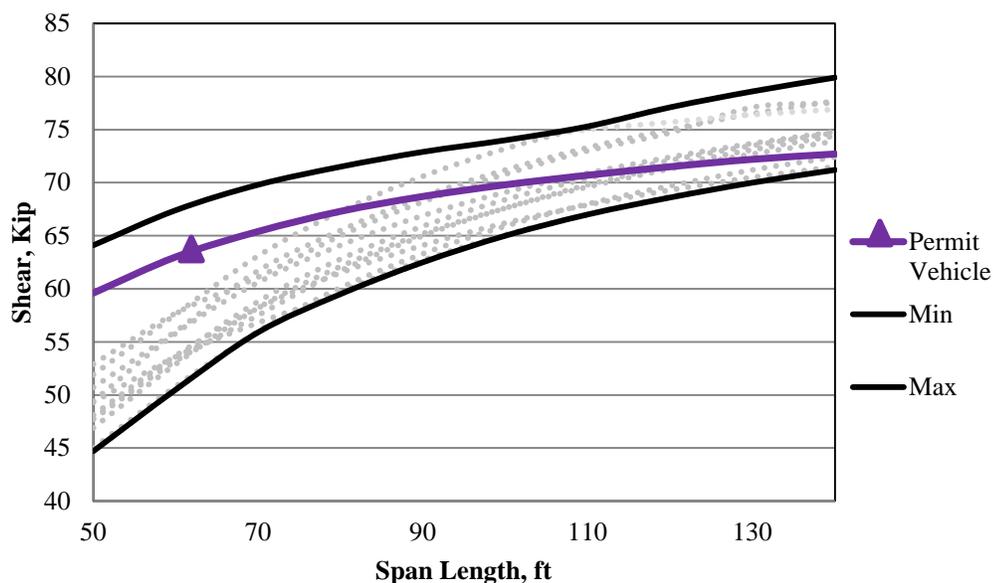


Figure 6-32: Shear Envelope of five-axle truck compared to the permit vehicle

The results shown in Figure 6-32 are similar to the shear comparisons with the HS20. For short spans less than 80ft, all the five-axle vehicles, except the overloaded vehicle, have shear effects less than the Wisconsin permit vehicle. They begin to exceed the shear effects of the Wisconsin permit vehicle at span lengths of 80ft.

The same comparisons were completed for the six-axle tractor trailer vehicle. Initially, the six-axle vehicles were compared to the average six-axle vehicle. This was done to check how well the average vehicle represented the vehicles measured and also to see if the average vehicle was conservative in comparison to the individually measured vehicles. Figure 6-33 shows the moment comparison between the average vehicle and the individual vehicles.

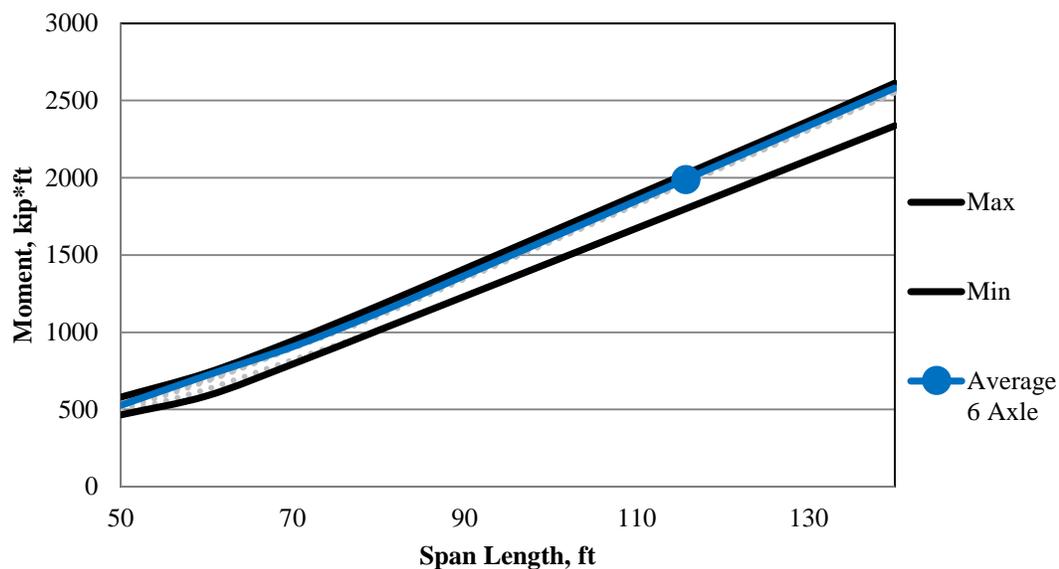


Figure 6-33: Moment Envelope for six-axle vehicles compared to the average six-axle vehicle

Nine six-axle tractor trailer combination vehicles were measured. Compared to the five-axle moment plots, the six-axle moment curves are more closely related and tightly grouped. The average six-axle vehicle appears to be a good representation of the all the individual six-axle trucks measured. Initially, at a span of 50ft, five of the individual vehicles produced slightly higher moments than the average six-axle vehicle. Only one vehicle consistently had a higher moment. This vehicle did not weigh more than the allowable permit gross weight and the overall length of the vehicle was 55.167ft which is near the length of the average six-axle vehicle. It is likely the increase in moment occurred as a result of closely spaced axles within the configuration.

In addition to the moment comparison, shear effects were also compared. The shear comparison is shown in Figure 6-34.

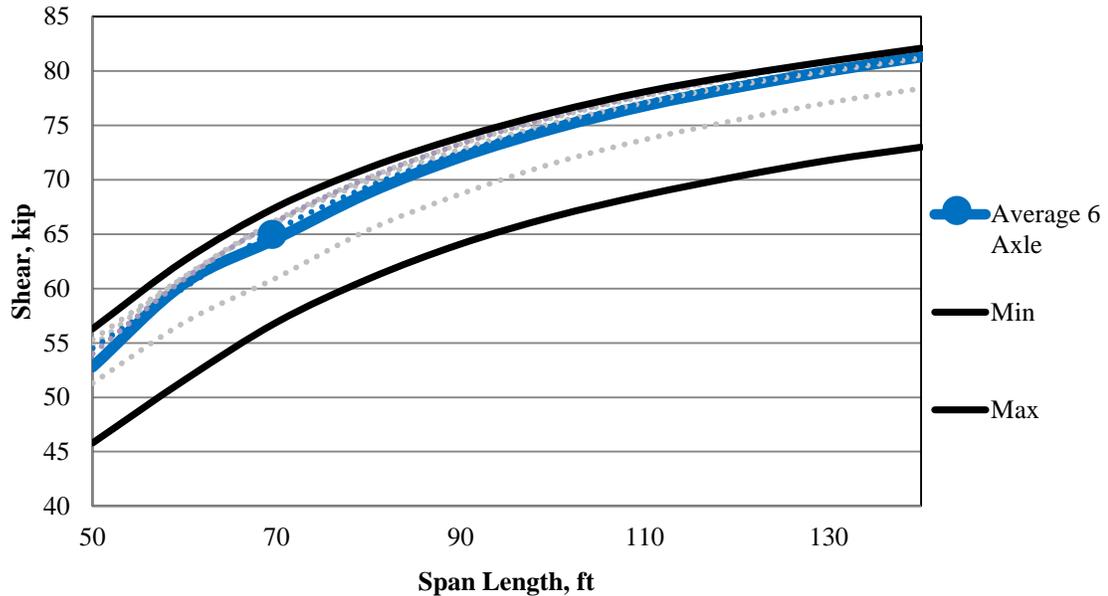


Figure 6-34: Shear envelope of the six-axle logging trucks compared to the average six-axle vehicle

There is more scatter in the shear plot between individual vehicles compared to the moment plot. Two vehicles had considerably lower shear effects than the other vehicles. As with the moment comparisons, the average six-axle vehicle looks to be a good average representation of the vehicles. It has a shear effect very similar to the majority of the vehicles measured, however, it is not conservative.

Next, the HS20 and the Wisconsin permit vehicle were compared to the six-axle individual logging trucks as shown in Figure 6-35. The main thing to note is that for small spans less than approximately 70ft, all the six-axle logging trucks produce a moment less than the HS20. Additionally, for span ranges greater than 120ft, all the six-axle logging vehicles produce moments greater than the HS20.



Figure 6-35: Moment envelope of the six-axle vehicles compared to the HS20

Figure 6-36 shows the shear comparison between the individual logging vehicles and the HS20. For very small spans, less than approximately 53ft, all the logging trucks produce shear effects less than the HS20. For longer spans, exceeding 90 ft. the individual logging trucks have shear effects that exceed the HS20. The majority of the vehicles are grouped together and begin to exceed the HS20 in terms of shear effects around a span range of 55ft-65ft.

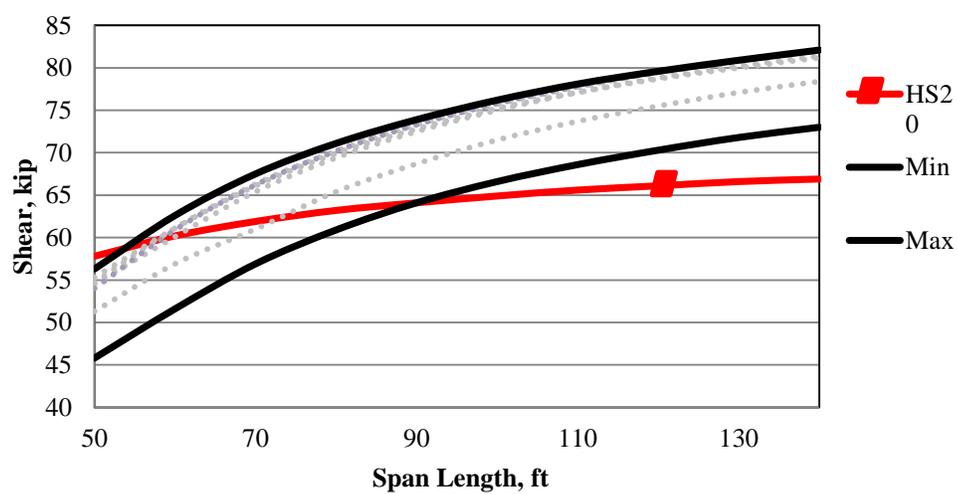


Figure 6-36: Shear envelope for the six-axle logging vehicles compared to the HS20

The final comparison completed with the individual logging trucks was comparing the six-axle vehicles to the Wisconsin permit vehicle, which was designed to simulate the weight effects allowed by annual permits. Figure 6-37 shows the moment comparison of these vehicles.

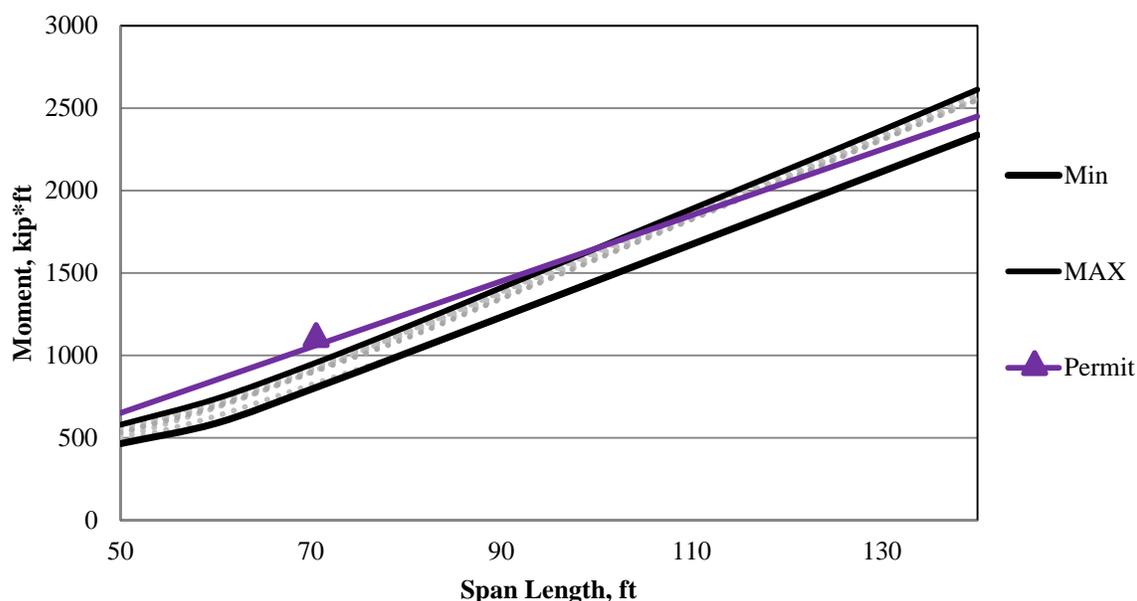


Figure 6-37: Moment envelope for six-axle vehicles compared to the Wisconsin permit vehicle

For small span ranges, the Wisconsin permit vehicle has moment effects that exceed all the individual vehicles. This is accurate for spans to approximately 90ft. The majority of the individual logging trucks do begin to exceed the permit vehicle at spans around 100ft. One measured vehicle, however, had moment effects that are still less than the permit vehicle up to spans of 140ft.

Lastly, the shear comparisons were done between the individual six-axle logging trucks and the Wisconsin permit vehicle. This comparison is shown in Figure 6-38.

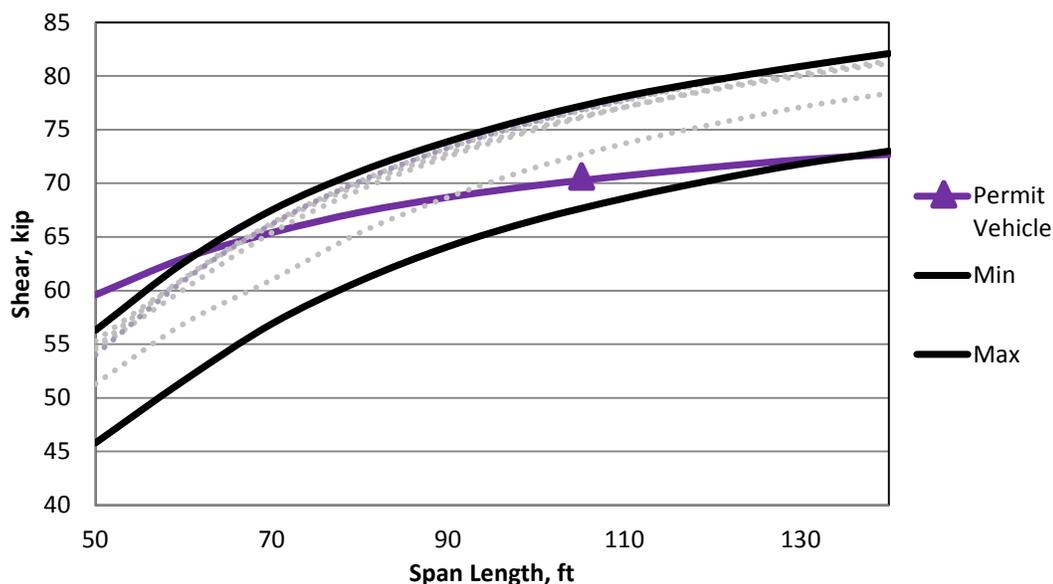


Figure 6-38: Shear envelope for six-axle vehicles compared to the Wisconsin permit vehicle

Similar to all permit vehicle shear comparisons, the Wisconsin permit vehicle has a larger shear force than all the measured six-axle vehicles up to spans of approximately 60ft. As span length increases, more and more of the six-axle vehicles begin to exceed the permit vehicles shear effects and at a span of 140 ft all nine of the measured vehicles have larger shear forces.

6.7 Comparison of Wisconsin Truck Size and Weight Study Proposed Vehicles

In Section 3.4 six candidate vehicles were discussed which were created as part of the Wisconsin Truck Size and Weight study completed on January 1, 2009. Of these six vehicles, two had similar configurations to the logging trucks measured in the field for this project. Those two configurations are the six-axle 98,000-lb. tractor semi-trailer and the six-axle 98,000-lb. straight truck and pup trailer.

For the 98,000-lb, six-axle tractor semi-trailer, comparisons were made with the average six-axle logging vehicle as well as the HS20 and the Wisconsin permit vehicle.

Figure 6-39 shows the maximum moment vs. span length curve for these four vehicles for spans from 20ft to 170ft.

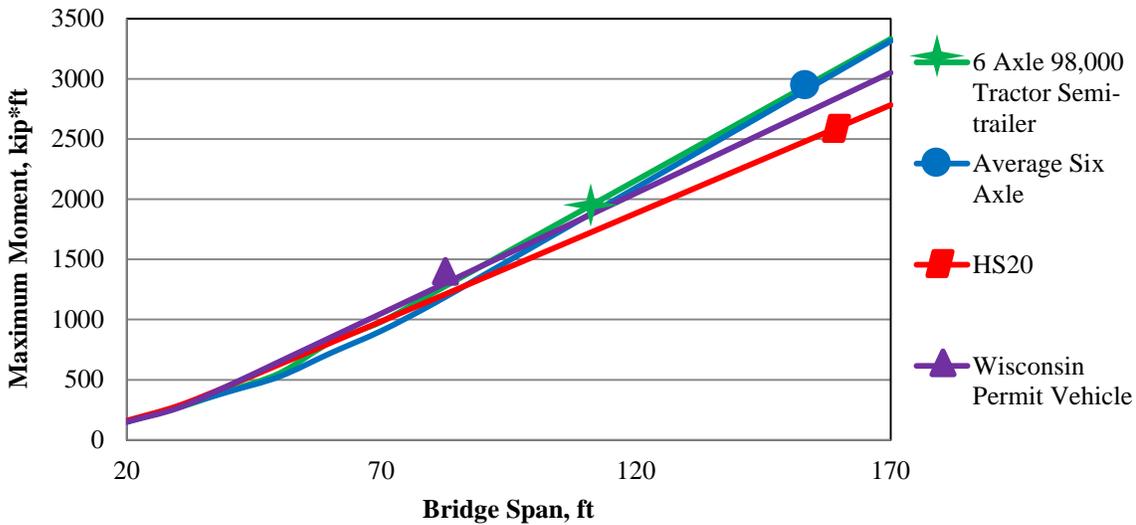


Figure 6-39: Moment Comparison including the proposed 6 Axle 98,000lb Tractor Semi-trailer

From this figure, it can be seen that for spans greater than 60ft the average six-axle logging vehicle produces a greater moment than the proposed six-axle tractor semi-trailer vehicle. Additionally, the proposed vehicle appears to have upper and lower bounds when compared to the HS20 design vehicle and the Wisconsin permit vehicle. The next two figures look more closely at these upper and lower bound limits.



Figure 6-40: lower bound limits for TSW Six-axle 98,000-lb Tractor Semi-Trailer

Figure 6-40 shows the lower bound limits for the proposed six-axle 98,000-lb tractor semi-trailer vehicle. Compared to the HS20 design vehicle the proposed vehicle produced larger moments up to spans of 34.5ft. For spans larger than 34.5ft, the HS20 produces a higher moment up until the upper bound limit. When comparing the proposed vehicle to the Wisconsin permit vehicle the proposed vehicle creates a higher moment for spans up to 36.2ft.

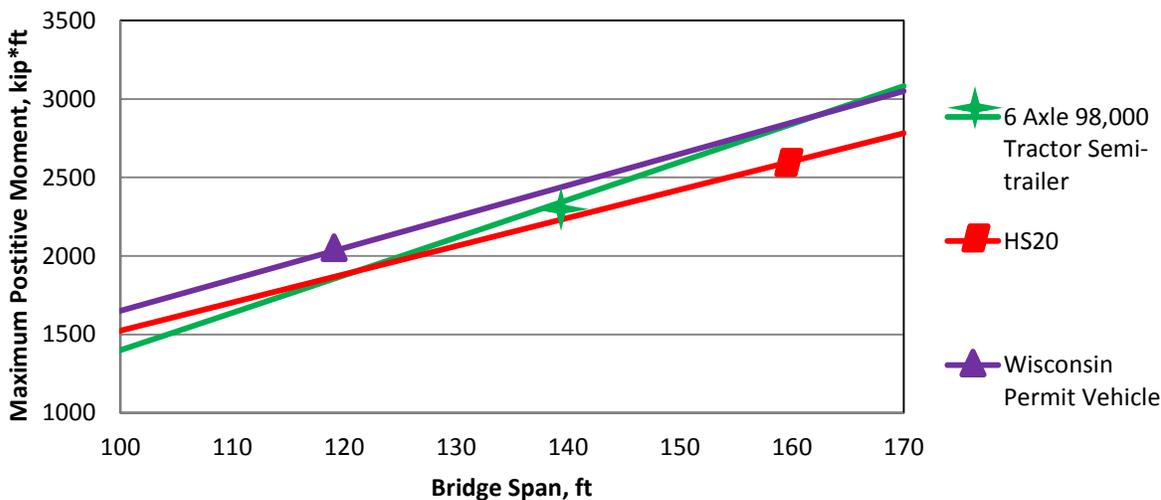


Figure 6-41 shows the upper bound limits of the proposed vehicle.

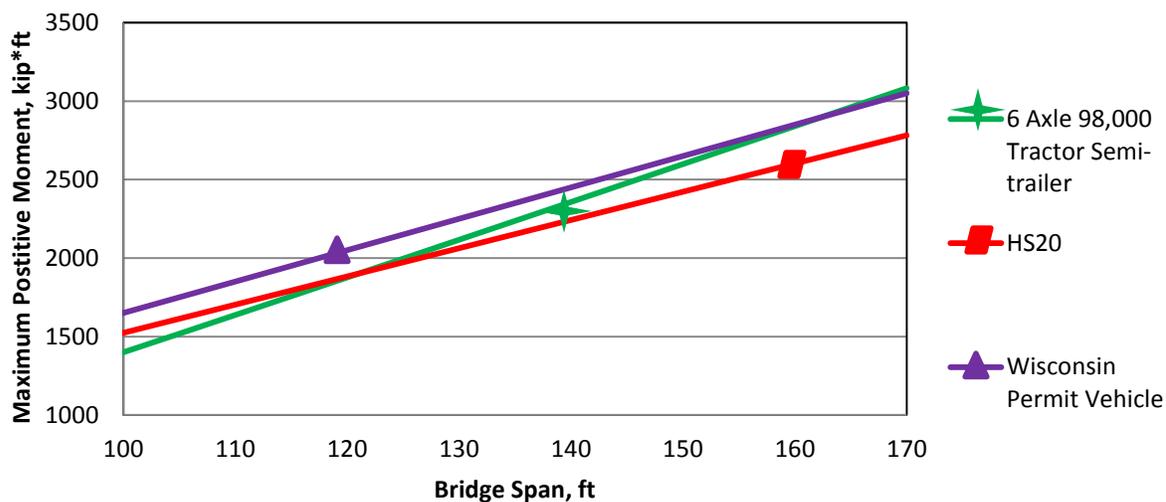


Figure 6-41: upper bound limits for proposed Six-axle 98,000lb tractor Semi-Trailer

The upper bound limits occur at a span range of approximately 120ft when comparing to the HS20 and approximately 165ft when comparing to the Wisconsin permit vehicle.

In conclusion, looking at moment effects alone, the proposed vehicle is acceptable on larger span ranges than the average six-axle logging vehicle. The span range when the proposed vehicle creates a smaller moment compared to the HS20 is from 34.5ft to 120ft and when comparing the proposed vehicle to the Wisconsin permit vehicle the span range for which the proposed vehicle is acceptable is from 36.2ft to 165ft. Looking at the average logging vehicle, the proposed vehicle creates a lower moment than the average logging vehicle from spans greater than 60ft.

Next, shear comparisons were made between the same four vehicles which are shown in Figure 6-42. First, a comparison between the average six-axle logging vehicle and the proposed six-axle vehicle was made. The proposed vehicle produces a higher shear force up to span lengths of approximately 40ft and from then on the average six-axle logging vehicle produces a higher shear force.

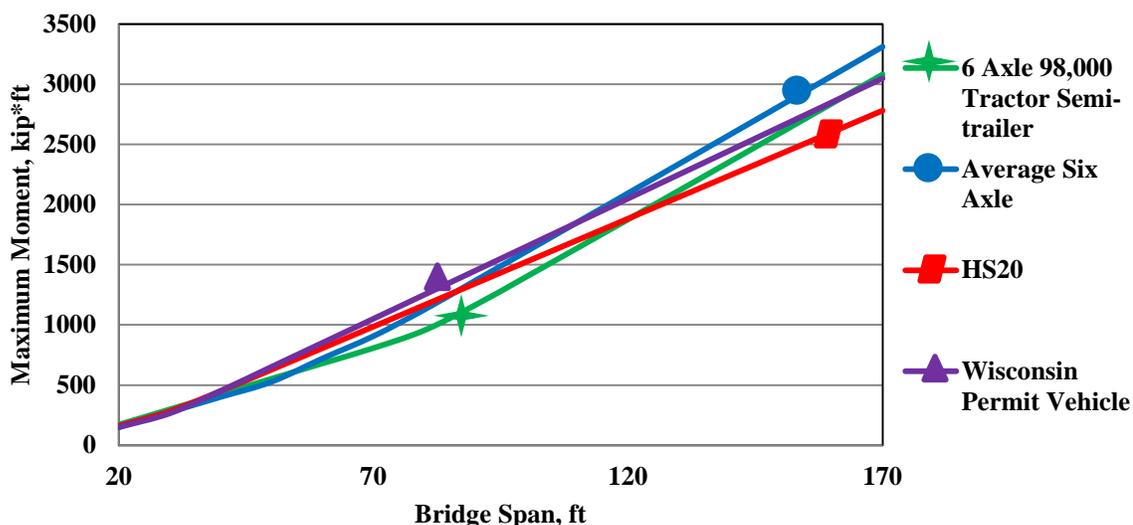


Figure 6-42: Shear Comparison including the proposed Six-axle 98,000lb Tractor Semi-trailer

Next, the HS20 and the proposed vehicle were compared. The proposed vehicle produces a smaller shear force up to spans of 80ft. After 80ft the proposed vehicle's force effects are larger. The final shear comparison was between the proposed vehicle and the Wisconsin permit vehicle. The proposed vehicle produces a smaller shear force than the Wisconsin permit vehicle up to spans of just under 100ft. Once span lengths exceed 100ft the proposed vehicle will have higher shear forces.

In terms of moment effects, the proposed vehicle creates a smaller moment compared to the average six-axle logging vehicle for spans greater than 60ft. The average six-axle logging vehicle only produced a smaller moment for spans ranging from 20ft to 60ft. Similar results were found for the shear comparison. The proposed vehicle produces higher shears at low span lengths, up to 40ft. For larger spans the average six-axle logging vehicle produces the larger shear effects. Overall, the two vehicles are producing moment and shear effects that are relatively close to one another for spans ranges from 20ft to 170ft. Although slightly

not conservative for long spans, the proposed vehicle is a reasonable representation of the logging vehicles that were measured.

The next vehicle that resembled the logging vehicles configurations was the six-axle truck and pup configuration proposed within the Wisconsin TSW study. For this vehicle, ranges for the axle spacing were provided for the back two axles. For this analysis, the smallest possible axle spacing was used as this was the conservative approach. Table 6-6 shows the distances and loads used for this analysis.

Table 6-6: Axle Load and Spacing for the TSW study Proposed Six-axle 98,000-lb Truck and Pup

Distance from previous axle (ft)	Load on Axle (kip)
0.00	18.00
11.00	15.33
9.00	15.33
4.33	15.33
8.00	17.00
16.00	17.00

Only three six-axle truck and pup vehicles were measured in the field and therefore an average vehicle was not created because there was not enough data to get an accurate representation. That being said, the proposed vehicle was compared to the three individual vehicles which had the six-axle truck and pup configuration. First, a moment comparison was done for spans ranging from 40ft to 150ft as shown in Figure 6-43.

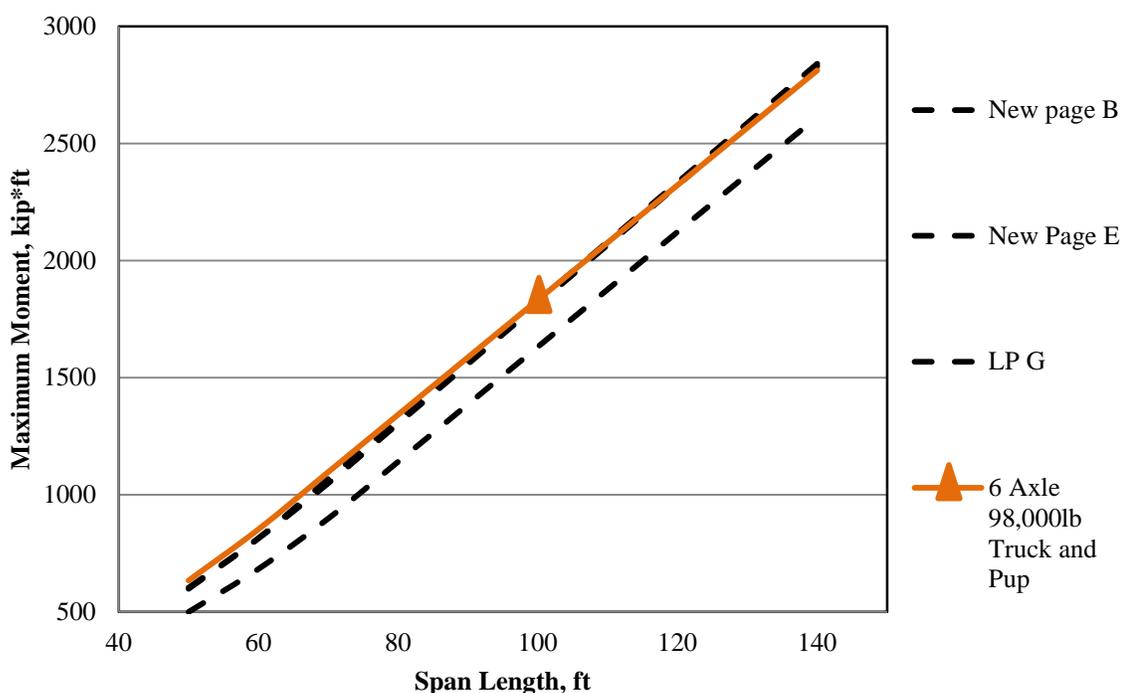


Figure 6-43: Moment Comparison of TSW Proposed Six-axle 98,000-lb Truck and Pup with individual Six-axle Truck and Pup vehicles (indicated by dashed lines)

The three dashed lines, two of which mostly overlap, represent the moment curves for the three six-axle logging truck and pup vehicles measured in the field. The solid line represents the TSW study proposed 98,000-lb truck and pup vehicle configuration. From this figure it is evident that one of the measured vehicles creates a smaller moment than the rest. The proposed vehicle produces a higher moment than all of the measured vehicles up to a span of 110ft. For spans greater than 110ft the proposed vehicle and two of the measured trucks produce a very similar moment effect. Once spans exceed 130ft, however, it becomes clear that the two measured vehicles produced moments which exceed the proposed 98,000-lb truck and pup configuration.

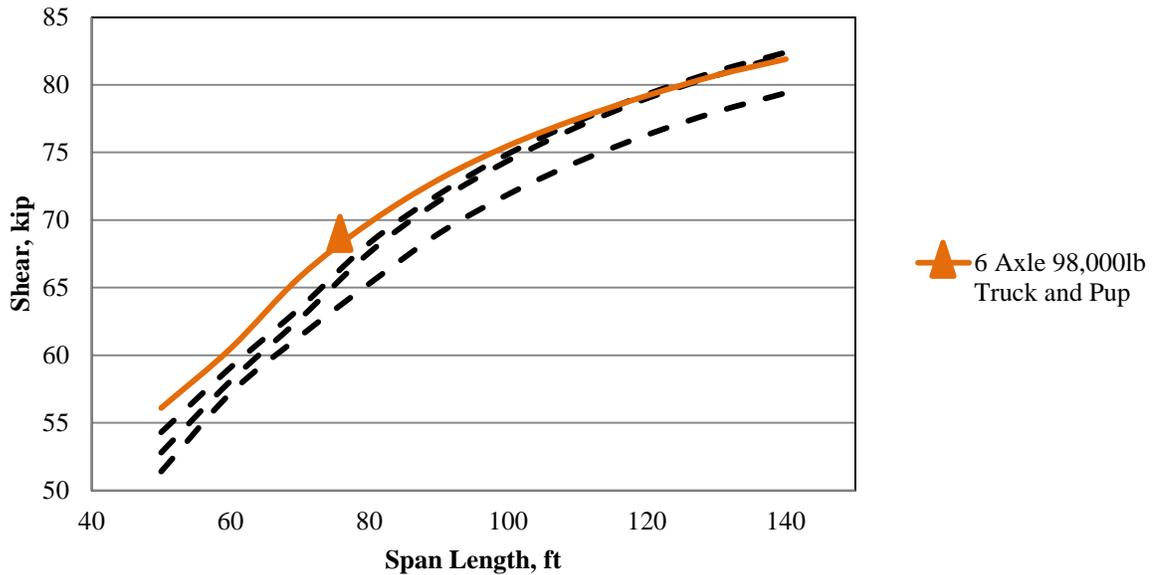


Figure 6-44: Shear Comparison of TSW Proposed Six-axle Truck and Pup with measured Six-axle Truck and Pup vehicles

Next, shear comparisons were completed. The results from this comparison are found in Figure 6-44. As with the moment plots, one of the measured trucks appears to have a much lower shear force than the other two. The remaining two measured vehicles have very similar shear force curves which are also relatively similar to the proposed six-axle, 98,000-lb truck and pup vehicle. The proposed truck creates a higher shear force than all the measured logging vehicles up to spans of approximately 120ft.

In order to determine if this proposed vehicle configuration is a good representation of six-axle truck and pups using in the timber industry more logging vehicles would need to be measured. Based on the comparisons that were completed the proposed six-axle truck and pup in the configuration used was a slightly conservative model for shorter spans. For longer spans the proposed vehicle produced moment and shear forces very similar to the measured logging vehicles.

6.8 Comparison of Wisconsin DOT Specialized Permit Vehicles

In addition to the AASHTO vehicles used in load rating bridges, Wisconsin has two state vehicles that are also used to determine load posting. Both of these vehicles have a gross weight of 98 kip. The first vehicle is the PUP unit weight vehicle and the second vehicle is the Semi Unit weight vehicle. Configurations for both of these vehicles are shown in the figure below.

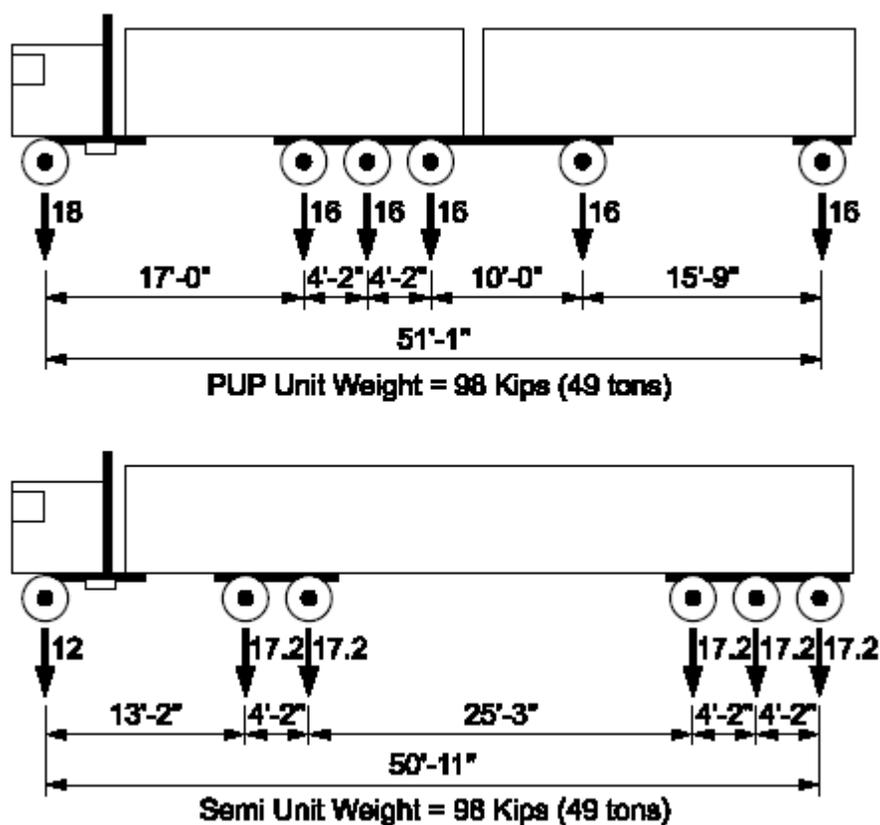


Figure 6-45: Configuration of WisDOT Specialized Permit Vehicles (Wisconsin Department of Transportation, 2008)

A comparison of these two vehicles and the logging vehicles that were measured was completed to see if these vehicles are an accurate representation of the logging vehicles currently being used. First, the PUP unit weight vehicle was compared to the three six axle

truck and pup vehicles. Looking at moment effects, the WisDOT PUP vehicle produced similar effects to the three measured vehicles and is a good representation of what is currently being used. The figure below shows the moment comparison results. The three logging vehicles that were measured are shown in black dashed lines and the WisDOT vehicle is shown in a solid red line.

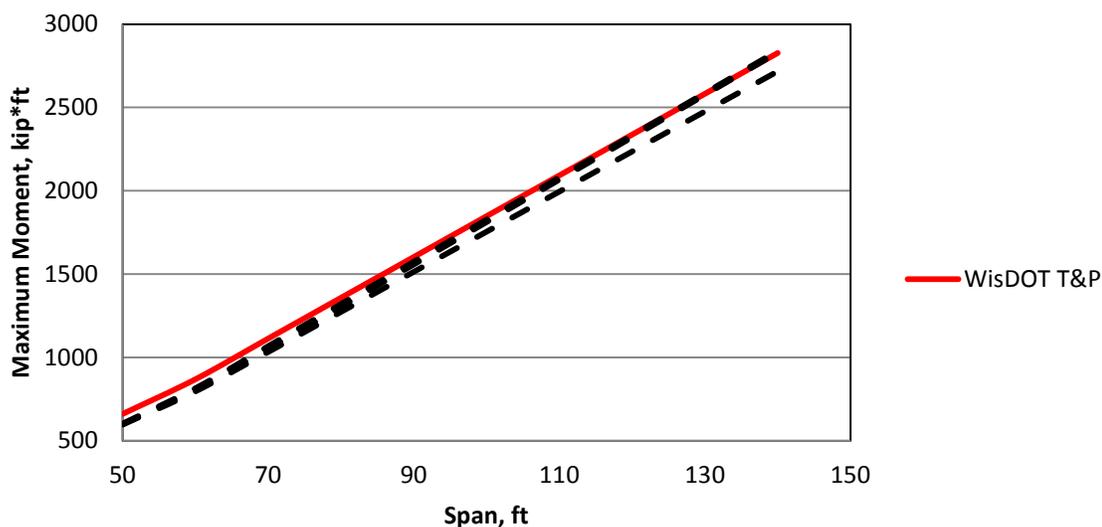


Figure 6-46: Moment Comparison of the WisDOT Specialized PUP Vehicle and the Measured Six Axle Truck and Pup vehicles

Shear effect comparisons of these same vehicles were also completed. The WisDOT vehicle creates shear forces similar to the other measured vehicles, however, at larger spans, two of the measured vehicles created slightly higher shear effects. Based on this, the WisDOT PUP vehicle is not conservative for all six axle truck and pup. The following figure shows the shear comparison results.

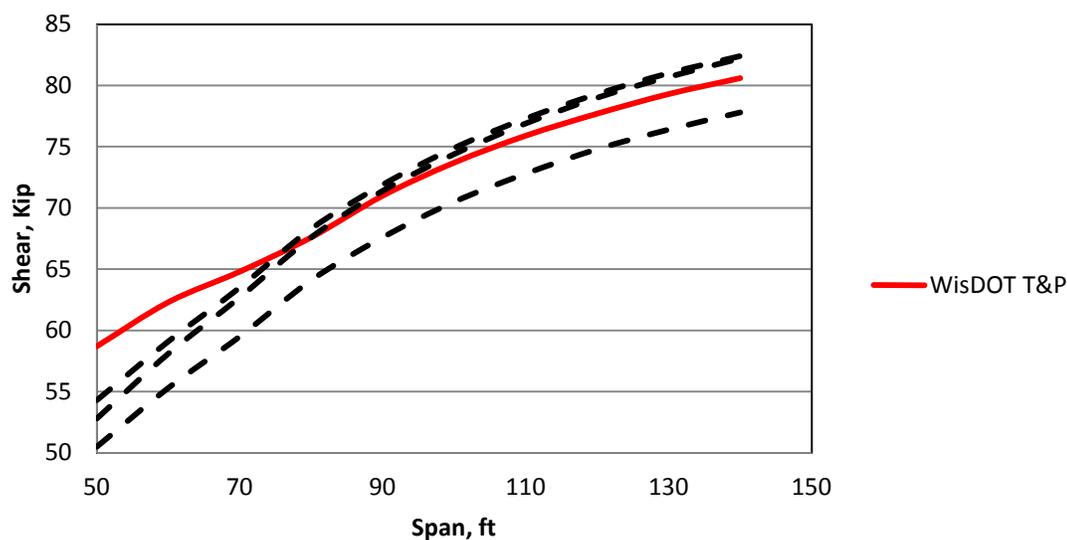


Figure 6-47: Shear Comparison of the WisDOT Specialized PUP Vehicle and the Measured Six Axle Truck and Pup vehicles

The second vehicle that is used in addition to the AASHTO load rating vehicles is the six-axle, 98,000-lb semi unit. This vehicle was compared to the average six axle vehicle used in previous comparisons. Both the shear and moment results showed that for spans from 50ft to 140ft, the WisDOT semi produced higher moment and shear effects than the average six axle vehicle. The increase in effects was minimal and based on these results, the WisDOT semi appears to be a good representation of the six axle semi tractor trailer vehicles currently being used. Figure 6-48 and Figure 6-49 show the moment and shear comparison results for the WisDOT Specialized Semi vehicle and the average six axle vehicle.

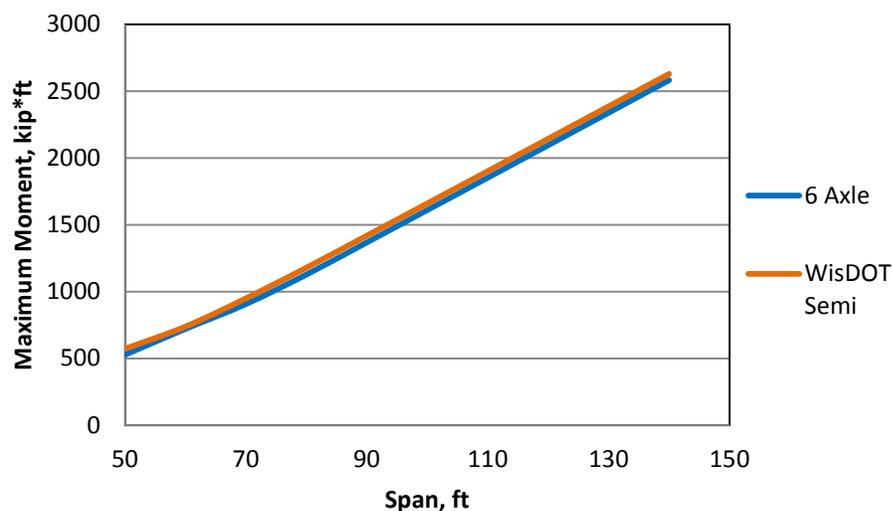


Figure 6-48: Moment Comparison of WisDOT Specialized Semi Vehicle and the Average Six Axle Truck

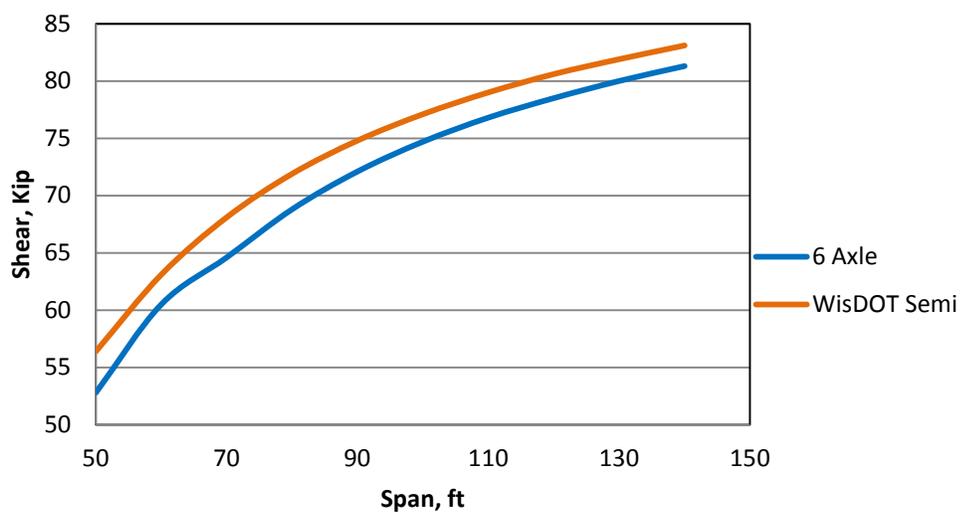


Figure 6-49: Shear Comparison of WisDOT Specialized Semi Vehicle and the Average Six Axle Truck

6.9 Comparison of Factored Design Vehicles and Average Logging Trucks

Up to this point, all the vehicles used in this analysis have been unfactored or service live loads. When looking at bridges that are load rated, the unfactored weight is the maximum allowable load that can travel on the bridge. This is the weight guaranteed to not

cause permanent damage to the bridge if the load is carried on the bridge indefinitely. However, when a bridge is designed, it is designed using a factored live load, which has effects on the bridge significantly higher than the unfactored load. This means that a bridge that is designed for an HS20 load can actually withstand much larger loads because it is designed for an amplified factored load as a factor of safety. With this in mind, the purpose of this comparison was to see how the effects of the average logging vehicles compare to the factored and unfactored live loads. The unfactored live loads show the legal limit for weight on a bridge, whereas the factored loads are likely the actual limits where damage to the bridge will occur. Both the unfactored and factored load effects were looked at for the HS15 and HS20 design vehicles.

In order to determine the factored loads for both the HS20 and the HS15, the load factors found in the 1994 AASHTO Bridge Specifications were used (American Association of State Highway and Transportation Officials, 1994). This older method of factoring was used because the older design vehicle, the HS20, was used in this analysis, rather than the current HL-93 design vehicle. The controlling load equation for the HS20 vehicle is as shown below.

$$N = \gamma * \beta_L * (M_{ll} + I) \quad (3)$$

The γ factor is the load factor which was equal to 1.3 and β_L is the live load coefficient which was taken as 1.67. Both of these factors were determined using Table 3.22.1A in the 1994 AASHTO Highway Bridge Specifications. The M_{ll} is simply equal to the moment effect for the given vehicle for a given span length and I is the impact factor. The same equation could be used calculating shear factored loads where M_{ll} would be replaced with V_{ll} . Using the 1994

AASHTO code, the impact factor is a function of the span length, L , of the bridge and is calculated using the following equation.

$$I = \min\left(0.3, \frac{50}{L+125}\right) \quad (4)$$

First, the moment comparison between the unfactored and factored HS20 and the average logging vehicles was completed. This comparison can be found in Figure 6-50.

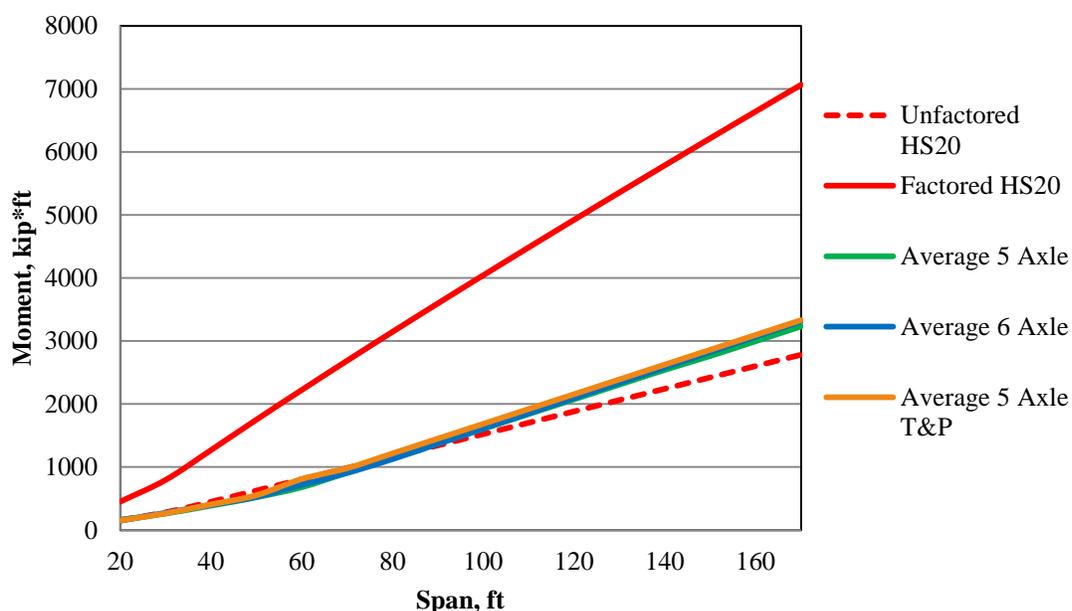


Figure 6-50: Moment Comparison of Unfactored and Factored HS20 and the Average Logging Vehicles

Based on Figure 6-50, it is evident that the average logging vehicles are not producing effects near the factored HS20 vehicles for moment. The comparisons between the average logging vehicles and the unfactored HS20 were made previously.

Next, the same vehicles were compared for shear effects as shown in Figure 6-51. The results were very similar to the moment results in that the average logging trucks produce much smaller effects than the factored HS20 vehicle.

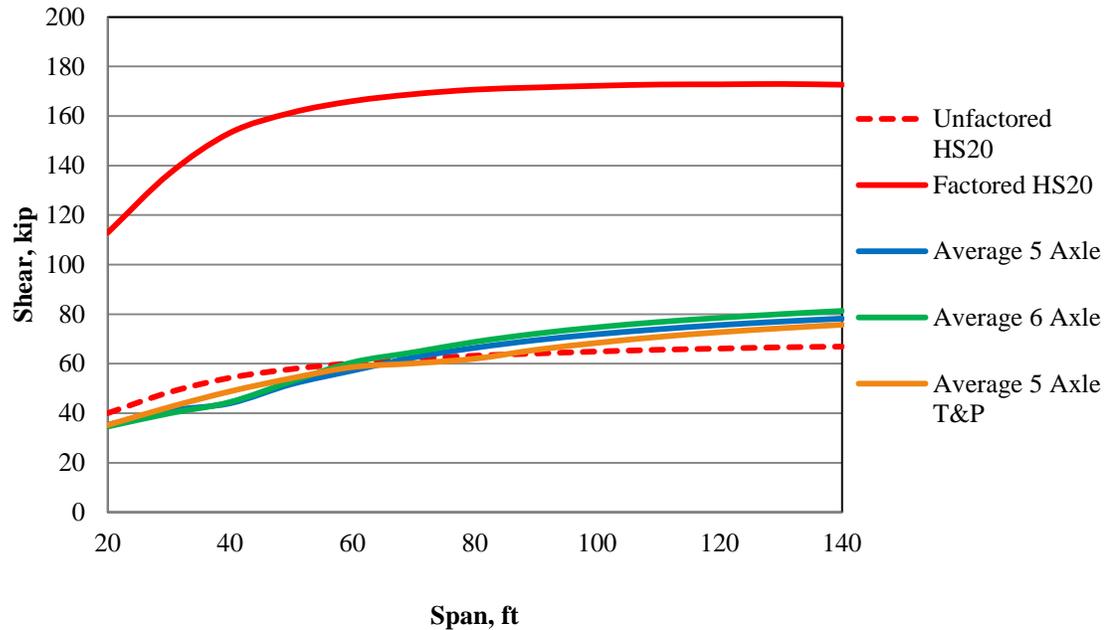


Figure 6-51: Shear Comparison of Unfactored and Factored HS20 and the Average Logging Vehicles

Based on the comparisons with the HS20 vehicle, it is very evident that the logging vehicles effects are magnitudes smaller than the factored HS20 vehicle meaning bridges designed for HS20 loads could safely carry the logging vehicles. This statement is valid for spans up to 140ft, however, based on the vehicle trend lines shown, it is likely this statement would hold true for all spans.

Next, similar comparisons were completed using the HS15 vehicle. The HS15 vehicle was chosen because it is a common load rating found on older deteriorated bridges. First, a moment comparison between the unfactored and factored HS15 was done with the three average logging vehicles as shown in the figure below.

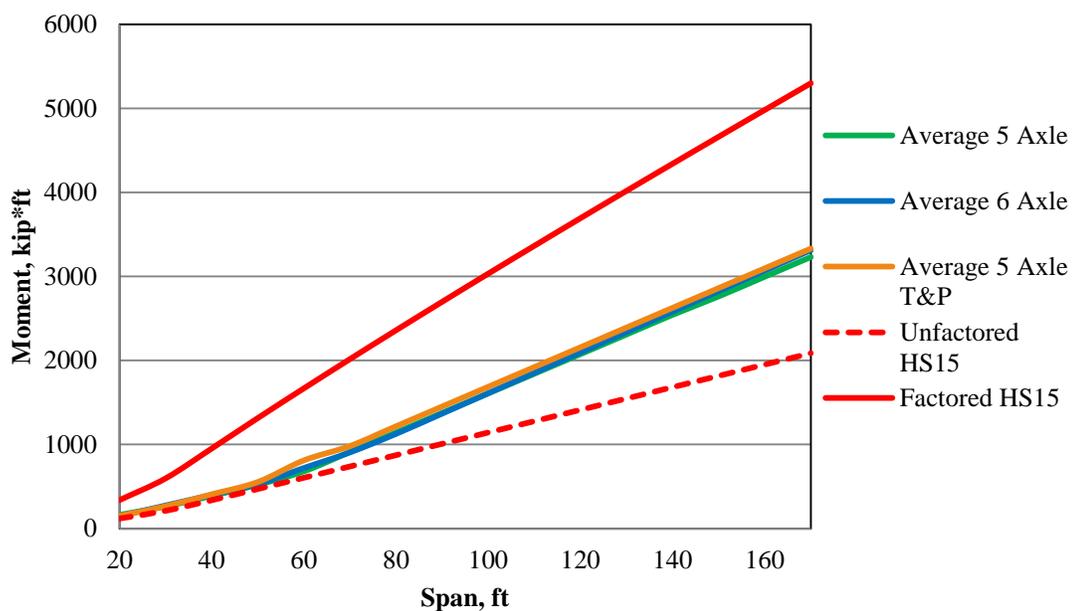


Figure 6-52: Moment Comparison of Unfactored and Factored HS15 and the Average Logging Vehicles

The results shown in Figure 6-52 showed that the logging vehicles never produce moment effects near the magnitude of the factored HS15 vehicle. The final comparisons which were done for this analysis looked at the shear effects of the unfactored and factored HS15 and the average logging vehicles. The results of this comparison are shown in Figure 6-53.

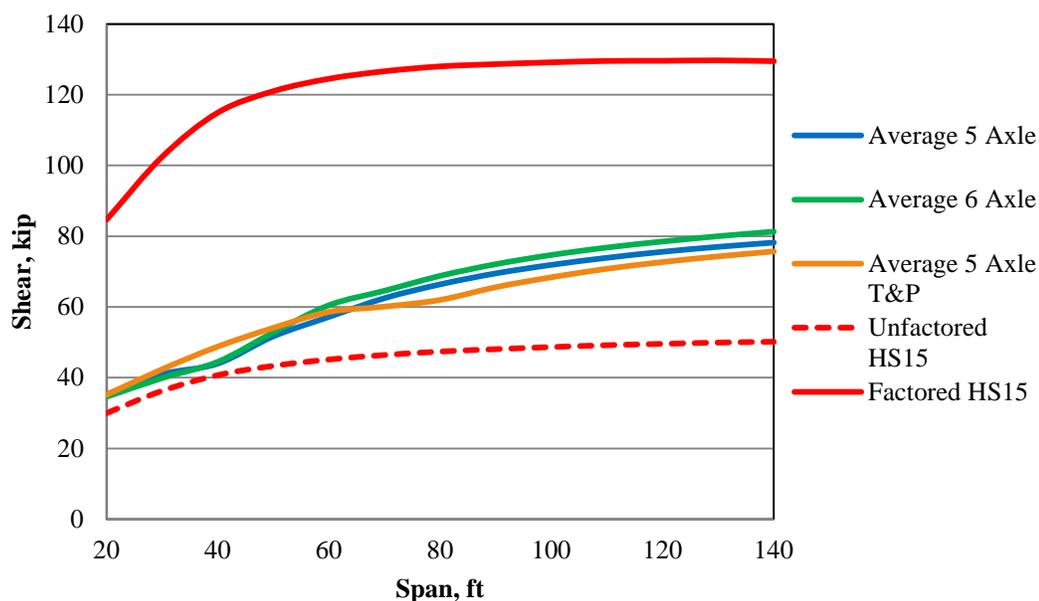


Figure 6-53 Shear Comparison of Unfactored and Factored HS15 and the Average Logging Vehicles

The results shown in Figure 6-53 were very similar to those found in the previous moment comparison. The shear effects of the average logging vehicles are always less than the factored HS15 vehicle and are always greater than the unfactored HS15 vehicle. Overall, all the results from this analysis were promising. The effects of the average logging vehicles are much less than both the shear and moment effects produced by the factored HS15 and HS20 design vehicles. This implies that although the logging vehicles may exceed the allowable load based on the unfactored service loads posted, they have effects much less than the factored posted loads and as a result may not cause permanent damage to the bridge.

6.10 Percentage of Force Effects of Average Logging Trucks

In previous comparisons, the average logging trucks moment and shear effects were compared to the effects of the HS20 design vehicle and the Wisconsin permit vehicle. From these comparisons, the span ranges for which each of the logging vehicles began to have effects exceeding the HS20 and permit vehicle were determined. The previous comparisons

did not look at how much the logging effects were larger than the HS20 and permit vehicle. In this section, the percent difference between the effects of the logging trucks and the HS20 and the permit vehicle were analyzed. This analysis looked at the maximum effects for a given span and was independent of moment or shear. This was completed for both the HS20 and permit vehicle unfactored and factored loads. The factoring loads were calculated using the same method described in Section 6.8.

First, the three average logging trucks were compared to the unfactored HS20 and Wisconsin permit vehicle. The following table shows the results. A value in red indicates that the logging vehicle produces a higher effect than the HS20 or permit vehicle at that given span length and will be equal to a percentage greater than 100%.

Table 6-7: Percentage of Force Effects Results for Unfactored Service Loads

Span	5 Axle		6 Axle		5 Axle T&P	
	HS20	Permit	HS20	Permit	HS20	Permit
20	102.19%	109.00%	93.00%	99.20%	95.38%	101.73%
30	93.37%	99.58%	97.41%	103.89%	94.40%	100.68%
40	86.53%	86.20%	89.13%	88.79%	90.66%	90.32%
50	89.45%	86.74%	91.35%	88.59%	93.60%	90.77%
60	95.02%	90.79%	100.50%	96.03%	100.60%	95.33%
70	100.97%	95.57%	104.36%	98.78%	99.67%	93.47%
80	105.06%	98.66%	108.86%	102.23%	104.48%	97.31%
90	108.42%	101.16%	112.48%	104.95%	108.00%	100.08%
100	110.79%	103.01%	115.10%	107.02%	110.69%	102.19%
110	112.65%	104.53%	117.07%	108.63%	112.80%	103.84%
120	114.37%	105.73%	118.76%	109.79%	114.51%	105.17%
130	115.62%	106.65%	120.12%	110.80%	115.92%	106.27%
140	116.89%	107.57%	121.52%	111.83%	117.10%	107.18%

One thing to note from this table is that the maximum percentage of force effect is equal to 121.52%. This is for a 140ft span comparing the force of the HS20 to the average six axle vehicle. This indicated that up to spans of 140ft, the logging trucks largest force effects

is only 22% larger than the HS20 design vehicle. Although it is producing effects higher than the HS20, the force effects of the average logging vehicle are not excessively larger.

In addition to the table, the following figures show the force effects of the average logging vehicles compared to the HS20 and the Wisconsin permit vehicle. A red line was drawn on the graph to indicate when the logging vehicles and the comparison vehicle have equal effects. If the logging vehicle trend lines go above this line, this means the effects of the logging vehicle are higher than the comparison vehicle, and if they are below the red line, this means the effects of the logging vehicle are less than the vehicle it's being compared to.

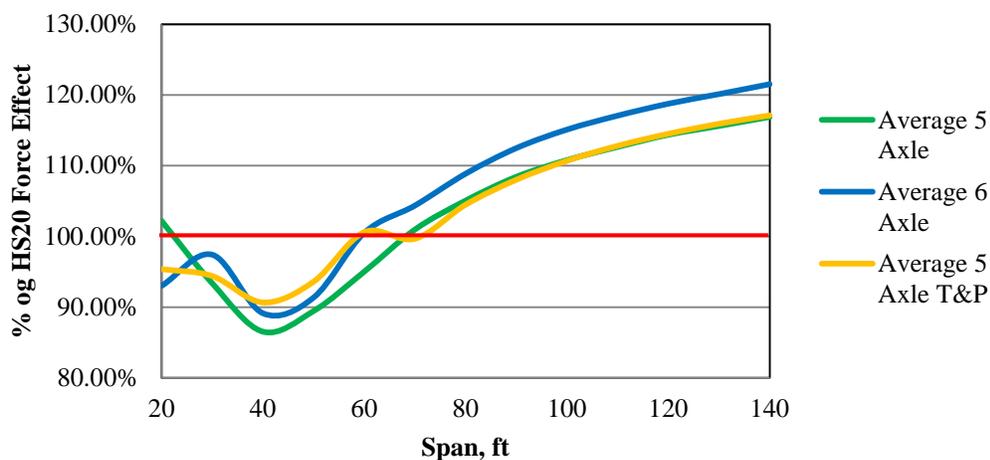


Figure 6-54: Percentage of Force Effects of the Average Logging Vehicles compared to the HS20 Vehicle

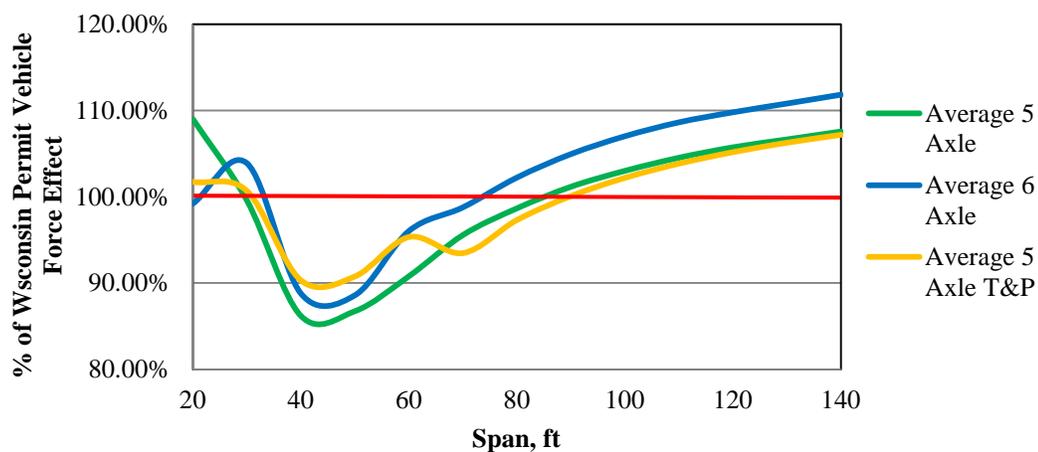


Figure 6-55: Percentage of Force Effects of the Average Logging Vehicles compared to the Wisconsin Permit Vehicle

In addition to comparing the logging vehicles to the service loads of the HS20 and the Wisconsin permit vehicle, a force effects comparison was also done using factored loads for the HS20 and the Wisconsin permit vehicle. The following table shows the results for this comparison. The main point to take away from this table is that for spans from 20ft to 140ft, the average logging vehicles always have force effects less than the HS20 and Wisconsin permit vehicle. Additionally, the logging vehicles effects are less than 50% of the effects for the factored loads meaning the logging trucks are creating half of the effects of the factored HS20 and Wisconsin permit vehicle.

Table 6-8: Percentage of Force Effects Results for Factored Loads

Span	5 Axle		6 Axle		5 Axle T&P	
	HS20	Permit	HS20	Permit	HS20	Permit
20	36.21%	38.62%	34.52%	35.15%	33.79%	36.05%
30	33.08%	35.28%	31.58%	36.81%	33.45%	35.67%
40	30.66%	30.54%	30.07%	31.46%	32.12%	32.00%
50	32.04%	31.08%	32.73%	31.74%	33.53%	32.52%
60	34.45%	32.92%	36.44%	34.82%	36.48%	34.57%
70	37.02%	35.04%	38.26%	36.21%	36.54%	34.27%
80	38.90%	36.53%	40.31%	37.86%	38.69%	36.03%
90	40.52%	37.81%	42.03%	39.22%	40.36%	37.40%
100	41.75%	38.82%	43.38%	40.33%	41.72%	38.51%
110	42.79%	39.70%	44.47%	41.26%	42.84%	39.44%
120	43.75%	40.45%	45.43%	42.00%	43.80%	40.23%
130	44.52%	41.07%	46.26%	42.67%	44.64%	40.92%
140	45.30%	41.68%	47.09%	43.33%	45.38%	41.53%

In addition to the table above, Figure 6-56 and Figure 6-57 also show the percentage of force effects of the logging trucks compared to the factored HS20 and the factored Wisconsin permit vehicle. As was also identified in the table, the logging trucks are always less than the effects of the HS20 and the Wisconsin permit vehicle and additionally, the effects are always less than half of the design vehicle effects.

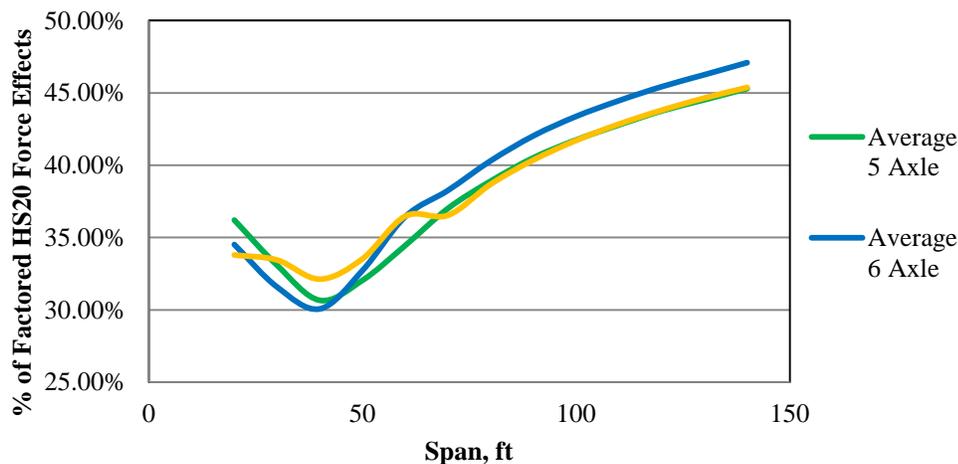


Figure 6-56: Percentage of Force Effects of the Average Logging Vehicles compared to the Factored HS20 Vehicle

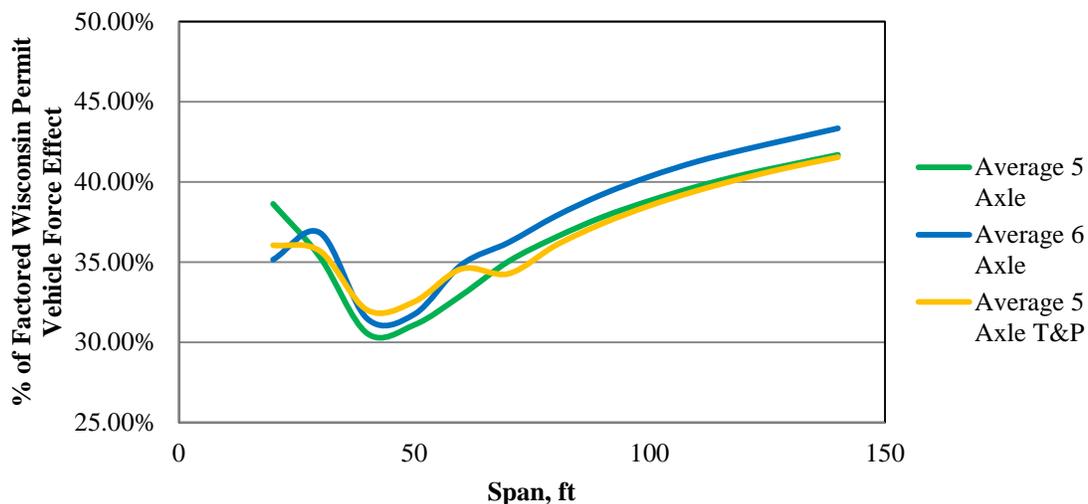


Figure 6-57: Percentage of Force Effects of the Average Logging Vehicles compared to the Factored Wisconsin Permit Vehicle

Based on this analysis, a few conclusions can be made. First, when looking at the unfactored analysis, there are spans which the logging vehicles will exceed the force effects of the HS20 and the Wisconsin permit vehicle, however up to spans of 140ft, the logging truck effects are never more than 20% larger than the design vehicles. Next, when looking at the logging vehicle compared to the factored loads, it was identified that the logging vehicles force effects are less than 50% of the factored loads up to spans of 140ft. Based on this trend, it is likely the logging vehicles will produce effects less than the factored loads for all span lengths, which means that bridges designed for the HS20 loading will not experience damage due to the logging truck force effects.

6.11 Summary of Logging Truck Analysis

Looking at the comparison of design vehicles, it was common that the effects produced by the HL-93 were greater than those produced by the average logging vehicles. This is promising in that new bridges will be designed to withstand the effects of most logging vehicles without experiencing damage. Two main conclusions were made based on the average logging truck analysis. First, for long span ranges, the average logging truck's effects on bridges will exceed those of the HS20 and the Wisconsin permit vehicle. The second conclusion is that the effects of the logging trucks are always greater than the HS15 truck regardless of the span length. This means that logging trucks will not be able to travel on bridges load rated at HS15 without the possibility of risking permanent damage to the bridge.

Several conclusions were also made based on the individual logging truck comparisons. First, it was determined that both the five and six-axle average logging trucks were good representations of the logging trucks measured in the field. Additionally, these average vehicles were slightly conservative for the majority of the actual trucks. There was one outlier in the five-axle comparisons, which had a gross vehicle weight of 96,660 lbs. This vehicle clearly demonstrated that overload vehicles have substantially higher effects on bridges.

Additionally, comparisons were completed between the measured vehicles and two of the proposed candidate vehicle configurations as a part of the Wisconsin TSW study, the six-axle 98,000-lb. vehicle and the six-axle 98,000-lb truck and pup configuration. Both of the vehicles were compared to the measured logging vehicles that matched the configuration. In the truck and pup configuration only three six-axle truck and pup's were measured. For most

span lengths these three logging vehicles produced moments and shear effects less than the proposed vehicle. For the six-axle 98,000-lb tractor trailer combination the main comparison was done using the average six-axle logging vehicle. Except for small span ranges less than 50ft, the proposed six-axle tractor trailer combination produced smaller moment and shear effects than the average six-axle logging vehicle. This means that based on the trucks measured for this project, the proposed six-axle 98,000-lb truck and pup appears to be a conservative model and the six-axle tractor trailer proposed vehicle seems to be non-conservative, especially with large span lengths.

The next comparisons that were done looked at factored loadings for the HS20 and the Wisconsin permit vehicle. When compared to the average logging vehicles, it was identified that the logging trucks produce effects which are much less than the factored load effects from the HS20 and the Wisconsin permit vehicle.

The final comparisons looked at the percentage of force effects of the logging vehicles compared to the HS20 and the Wisconsin permit vehicle. This comparison was completed independent of moment or shear and looked specifically at how the effect of the logging vehicles compared to the effects of the comparison vehicles. The results for the unfactored HS20 and Wisconsin permit vehicles showed that the effects of the logging trucks are larger for some spans, however the percentage of effects is not more than 20% larger to spans of 140ft. Lastly, looking at factored loadings for the comparison vehicles, the effects of the logging vehicles was always less than 50% of the HS20 and Wisconsin permit vehicle up to spans of 140ft.

7 Study of Current Load Ratings of bridges in Two Wisconsin Counties

Based on the truck analyses discussed previously it was determined that for some span ranges the moment and shear effects of the logging trucks were less than the HS20 design vehicle. Additionally, it was determined that the logging trucks would always exceed the allowable moment and shear effects of the HS15 vehicle. Which leads to the question of how many bridges in Wisconsin have a rating of HS20 and how many are at HS15 or less?

To go through inspection reports for all bridges in the State of Wisconsin was outside the scope of this project, however, two counties that have a good deal of logging traffic were chosen to assess the state of their bridges. Marathon County and Lincoln County were chosen and inventory ratings for the bridges in each of these counties were tabulated and analyzed.

Marathon County has a total of 360 bridges according to the Highway Structures Information HSI system database. This database is found on the Wisconsin DOT website. Of the 360 bridges, 96 had an inventory rating less than an HS20. This is equal to 26.7% of the bridges. Figure 7-1 shows the breakdown of the inventory ratings for every bridge in the county. The bottom axis lists the equivalent load capacity, i.e. HS16-17 indicates the inventory capacity is between an HS16 and HS17 truck load.

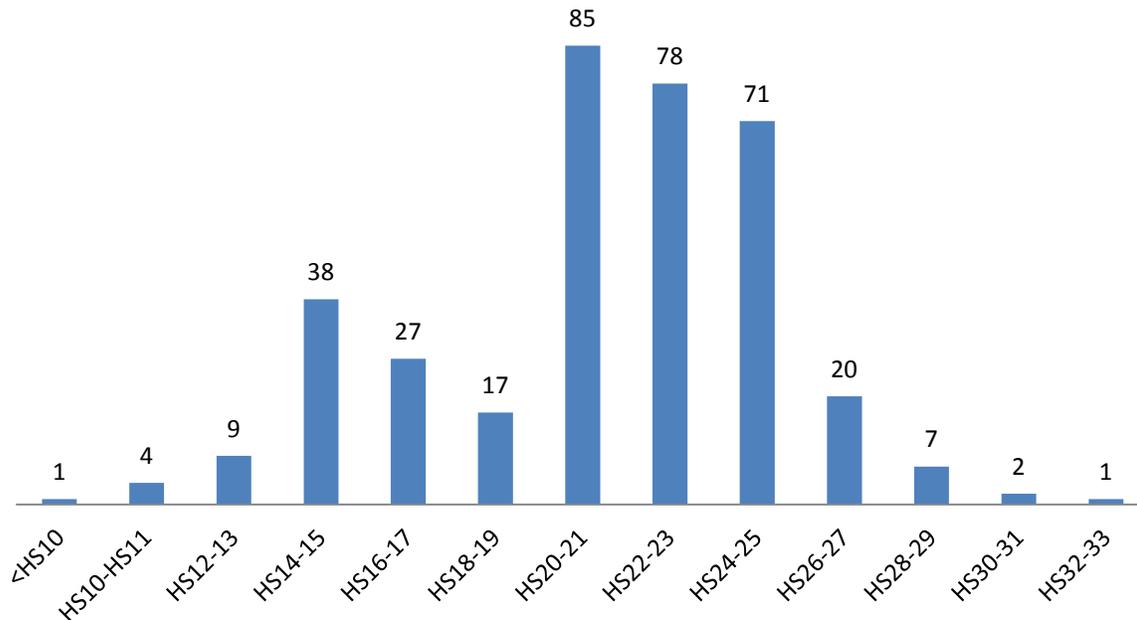


Figure 7-1: Marathon County Bridge Inventory Ratings

Marathon County includes the City of Wausau. It was considered that this could skew the results towards newer construction bridges. These new bridges could potentially balance out the older rural bridges and make the percentage of bridges with a rating of less than HS20 smaller. With this in mind Lincoln County was also chosen as it doesn't contain any large cities. In total, 121 bridges are reported in the HSI database for Lincoln County. Of these 121 bridges, only 22 of them were found to have load ratings less than an HS20. This is equivalent to 18.2% of the county bridges. Figure 7-2 shows the break down of the inventory load rating for bridges in Lincoln County.

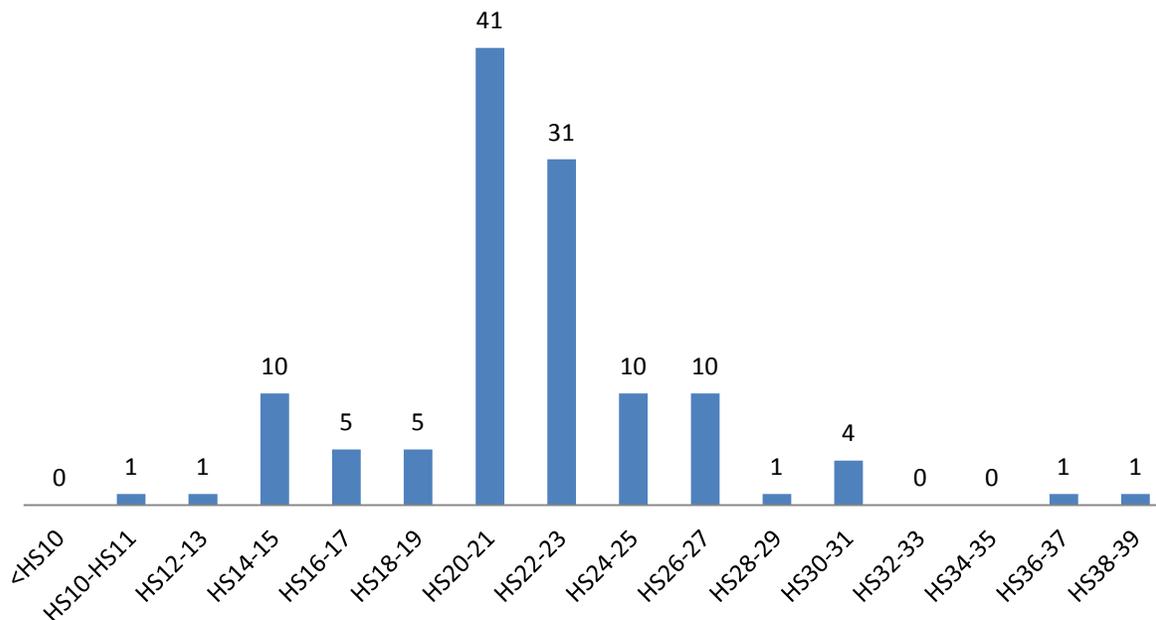


Figure 7-2: Lincoln County Bridge Inventory Ratings

The findings from both Marathon County and Lincoln County were surprising. It was originally expected that a higher percentage of bridges would have inventory load ratings less than an HS20. There was one final factor that may have skewed the results. A major highway with new bridges and overpasses, US-51, runs directly through both of these counties. It was questioned whether the highway bridges may be skewing the results and counterbalancing the deteriorated bridges in the more rural areas of the Counties. In order to check this hypothesis both of the County's bridges were tabulated again excluding bridges on US-51. Looking at the results from Marathon County, the percentage of bridges with inventory ratings less than an HS20 decreased to 23.7%, down from 26.7%. This change in percentage disproves the hypothesis. There are 238 bridges in Marathon County excluding US-51 bridges. Figure 7-3 shows the results of the inventory load rating for bridges in Marathon County excluding US-51 bridges.

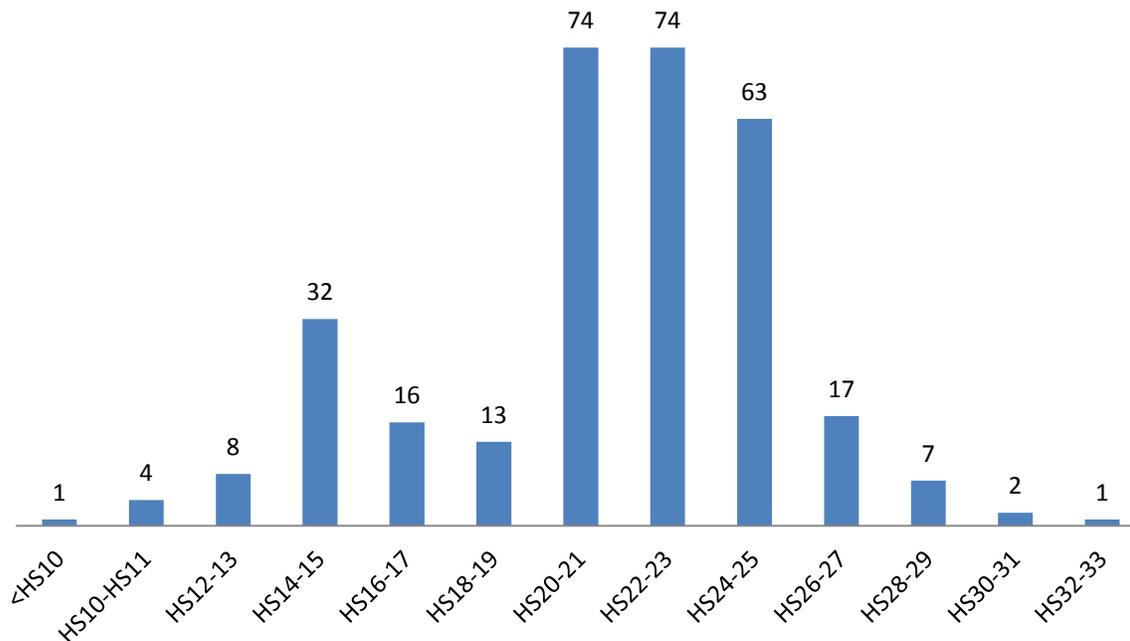


Figure 7-3: Marathon County Bridge Inventory Ratings Excluding US-51 Bridge

Lastly, the inventory ratings of bridges in Lincoln County were again analyzed, this time excluding bridges on US-51. The results yielded 20.2% of the 79 bridges had a rating of less than an HS20. This was just over two percent higher than the percentage when US-51 bridges were included. Overall, this is a small change and the bridges on US-51 do not have a large effect on the change in percentage of inventory ratings less than HS20. Figure 7-4 shows the results for Lincoln county inventory ratings excluding bridges on US-51.

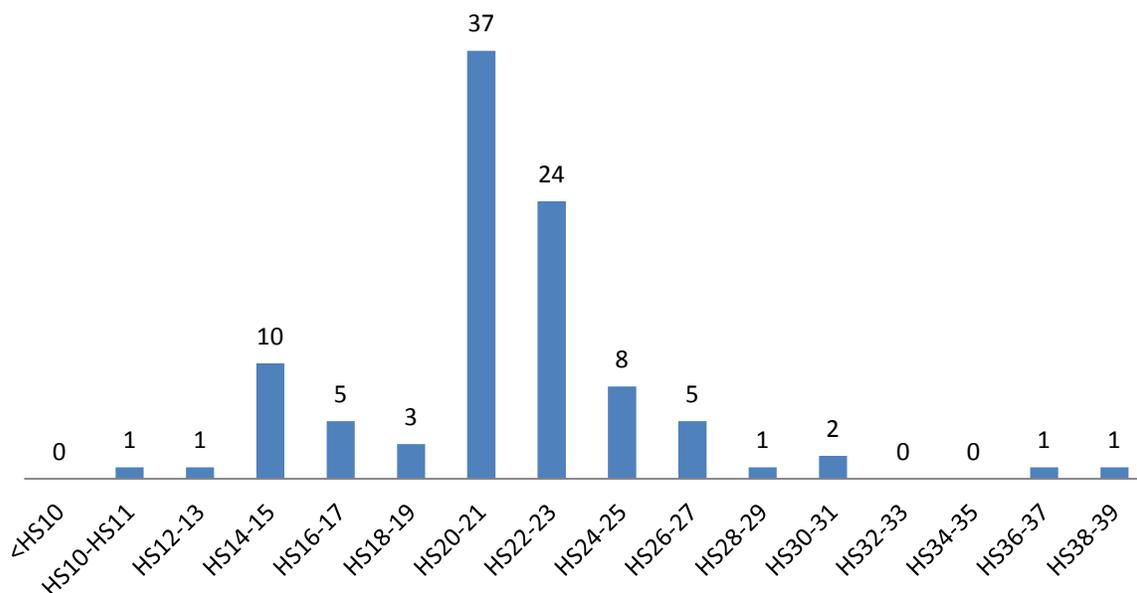


Figure 7-4: Lincoln County Bridge Inventory Ratings Excluding US-51 Bridges

Looking at the results from both Counties it doesn't appear as if the bridges on US-51 have a large effect on the percentage of bridges with load ratings less than an HS20. Additionally, the number of bridges with load ratings less than HS20 is lower than expected. This means that for the allowable span ranges discussed in Chapter 6, the logging trucks should not cause structural damage on a large percentage of bridges and the trucks should be able to use the majority of bridges in each county.

8 Specific Bridges of Interest to the Timber Industry

All of the bridges that were investigated for this project are bridges of concern to the Great Lakes Timber Professionals Association. Originally, a description of seventeen bridges that are currently affecting logging truck haul routes was provided by the Association. Based on these descriptions and using the Highway Structures Information (HSI) system, attempts were made to match each bridge description with a bridge identification number. Most of the descriptions included information such as the roadway it was on, or the county or city it was in. A few of the descriptions also included what material the bridge was made of such as steel or concrete.

Some of the bridge descriptions matched several different bridges listed in the HSI and a select few were not matched with any of the bridges in the database. At this point, the first elimination of bridges which would not be load rated was done. Bridge descriptions which could not be matched to a bridge identification number were eliminated. The next step was to find plans for each of the bridge identification numbers that was identified. Some of the bridges were found to be county owned and the HSI database did not have plans for these bridges. This again eliminated a few of the bridges as having available plans was crucial to the load rating process and contracting counties to obtain plans was beyond the project scope.

A final list of seven bridges were chosen to be investigated in more detail and load rated. These seven bridges were all bridges that had available plans. One of the bridges, B380513, was chosen because of how critical it is to the Great Lakes Timber Professionals Association haul routes. Another factor that was considered in picking these seven was the current load posting according to the inspection reports. The bridges chosen include some bridges with inventory ratings as low as HS09 and up to one bridge which has a rating of

HS41. This wide range was chosen to test the accuracy of the load rating templates which were created for this project and also to verify the need for load posting on some of the bridges.

Table 8-1 provides information on each of these seven bridges which will be load rated. Included in the table is the number of spans, the lengths of each individual span, the superstructure material, the year built and the current posting on the bridge. The current posting on the bridge indicates whether or not the bridge is posted either for dimensions or weight limits. A load posting of 45ton means that the maximum gross weight allowed on the bridge is 45ton. The HS20 design vehicle has a gross weight of 36tons so this design vehicle would still be allowed on the bridge. However, the average five and six-axle logging vehicles used in Chapter 6 have gross weights of 47ton and 49ton respectively. These logging vehicles exceed the posted weight limit and would not be allowed on the bridges posted at 45ton.

Additionally, the inventory and operating load factor ratings are included and were taken from the most current routine inspection information from the Wisconsin Department of Transportation. The trucks used for ratings, (i.e. HS41) are related to the HS20 design vehicle described earlier. For example, the HS41 vehicle has the same axle configuration as the HS20 except it has higher loads on each axle resulting in a higher gross weight. The gross weight of the HS41 is equal to the gross weight of the HS20 times the ratio (41/20) between the two truck ratings. The gross weight of the HS41 is 73.8ton. A short description of each bridge follows the table.

Table 8-1: Seven Bridges of Concern for the Great Lakes Timber Professionals Association

ID #	Span	Length (ft)	Structure Material	Year Built	Current Posting	Inventory Rating	Operating Rating
B06-0013	2	41.8	Continuous Steel Deck Girder	1951	45Ton	HS41	HS58
B26-0002	1	61.5	Steel Girders	1948	45Ton	HS21.6	HS36
B37-0094	2	54	Continuous Steel Deck Girder	1962	---	HS15	HS24
		54					
B37-0043	2	92	Continuous Steel Deck Girder	1958	Narrow bridge	HS13	HS21
		92					
B60-0005	2	45	Continuous Steel Deck Girder	1960	Narrow bridge	HS14	HS27
		45					
B37-0006	1	51.5	Steel Deck Girder	1951	---	HS19	HS32
B38-0513	1	43	Concrete Girder	1925	45Ton	HS09	HS24

8.1 Bridge Number B06-0013

Bridge B06-0013 was built in 1951 and is located in Buffalo County in the town of Buffalo, WI. The bridge services STH 35 and runs over a Mississippi River Tributary. B06-0013 is a two span steel girder bridge with a total span length of 40ft. The bridge is composed of seven W18x60 rolled steel girders. The current existing posting on the bridge is 45ton. The inspection inventory and operating load factor ratings were HS41 and HS58 respectively. If these ratings are correct, then load posting of this bridge would not be necessary as the gross weight of the inventory rating of HS41 is over 70ton. This discrepancy between the rating and the posting is one of the main reasons this bridge was chosen for load rating.

In 1985, the bridge underwent a deck replacement and a new steel railing Type W was installed on both sides of the bridge. The bridge deck is concrete and composite with the steel girders. The national bridge inventory (NBI) ratings for the bridge were seven for deck and six for superstructure.

The NBI ratings are visual inspection ratings determined by the bridge inspector. For this project, ratings were taken from the most recent inspection report. Ratings are on a scale of 0-9. The following table describes the relationship between the NBI condition rating and the actual condition of the bridge. This table is based on the Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges. This code is a guide used in bridge inspection (U.S. Department of Transportation: Federal Highway Administration, 1995).

Table 8-2: Description of National Bridge Inventory (NBI) Condition Ratings

Rating	Condition	Additional Comments
9	Excellent	
8	Very Good	No problems noted
7	Good	Some minor problems
6	Satisfactory	Structural elements show some minor deterioration
5	Fair	All primary structural elements are sound but may have minor section loss, cracking, spalling or scour
4	Poor	Advanced section loss, deterioration, spalling or scour
3	Serious	Loss of section, deterioration, spalling or scour have seriously affected primary structural components
2	Critical	Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken
1	"Immanent" Failure	Major deterioration or section less present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service
0	Failed	Out of service, beyond corrective action

8.2 Bridge Number B26-0002

Structure B26-0002 is a single span bridge originally built in 1948. In 1989, the deck was replaced and the structure had painting work done in 1990. B26-0002 is a single, 60ft span bridge located in Iron County in the town of Sherman, WI. This bridge is a very low traffic bridge with a reported average daily traffic (ADT) of 770 in 2005. The current posting for the bridge is 45ton. The inventory and operating load factor ratings from the most current inspection report are HS21.6 and HS36 respectively. The bridge is made up of five W33x141 steel rolled girders spaced at 5ft 5in on center. Based on the plans for this bridge, there was no indication as to whether composite action between the steel girders and

the concrete deck was developed. Because of this, the capacity of this bridge will be taken as the capacity of the girders alone with no contribution from the concrete deck. The NBI ratings for the bridge were six for deck and five for superstructure.

8.3 Bridge Number B37-0006

Structure B37-0006 is a single span steel girder bridge located in Marathon County. The bridge is located in the village of Stratford on STH 153-Fir St. The bridge runs over the Big Eau Pleine River. B37-0006 was built in 1951 and originally consisted of five W30x116 girders. Small maintenance repairs were done on the bridge after it was constructed including painting in 1979 and a bituminous overlay in 1989. In 2001, the bridge underwent a large change which included a new concrete deck as well as widening of the bridge. Four new girders were added to the bridge and the structure is now 46.3ft wide. According to the most recent inspection report, the bridge has an inventory rating of HS19 and an operating rating of HS32. This bridge has a NBI rating of eight for both the deck and the superstructure.

8.4 Bridge Number B37-0094

Bridge B37-0094 was built in 1962 and is a two-span continuous rolled steel girder bridge. The bridge is located in the town of Easton, in Marathon County and is on CTH. Z. The Big Sandy Creek River runs under the bridge. B37-0094 is 108ft in length and is 26ft wide. B37-0094 consists of four rolled steel girders which were W27x84. The only recorded maintenance on the bridge was the placement of a new deck in 2002. The current load factor ratings for this bridge are HS15 for inventory and HS24 for operating. The deck has a NBI rating of eight and the superstructure has a NBI rating of five.

8.5 Bridge Number B60-0005

Bridge B60-0005 is a two span continuous steel deck girder bridge located in Taylor County in the town of Westboro. The bridge was built in 1960 and is on County Road D. The bridge runs over the Silver Creek. The bridge is 90ft. in length; each span is 45ft long. The deck is made of concrete and is composite with the four W24x76 steel girders. A concrete overlay was completed in 1997. This bridge is posted as a narrow bridge as it is only 24.5ft wide. Based on the most recent inspection report, the NBI rating is a six for both the deck and the superstructure. The load factor ratings are recorded as HS14 for inventory and HS27 for operating.

8.6 Bridge Number B37-0043

Bridge B37-0043 is located in Marathon County in the town of Easton. The bridge is a two span continuous steel girder bridge and each span is 92ft long. B370094 runs over the Eau Claire River and is on County Road Z. The bridge consists of four W36x150 steel rolled girders. No rehabilitation has been done to this bridge since it was built in 1958. The only existing posting is that it is a narrow bridge. Lastly, the NBI rating for the superstructure as of September of 2007 was a five and the rating for the deck was a four. The current load ratings for the bridge are HS13 for inventory and an HS21 for operating.

8.7 Bridge Number B38-0513

Structure B38-0513 is the only reinforced concrete bridge that was chosen for load rating. This bridge was originally built in 1925 and is a single span concrete T-girder bridge. This bridge is located in Marinette County in the city of Wausaukee. The bridge is on US - 141 and runs over the Wausaukee River. Currently, this bridge is load posted at 45tons.

When the bridge was originally constructed in 1925, this bridge consisted of six concrete girders which were 40ft long. This bridge has greatly evolved over the years. Major construction was completed on the bridge in 1948. The structure was widened and a new deck was poured. In 1948, the structure was widened with five new girders. These girders all range in length due to a skew at both ends. It was presumed that the skew are in order to stay clear of the flow of the river below. Figure 8-1 shows the 1948 plan view of the superstructure with the new added girders in bold red.

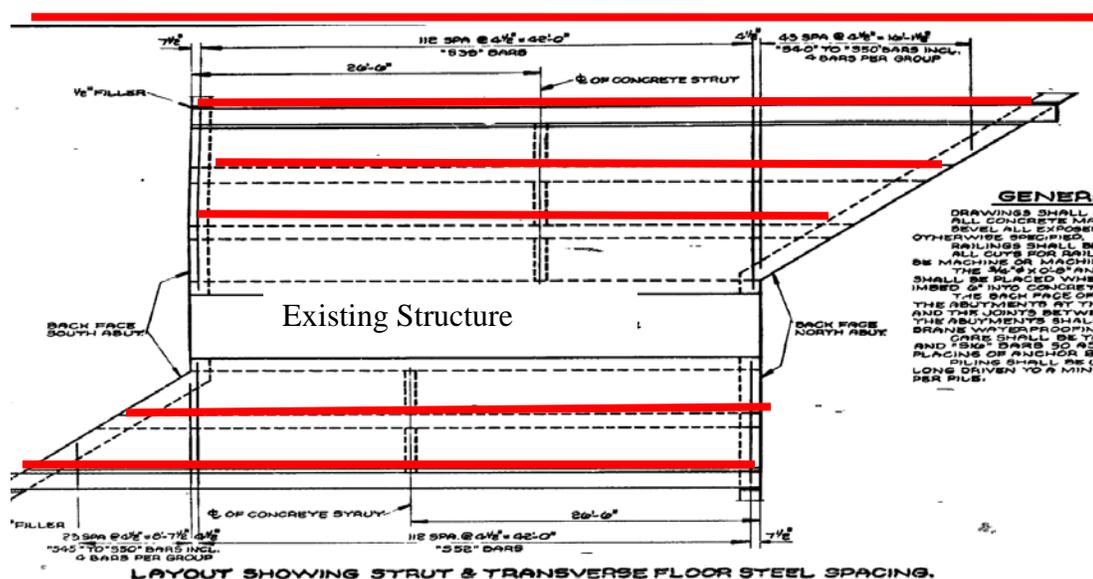


Figure 8-1: B380513 Superstructure 1948 Plan View

As can be identified in Figure 8-1, the new girders which were added to the bridge are all different span lengths. Because of this, the bridge had to be modeled using computer analysis in order to determine the moment and shear distribution factors (i.e. percentage of a truck load carried by one girder) that the individual girders are experiencing as a result of the live loads. The shear and moment live load distribution factors provided in the AASHTO LRFD Bridge Specifications require that all the girders in the bridge have the same span length (American Association of State Highway and Transportation Officials, 2004).

Following the 1948 construction, only minor rehabilitation was done on the bridge. In 1990, a bituminous overlay was placed on the deck and lastly in 1997, a new railing was placed on the bridge. Based on the most recent inspection report from 2008, the inventory rating of this bridge is HS09 and the operating rating is HS24. The NBI rating for the bridge deck is a seven and the rating for the superstructure is a six. This bridge is load posted at 45 ton.

8.8 Length of Bridges Compared to Vehicle Effects

As discussed in Chapter 5, there are spans ranges for which the logging vehicles produce less of an effect than the comparison vehicles such as the HS20 and the Wisconsin permit vehicle. This led to the question of whether or not the bridges of concern fall into these acceptable span ranges.

Figure 8-2 shows the moment effects of several of the vehicles used in the previous truck analysis. The figure also includes black lines that are the span lengths of the seven bridges that were described previously. From this figure it can be identified that the majority of the bridges of concern fall into a range of 40 to 65ft. with one bridge which had spans exceeding 90ft.

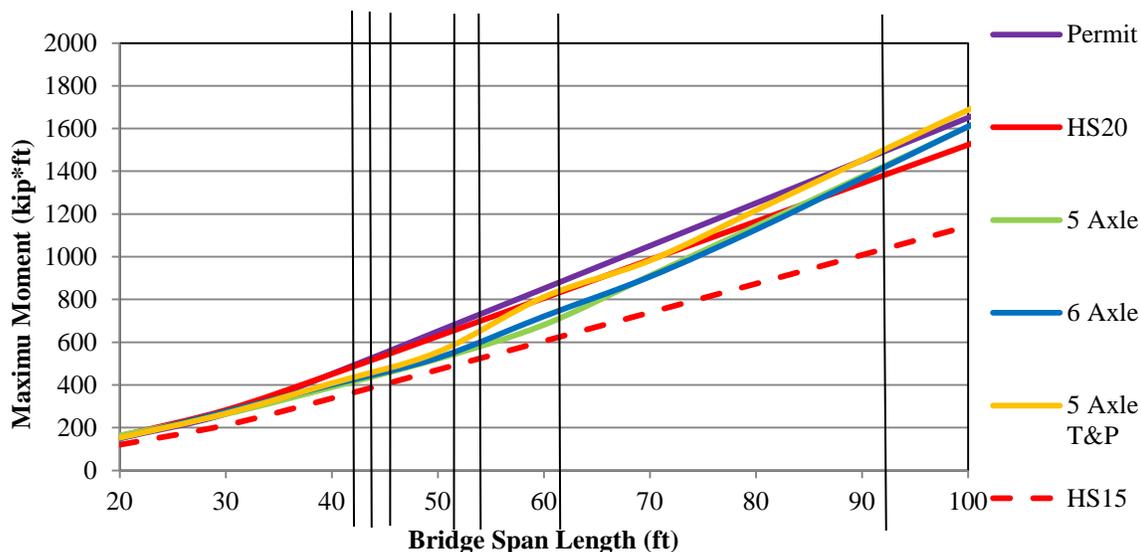


Figure 8-2: Moment Vehicle Effects and Bridge Span Lengths of Interest

When looking at how logging trucks moment effects would compare to the HS20 and the Wisconsin permit vehicle at the span lengths of the bridges of concern, it was found that the majority of the bridges fall within the acceptable ranges. Similar results were found for shear effects. Only the two longest spans exceeded the allowable span ranges for the logging vehicles compared to the HS20 and the Wisconsin permit vehicle. For more information regarding the acceptable span ranges for each of the average logging vehicles, see Chapter 6.

8.9 Summary of Specific Bridges of Interest to the Timber Industry

A wide range of types of bridges were chosen for load rating through this project. The seven chosen include both steel and concrete bridges and also include single and double span bridges. Additionally, the bridges have a wide variety in the complexity of the structures. Some structures, such as B260002 are simple, single span rolled girders. Others, such as B380513, are complex structures with varied girder size, reinforcement and length. Results from the load rating of these bridges can be found in Chapter 10.7.

9 SAP2000 Modeling of Bridge B380513

As stated before, due to the unique geometry of bridge number B380513, the live load distribution factor equations provided in the AASHTO bridge manual are not valid. Because of this, the bridge was modeled using SAP2000 in order to find the maximum moment and shear effects that the bridge will experience due to truck loads on the structure. (Computers and Structures Inc.)

The first step in modeling was to build the geometry of the bridge in the X-Y plane. An attempt to model this bridge had been recently completed in SAP2000 as an undergraduate project (Bruenig, 2009)so the geometry of the bridge in this direction was pulled from that model. The dimensions were based on the center to center spacing of the rectangular section of the girders. Nodes were placed to represent each end of the eleven beams, totaling 22 nodes. Originally, the nodes were all based on the same plane and needed to be adjusted so that they were placed at the strong axis neutral axis.

In order to determine the location of the beams in the vertical z-plane, first, each individual girder's strong direction neutral axis was calculated. Next, a base line was defined and each girder's strong direction neutral axis was determined relative to the other girders and a distance to the neutral axis from the base line was determined. Figure 9-1 shows the distance each beam was from the defined base line.

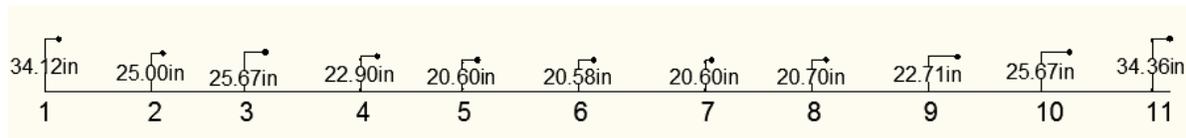


Figure 9-1: Cross Section of Bridge showing distance from baseline to strong direction neutral axis

Lastly, the neutral axis of the beam in the weak direction was calculated and the distance that it was from the center of the rectangular section of the beam was determined. Figure 9-2

shows an example of this dimension. As stated earlier, the beams were originally modeled from the neutral axis of the rectangular section so they needed to be adjusted to represent the neutral axis of the entire composite section.

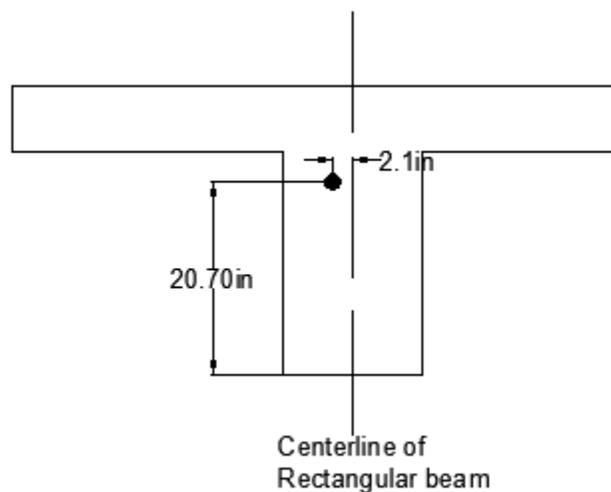


Figure 9-2: Beam 8 showing Location of both Strong and Weak Neutral Axis

At this point, the nodes were in the correct place in all three directions and the frame elements could be added. The beams were modeled using t-shape frame sections which represented the composite beams. In some cases, approximations had to be made as the beams were not exactly symmetrical due to different spacings between the beams. The t-shape frame section dimensions require that the frame elements are symmetrical. Additionally, beams four and nine consist of an original beam connected to a newer beam added when the bridge was widened. These beams still form an approximate t-shape and a conservative t-shape was used in the model. Dimensions and model inputs used for the frame elements can be found in Appendix A4. The plan sets for the bridge superstructure from 1925 and 1948 can found in Appendix A9.

The next step in the modeling was to connect the beams using frame elements to represent the stiffness of the deck. These were created by dividing the beam frame elements

so that there was a deck (frame) element approximately every 6-8ft. With these frames, the weight was excluded in the analysis as the weight of the deck would already be incorporated in the beam-frame elements. These elements were created using rectangular sections. The dimensions of the deck frame elements were equal to the size of the deck that each element was representing. SAP would then determine the stiffness of the section which would allow for load to be transferred between girders. Figure 9-3 shows the full model of the bridge.

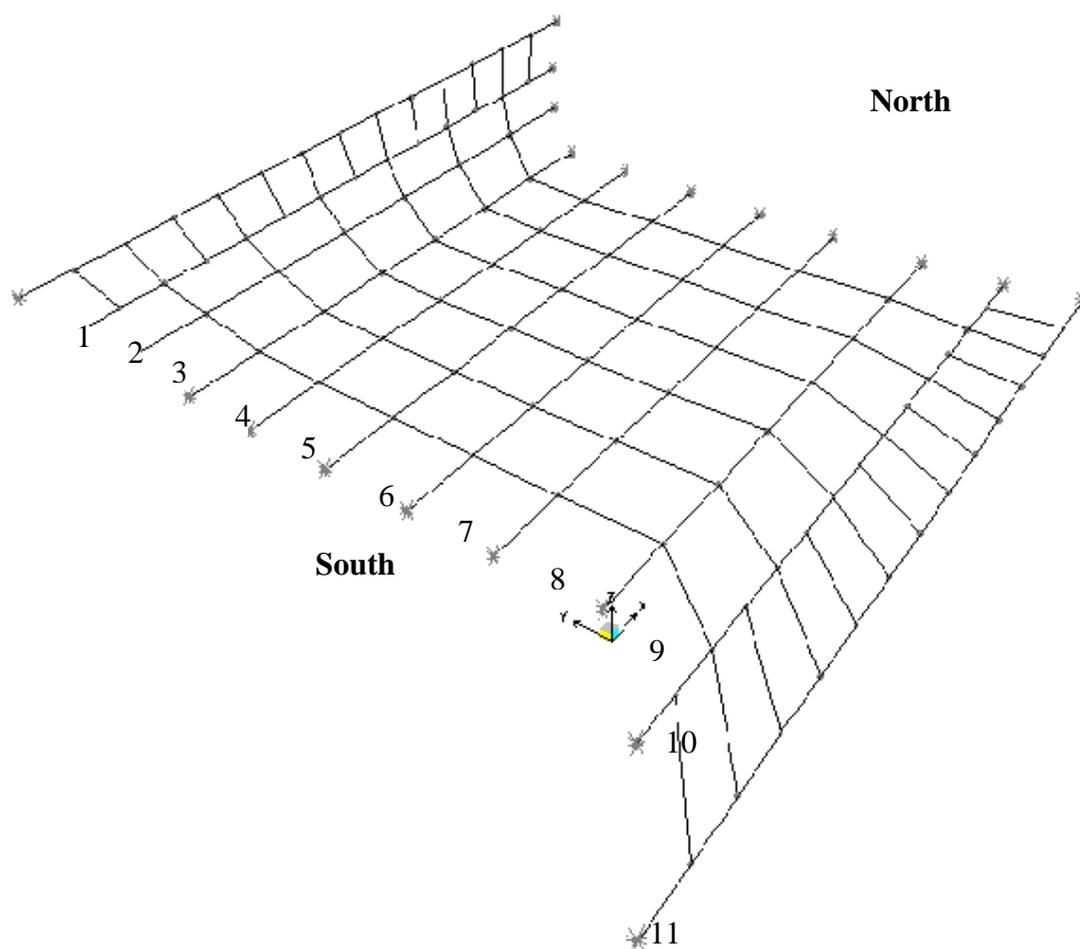


Figure 9-3: Snapshot of SAP 2000 Model of Bridge B38-0513

The final step in the modeling of the structure itself was to define the end restraint conditions. Beam 11 had different restraints from all the other beams. On the south

abutment side of beam 11, the translation is fixed in all three directions and rotation is fixed in the x-direction. On the north abutment side of beam 11, the translation is fixed in the y and z direction and the rotation is fixed in the x-direction (see axes in Figure 9-3 for directions). Beams one through ten have the same support conditions on either side of the abutment. On the south side of the abutment, beams one through ten are rotationally fixed in the x-direction (i.e. torsionally) and fixed against translation in the x and z direction. Lastly, on the north side of the abutment, beams one through ten are fixed against translation in the z-direction and fixed against rotation in the x-direction (torsion).

One issue that was discovered in the model was that the stiffness of the deck was actually being accounted for twice in the design. The stiffness of the deck was included in the beam frame element as well as the deck frame elements that were connecting the beams. In order to resolve this issue, the torsional constant of the beam-frame elements was modified to only represent the stiffness of the rectangular section and not to include the deck stiffness. By doing this, only the deck frame elements connecting the beams represented the deck stiffness.

Lastly, in order to check the accuracy of this model, a simplified two girder model was created and the HS-20 vehicle was loaded on the bridge in a position between the two girders. The magenta middle line shown in Figure 9-4 represents the lane the HS20 traveled on. The girders were modeled as t-sections and they were connected with frame elements to represent the deck. As with the real model, the torsional constant of the frame girder elements was modified to only represent the stiffness of the rectangular section of the composite girder. If the model was working correctly, the moment effects due to the truck would be evenly distributed by the two girders. Using PCBRIDGE, the total moment due to

the HS-20 for a span length of 42ft was determined. 42ft was used because this is the length of the original girders in bridge B380513. The total moment due to the truck on a single span bridge was 485.3 kip*ft. This meant that in order for the model to check, the moment on each of the two girders should be half of this total moment. Figure 9-4 shows the model that was used for this check and it did check accurately with a maximum moment of 2910kip*in (242.6kip*ft) on each girder at midspan. This check meant that the elements connecting the girders were correctly simulating the stiffness of the deck and the girders were accurately simulating the stiffness of the rectangular girder section.

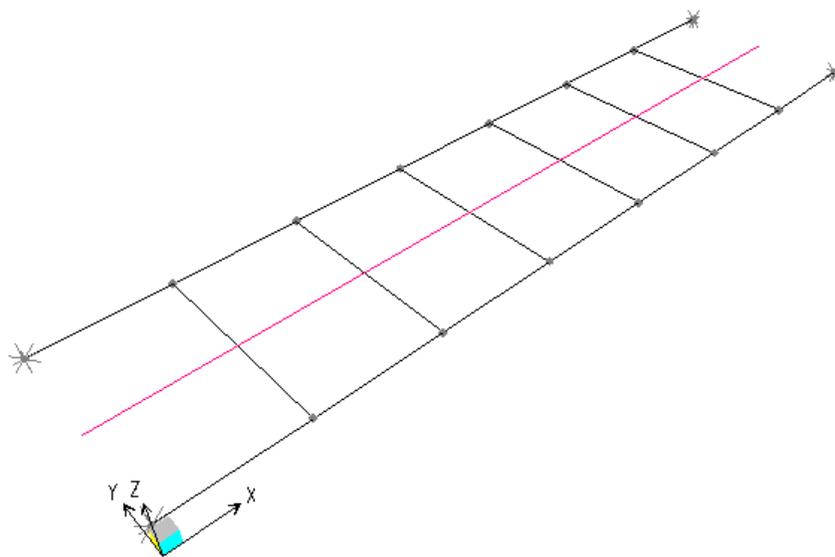


Figure 9-4: Simple Model to check accuracy of modeling methods

With the structure of the bridge now modeled and also checked for accuracy, the next step was to model the loads that are on the bridge. A new Type W railing was installed in 1997 which has a weight of 45lb/ft. (Wisconsin Department of Transportation, 2008). This load was applied to girders one and eleven since the railing is connected to these girders. When the bridge was widened, the new girders were connected with struts. The weight of

these struts was not included in the model but was added later in the load rating process. The weight of the struts was evenly distributed among all the girders. Additionally, there were some discrepancies as to how the deck was completed following the widening of the bridge. The top of the deck of the original bridge is lower than the height of the portions of the new deck proposed in 1948. The 1944 plans do not show how these two decks were connected to make a level surface. Due to this discrepancy, a conservative assumption had to be made. The original girders have an additional dead weight on the bridge to represent the possible concrete overlay on the original deck that would make the full deck level. This load was considered a wearing course load.

In addition to the dead loads on the bridge, live loads also had to be assessed. With the LRFR load rating, the design vehicle is the HL-93. Instead of using this vehicle in the SAP model, the HS20 vehicle was used. The tandem vehicle was not included because all of the girders have spans which exceed 40ft, so the HS20 will control. The lane load effects also associated with the HL-93 design vehicle were not included in the SAP model but were accounted for in the load rating. The first step in placing the live loads on the bridge was determining the lanes on the bridge. Two moment and shear effects needed to be calculated for each beam being load rated, for a single loaded lane and for two loaded lanes on the bridge. The moment/shear with only one vehicle on the bridge is multiplied by a multiple presence factor, m which is equal to 1.2. The controlling effects between a single loaded lane and two loaded lanes are then compared and the maximum moment/shear effects of the two are used in design or rating.

One of the modeling obstacles to overcome for this design was to determine where the bridge lane should be placed in order to create the maximum effects on the girder in

question. The maximum effects do not always occur when the center of the lane is directly over the girder. For the single loaded lane case, this was fairly simple. The lane was modeled 150 in wide. 150in was chosen because this is equal to just over two girder spacings and the location of the lane to result in the highest effects will be within that width. When the lane is wider than 72in, which is the standard width of the design vehicle, during the analysis, SAP will move the vehicle around horizontally to find the location of the lane that creates the highest forces. The results then just show the maximum forces based on the lane location determined during the analysis. By doing this, several trials of moving the lane to find the maximum effects is avoided.

Determining the maximum effects due to two vehicles on the bridge was more difficult. Both of the lanes that were modeled were 72in wide and the two lanes were modeled next to one another. Trial and error was used by moving the lanes together to determine the lane locations to create the maximum effects on each of the three girders. This loading is different than expected by AASHTO where it is suggested that 12ft lanes are used.

Results from three different girders were extracted from this model. Due to constraints, every beam could not be analyzed and load rated. Therefore the girders most likely to control the design were analyzed. The girders chosen were 2, 6 and 10. Girder 6 was chosen to represent the original girders built in 1925. This girder has the smallest deck flange of the interior girders and is expected to have the lowest capacity. Girders 1 and 11 are taller than the top of the driving surface, not directly loaded with live load and are deeper and stronger than the other girders. Because of this, these girders will not control. Instead, girders 2 and 10 were analyzed because they were thought to be the most likely to control of the girders added when the bridge was widened.

For the load rating, the moment and shear effects due to the three different types of loading were kept separate. This is necessary because different load factors are used for different types of loads for the Load and Resistance Factor Rating method. The final results from the model were maximum moment and shear effects for composite dead loads, dead loads due to wearing course and live loads. Table 9-1 to Table 9-4 show the results for each type of loading that was modeled and analyzed for bridge B38-0513. These results were later used in the load rating of B38-0513.

Table 9-1: SAP 2000 Dead Load Results for Composite Dead Loads (DC)

Dead Loads (DC)		
Beam	Moment (kip*ft)	Shear (kip)
6	170.54	17.14
2	223.98	24.94
10	162.67	18.25

Table 9-2: SAP 2000 Dead Load Results for loads due to Wearing Course (DW)

Dead Loads Due to Wearing Course		
Beam	Moment (kip*ft)	Shear (kip)
6	92.28	9.40
2	18.08	1.42
10	36.95	2.90

Table 9-3: SAP 2000 Live Load Results for Single Loaded Lane

Single Loaded Lane Live Load Effects:		
Beam	Moment (kip*ft)	Shear (kip)
6	116.46	23.05
2	161.63	41.52
10	127.28	37.91

Table 9-4: SAP 2000 Live Load Results for Two or More Loaded Lanes

2 Lanes Loaded: live Load Effects		
Beam	Moment (kip*ft)	Shear (kip)
6	224.29	45.15
2	273.31	66.35
10	219.96	62.30

10 Load Rating of Bridges

In Chapter 8, the seven bridges which were chosen for load rating are described and the method for choosing those seven bridges is explained. Each of these seven bridges was individually load rated using two methods. These methods, the Load Factor Rating (LFR) method and the Load and Resistance Factor Rating (LRFR) method are both used today in practice. The LRFR method is the newer method and was created in conjunction with the LRFD design method.

10.1 Terms and definitions

10.1.1 Inventory Ratings and Operating Ratings

The primary step in load rating that is completed in both LFR and LRFR method is the design vehicle load rating. There is not an actual design load rating, but this rating consists of the inventory and operating ratings. The bridge inventory and operating ratings from the LFR method are not directly comparable with the LRFR method results, however the ratings have similar meaning in terms of the load capacity of the bridge. The main difference in the two ratings is the design vehicle used. The LFR is based on an HS20 vehicle and the LRFR is based on the HL-93 vehicle. Typically, the LRFR rating will be slightly more conservative.

The inventory rating for the LFR method is the maximum load that the bridge can handle indefinitely without experiencing any permanent damage to the bridge. This loading can also be on multiple lanes simultaneously daily. For example, if the inventory rating from the LFR method was an HS20 that would mean that the bridge can withstand load effects equal to the HS20 indefinitely without compromising the structural integrity.

In LRFR, the AASHTO definition of the inventory rating is “Generally corresponds to the rating at the design level of reliability for new bridges in the LRFD Bridge Design Specifications but reflects the existing bridge and material conditions with regard to deterioration and loss of section. (American Association of State Highway and Transportation Officials , 2003). The LRFR inventory rating differs from the LFR in that it isn’t a HS truck but rather a number around 1.0. An inventory rating of 1.0 means that the bridge can handle loads equal to the HL-93 indefinitely without permanent damage. A value less than 1.0 indicates that the bridge can only sustain loads lower than the HL-93 indefinitely without damage.

The operating rating is the maximum load that the bridge can withstand without the possibility of permanent damage. The AASHTO definition of the inventory rating for LRFR rating is “absolute maximum load level to which a structure may be subjected for limited passages of load. Generally corresponds to the rating at the Operating level of reliability in past load rating practice.” (American Association of State Highway and Transportation Officials , 2003).

10.1.2 Legal Load Rating Vehicles

The legal load rating is the secondary level of the load rating process. In the case of using the LRFR method, if the design vehicle ratings are greater than or equal to one, then all the legal loads will be acceptable and legal load rating is not necessary. However, if the design loads are not acceptable, then legal load rating is necessary to determine if the bridge needs to be load posted or strengthened.

For this project, nine vehicles were used for legal load rating. These vehicles are the same vehicles that are used by the State of Wisconsin to determine load posting of bridges using the LRFR method. These vehicles consist of three AASHTO commercial vehicles, four AASHTO single unit vehicles and two Wisconsin legal vehicles. The AASHTO commercial legal loads consist of three vehicles, Type 3, Type 3S2 and Type 3-3, which were created to represent the majority of truck configurations on the roads today. Figure 10-1 shows the load and axle configurations for the three AASHTO legal vehicles.

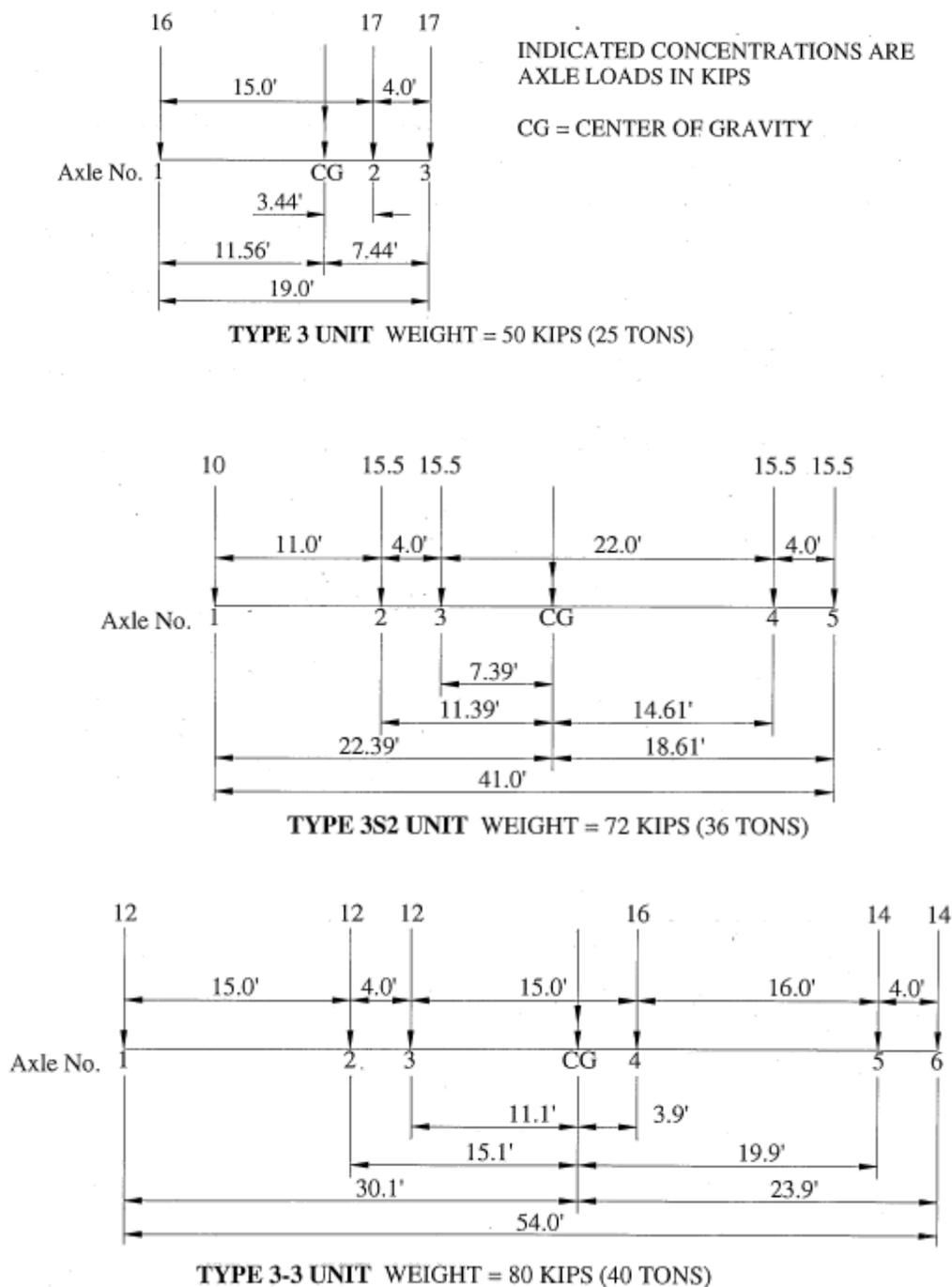


Figure 10-1: AASHTO Legal Vehicles: Type 3, Type 3S2 and Type 3-3 (American Association of State Highway and Transportation Officials, 2003)

The second set of vehicles that was used in the load rating were the four single unit AASHTO vehicles. Figure 10-2 shows the vehicle configurations for each of the four AASHTO single unit vehicles.

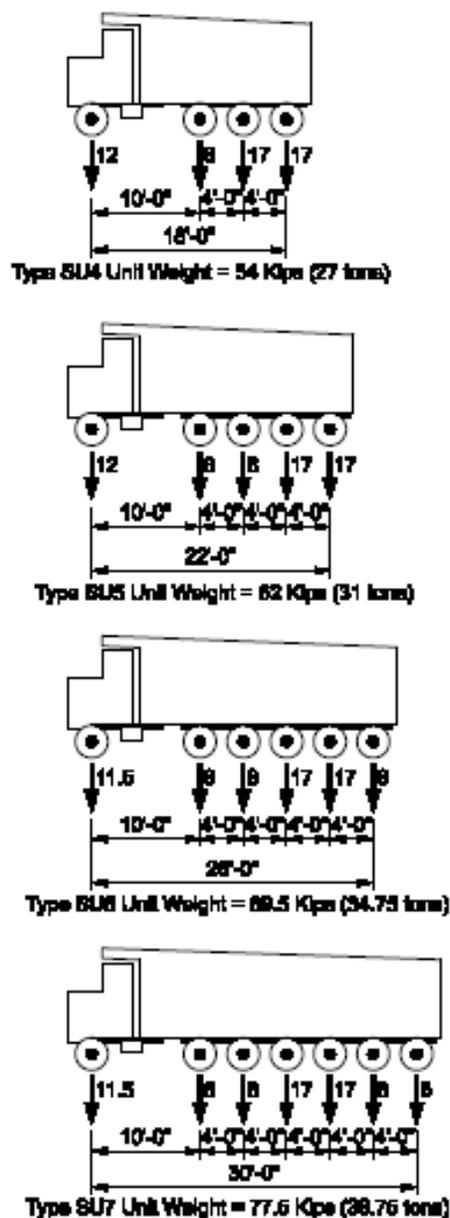


Figure 10-2: AASHTO Single Unit Vehicles (Wisconsin Department of Transportation, 2008)

The final two vehicles used in the legal load rating were Wisconsin State legal vehicles. Both of these are six axle vehicles, one in the tractor-trailer combination and the other a truck and pup configuration. Additionally, both vehicles have a gross weight of 98 ton. Figure 10-3 shows the configurations for both the two Wisconsin legal vehicles.

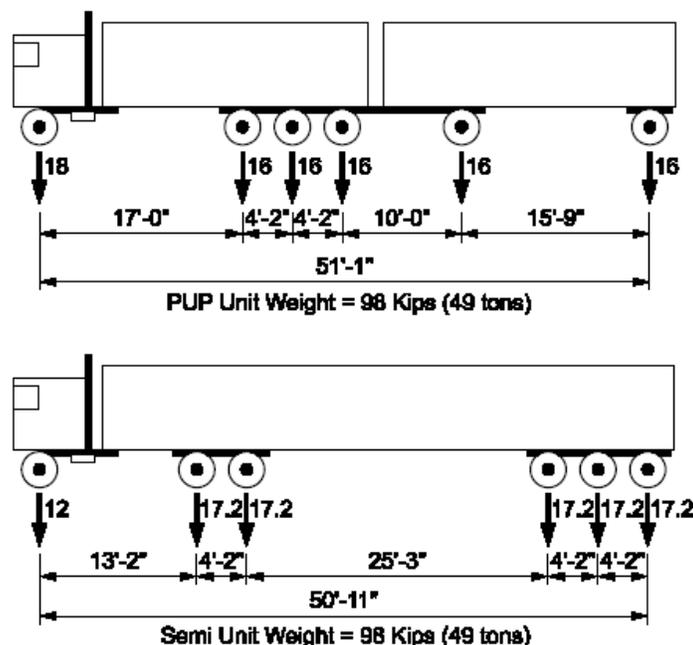


Figure 10-3: Wisconsin Legal Vehicle Configurations (Wisconsin Department of Transportation, 2008)

10.2 Analysis Methodology of Structure

Before load rating of a structure can be done, the structure must be analyzed in its current state to determine the capacity of the bridge. The first step to completing this analysis is to inspect the bridge and to assess any damage or other issues which could affect the strength of the bridge. Based on the assessment, the national bridge inventory (NBI) ratings can be determined as described earlier. Exact instructions on how to determine the NBI ratings based on a visual inspection can be found in the Recording and Coding guide for the

Structure Inventory and Appraisal of the Nation's Bridges. (U.S. Department of Transportation: Federal Highway Administration, 1995). In addition to the NBI ratings, the visual inspection can also determine if any section loss has occurred over time that could affect the capacity of the bridge. This can be especially important when looking at old deteriorated reinforced concrete bridges where spalling and section loss may be more common. Due to constraints associated with this research project, visual inspections of the bridges could not be completed. As a result, inspection reports were used to obtain necessary information about the current state of the bridge including NBI ratings. The inspection reports as well as the plans for the bridges which were load rated were from the Highway Structures Information (HSI) system (Wisconsin Department of Transportation, 2003).

For each bridge, a minimum of two girders must be analyzed for load rating. In the case of a single span bridge, an interior girder and an exterior girder was analyzed. In the cases of multi span continuous bridges, one interior and one exterior girder was checked for both positive and negative moment load effects.

10.2.1 Determining Bridge Properties

The first step in the analysis of the bridge was to obtain all the necessary information about the bridge from the plans. This information included but was not limited to the size and weight of the girders and deck, the strength of materials, and weight of additional dead loads on the bridge. Additionally, knowing whether the girders and deck are composite was very important as a composite girder can have a much higher capacity than a non-composite girder. Many times complete plans are unavailable and some specific information was not known. For some types of information such as strength of concrete, there was information in

the AASHTO LRFR code which approximates the strength based on the time period the bridge was constructed. Once all the individual properties are known, if the girder is a composite girder, the composite properties such as the moment of inertia are calculated. When analyzing steel girders with concrete decks, two types of composite properties are calculated. First, short term composite properties were calculated. This is when the modular ratio of steel to concrete is equal to the ratio of steel moduli divided by concrete moduli only. The second type of property is the long term composite section when the modular ratio of the modulus of steel divided by the modulus of concrete is multiplied by three due to creep in the concrete. Based on these two modular ratios, individual moments of inertia and more importantly individual section moduli were calculated. The short term composite properties were used when dealing with the live loads on the bridge because they are not permanent. The long term properties were used with the dead loads on the structure as they will be permanent loads. Once all the properties were known or estimated as closely as possible, the moment and shear capacity of the bridge can be calculated.

10.2.2 Calculating Dead and Live Loads of a Structure

Before calculating the capacity of the bridge, the next major task in load rating was to determine all the loads on the bridge. The first loads that need to be determined are the dead loads already on the bridge. This includes but is not limited to curbs, railings, struts and wearing course overlays. Information on these loads was typically found in the plans. Once the dead loads were determined, the loads associated with the permanent loads were distributed evenly among the girders. Based on this load, which is calculated in kip/ft for each individual beam, the moment and shear effects due to the dead loads was calculated. In

composite bridges, two dead load moment and shear effects were calculated. The first for loads that were only applied to the beam directly such as the weight of the girder itself and the weight of the deck, cover plates and struts if applicable. The second set of dead load moment and shear effects from dead loads were for loads that are loaded onto the composite beam. These loads include the weight of railings, curbs, and parapets. As an end result, there were two sets of moment and shear's calculated, one for non-composite loads and one for composite loads. In the case of a non-composite structure, no modular ratio is needed and all the loads were lumped together and one set of moment and shear effects are calculated.

Next, the moment and shear effects on the bridge due to the live load design vehicle, the HL-93, were calculated. The HL-93 consists of the controlling vehicle between the HS20 and the tandem vehicle plus a uniform load of 0.64kip/ft representing other vehicles on the bridge at the same time. The uniform load moment and shear effects were calculated based on the length of the bridge. The effects of the two vehicles, the HS20 and the tandem vehicle, were determined using PCBRIDGE. The larger effects of the two possible trucks were used in analysis. PCBRIDGE is a program which determines the maximum positive and negative moments and shears experienced on continuous span bridges. The user simply inputs the span lengths of the bridge and the axle spacing and loads for the vehicle and the program then steps the vehicles across the bridge in 0.5ft increments to determine the maximum moments and shears and the locations of these maximum effects.

10.2.3 Moment and Shear Live Load Distribution Factors

With all the live loads calculated, moment and shear live load distribution factors need to be calculated. These equations determine how much of the live load is applied to

each girder when a truck is on the bridge. All of the calculations used to determine the distribution factors were based on the AASHTO Bridge Design specifications (American Association of State Highway and Transportation Officials, 2004). These equations which are used to determine the distribution factors have limitations. For example, the equations are only valid if all the girders in the bridge are the same span length. This was an issue with bridge number B38-0513 which as a result had to be modeled in order to determine the distribution of live loads. In addition to the limitations of the equations, there are also separate distribution factors for interior and exterior girders.

The first distribution factors that were calculated were for interior girders. There are two moment distribution factors (MDF) which are calculated for an interior girder and the maximum of the two is used as the controlling MDF. The first equation is for a single lane load and the second equation is for a condition of two or more lanes loaded. Both of the MDF equations are functions of the spacing between girders, S (ft), the thickness of the deck, t_s (in), and the length of the bridge, L (ft). Additionally, both equations are a factor of the longitudinal stiffness parameter, K_g (in^4). The longitudinal stiffness parameter, K_g is a function of the following girder properties: the modulus of elasticity of both the beam material, E_B (ksi), and the deck material, E_D (ksi), the moment of inertia of the beam, I (in^4), the area of the beam, A (in^2), and the distance between the center of gravity of the deck and the center of gravity of the beam, e_g , (in.). Equation 5 below is the equation for calculating the longitudinal stiffness parameter of a girder.

$$K_g = \frac{E_B}{E_D} * (I_x + A * e_g^2) \quad (5)$$

Equation 6 is the MDF equation for a single loaded lane and equation 7 is the MDF equation for two or more lanes loaded. Both equations can be found in Table 4.6.2.2b-1 on page 4-31 in the AASHTO LRFD Bridge Design Specifications. (American Association of State Highway and Transportation Officials, 2004)

$$g_{m1} = 0.06 + \left(\frac{S}{14}\right)^{0.4} * \left(\frac{S}{L}\right)^{0.3} * \left(\frac{K_g}{12 * L * t_s^3}\right)^{0.1} \quad (6)$$

$$g_{m2} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} * \left(\frac{S}{L}\right)^{0.2} * \left(\frac{K_g}{12 * L * t_s^3}\right)^{0.1} \quad (7)$$

There are also ranges of these properties for which the equations are applicable. These ranges can be found in Table 4.6.2.2b-1 of the AASHTO LRFD Bridge Design Specifications. All seven of the bridges load rated were within the applicable ranges.

The shear distribution factors are much simpler and are only a function of the spacing between girders, S. Like the MDF, two SDF are calculated, one for a single lane loaded case and one for a case of multiple lanes loaded on the bridge. Equation 8 is the SDF equation for a single loaded lane and equation 9 is the SDF equation for the case of two or more lanes loaded. Both of these equations can be found in the AASHTO Bridge Design Specifications in Table 4.6.2.2.3a-1.

$$g_{v1} = 0.36 * \frac{S}{25} \quad (8)$$

$$g_{v2} = 0.2 + \frac{S}{12} - \frac{S^{2.0}}{35} \quad (9)$$

The moment and shear distribution factors for exterior girders are different from the interior girders. There are other methods that are used to determine the exterior girder distribution factors. First, the equations and methods used to calculate the exterior girder

MDF will be discussed. With exterior girders, the lever rule is used to determine the single loaded lane MDF and an equation based on the interior MDF is used for two or more lanes loaded. In addition to these equations, the MDF may also be calculated using the cross frame method. The controlling MDF is the maximum of all three of these distribution factors.

In order to determine the MDF for a single loaded lane, the lever rule is used. A figure showing an example of the lever rule is shown in Figure 10-4.

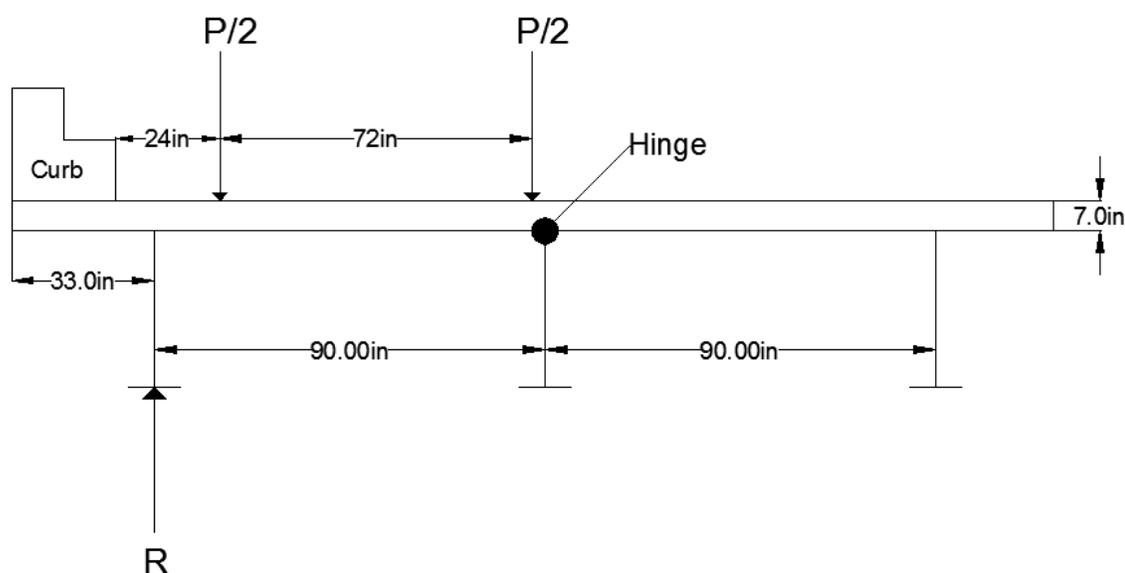


Figure 10-4: Lever Rule Example: Bridge B37-0043

The MDF based on the lever rule is calculated by summing moments around the hinge point (point at which the deck is assumed to have an internal hinge). The hinge point is the center of the first interior beam of the bridge. In the case shown in Figure 10-4, the second wheel load is three inches to the left of the second girder, and the second girder is the hinge point. Once moments are summed about the hinge point, the reaction at the girders under the loads, which in this case is only the exterior girder is solved for in terms of the axle

load P . At this point, the resultant $R = \text{coefficient} * P$. The coefficient in front of P is then multiplied by the multiple presence factor, m to get the MDF based on the lever rule.

For two or more lanes loaded, the MDF is calculated by multiplying the controlling interior MDF by e , which is a function of d_e . d_e is the distance from the exterior of the web of the exterior beam to the interior edge of the curb or traffic barrier. Figure 10-5 shows an example of this measurement.

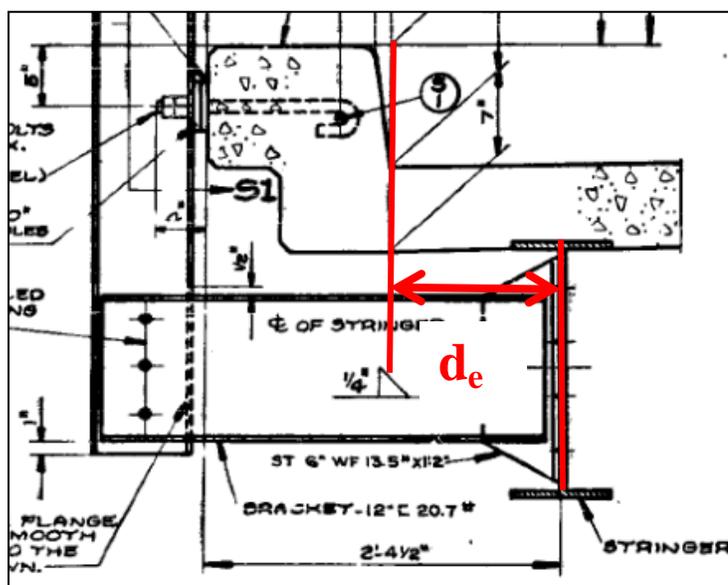


Figure 10-5: Example of d_e measurement on an exterior girder

The equations for e and for the MDF for two or more lanes loaded are shown in equations 10 and 11.

$$e = 0.77 + \frac{d_e}{9.1} \quad (10)$$

$$g_{m2} = e * g_{m\text{-interior}} \quad (11)$$

The final method for calculating the moment distribution factor for an exterior girder is using the cross frame method. The cross frame method consists of one equation, which is

used twice, once for a single loaded lane and another for two or more lanes loaded. The equation is a function of the following bridge properties listed in Table 10-1.

Table 10-1: Factors Included in Cross Frame Method for Calculating the MDF on Exterior Girders

N_L	Number of Loaded Lanes
N_B	Number of Beams or girders
x	Horizontal distance from center of gravity of the pattern of girders to each girder (ft)
X_{ext}	Horizontal distance from center of gravity of the pattern of girders to the exterior girder (ft)
e	Eccentricity of a design truck or a design lane load from the center of gravity of the pattern of the girders.
Σx	Sum of all girder x 's
Σe	The sum of the two lane eccentricity's

Equation 12 is the MDF equation based on the cross frame method which is equation C4.6.2.2.2d-1 on page 4-33 in the AASHTO LRFD Bridge Design Specifications. In the case of one lane loaded, the N_L is equal to one and when two or more lanes are loaded, then N_L is equal to two or the number of lanes which are loaded. Additionally, in the single loaded lane case, the R value is multiplied by the multiple presence factor, m which is equal to 1.2.

$$R = \frac{N_L}{N_b} + \frac{X_{ext} * \Sigma e}{\Sigma x} \quad (12)$$

In order to better understand the cross frame method, the following figure shows the bridge dimensions that were used in calculating the MDF for bridge B37-0043.

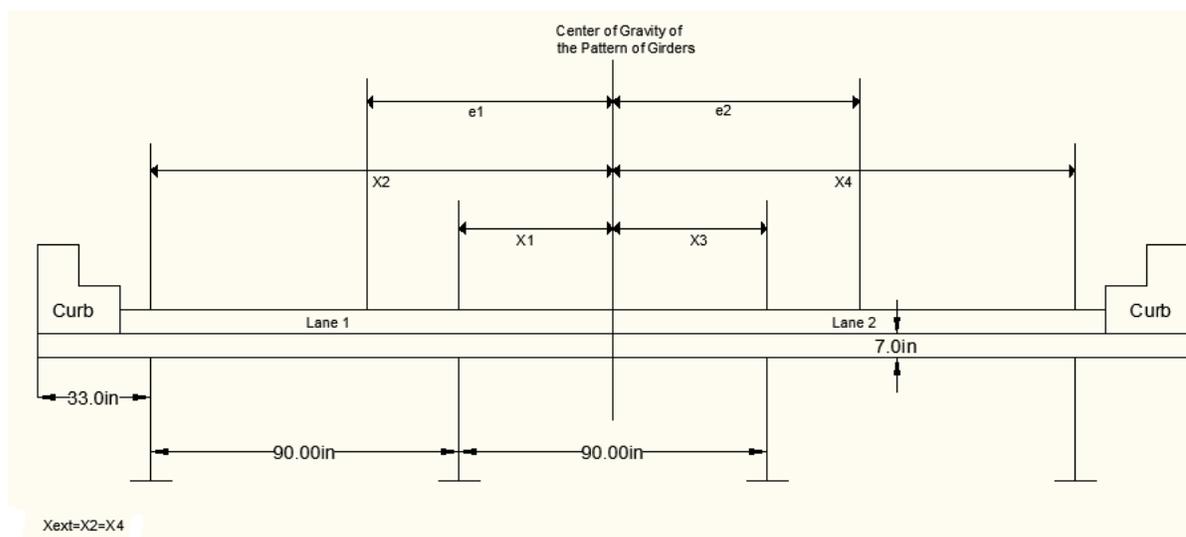


Figure 10-6: Example of Cross Frame Method for Bridge B37-0043

With the cross frame method complete, in total, four MDF have been calculated for exterior girders. The controlling moment distribution factor which will be used in the load rating is the largest distribution factor.

The final distribution factor that was calculated was the shear distribution factor of exterior girders. As with the moment exterior distribution factors, four different distribution factors were calculated and the maximum was the controlling SDF that was used in the load rating. There is a SDF that is for two or more lanes loaded. Similarly to the MDF, this distribution factor is the controlling SDF for an interior span multiplied by e . The equation for e is a function of the distance d_e and is labeled equation 13.

$$e = 0.6 + \frac{d_e}{10} \quad (13)$$

The final three SDF are from the lever rule and the cross frame method. These three SDF are all the same as the previously calculated MDF for an exterior span. With the controlling shear distribution factor for exterior girders determined, all of the controlling live load distribution factors were compiled and used in the load rating.

10.2.4 Calculating the Moment and Shear Strength Capacity of the Girder

The final necessary step to be completed prior to load rating the bridge was to determine the actual moment and shear capacity of the bridge. The method for doing this is based on the type of material the girders are made out of and whether or not there is composite action between the deck and the girder.

First, looking at non-composite steel w shape girders, the plastic nominal moment capacity was determined based on the strength of the steel and the section modulus as shown in the equation below.

$$M_n = F_y * Z_x \quad (14)$$

For composite beams, the calculations to determine the moment capacity are more complex. First, the location of the plastic neutral axis (PNA) of the section is determined. The PNA is either in the slab, the top flange or in the web of the steel section. The total moment capacity of the girder is equal to the compression or tension force in the beam since they must be equal. The templates which were created as part of this project for load rating were programmed so that the location of the PNA would be calculated and then based on this location, the appropriate equation would be used to calculate the over all nominal moment capacity of the beam. These equations were based on internal equilibrium, the location of the concrete deck compression block and other dimensional properties of the girder. The template showing this process including all the equations used can be found in the Appendix A5. Appendix A5 includes full load rating templates for interior and exterior steel beams looking at both positive and negative moment.

In addition to the steel beam template, a template for reinforced concrete beams was also made. The moment capacity calculations for a reinforced concrete beam are similar to a

composite steel beam. The capacity is again based on internal equilibrium between the concrete and the steel reinforcing bars. Appendix A_ consists of the interior and exterior templates for reinforced concrete girders, which shows detailed calculations used to determine the moment capacity of a reinforced concrete girder.

The shear capacity for all steel beams is computed the same. The shear capacity is a function of the strength of the steel, the depth of the fillet and the thickness of the web and is always based on the non-composite section. The composite concrete deck in composite construction does not add any additional strength to the shear capacity of a member. Equation 15 is the equation used to determine the shear strength capacity of the steel section.

$$V_n = .58 * D_{fillet} * t_w \quad (15)$$

The calculation of the shear capacity of a reinforced concrete beam is based on the shear strength provided by the concrete (V_c) and the shear strength provided by the reinforcing steel stirrups (V_s). The total nominal shear (V_n) is calculated by adding together the strength provided by the steel and the concrete shear strengths. The strength provided by the concrete is a function of f'_c , the compressive strength of the concrete, in ksi, b_v , the width of the web of the beam and d_v , the effective shear depth. A conservative approach to determining the effective shear depth is to take the maximum value between 0.9 times the depth from the top of the beam to the centroid of the flexural steel and 0.72 times the height of the beam. These conservative limits can be found in the AASHTO LRFD Bridge Design Specifications in section 5.8.2.9 (American Association of State Highway and Transportation Officials, 2004). The equation for the concrete contribution to the total shear strength of the section is shown in equation 16.

$$V_c = 2 * \sqrt{f'_c} * b_v * d_v \quad (16)$$

As stated previously, the other contribution to the shear strength of a member is the strength from the steel stirrup reinforcement. This strength is based on the area of reinforcement, A_v , the depth of the beam, d , the strength of the reinforcing steel, f_y , and the spacing of the stirrups, s . Equation 17 is the equation used to determine the shear strength contribution from the steel stirrups.

$$V_s = \frac{A_v * f_y * d}{s} \quad (17)$$

The final step in determining the shear strength of a reinforced concrete beam is to add the two contributions together. The complete load rating templates for reinforced concrete beams can be found in Appendix A6.

$$V_n = V_c + V_s \quad (18)$$

At this point in the process, all the necessary calculations have been completed and all the necessary information has been compiled in order to load rate the bridge. As stated earlier, two methods are available for load rating and Sections 10.3: Load Factor Rating Method and Section 10.4: Load and Resistance Factor Rating, describe the methodology of load rating the structure using the respective method once the analysis of the structure has been completed.

10.3 Load Factor Rating (LFR) Method

The Load Factor Rating (LFR) is the older of the two methods used in this research. This method is still commonly used as the main method of posting bridges in many states. In Wisconsin, bridges are still load rated using the LFR method however a project for load rating state bridges using both the LFR method and the Load and Resistance Factor (LRFR) method is currently in progress.

The main reason that the LFR method was used in this project was to be able to compare the results from this project with the current results found in the bridge inspection reports which are only reported for LFR results. As stated previously, these results can not be directly compared with LRFR results. The inspection reports give results for the LFR inventory and operating rating. For this reason, only the inventory and operating portion of the LFR load rating process was used and legal load rating using this method was not completed.

The equation for calculating the inventory and operating ratings using the LFR method are a function of the capacity of the girder, either moment or shear, and the dead (D) and live loads on the bridge. Equation 19 shows the equation used for both the inventory and operating ratings in terms of moment. The same equation is used when looking at shear capacity except the nominal shear capacity (V_n) replaces the nominal moment capacity (M_n), the dead load effects, D are in terms of shear, and the moment effects from the live load (M_{LL_IM}) is replaced by the shear effects from the live load (V_{LL_IM}).

$$\text{LFR Inventory/Operating Equation} \quad RF = \frac{M_n - A_1 * D}{A_2 * M_{LL} * (1 + I)} \quad (19)$$

In equation 19, there are two factors, A_1 and A_2 . A_1 is equal to 1.3 for both inventory and operating ratings and A_2 is equal to 2.17 for inventory ratings and 1.3 for operating ratings. The results for all seven bridge load rated using the LFR method can be found in Section 10.7.

10.4 Load and Resistance Factor Rating (LRFR) Method

The Load and Resistance Factor Rating (LRFR) method was created based on the LRFD design philosophy. A flow chart showing the load rating process for the LRFR procedure is shown in Figure 10-7.

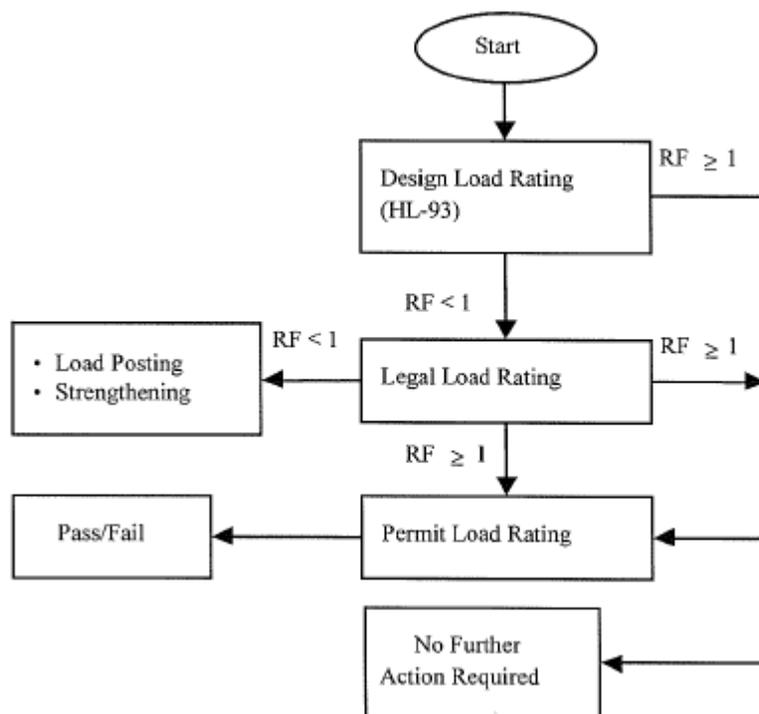


Figure 10-7: Flow Chart of the Load and Resistance Factor Rating Method (American Association of State Highway and Transportation Officials , 2003)

The first step in LRFR rating is to check the design load rating. This rating uses the HL-93 design vehicle as the live load and results in a factor relative to 1.0. If the inventory and operating load rating for a given bridge exceeds one, then the bridge will be able to withstand the AASHTO legal loads also, so no further load rating needs to be done, and no posting is required. If the design load ratings are less than one, then legal load rating needs to be completed. If the legal load ratings are greater than one, then the bridge does not require posting however if the load ratings are less than one, then load posting or

strengthening is necessary. Legal load ratings less than one means the bridge is unable to carry the loads which simulate a large range of the truck traffic on the roads today. These two load ratings determine whether or not a bridge needs to be load posted for typical traffic. In addition to these load ratings, the LRFR also has permit load rating which can be used when issuing permits to vehicles. The permit loading is a pass/fail based on whether or not the load rating is greater than or less than one. A permit load rating greater than one means the permit is allowable for the bridge load rated. All bridges on the haul route would need to be checked for the permit to be approved. This is true even if a bridge is acceptable for the design load ratings.

There is one main equation used in the LRFR method. This equation is valid for design load rating, legal load rating and permit load rating. Additionally, this equation is valid for all the various limit states. Equation 20 is the load rating equation for the LRFR method and each variable in the equation is described beneath the equation.

$$RF = \frac{C - \gamma_{DC} * DC - \gamma_{DW} * DW + \gamma_P * P}{\gamma_L * (LL + IM)} \quad (20)$$

C = capacity of the bridge

γ_{DC} = LRFD load factor for structural components and attachments

DC = Dead-load effect due to structural components and attachments

γ_{DW} = LRFD Load factor for wearing surfaces and utilities

DW = Dead-Load effect due to wearing surfaces and utilities

γ_P = LRFD load factor for permanent loads other than dead loads = 1.0

P = Permanent Loads other than Dead Loads

γ_L = Evaluation Live-Load Factor

LL = Live Load Effects

IM = Dynamic Load Allowance

The first variable in the load rating equation is the capacity of the bridge. When looking at strength limit states, there are three evaluation factors which are multiplied by the

capacity of the bridge which can reduce the strength of the bridge. These factors include the resistance factor, the condition factor and the system factor. Equation 21 shows the equation used to determine the capacity of the bridge for strength limit state load ratings. One limit to these evaluation factors is that $\phi_s * \phi_c$ must be greater than or equal to 0.85.

$$C = \phi * \phi_s * \phi_c * R_n \quad (21)$$

The first factor is the LFRD resistance factor, ϕ which is the same resistance factor that would be used for new bridge design. For steel bridges, the resistance factor is 1.0 for both moment and shear calculations. This information can be found in Section 6.5.4.2: Resistance Factors in the AASHTO Bridge Manual. For reinforced concrete girders, the resistance factors are 0.9 for moment and shear calculations for normal weight concrete. This can be found in Section 5.5.4.2.1 of the AASHTO Bridge Design Specifications Manual (American Association of State Highway and Transportation Officials, 2004).

The next factor that must be determined is the condition factor. This factor is based on the NBI superstructure rating which are based on a visual inspection of the current condition of the bridge. Two steps are involved in the process of determining the condition factor. First, based on the superstructure NBI rating, an equivalent member structural condition is assigned to the bridge, either good, fair or poor. Next, these structural condition factors are matched with a condition factor which is used in the load rating. If the bridge is considered in good condition, the factor is one and the capacity of the bridge is not reduced. Table 10-2 shows what equivalent member conditions are associated with what NBI ratings and Table 10-3 states what condition factor is associated with the structural condition of the member. Both of these tables can be found on page 6-15 in the AASHTO Condition Rating Manual (American Association of State Highway and Transportation Officials, 2003).

Table 10-2: Equivalent Condition of Structural Member Based on the NBI superstructure Rating

Superstructure NBI Rating	Structural Condition of the Member
≥ 6	Good/Satisfactory
5	Fair
≤ 4	Poor

Table 10-3: Condition Evaluation Factor Based on Structural Condition of the Member

Structural Condition of the Member	ϕ_c
Good/Satisfactory	1
Fair	0.95
Poor	0.85

The final evaluation factor is the system factor which is based on the redundancies within the structure. Table 10-4 is a simplified list of the different system factors associated with different bridge types. For more exact system factors, the LRFD Bridge Specifications manual system factors should be used. For shear strength calculations, a system factor of 1.0 should be used.

Table 10-4: System Evaluation Factor for LRFR Load Rating

Superstructure Type	ϕ_s
Welded Members in Two-Girder/Truss/Arch Bridges	0.85
Riveted members in Two-Girder/Truss/Arch Bridges	0.9
Multiple Eyebar Members in Truss Bridges	0.9
Three-Girder Bridges with Spacing \leq 6ft.	0.85
Four-Girder Bridges with Spacing \leq 4ft.	0.95
All other Girder Bridges and Slab Bridges	1
Floor beams with Spacing greater than 12ft and Non-Continuous stringers	0.85
Redundant Stringer Subsystems between floor beams	1

When performing load rating calculated for service limit states, the capacity, C is simply equal to the allowable stress in the beam. No factors are included in the service limit states.

As stated previously, the main thing that distinguishes the different load ratings, i.e. design load rating from legal load rating is the load factors. The load factors differ based on bridge type, the limit state being used and the type of load being applied. Additionally, for the design load rating, there are separate live load factors for inventory and operating. Table 10-5 from the AASHTO Condition Manual displays all the load factors that are used in LRFR load rating.

Table 10-5: Dead and Live Load Factors for LRFR Load Rating (American Association of State Highway and Transportation Officials, 2003)

Bridge Type	Limit State*	Dead Load <i>DC</i>	Dead Load <i>DW</i>	Design Load 6.4.3.2.1		Legal Load 6.4.4.2.1	Permit Load 6.4.5.4.1
				Inventory	Operating		
				<i>LL</i>	<i>LL</i>	<i>LL</i>	<i>LL</i>
Steel	Strength I	1.25	1.50	1.75	1.35	Table 6-5	–
	Strength II	1.25	1.50	–	–	–	Table 6-6
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.75	–	–	–
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Table 6-5	–
	Strength II	1.25	1.50	–	–	–	Table 6-6
	Service I	1.00	1.00	–	–	–	1.00
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35	Table 6-5	–
	Strength II	1.25	1.50	–	–	–	Table 6-6
	Service III	1.00	1.00	0.80	–	1.00	–
	Service I	1.00	1.00	–	–	–	1.00
Wood	Strength I	1.25	1.50	1.75	1.35	Table 6-5	–
	Strength II	1.25	1.50	–	–	–	Table 6-6

* Defined in the *AASHTO LRFD Bridge Design Specifications*.

In addition to looking at strength limit states, when using the LRFR method, each steel bridge was also load rated for service II loads. The load rating equation is the same as when looking at strength limits, except using stress limits instead of strength limits.

$$RF = \frac{f_R - \gamma_D * f_D - \gamma_{DW} * f_{DW}}{\gamma_L * f_{LL_IM}} \quad (22)$$

For steel bridges, only the tension flange was checked for service as the compression flange does not typically govern. (LRFR code) The flange stress due to service two loads is a function of whether the girder and deck are acting composite. If there is composite action, the flange stress is based on equation 23. If the capacity of the bridge is solely based on the steel girders, meaning there is no composite action, the flange stress is based on equation 24. The flange stress is a function of the yield strength of the steel as well as the R_h , the hybrid factor. For rolled shapes, the hybrid factor is taken as 1.0.

$$f_R = 0.95 * R_h * F_y \quad (23)$$

$$f_R = 0.80 * R_h * F_y \text{ (24)}$$

The stresses imposed on the bridge due to dead and live loads are determined by dividing the moment as a result of the type of load by the section modulus of the girder. The non-composite dead load stresses are calculated using the non-composite section modulus and the composite dead loads are calculated using the long term composite section modulus. The long term section modulus is calculated by multiplying the modular ratio by three. Lastly, live loads are calculated using the short term section modulus. The short term modulus is calculated using the original modular ratio.

10.5 LFR vs. LRFR

There are several differences between the current two available methods for load rating. One main difference is in the type of design vehicle used for the load rating. The older LFR method uses the larger moment/shear effects between the HS20 and the tandem design vehicles where as the LRFR method is based on the HL-93 vehicle which is a combination of the HS20/tandem vehicle plus a uniform lane load to represent other traffic on the bridge simultaneously with the design vehicle.

One large benefit to the LRFR method is that all the load ratings are enveloped by the design load ratings. This means that if the design load ratings are greater than one then the legal load ratings will also be greater than one. This can eliminate calculations that need to be done when load rating a bridge. When using the LFR method, the design and legal loads must both be checked because acceptable design load ratings do not always result in legal load ratings that are also acceptable.

Another benefit to the LRFR method is that the legal load ratings take into account the AADT on the bridge. The live load factor is a function of the AADT. This could be beneficial for low level traffic bridges because the live load factor decreases as the traffic volume on a bridge decreases.

In addition to the few beneficial aspects of the LRFR method, there are several differences which are neither beneficial or a hindrance to the load rating, but are solely just differences between the two methods. A sample of these differences is described here. In the LFR rating method, there are dead load effects and live load effects with factors connected to each type of effect. In the LRFR method, there are still dead and live load effects however the dead load effects are broken up into three separate variables, DC, DW and P. These variables were previously described in Section 10.4. Each of these dead load variables also have unique dead load factors associated with them.

The method for calculating the dynamic allowance factor differs between the two methods. In the LRFR method, the dynamic allowance is always calculated at 33% however when load rating using the LFR method, the dynamic allowance is based on the span length and is always less than or equal to 30%.

As stated before, the two load rating methods can not be directly compared. This is due in part to all the differences in the two methods that have been described above. The main difference between the method remains that different design vehicles are used for the two methods.

10.6 Templates for future use

In order to load rate the bridges, templates were created using MathCAD. These templates are set up so that the user can put in data that is specific to the bridge and then eventually the template will yield results for load ratings for both the LFR and LRFR methods. The template was set up to allow for ease of use for future users. Instances where inputs need to be put in based on specific bridge information are highlighted. Additionally, the templates include tables from the AASHTO LRFR manual (American Association of State Highway and Transportation Officials, 2003). Having these tables within the template helps eliminate the need for extra references while load rating. One example of the information provided in one of the tables is the strength of concrete which should be used based on the date the bridge was built if the actual strength is unknown.

The templates were originally created based off design examples found in appendices A1 and A2 in the AASHTO LRFR manual. Using the examples in the manual allowed for checking of the accuracy of the templates. After using the design examples as a starting point, additional changes were made to the templates in attempt to make the load rating process more user friendly.

Two concrete bridge templates were completed, one for an exterior girder and one for an interior girder. Both templates are set up for positive moment, so additional templates would need to be made in order to load rate multi-span structures.

Four steel templates were made so that they can be applied to multi-span bridges. Two of the templates are set up to load rate exterior girders, one for positive moment and the other for negative moment. The two other templates are for interior girders and are set up

similarly, one for positive moment and the other for negative moment. An example of these four templates load rated for bridge B60-0005 can be found in Appendix A5.

10.7 Load Rating Results for Selected Bridges

As stated previously, seven bridges were load rated. All seven of these bridges are of concern to the logging industry due to load postings. Four of these bridges are double span steel girder bridges. For these double span bridges, both positive and negative moment were checked for interior and exterior spans. The three remaining bridges were single span structures, two of which are steel girder bridges and one which is a concrete t-beam bridge. These structures were checked for positive moment on both interior and exterior spans. For each bridge, LRFR ratings and LFR ratings were computed. The results for each of the seven bridges are summarized below. Tables including the load rating results for both load rating methods for each bridge can be found in Appendix A7. Appendix A8 lists all the inputs that were used in the load ratings for each bridge.

10.7.1 Results for B37-0006

The primary load rating check is the design load rating which uses the HL-93 vehicle as a live load on the bridge. The LRFR design inventory and operating results for bridge B37-0006 are greater than one for moment, shear and service checks. Because of this, the legal load rating checks are not necessary but these load ratings were still calculated as a check. The legal load ratings are also greater than one with the moment check equal to 1.42. Overall, the LRFR results calculated for this project show that the bridge is capable of carrying loads with effects equal to the HL-93 indefinitely without decreasing the life of the

bridge. Table 10-6 summarizes the load ratings for both the LRFR method and the LFR method.

Table 10-6: Summary of Load Rating Results for Bridge B37-0006

B370006			
LRFR	Inventory	Moment	1.314
	Operating		1.703
	Inventory	Shear	2.506
	Operating		3.248
	Inventory	Service	1.05
	Operating		1.365
	Legal Loads	Moment	1.42
	Legal Loads	Service	1.167
LFR	Inventory	Moment	HS27
	Operating		HS46
	Inventory	Shear	HS50
	Operating		HS84
Inspection Report	Inventory		HS19
	Operating		HS32

Next, LFR results were calculated and compared to the LFR results found in the most recent inspection report for the bridge. The calculated results showed that moment controlled with an inventory rating of HS27 and an operating rating equal to HS46. These results differed greatly with the inspection report ratings which had an inventory rating of HS19 and an operating rating of HS32. Because of this large difference, several different variations of the original load rating were done to try to find out what properties were used in the inspection report load rating. Table 10-7 shows the results from these variations.

Table 10-7: LFR Load Rating Variation for Bridge B37-0006

	WisDOT Results	Composite Results	Non-Composite Results	5 Girders Composite
Inventory	HS19	HS27	HS14	HS19
Operating	HS32	HS46	HS24	HS32

Initially, the bridge was load rated as composite. The bridge underwent large improvements in 2001 included a new deck so it was assumed the new deck would be composite to the steel girders. The first variation completed was to load rate the bridge as non-composite. This yielded an inventory rating of HS14 and an operating rating of HS24. These ratings are closer to the inspection report results, however there is still a large difference in the ratings. In addition to the deck improvements in 2001, the bridge was also widened to include nine girders. Previous to 2001, the bridge consisted of five girders. The final variation which was load rated used five composite girders. This variation yielded inventory and operating ratings which matched with the inspection report exactly. When load rating the bridge as a five girder structure, the live load moment and shear distribution factors increased and the composite dead loads on each girder also increased. This resulted in the load ratings less than the ratings for a nine girder structure.

Based on these results, it is probable that the load rating reported on the inspection report is out of date and is based on the structure pre 2001 when the bridge underwent large renovations. The load ratings calculated for this project show that the bridge does not need to be load posted and is acceptable for carrying the design loads of the HL-93 vehicle.

10.7.2 Results for B26-0002

The first results for bridge B26-0002 that were calculated were the LRFR results. For the design load rating the service checks controlled, resulting in inventory ratings that were less than one and an operating rating of 1.014. The design load rating checks for moment were very similar to the service checks as the inventory rating was less than one and the operating rating was 1.087. The shear checks had an inventory rating of 3.393 and an

operating rating of 4.398. Because both the moment and service design ratings were less than one the legal loads were checked next. For both moment and service, the legal loads checks were found to be less than one so this bridge was found to either require load posting or bridge strengthening.

Next, the LFR results were calculated and compared to the inspection report's LFR load ratings. The results calculated for this project were controlled by moment with an inventory rating of HS19 and an operating rating of HS32. The inspection reports yielded slightly higher ratings with an inventory rating of HS21.6 and an operating rating of HS36. The discrepancy in these results is minimal due to small differences in load rating techniques. A summary table of the controlling results can be found below in Table 10-8.

Table 10-8: Summary of Load Rating Results for Bridge B26-0002

B260002			
LRFR	Inventory	Moment	0.838
	Operating		1.087
	Inventory	Shear	3.393
	Operating		4.398
	Inventory	Service	0.78
	Operating		1.014
	Legal Loads	Moment	0.833
	Legal Loads	Service	0.798
LFR	Inventory	Moment	HS19
	Operating		HS32
	Inventory	Shear	HS93
	Operating		HS155
Inspection Report	Inventory		HS21.6
	Operating		HS36

10.7.3 Results for B60-0005

Looking at the LRFR results, the bridge does not have load ratings which exceed one when checking moment and service for the design load rating. Checking shear, the bridge

does have load ratings exceeding one for both the inventory and operating ratings. Next, the results for the legal loads for both moment and service are less than one. These results conclude that when using the LRFR method for load rating, this bridge should be load posted or strengthened to be able to withstand the legal and design loads.

Next, LFR load rating was also completed. The most recent inspection report reported that the inventory rating of the bridge was an HS14 and the operating rating was a HS27. Based on the inspection report, it can not be determined what check controls, whether it be moment or shear. For this project, both moment and shear were checked and for B60-0005, moment controlled the load ratings. The load ratings for moment using the LFR method were HS16 for inventory and HS27 for operating. These results are very close to the results found in the inspection report.

When comparing the two methods results, both the LFRF and the LFR methods are giving ratings which tell that the bridge is not capable of carrying the design loads and needs to either be load posted or strengthened. Table 10-9 summarizes the load rating results for bridge B60-0005 by showing the load rating which controls for a given check.

Table 10-9: Summary of Load Rating Results for Bridge B60-0005

B600005			
LRFR	Inventory	Moment	0.574
	Operating		0.744
	Inventory	Shear	1.138
	Operating		1.475
	Inventory	Service	0.713
	Operating		0.927
	Legal Loads	Moment	0.534
	Legal Loads	Service	0.682
LFR	Inventory	Moment	HS16
	Operating		HS27
	Inventory	Shear	HS24
	Operating		HS41
Inspection Report	Inventory		HS14
	Operating		HS27

10.7.4 Results for B06-0013

The results for bridge B06-0013 are promising for the logging industry. The LRFR design vehicle load ratings were greater than one for moment, shear and service checks. Additionally, the LFR results were greater than or equal to the HS20 for the inventory rating for both moment and shear.

First, looking at the LRFR results, the inventory ratings for moment, shear and service are all very close to one but they are also all greater than one. In this situation, the load rating could be considered complete as all the load ratings are enveloped by the design loads. This means that if it passes the design loads, it will be greater than one for all the loads. Even though it wasn't needed, the legal loads were still checked to ensure the load rating was successful. Both the moment and service results were greater than one and also were greater than the design load rating results.

Lastly, the LFR results showed that for this method shear controlled with an inventory rating of HS20 and an operating rating of HS34. These results match up fairly well with the LRFR ratings in that the bridge is acceptable for HS20 loads indefinitely but not loads any heavier. The LRFR ratings also allowed for the design vehicle, the HL-93, to run over the bridge indefinitely, however it couldn't handle loads much heavier than that vehicle. In comparing the LFR ratings done for this project to the inspection report ratings, it can be identified that there is a very large gap between the two ratings. The inspection report has an inventory rating of HS41; however it is also a load posted bridge at 45ton. It is likely that there is an error in the inspection report ratings because bridges built in the early 1950's were unlikely to be designed to carry loads as large as what the HS41 vehicle would have. A summary table of the load rating results for B06-0014 can be found in Table 10-10.

Table 10-10: Summary of Load Rating Results for Bridge B06-0013

B060013			
LRFR	Inventory	Moment	1.115
	Operating		1.446
	Inventory	Shear	1.098
	Operating		1.423
	Inventory	Service	1.06
	Operating		1.378
	Legal Loads	Moment	1.9
	Legal Loads	Service	1.294
LFR	Inventory	Moment	HS22
	Operating		HS37
	Inventory	Shear	HS20
	Operating		HS34
Inspection Report	Inventory		HS41
	Operating		HS58

10.7.5 Results for B37-0043

Looking first at the LRFR results for moment and service loads, bridge B37-0043 had inventory ratings less than one. The operating result for moment was .695 and the operating rating for service loads was 1.059. The shear design load ratings are acceptable at 1.667 and 2.161 for inventory and operating respectively. Because three out of the four moment and service load ratings were less than one, the legal loads were checked. The legal loads for bridge B37-0043 were found to be less than one for both flexure and service loads. Based on these LRFR results, load posting or bridge strengthening is necessary.

Looking at the LFR results, it can be identified that moment again controls the load ratings. The moment load ratings were HS24 for inventory and HS40 for operating. These were higher than the load ratings found in the most recent inspection reports for this bridge which was HS13 for inventory and HS21 for operating. There was nothing found as to why this discrepancy occurred between the ratings calculated for this project and the reported ratings found in the inspection report. Table 10-11 shows a summary table of the controlling load ratings for both LRFR and LFR.

Table 10-11: Summary of Load Rating Results for Bridge B37-0043

B370043			
LRFR	Inventory	Moment	0.536
	Operating		0.695
	Inventory	Shear	1.667
	Operating		2.161
	Inventory	Service	0.814
	Operating		1.059
	Legal Loads	Moment	0.772
	Legal Loads	Service	0.848
LFR	Inventory	Moment	HS24
	Operating		HS40
	Inventory	Shear	HS47
	Operating		HS79
Inspection Report	Inventory		HS13
	Operating		HS21

10.7.6 Results for B37-0094

The controlling LRFR design load rating results for this bridge are much less than one meaning that legal load rating must be done in order to check if load posting of the bridge is necessary. Both the moment and service load ratings were less than one with only the shear checks resulting in load ratings which exceeded one. In the case of this bridge, the moment load ratings controlled with an inventory rating of .353 and an operating rating of .457. Following the design load ratings, legal load ratings were calculated for moment and service. Again, both load ratings were less than one which means that based on these results, load posting or strengthening is necessary.

Next, looking at the LFR results, it is clear that moment again controls over shear. The moment load ratings were calculated to be HS13 for inventory and HS22 for operating. These results were very close to the load ratings reported in the inspection reports with an inventory rating of HS15 and an operating rating of HS24. Small differences between the

two ratings is expected as there are likely to be small differences in the load rating process use by two different people performing load ratings. The LFR ratings also show that the bridge can not withstand the design loads and that load posting may be necessary. A summary table of the results for bridge B37-0094 can be found in Table 10-12.

Table 10-12: Summary of Load Rating Results for Bridge B37-0094

B370094			
LRFR	Inventory	Moment	0.353
	Operating		0.457
	Inventory	Shear	1.183
	Operating		1.533
	Inventory	Service	0.567
	Operating		0.736
	Legal Loads	Moment	0.373
	Legal Loads	Service	0.562
LFR	Inventory	Moment	HS13
	Operating		HS22
	Inventory	Shear	HS28
	Operating		HS48
Inspection Report	Inventory		HS15
	Operating		HS24

10.7.7 Results for B38-0513

Bridge B38-0513 was load rated slightly differently than the other bridges. Due to its complicated geometry, this bridge was modeled in SAP2000. The effects on individual girders to dead and live loads were then pulled from the model and used in the load rating. Three girders were load rated for this bridge. This is because the girders had different span lengths and reinforcement making it difficult to determine which beam controlled. The first beam load rated was an original interior girder. This girder was labeled girder six. Additionally, girders added during the widening of the structure in 1948 were also load rated. The exterior girders were not load rated because they are not subjected to direct live loads as

they come up above the driving surface. Instead, girders two and ten were load rated. Figure 10-8 shows the three girders which were load rated.

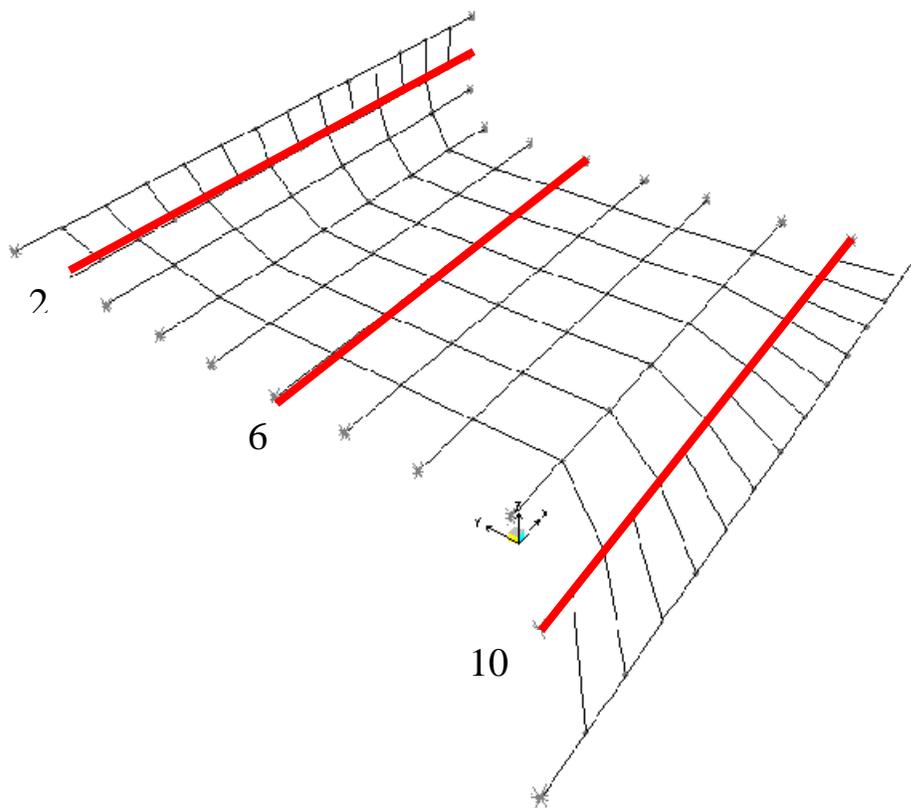


Figure 10-8: Figure of Girders Load Rating on Bridge B38-0513

Table 10-13 shows the controlling results for bridge B38-0513. The LRFR results for moment and shear that are less than one however the LFR results for moment are above an HS20 rating with an inventory rating of HS21 and an operating rating of HS36. Next, looking at the shear results, both the LRFR and LFR results give ratings which are undesirable. The LRFR results for shear are 0.237 for inventory and 0.307 for operating. The LRF ratings are HS04 and HS07. Based on these results, it is likely that shear also controlled the load ratings which are found in the inspection report. The reported load ratings from the inspection report are HS09 for inventory and HS24 for operating. These

ratings don't match exactly with the inspection report ratings but nothing was found as to why they differ slightly. Due to the complexity of this bridge, legal load ratings were not completed but in actual load rating of this bridge, they would need to be checked as the LRFR ratings were less than 1.0 and the LFR ratings were less than an HS20 for some of the checks.

Table 10-13: Load Rating Results for Bridge B38-0513

B38-0513			
LRFR	Inventory	Moment	0.742
	Operating		0.977
	Inventory	Shear	0.237
	Operating		0.307
LFR	Inventory	Moment	HS21
	Operating		HS36
	Inventory	Shear	HS04
	Operating		HS07
Inspection Report	Inventory		HS09
	Operating		HS24

10.8 Load Posting of Bridges

Based on the LRFR ratings, four of the seven bridges had legal load ratings which were less than one. This means they require load posting. One of the seven bridges was not checked for legal load rating, which was B38-0513 due to the complex geometry of the bridge. Determining the load posting for each bridge was completed based on the Wisconsin Bridge Manual methods (Wisconsin Department of Transportation, 2008).

$$Posting = \frac{W}{0.7} * [(RF) - 0.3] \quad (25)$$

The RF is the legal load rating factor calculated and the W is the weight of the rating vehicle that was used for the load rating. Based on Equation 25, the posting weight for a bridge can be determined. The posting is equal to the lowest posting weight of the nine trucks that are

checked in the legal load rating process. Only trucks which yield a load rating less than one need to have a posting weight calculated. Table 10-14 shows the posting weight for each of the six bridges which were load rating for legal vehicles. Only vehicles which yielded a legal load rating less than one were included in determining the bridge posting.

Table 10-14: Load Posting Results

Bridge	Controlling Vehicle	Gross Weight of Vehicle (ton)	LRFR Rating	Posting (Ton)	Bridge Posting (Ton)
B37-0006	ALL OK				
B26-0002	SU6	34.75	0.905	30.0339	29.51
	SU7	38.75	0.833	29.5054	
	PUP	49	0.86	39.2	
B60-0005	Type 3S2	36	0.75	23.1429	16.38
	Type 3-3	40	0.747	25.5429	
	SU4	27	0.985	26.4214	
	SU5	31	0.898	26.4829	
	SU6	34.75	0.811	25.3675	
	SU7	38.75	0.75	24.9107	
	Pup	49	0.665	25.55	
Semi	49	0.534	16.38		
B06-0013	ALL OK				
B37-0043	SU6	34.75	0.981	33.8068	32.44
	SU7	38.75	0.886	32.4393	
	Pup	49	0.772	33.04	
	Semi	49	0.808	35.56	
B37-0094	Type 3	25	0.732	15.4286	5.11
	Type 3S2	36	0.542	12.4457	
	Type 3-3	40	0.51	12	
	SU4	27	0.657	13.77	
	SU5	31	0.586	12.6657	
	SU6	34.75	0.528	11.3186	
	SU7	38.75	0.483	10.1304	
	Pup	49	0.435	9.45	
Semi	49	0.373	5.11		

Based on the results, the load posting for the five bridges that require posting range from 5.1 to 32.4 ton. Bridge B37-0094 had the lowest posting at just over five ton. One thing to note is that by excluding the WisDOT semi vehicle, the posting weight could be almost doubled. This is an excellent example of how certain configurations can have much more adverse effects on a bridge and why posting solely based on gross weight is not always beneficial to all truck configurations.

10.9 Comparison and Summary of Results

Seven bridges in total were load rated in conjunction with this project. Two of these bridges were single span steel girder bridges. For both of these bridges the LRFR ratings were controlled by the exterior girder service ratings. When looking at LFR ratings, both were governed by flexure ratings.

Four double span bridges were also load rated for this project. All of these bridges were controlled by negative moment, either on an interior girder or an exterior girder. Three of the four girders were controlled by moment, the fourth bridge, B06-0013 being controlled by service on the exterior girder. Looking at the LFR results, there was no trend in what controlled the rating. Two of the bridges were controlled by the negative flexure rating on an interior beam and one was controlled by the shear on an exterior beam. The final of the four bridges was controlled by flexure but had the same rating for both negative interior and negative exterior beams.

The final bridge that was load rated was B38-0513. This bridge was a reinforced concrete t-beam single span bridge and the LRFR ratings were controlled by shear. The

same controlled for LFR ratings. Only interior beams were load rated for this bridge as the exterior girders are not subjected to direct live load.

Additionally, once the load ratings were completed, a meeting was held with the Wisconsin DOT to discuss and compare the results. Table 10-15 looks just at the controlling LFR results for each bridge calculated for this project and the LFR results from the most recent inspection report.

Table 10-15: Comparison of Load Factor Rating Results

		B370006	B260002	B600005	B060013	B370043	B370094	B380513
Inspection Ratings	Inventory	HS19	HS21.6	HS14	HS41	HS13	HS15	HS09
	Operating	HS32	HS36	HS27	HS58	HS43	HS24	HS24
LFR Ratings	Inventory	HS27	HS19	HS16	HS22	HS25	HS13	HS04
	Operating	HS46	HS32	HS27	HS37	HS43	HS22	HS07

Three of the bridges load rated matched up very closely to the inspection report ratings. The bridges were B26-0002, B60-0005 and B37-0094. The rating are within two ratings, i.e. HS13 to HS15, and this small variance is expected due to differences in the engineer's load rating process.

Bridges B37-0006, B060013 and B370043 have the largest variance between the ratings completed for this project and the ratings found in the inspection reports. It is expected that the difference in the B37-0006 bridge is due to a mistake in the inspection report ratings. The bridge was widened in 1997 and it is possible these new girders were not included in the most recent rating. Next, B06-0013 is the bridge which has the most discrepancy between ratings. It is very unlikely the bridge actually has an inventory rating of an HS41 so it is again likely there is a mistake in the rating. Additionally, this bridge is posted at 40ton and this would not be necessary if the inventory rating was truly an HS41.

The last bridge with a discrepancy in ratings is B370043. Nothing was found as an indicator to why these ratings differ.

The final bridge that was load rated was B38-0513. This bridge matched up fairly closely for the inventory rating, however, the operating ratings were very different. The reason for this discrepancy is unknown.

Based on the inspection reports, three of the seven bridges that were load rated were load posted at 45ton. As stated in Chapter 4, the load posting of bridges in Wisconsin is based on nine different vehicles. The load posting is equal to the lowest restricted gross weight of these nine vehicles. Based on the LRFR results from this project, all but two of the bridges would require load posting or strengthening. B06-0013 and B37-0006 were the two structures that were had all legal load ratings and design load ratings greater than one. Based on the inspection reports, B06-0013 is a load posted bridge at 45ton which does not require posting based on the load ratings completed for this project. One of the seven bridges was not load rated for legal loads due to the complicated geometry. This bridge was B38-0513. Only the HL-93 vehicle was used in the LRFR ratings so it is unknown how the nine legal vehicles effect the bridge. The load ratings that were completed using the HL-93 were very low and it is likely the legal load ratings which use the AASHTO commercial vehicles would also be less than one, meaning load posting would be necessary.

11 Vehicle Configuration Optimization: Solution 1

As noted earlier, there are long spans beyond which the effects from the average logging truck on a bridge will exceed the design vehicle effects. It was also seen that the effects of the average logging truck are always greater than those of the HS15 design truck regardless of span length. Those conclusions clearly indicate that there are bridges that the average logging truck will not be able to use. One possible solution to this dilemma is to decrease the effects the logging vehicles have on bridges. This could occur if optimization of the logging vehicle shape and weight decreased the moment and shear effects the trucks will have on bridges. Three different optimization options were utilized and are described below.

11.1 Percent Reduction in Gross Vehicle Weight to Reduce effects to equal the design vehicles

One possible solution for reducing the effects the logging trucks have on bridges is to reduce the gross weight of the vehicle. This would only be a viable solution if the reduction of load was minimal so that it was still economical to use the vehicles. The HS20 and HS15 design vehicles were used in this analysis as a basis of comparison because many bridges have load ratings which accept these design vehicles. Three average logging trucks were also included in the analysis, the average five-axle truck, the average six-axle truck and the average five-axle truck and pup.

The first analysis was done using the average five-axle logging vehicle. With its current gross weight of 94,000lb, the average five-axle vehicle exceeds the moment of the HS20 at a span of 84ft and exceeds the HS20's maximum shear at a span of 68ft. Additional analyses of the effects of that truck were conducted with reduced gross weight, but same proportion of weight on each axle. This assumption simplifies the analysis because in reality

the weight of the front two axles would not change much even if the pay load weight decreased. The front two axles support the cab of the truck and don't carry much of the weight from the timber. With this in mind, the analysis is still is a good representation of how the effects change when the gross weight is reduced. Figure 11-1 shows the moment comparison of the HS20 to the five-axle vehicle at different percentages of the allowable gross weight of 94,000lbs.

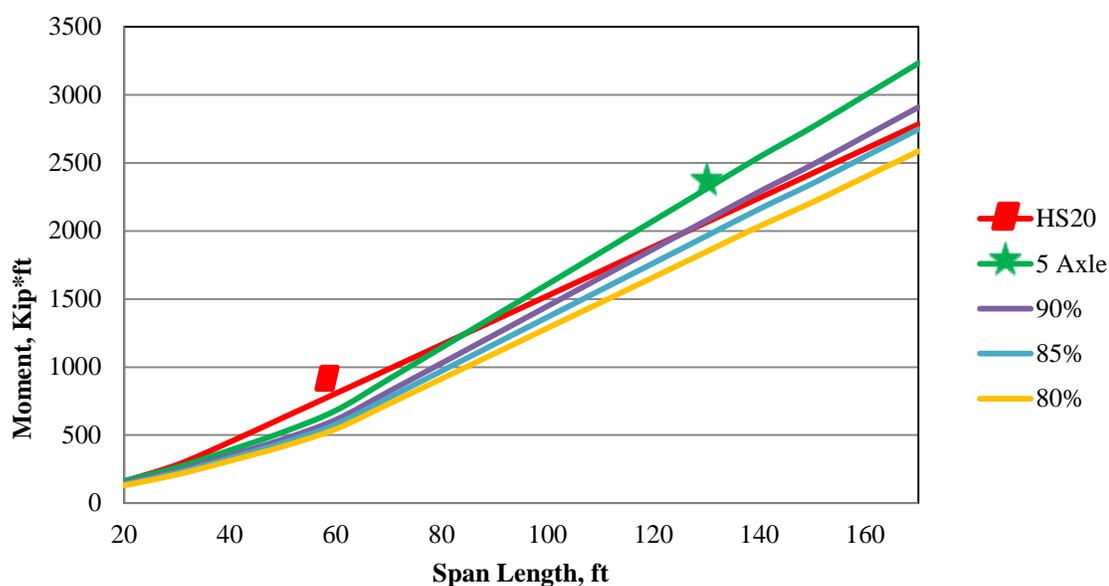


Figure 11-1: Percentage of Gross Weight of Average Five-axle Vehicle Moments Compared to HS20

From Figure 11-1, it was identified that the gross weight of the average five-axle vehicle would have to be reduced to 85%, equal to 79,900lbs, to make it an acceptable truck on HS20 rated bridges up to 170ft in span. Spans longer than 170ft were not investigated. On spans shorter than 170ft the reduction would be less. Next, the same comparison was done with the HS15 design vehicle.

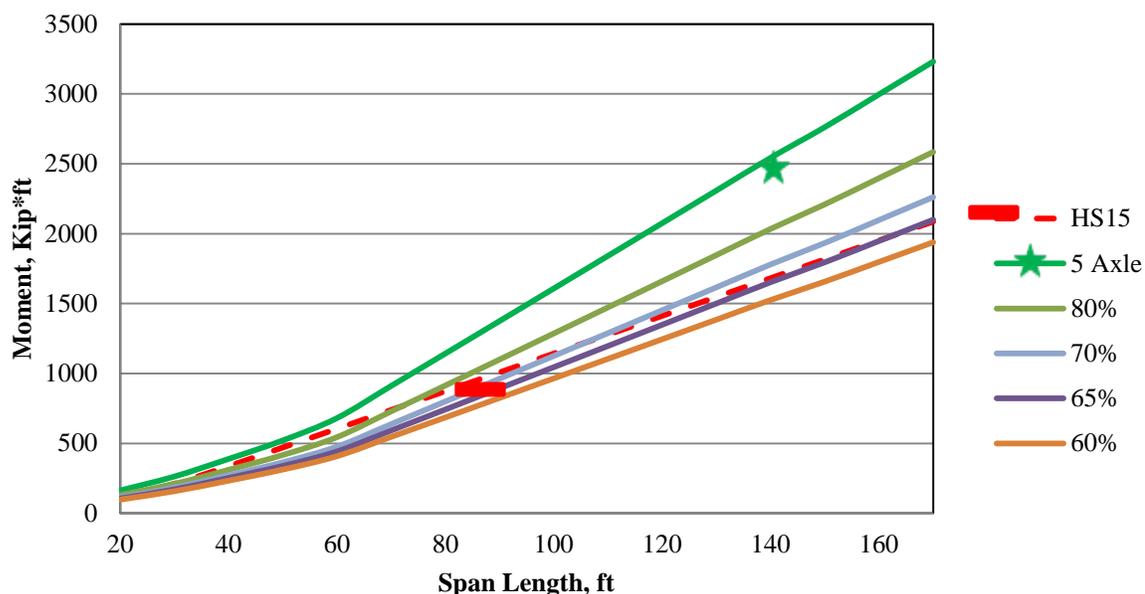


Figure 11-2: Percentage of Gross Weight of Average Five-axle Vehicle moments compared to HS15 Design Vehicle

In order to reduce the moment of the logging vehicle to be less than the HS15, the overall reduction in gross weight was much larger. The gross weight of the logging truck would have to be 60% of 94,000lbs which is 61,100lbs. With a gross weight of 61,100lbs, the logging truck would have less of an effect on the bridge than the HS15 design truck for spans ranges of up to 170ft.

Similar comparisons were completed with the average five-axle truck for shear effects on bridges. Figure 11-3 and Figure 11-4 show the results of the shear comparison of the average five-axle vehicle.

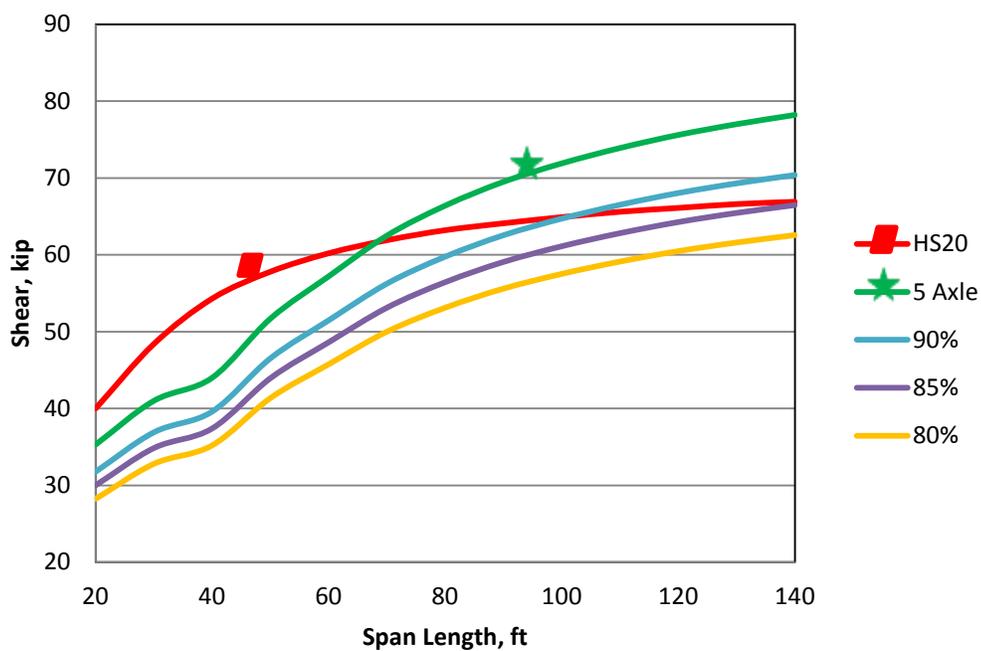


Figure 11-3: Percentage of Gross Weight of Average Five-axle shears Compared to HS20

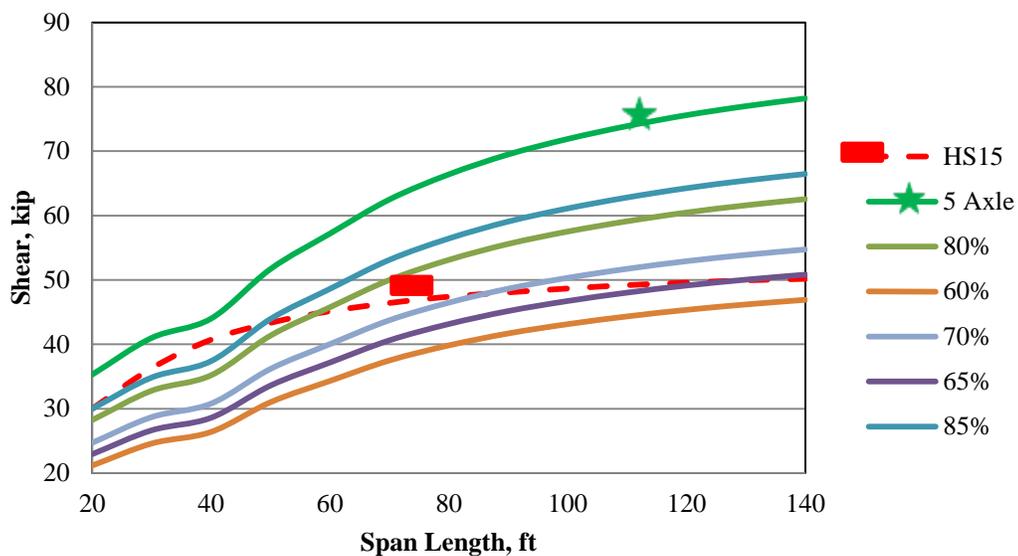


Figure 11-4: Percentage of Gross Weight of Average Five-axle Shears Compared to HS15

Very similar results were discovered when looking at shear comparisons. The first shear comparison looked at the HS20 vehicle. The gross weight of the logging vehicle would have to be reduced to 85% to have shear effects less than the HS20 for spans up to 170ft. Next, when looking at the HS15 design vehicle, the gross weight of the logging vehicle would have to be reduced to 60% in order to have less of a shear effect than the HS15.

The same analysis was done for the average six-axle vehicle and the average five-axle truck and pup. Table 11-1 summarizes the results and percentage of reduction that would be needed in order to meet or be less than the design load limits of HS20 and HS15.

Table 11-1: Summary of Gross Weight Reduction Analysis Results: reduced capacities needed to meet design load limits

	HS20 load limit		HS15 load limit	
	Moment	Shear	Moment	Shear
Span Length	170ft	140ft	170ft	140ft
5 Axle	85%	85%	60%	60%
6 Axle	80%	80%	60%	60%
5 Axle T&P	80%	85%	60%	65%

It is evident from Table 11-1 that regardless of the vehicle configuration, the percent reduction in gross weight is fairly consistent for either of the design load limits. Graphical results for the average six-axle vehicle and the average five-axle truck and pup can be found in Appendix A10.

11.1.1 Conclusions

As stated earlier, in order for the gross weight reduction solution to be acceptable it must be economical. In reality, reducing the gross weight of the vehicle by as much as 40% (for bridges rated at HS15) is not economical because it would require almost twice as many haul trips to carry the same amount of timber. When comparing the reduced logging vehicles

to the HS20, on average the gross weight is being reduced by 15-20%. Depending on the length of the detour to avoid a posted bridge, it may or may not be economically logical to reduce the truck load. Overall, the cost associated with a ten to fifteen mile detour due to a posted bridge is likely cheaper than the cost associated with having to take multiple trips to haul the same amount of timber. Based on these assumptions, this solution will probably not be cost effective for the timber haulers.

11.2 Optimization of Gross Weight Distribution on Individual Axles

The next viable solution investigated to reduce the effects of the logging vehicles on bridges was to look into optimizing the vehicle gross weight distribution among the individual axles, but not changing the total weight. The average five-axle vehicle was used as a base model for this analysis as shown in Figure 11-5.

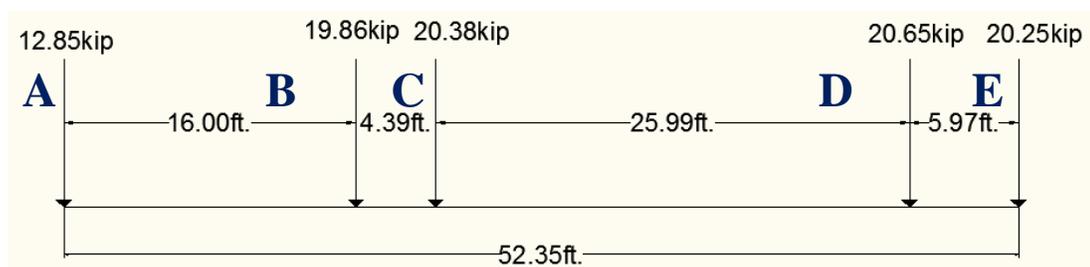


Figure 11-5: Average Five-axle Logging Vehicle

The first step in this analysis was to determine constraints. The first two axles typically carry the load from the cab of the vehicle and it would be difficult to distribute the haul load to those axles. Additionally, based on the measurements taken in the field, the weight distributed to the first and second axles seemed fairly consistent for all vehicles. For these reasons, the weight distributed on axles A and B were held constant at 12.85kip (14%) and 19.87kip (21%) respectively. Next, to determine how much weight could be applied to

an axle, all the individual five-axle vehicles were reviewed and the minimum and maximum percentage of weight on a single axle was recorded. In order to get reasonable results, these ranges were used as boundaries for the maximum and minimum amount of weight that can be distributed on a single vehicle axle. Table 11-2 shows the maximum and minimum percentages of weights allowed on the third, fourth and fifth axles based on the five-axle trucks measured.

Table 11-2: Minimum and Maximum Percentage of Weight observed on Five-axle Logging Vehicles

	3rd Axle	4th Axle	5th Axle
MIN %:	19.75%	20.66%	16.23%
MAX %:	22.57%	24.00%	23.29%

A final constraint was that the overall gross weight must remain at 94,000lbs. Additionally, the effects of all trial trucks were analyzed for an eighty foot bridge span.

The goal of this analysis was to find the weight distribution which had that lowest effect on the bridge, meaning the lowest moment and shear effects. Six trials were run in total. PC Bridge was used to determine the maximum shear and moment effects each trial would have on the eighty foot span. The first, second, and third trials maximized the third, fourth and fifth axle weights respectively. Trial four maximized the fourth axle load and equalized the loads on the third and fifth axles. The fifth trial maximized the fourth axle and minimized the third axle. The final trial, trial six, maximized the third and fourth axle and minimized the fifth axle as much as possible. Table 11-3 summarizes the results.

Table 11-3: Weight Distribution Optimization Results

Trials		Moment (kip*ft)	Shear (kip)
Base	Average 5 Axle Vehicle	1143.00	66.40
1	Maximize 3rd Axle Load	1153.80	66.10
2	Maximize 4th Axle Load	1125.80	66.90
3	Maximum 5th Axle Load	1135.70	66.60
4	Maximize 4 th Axle; equalize 3rd and 5th Axle Loads	1132.5	66.7
5	Maximize 4th Axle; minimize 3rd Axle	1189.1	69.2
6	Maximizes 3rd and 4th Axles; minimize 5th as much as possible	1223.3	69.2

Before any of the trials were completed, a base analysis using the average five-axle vehicle was conducted to form a comparison basis. The results from this base trial can be found in the first row in Table 11-3. Looking at trials one through six, it was determined that two different trials had optimal results, one being optimal for shear and the other optimal for moment. Trial 1 was optimal for shear and had a reduction of 0.3 kips or 1% compared to the base trial. However, Trial 1 also resulted in an increase in moment of 10.8kip*ft or a 1% increase. Table 11-4 shows the weight distribution on each of the five-axles associated with Trial 1 and also reports the shear and moment effects of Trial 1 on an 80ft span.

Table 11-4: Weight Distribution Results for Trial 1

Trial 1: max 3rd Axle	
13.67%	12.84 Kip
21.13%	19.86 Kip
22.57%	21.21 Kip
21.50%	20.21 Kip
21.13%	19.86 Kip
100.00%	
Moment:	1153.8 kip*ft
Shear:	66.1 kip

The next optimal trial was Trial 2. Trial two was optimal in terms of moment with a 17.2 kip*ft reduction compared to the base trial. This is a 2% reduction in moment. Trial 2 also results in an increase in shear compared to the base model of 0.5 kip which is a 1% increase. Table 11-5 shows the axle weight distribution on each axle for Trial 2 and also the moment and shear imposed on an 80ft span by the trial vehicle.

Table 11-5: Weight Distribution Results for Trial 2

Trial 2: max 4th Axle	
13.67%	12.84 Kip
21.13%	19.86 Kip
20.00%	18.8 Kip
24.00%	22.56 Kip
21.20%	19.93 Kip
100.00%	
Moment:	1125.8 kip*ft
Shear:	66.9 kip

Overall, there was no trial which was optimal for both moment and shear. The trial optimal for shear resulted in an increase in moment and the trial optimal for moment resulted in an increase in moment. Additionally, the reductions seen in shear and moment for Trials 1 and 2 were relatively small and would result in very small improvements in the overall effects the five-axle vehicle would have on a bridge. Lastly, determining the truck weight distribution of a timber load in the forest is not an exact science; therefore accurately adjusting the weight by the small percentages presented may not be possible. In summary, the improvements obtained by optimizing the axle weight distribution were marginal and will not result in large improvements. Additionally, based on the trials, the current weight distribution is very near optimal. Optimizing the axle weight distribution is not a reasonable option to obtain noticeable reductions in shear and moment effects for the logging vehicles.

11.3 Axle Configuration Optimization

The final option for optimizing the logging vehicles is to look at the axle configurations of the vehicle. The analysis for this work was completed as part of a UW undergraduate independent study project. For this analysis, the bridge force effects caused by a five-axle logging truck (Figure 11-5) were used as a basis for comparison. Several constraints were used in this analysis. A complete report on this analysis is provided (Hagar, 2009).

Several constraints were used in the analysis. First, the overall length of the truck was held constant at 52.34ft. This is the total length of the average five-axle vehicle. Next, the axle spacing between the first two axles was fixed at 16ft. This constraint was made because the first two axles typically support the cab of the truck and therefore this spacing can not be adjusted. A bridge with a fifty foot span was used to examine the effects of the trial vehicles. PC Bridge was again used to determine the effects for each trial. In total, twenty trial cases were analyzed. Table 11-6 shows the spacings used between the axles and also the maximum moment and shear the trucks imposes on a fifty foot span bridge.

Table 11-6: Axle Spacing Optimization Results

Trial	BC spacing (ft)	CD spacing (ft)	DE spacing (ft)	Maximum Moment (kip*ft)	Maximum Shear (kip)
Base	4.39	25.99	5.97	521.60	51.70
2	3.00	25.00	8.34	532.30	53.00
3	5.00	24.00	6.34	511.00	52.10
4	8.00	24.00	4.34	494.50	53.00
5	10.00	4.00	22.34	627.90	58.10
6	10.00	20.00	6.34	508.10	53.00
7	13.00	6.00	17.34	597.10	54.00
8	13.00	17.34	6.00	536.10	54.20
9	15.00	15.00	6.34	555.20	55.00
10	15.00	6.34	15.00	596.10	52.70
11	20.00	5.00	11.34	612.30	55.00
12	20.00	11.34	5.00	604.50	57.40
13	20.00	4.00	12.34	619.80	54.60
14	20.00	12.34	4.00	605.40	57.90
15	25.00	5.00	6.34	654.30	59.10
16	4.00	12.34	20.00	630.30	57.50
17	12.34	4.00	20.00	617.60	56.80
18	4.00	7.34	25.00	683.80	59.60
19	7.34	4.00	25.00	671.60	60.10
20	12.34	12.00	12.00	534.40	49.30

Based on these results, two different axle configurations were found as optimal. Trial four is optimal in terms of moment and trial twenty is optimal for shear effects. In comparison to the average five-axle vehicle, the configuration used in trial four produces a moment that is 5% less than the average vehicle, however the shear effects on the bridge increase and are 3% larger than the average five-axle vehicle. Trial 20 has similar results. The maximum shear produced by the trial twenty configuration is 2.4kip less which is 5% less than the average five-axle vehicles however the maximum moment caused by the configuration from trial twenty is 12.8kip*ft (2%) larger than the average five-axle. Looking at the two optimized cases, both have drawbacks in that whichever force it is able to be reduced compared to the average vehicle, the other force is increased.

In addition to comparing overall configurations, the effects and trends that each axle spacing has on the overall moment and shear was analyzed. First the axle spacing between B and C was compared to the maximum moment produced by the vehicle on a fifty foot simply supported span.

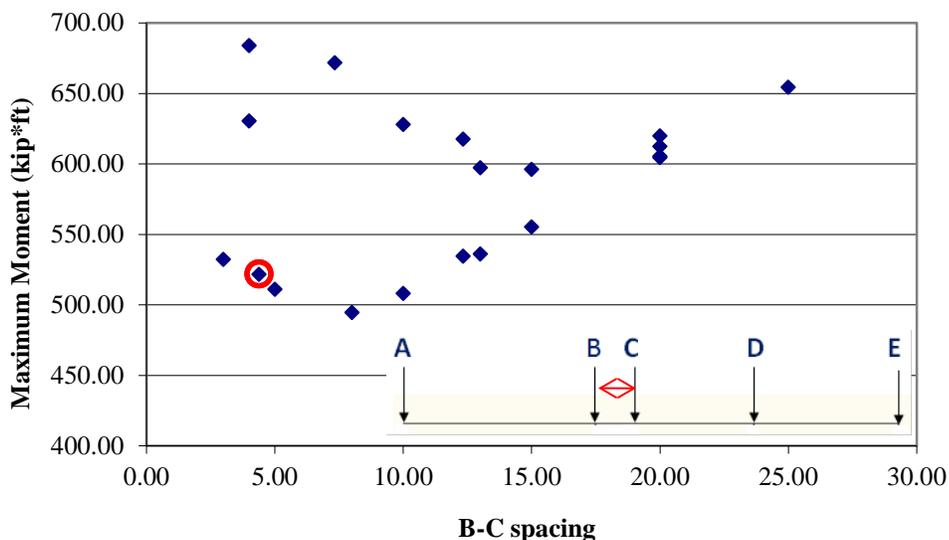


Figure 11-6: Axle Spacing between B-C vs. Maximum Moment

This data has a lot of scatter and no general trend. The data point circled in red signifies the moment produced by the average five-axle vehicle for a B-C spacing of 4.39ft. From Figure 11-6, it can be identified that very few spacings result in a lower moment effect and the average five-axle vehicle spacing between B-C is already very near being optimized.

Next, the spacing between C-D was compared to the maximum moment produced. This overall trend was opposite the one seen for the B-C spacing. As the spacing between C-D increases, the maximum moment decreases, meaning that the larger the spacing between axles C and D, the smaller the overall effects. The red data point again shows the average five-axle vehicles which is nearly optimized with its current spacing of 25.99ft.

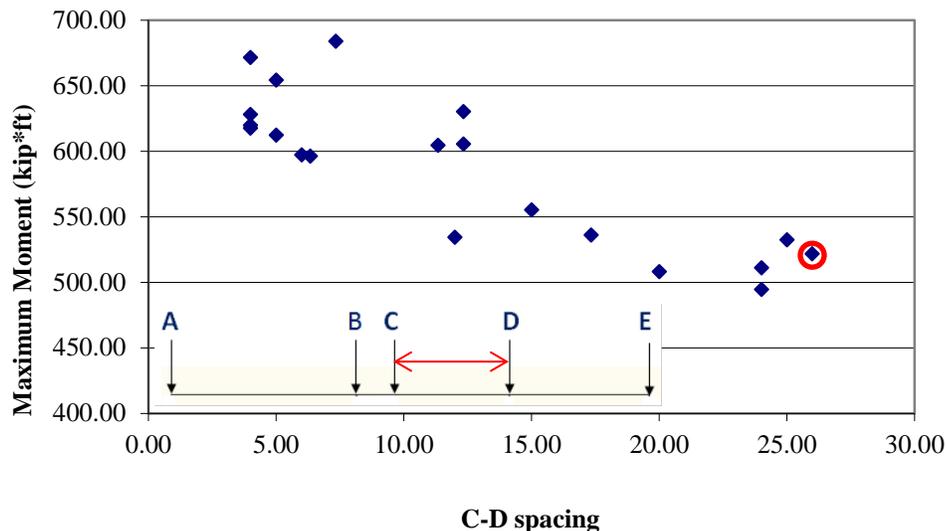


Figure 11-7: Axle Spacing between C-D vs. Maximum Moment

Next, the spacing between D-E was compared to the maximum moment on the bridge. This overall trend was similar to the B-C spacing trend. As the spacing D-E increases, the maximum moment also increases. The average five-axle vehicle, circled in Figure 11-8 below, is near the bottom of the trend at a D-E spacing of 5.97ft.

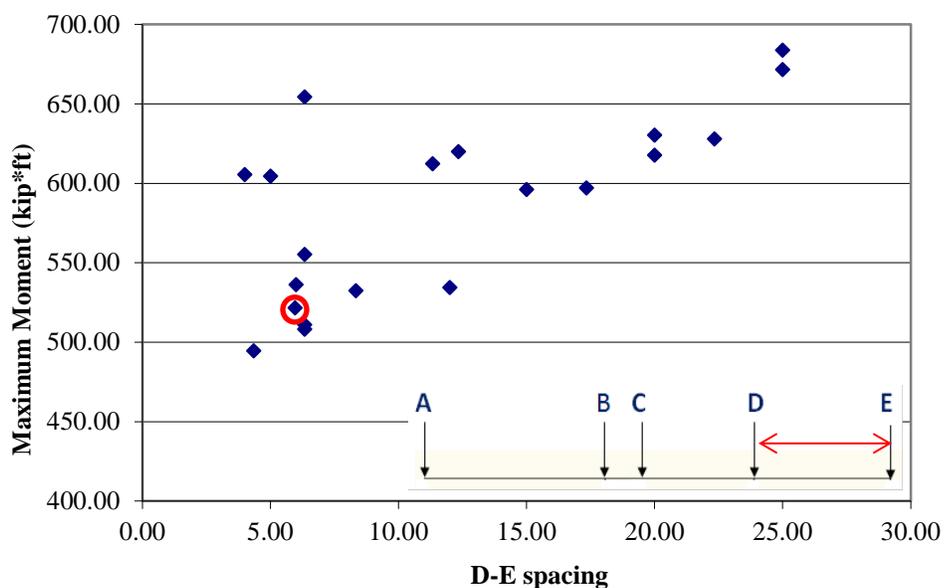


Figure 11-8: Axle Spacing Between D-E vs. Maximum Moment

Next, the trends between the axle spacing's and maximum shear were analyzed. First, the spacing between B-C was compared to the maximum shear. There is no strong relationship between spacing and shear for the B-C spacing; however the average five-axle vehicle does have one of the lowest shear values with a B-C spacing of 4.39ft.

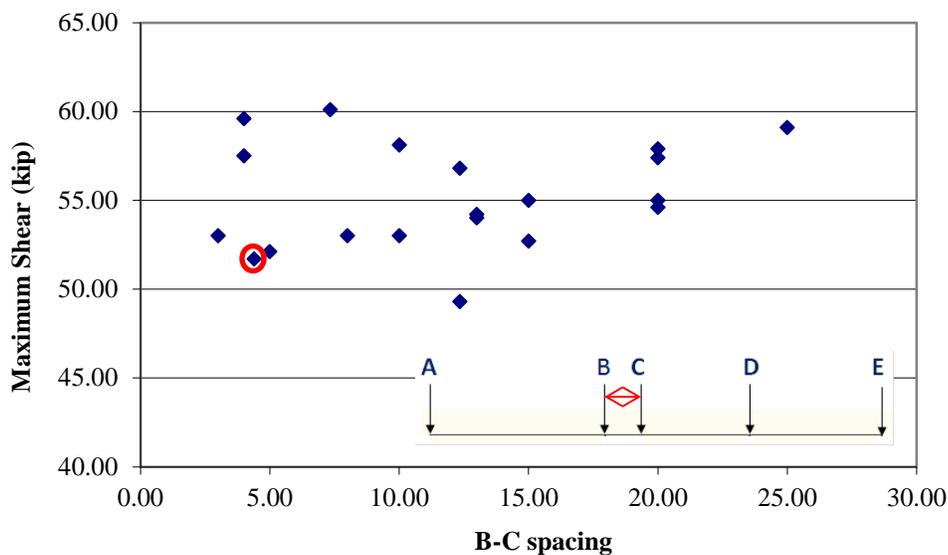


Figure 11-9: Axle Spacing between B-C vs. Maximum Shear

Next, the axle spacing C-D was compared to the maximum shear. The overall trend was that as the C-D spacing increases, the maximum shear decreases. Looking at the average five-axle vehicle, it is the optimal vehicle in terms of the trend, however there is one vehicle that had a lower shear with a C-D spacing of 12.00ft which does not follow the overall trend.

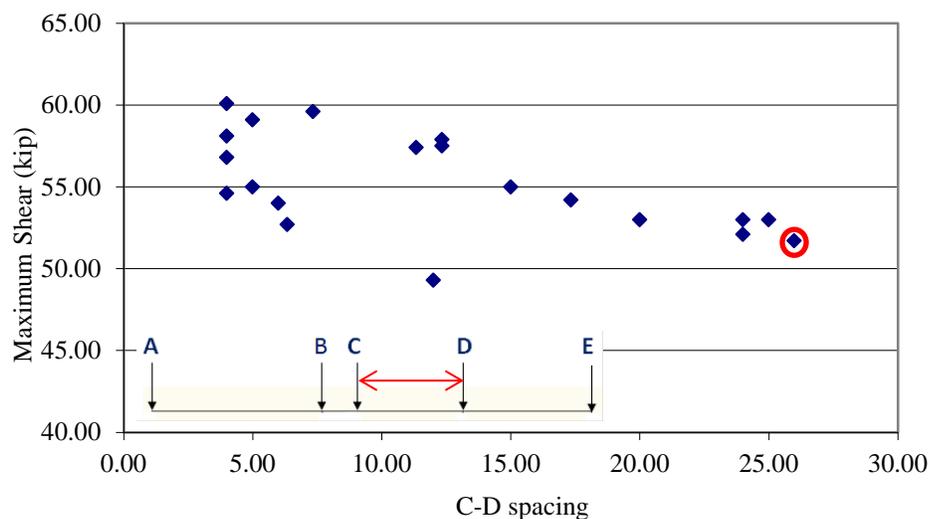


Figure 11-10: Axle Spacing between C-D vs. Maximum Shear

Lastly, the spacing between D-E was compared to the maximum shear. There was a trend seen from spacing of ten feet or greater which was that as the D-E spacing increased, the maximum shear also increased. For D-E spacing's less than ten feet, there was no strong correlation between D-E spacing and maximum shear. The average five-axle vehicle has a D-E spacing of 5.97ft which is does not fall in the range where the trend is apparent but does produce one of the lowest shear of all the trials.

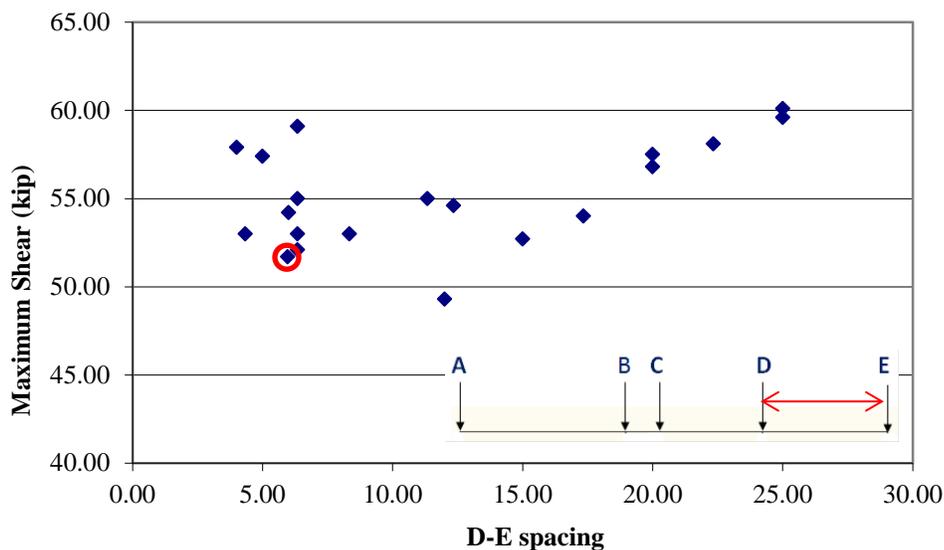


Figure 11-11: Axle Spacing between D-E vs. Maximum Shear

11.3.1 Axle Configuration Optimization Summary and Conclusions

Based on the analysis, it was determined that the current vehicle has effects that are fairly low in comparison to all the trials completed. In many of the comparisons, the average five-axle vehicle produced one of the lowest moments or shear effects. That being said, there are configurations that are slightly better in terms of moment or shear effects, but they are trades off in that there isn't one configuration which produces lower moments and shears. Additionally, the overall reductions in either moment or shear are relatively small in comparison to the overall effects caused by the vehicles. Due to the large cost associated with adjusting the axle configuration of logging vehicles, none of the configurations used in the trials can be economically justified.

11.4 Logging Truck Optimization Conclusions

Three main aspects of optimizing the current logging trucks used for carrying raw timber were analyzed. The first was to optimize the weight distribution of the timber load on the axles. The average five-axle logging truck was used as a base model and the results showed that the current average weight distribution was already close to optimal considering the level of accuracy possible for the logging vehicle operators. Next, the optimization of the axle spacing was analyzed again using the average five-axle vehicle as a base configuration. The results again yielded that the truck was fairly close to optimized and for the cost that would be required to adjust the axle spacing of the vehicle, changes to the configuration would not be beneficial. The last consideration was looking at decreasing the gross weight of the vehicle to match the effects of the HS20 design vehicle and the Wisconsin permit vehicle. The results of this analysis showed that in order to match the effects of these vehicles for spans up to 140ft, too much of the payload would have to be removed and taking the detour route would most likely be less expensive. In all, none of the optimization analyses provided promising results that could help lower the effects current logging vehicles have on bridges.

12 Strengthening/Rehabilitation: Solution 2

One main conclusion that was made based on the logging truck comparisons was that if bridges are load rated at an HS15 or lower, the logging trucks in use today will exceed the stresses that the bridge can safely handle. Previously, it was seen that optimizing the logging trucks doesn't seem to be a viable solution to this issue. Another possible solution is to strengthen the current bridges so that the allowable gross weight on the bridge increases, therefore allowing for logging trucks to use the bridges.

12.1 Implementation/Future Implementation

Bridge B38-0513 is one of the bridges which was load rated because it is a posted bridge which greatly impacts the timber industry. Figure 12-1 is a photo of bridge B38-0513.



Figure 12-1: Photo of Bridge B38-0513

Having this bridge serviceable to the timber industry would reopen an artery for transportation of timber in Northeast Wisconsin. Because of its importance to the timber industry and the Great Lakes Timber Professionals, it has been chosen as a candidate bridge for strengthening using MF-FRP strips. In addition to its importance to the timber industry, this bridge is also a strong candidate for this method of strengthening because it is a bridge in good condition. Figure 12-2 is a photo of the underside of the bridge. Based on the photos taken, no spalling of the concrete or visible reinforcement is seen.



Figure 12-2: Photo of the underside of bridge B38-0513

In Section 8.7, the results from the load rating of this bridge are reported. Looking at the LRFR results, shear was the controlling factor however the moment load ratings were also less than one. This implies that the bridge needs strengthening in terms of both moment and shear. This project will look at the moment strengthening design and the shear design for this bridge will be looked at in the future.

Six of the girders included in the bridge are original girders placed in 1925. These girders need moment strengthening in addition to shear strengthening and were the focus of this design. Figure 12-3 shows a plan view of the bridge and highlights the girders which will be designed for moment strengthening.

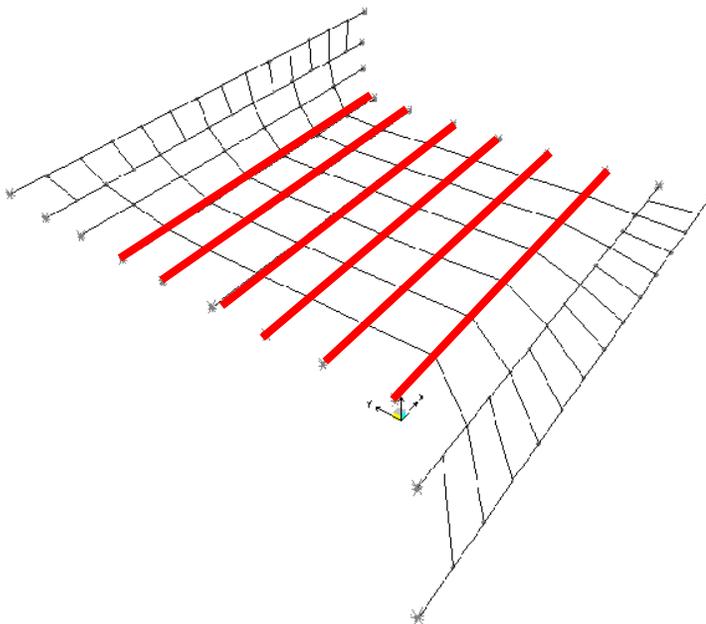
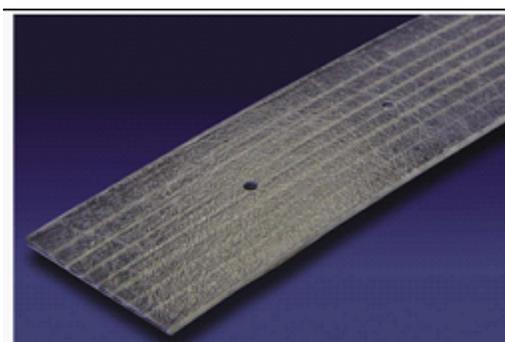


Figure 12-3: Diagram showing girders to be design for moment strengthening using MF-FRP

12.1.1 SAFSTRIP Design Program and Moment Strengthening Design 1

The University of Miami-Ohio created a design tool known as SAFSTRIP for the Strongwell Corporation. This tool is used to for flexural strengthening on simply supported concrete girder spans. Additional constraints of the program is that the loads that the girders are subjected to must be uniformly distributed and the design is limited to using the materials listed with in the program. The first material constraint is the FRP strip which is used. Figure 12-4 shows a picture of the FRP strip and also lists the material properties used in the design.



Dimensions	
Thickness: 0.125"	[3.175 mm]
Width: 4"	[101.6 mm]
Mechanical Properties in long. direction	
Ultimate Tensile Strength: 121Ksi	[836Mpa]
Elastic modulus: 9077 Ksi	[62 GPa]
Ultimate strain : 13809	

Figure 12-4: FRP Material Properties for SAFSTRIP design tool (University of Ohio-Miami, Strongwell Corporation, 2008)

The second component that is specified by the SAFSTRIP design tool is the wedge bolts and wedge anchors that are used in attaching the FRP strip to the concrete. Figure 12-5 shows required bolt and anchor.

	Anchor Fastener System	Washer	Gap Filler
 Powers Fasteners Wedge-Bolt®	Wedge Bolt #3	Steel Washer	
	Diameter: 3/8" [9.525 mm]	Outer Diameter: 1" [25.4 mm]	epoxy
	Length: as needed	Thickness ≥ 0.094" [≥2.38 mm]	
 Thunderstud® Concrete Wedge Anchor	Wedge Anchor #3	Steel Washer	
	Diameter: 3/8" [9.525 mm]	Outer Diameter: 0.81"[20.6 mm]	epoxy
	Length: as needed	Thickness ≥ 0.063" [2.38 mm]	

Figure 12-5: Anchor Wedge and Anchor Bolt Required with Design using the SAFSTRIP Design Tool (University of Ohio-Miami, Strongwell Corporation, 2008)

The SAFSTRIP design tool is a program that works using Excel. The first step in using the program was to input the geometry and properties of the girder or slab being strengthened. This also included inputting the flexural and shear reinforcement information. The second set of inputs into the program is the existing and anticipated loads, both dead and live, that the bridge is either currently experiencing or will be experiencing following strengthening. With all of these values inputted the program checks for shear to make sure it isn't controlling the design. If the shear is acceptable, a results tab yields design information for three different fastener patterns. The designer can input the number of FRP strips that will be on each girder, i.e. one or two strips. With each design option, the nominal moment capacity of the beam at midspan is calculated and the program outputs whether or not this strengthening will be able to withstand the anticipated loads.

For this design, girder six geometry and reinforcement was used for the design. As stated earlier, the program is limited to only rectangular cross sections, so in order to take into account the strength of the deck, the section was inputted as equal to the depth of the beam times the effective length of the deck flange. All the inputs used in the design can be found in Appendix A11.

The design chosen for flexural strengthening is as shown in Figure 12-6. The second of the three patterns was chosen because it uses fewer fasteners than pattern one but still offers the same amount of strengthening in terms of flexure. As a result of this strengthening, which consists of two FRP strips and 109 fasteners on the bottom of each girder, the nominal moment capacity of the girder increases to 1074.63 kip*ft. This capacity is acceptable to carry the loads of the HL-93 design vehicle.

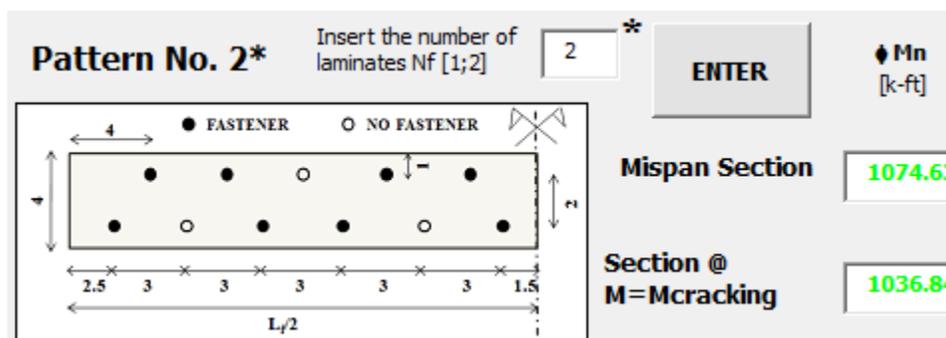


Figure 12-6: Design for MF-FRP Flexural Strengthening

12.2 Moment Strengthening Design 2

In addition to using the SAFSTRIP program, the design was also computed based on a strength model developed in previous work completed on MF-FRP Strengthening (Lamanna, 2002). First, the design was done only using one strip, however, the FRP strip was found to rupture before the concrete crushed so this design was unacceptable. Next, the strengthening system was designed using two strips attached to the bottom of the concrete beam. This design was found to provide adequate capacity and consisted of using 116 fasteners evenly spaced across the length of the strips in rows of two. The number of fasteners needed is based on the load per fastener of each strip to be 1000lb. This design yields a moment capacity of 1495.87 kip*ft. This is a much larger capacity than the previous design which only yielded a capacity of 1074.63 kip*ft. Very little is known about the equations and calculations used in the SAFSTRIP program, so it is difficult to tell why these two designs vary so much in allowable capacity. Calculations for this second strengthening design can be found in Appendix A11.

12.3 Bridge Strengthening/Rehabilitation Conclusions

Based on the literature review, this strengthening option is very promising. It has been proven to remove load postings and is all a cost effective solution compared to

replacing the bridge. Additionally, the option is quick and can be installed with unskilled labor. Very little surface preparation is necessary before installation and the bridge is serviceable immediately following installation. All of these reasons make this strengthening option a viable avenue for the timber industry to strengthen critical bridges cost effectively and quickly.

13 Summary and Conclusions

The first objective of this project was to analyze the effects that logging trucks have on single span bridges and compare those effects to design vehicles. In order to simplify the analysis, three average logging trucks were created that were representative of the logging vehicles which were measured in the field. These three average logging trucks included a five-axle tractor trailer, a six-axle tractor trailer and a five-axle truck and pup vehicle. Based on the comparison analysis, it was concluded that there were span ranges that the average logging vehicles had less of an effect than the design vehicles. The table below shows the span ranges for both moment and shear effects where the logging vehicles are compared to the HS20 design vehicle and the Wisconsin permit vehicle.

Table 13-1: Summary of Logging Truck Analysis Results

	Truck Weight (lbs)	Moment				Shear	
		HS20		Permit		HS20	Permit
		Lower bound	Upper Bound	Lower Bound	Upper Bound		
5 Axle Truck	94000	21.75	84.09ft	29	112.6ft	68ft	85ft
6 Axle Truck	98000	none	86.14ft	20.75ft	32.5ft	59ft	74ft
				32.5ft	109.6ft		
5 Axle Truck and Pup	94000	-----	59ft	30.75ft	89ft	84ft	108ft
		66ft	71ft				

(The average logging vehicles have less effect between the lower and upper bound span ranges, if only one bound is given then the vehicle is acceptable up to that span.)

In addition to the logging truck analysis, load rating of seven critical bridges to the timber industry was completed. Load ratings were completed in both LFR and LRFR. For LRFR ratings, load ratings were completed using the HL-93 design vehicle as well as the three AASHTO commercial vehicles. In Wisconsin, load posting is based on nine vehicles,

three of which are the AASHTO commercial vehicles. Because only six of the nine vehicles were not used in this load rating analysis, it can not be determined whether or not load postings are correct. However, based on the load rating that was completed, conjunctures as to the accuracy of the load postings could be made. Three of the bridges load rated are load posted according to the inspection reports. Of those three bridges, two of them, B06-0013 and B26-0002 both had legal load ratings greater than one, indicating that based on the AASHTO commercial vehicles, load posting is not necessary. The final load posted bridge B38-0513 was not load rated based on legal trucks due to the complicated geometry, however the design load ratings were very low indicating that load posting might be necessary. Lastly, two of the four bridges that were not load posted according to the inspection reports did have legal load ratings less than one. These were bridges B60-0005 and B37-0094. These conclusions are based on the AASHTO commercial vehicles alone. Load ratings using the AASHTO Specialized hauling vehicles and the WisDOT Specialized Annual Permit Vehicles would also have to be completed to determine the proper if necessary load posting for each bridge.

When the logging trucks were compared to the HS15 design vehicle, it was found that for both moment and shear effects of the average logging vehicles will exceed the HS15 effects. This means that the forces created by the logging vehicles could potentially cause permanent damage to bridges load rated at HS15 or lower. With this in mind, there are viable options for strengthening that are both cost effective and efficient that could help solve this issue. The use of MF-FRP strips was researched for this project and this method of strengthening has been proven successful on rural bridges in Missouri. Additionally, as a

result of this project, a design for flexural strengthening of one critical load posted bridge to the timber industry was created. This strengthening solution has potential to be very useful in removing load postings on load posted reinforced concrete bridges.

14 Primary Findings and Main Conclusions

The following is a summary of the primary findings of this research. A logging truck analysis was completed which looked at how the effects of the logging trucks compared to the effects of the modeled vehicles that are used to design bridges. Average logging trucks were created to represent the logging vehicles currently being used. It was determined that the logging trucks produce effects less than the design vehicles for some medium length span ranges. Additionally, as an upper bound, the average logging trucks never produced more than 20% greater effects than the HS20 design vehicle. This is valid for spans up to 150ft. When bridges are designed, the loads, such as the HS20, are factored to incorporate a safety factor into the design. When the logging trucks are compared to the factored HS20, the effects of the logging trucks are much less than the design vehicle.

Seven Bridges were load rated using two load rating methods. Of the seven, five bridges had load ratings which require load posting or bridge strengthening. Based on these results, it is clear that several bridges were not originally designed to carry the large loads now being used.

One solution to reducing the effects of logging trucks was to look at optimizing the current vehicles being used. Three different options were researched that would optimize the vehicle to reduce the effects it had on a bridge. None of these options proved to be viable solutions and it was determined the current vehicle used as a control, the average five axle vehicle, is already close to being optimal.

The final solution investigated was a cost effective bridge strengthening option. The option chosen is for reinforcing reinforced concrete beam through the use of Mechanically-

Fastened Fiber Reinforced Polymer (MF-FRP) strips. FRP strips are attached to the underside of concrete beams using powder actuated fasteners and increase the flexural or bending capacity of an individual beam. This method is not only cost effective, but can also be installed by unskilled labor and does not require very much surface preparation. Lastly, after the strips are attached to the bridge, the bridge can immediately be reopened for traffic. This method has successfully removed load postings on three bridges in Missouri and has been found to be an efficient and cost effective option for strengthening reinforced concrete bridges.

15 Recommendations for Future Research

A variety of logging trucks were analyzed for a large range of single spanned bridges. Based on these analyses, it was determined for what span ranges the logging vehicles were acceptable compared to the design and State vehicles. One suggestion for future work would be to look at the effects of logging trucks on multi-span bridges.

Logging truck traffic changes with the location of the timber being harvested. This means that often truck traffic may need to travel over a specific bridge for only a short period time, for example three months. Research on what type of permanent effects logging trucks will have on a bridge if only used for a short period of time could be very beneficial. It would lead to findings as to whether or not overloading a bridge for a short period of time impacts the bridge long term.

This project looked at a strengthening option for reinforced concrete bridges through the use of MF-FRP strips. Several of the bridges of concern to the logging industry are steel girder bridges which leads to the recommendation to look at strengthening options for these type of bridges. One option for this might be to look into using the same MF-FRP concept and apply it to strengthening steel bridges. Based on literature review, there are already fasteners which can be used in steel applications. One of the fasteners used in the Edgerton bridge strengthening project, the X-ALH fasteners, could be used in both concrete and steel applications (Arora, 2003).

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