Effect of Increasing Truck Weight on Bridges

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16. Abstract

Legislation has been proposed that will allow a 17,000 lb increase in the maximum gross vehicle weight on the Interstate Highway System. This project's main goal is quantify the effect of this increase on the internal forces to which typical slab-on-girder bridges are subjected. Both the shear and moment in the girders and the in the deck slab due to the truck loadings are investigated. To accomplish this, several configurations for these heavier trucks that have been proposed in the literature are evaluated. The HS20-44 loading with alternate military loading, the HL-93 design loading, and Alabama legal loads are used as baselines for comparison. The project focuses on short and medium span bridges with spans between 20 feet and 150 feet and girder spacings between 4 feet and 10 feet. By comparing the proposed truck configurations with the baseline configurations, the adequacy or deficiency of current design specifications and existing bridges are quantified. Recommendations for the implementation of a policy allowing specifically configured 97,000-lb, six-axle trucks are made. The results of this research will assist Alabama and other state DOTs in providing a path forward for the eventuality of heavier trucks.

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Executive Summary

Increasing the gross vehicle weight (GVW) limit on the interstate highway system above the current 80,000-lb federal limit is widely discussed at the federal and local levels. The aim of this study is to assess the internal force effects that simple and continuous span non-skewed bridges experience when travelled by six-axle semitrailers with a 97,000-lb GVW. The increase is quantified by comparing the shear and moment initiated by two individual 97-kip trucks (97-S and 97-TRB) to those from three base models: design live loadings from the *American Association of State Highway Transportation Officials* (AASHTO) *Standard Specifications*, *AASHTO LRFD Specifications*, and the envelope from five potentially critical Alabama legal loads.

The design live loadings from the AASHTO Standard Specifications do not generate adequate shear and moment to fully envelope the effects of the proposed 97-kip vehicles. Additionally, depending on bridge type and span length, both 97-kip vehicles demonstrate force effects greater than those from the envelope of the five Alabama legal loads. However, the shear and moment induced by the design loading of the LRFD Specifications completely envelope the effects of the proposed heavier trucks on each bridge type investigated. It is concluded that the LRFD notional loads represent significant benefits to bridge design practices concerning the potential for heavier trucks on the highway system. Furthermore, the overall vehicle length and axle spacing plays a vital role in the longitudinal force effects created in the bridge by the 97,000-lb trucks, and should these vehicles be permitted to operate, consideration should be given to limiting their use only to trucks that maximize the kingpin-to-rear axle spacing of the trailer for the jurisdiction in which the trucks operate. For many jurisdictions, Alabama included, the maximum permitted spacing from kingpin-to-center of rear axle group is 41 feet.

Additionally, the primary deck reinforcement specified by the *Alabama Department of Transportation* (ALDOT) *Bridge Bureau's* standard slab detail is evaluated using the LRFD Specifications and the critical 51-kip tri-axle load of the 97,000-lb tractor-trailer. It is concluded that the positive and negative reinforcement currently supplied satisfies the LRFD strength requirements for this axle group for all slab-on-girder bridges. This scope of this study focused only on the specific effects of the heavier trucks; and as such, did not verify the adequacy of the slab reinforcement for the barrier collision loads of AASHTO LRFD or other effects of AASHTO LRFD that are not included in the original design to develop ALDOT's standard slab details, which are based on the AASHTO Standard Specifications.

1.0 Introduction & Problem Statement

Introduction: Legislation has been introduced in Congress over the past several years to allow heavier trucks to operate on the Interstate Highway System (IHS). The most recent of these bills from the US House of Representatives (H.R. 763 "Safe and Efficient Transportation Act of 2011") has proposed to allow states to authorize the use of vehicles with a gross weight of 97,000 pounds on the IHS if: (1) the vehicle has a minimum of six axles, (2) single axles do not exceed 20,000 pounds, (3) tandem axles do not exceed 34,000 pounds, (4) any grouping of three or more axles does not exceed 51,000 pounds. The general intent of this legislation is to promote economical prosperity and uniformity among US states and bordering nations as described in the North American Free Trade Agreement established in the mid 1990's.

Problem Statement: The main objective of this study involves analyzing the critical shear and moment effects developed in simple and two-span continuous bridges that are subjected to truck configurations that represent the criteria of the proposed legislation. This is achieved by comparing the effects caused by two independent 97,000-lb, six-axle trucks to those from three base models: design live loadings from the *AASHTO Standard Specifications*, *AASHTO LRFD Specifications*, and the envelope from five potentially critical Alabama legal loads. Maximum shear and moment effects are quantified as the ratio of the effect of the proposed trucks to the effect of each base model to provide the bridge community with a tangible magnitude to the increase in effect bridges will see as a result of the heavier trucks.

Furthermore, a comparative analysis of the standard bridge slab design issued by Alabama Department of Transportation (ALDOT) is completed. The transverse reinforcement provided by the standard deck slab chart is investigated using Load and Resistance Factor Design methods and the critical axle grouping of the 97,000 pound vehicles. The results from this analysis will aid Alabama and other state departments of transportation with a few preliminary steps along the inevitable path to heavier trucks on the IHS.

Notes and Assumptions: All bridge analyses are performed under linear-elastic and static loading pretenses. Bridge supports are considered rigid without deflection. The live load force effects generated in longitudinal bridge models are analyzed utilizing the two-dimensional line-girder methodology. Dynamic load allowance and transverse load distribution are not included as they are assumed to be similar for all truck configurations and would therefore be applied the same to all trucks in accordance with the appropriate specification. The intent of this study is to compare the effect of the 97-kip truck with that of the design loads of the LRFD and Standard Specifications, separately, rather than to compare across the specifications. Therefore the differences in these parameters between the specifications can be neglected. The live load transverse moment effects for the deck slab are determined using three-dimensional finite element modeling.

2.0 Background

Reason for Gross Vehicle Weight Policy Change: Economic projections indicate that freight is rapidly on the rise. In the United States, 12.8 billion tons of freight was transported by truck in 2007. Due to lingering recession impacts, only 10.9 billion tons were moved in 2009, but 18.4 billion tons are expected in 2040, a 69% increase. Without expansion to the national highway system, roadway segments experiencing congestion are assumed to increase by nearly 400% between 2007 and 2040 (USDOT Freight Facts 2010). In Alabama, this heavy truck traffic will directly affect segments of I-59/I-20, I-65, and I-10 around the Birmingham, Montgomery, and Mobile areas respectively. Non-interstate highways expecting increased congestion include US 431 and US 280 (ALDOT Freight Study, 2010). As the economy improves, diesel fuel prices are expected to rise, which will raise the operating costs of shippers and eventually raise the cost of shipped goods unless freight can be moved more efficiently. Increasing efficiency of freight movement is the primary objective of truck size and weight limit reforms.

Along with easing congestion, an increase in gross vehicle weight (GVW) will help provide uniformity with neighboring countries Canada and Mexico. In part, the North American Free Trade Agreement (NAFTA) of 1994 was established for this reason, but the varying truck size and weight standards of each country confine the effectiveness of this agreement. Mexico has a maximum GVW limit around 107-kip while some provinces in Canada allow trucks to operate near 129-kip depending on axle spacing. The US has the lowest maximum GVW limit of 80-kip. Special NAFTA permits are issued for overweight loads, but this process restricts the overall efficiency of import/export trade scenarios (TRB 1990).

Impacts of Increasing Truck Weight: There are a multitude of impacts that increasing truck weight will have on trucking industries as well as the tangible impacts felt by others. Several key effects include, but are not limited to, economic productivity, environmental, safety, and highway infrastructure costs. Whether these impacts are considered beneficial or disruptive often depends on perspective.

Economic Productivity: The economic productivity deriving from increased GVW is a relative issue benefitting some while hindering others. Agencies that currently transport bulk commodities at a GVW near 80-kip will benefit from weight increases as their payload subsequently rises. This will reduce operating cost on a per trip basis. Due to the competitive nature of the shipping business, carrier operating cost savings would likely trickle down to the freight distributors because a reduction in vehicle miles of travel will be provided. A study done in the 1980's concluded that annual savings of $3.2 billion would result if the proposed 9-axle (one single and four tandem axles) Turner Double with a GVW of 105-kip became legal (Figure 2-1). Based on historical freight data it was estimated that one-fourth of the total miles traveled by combination trucks would take place in Turner Doubles. From another perspective, increased truck weight and lower shipping cost will reduce the volume of freight transported by rail as current manufacturers utilizing the railroad system will have cost incentives to make the switch to truck carriers (Cohen, Godwin, Morris, and Skinner 1987).

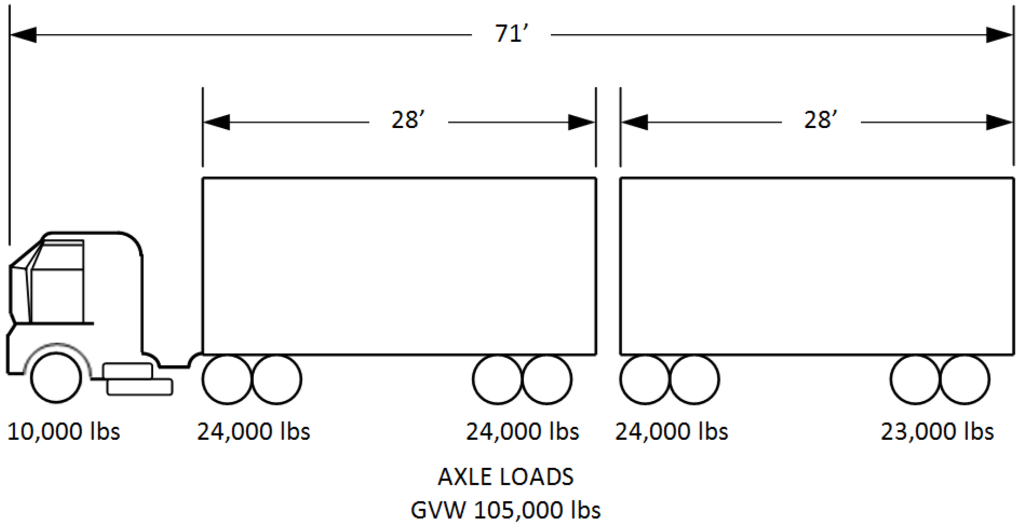


Figure 2-1. Turner Double

Environmental: Fuel consumption, on a freight ton hauled per gallon burned basis, will decrease if larger loads are permitted. Hauling an abundance of commodities from an arbitrary origin A to location B will reduce the total number of trips required hence limiting the number of vehicle miles of travel (VMT) and the fuel consumed. However, the added freight to truck transport switching from rail will increase annual gross fuel consumption. Comparative information on train versus truck emissions and efficiency was not investigated.

A slight drawback from increasing truck weight limits is the increased noise. Truck noise is a function of engine type, speed, and tire properties. No recent historical data on noise studies between truck types was discovered but it is rational to assume increased GVW will increase engine strain hence the noise level. As property value is affected by noise, it is predicted that noise will have an impact, but the degree of the impact is not apparent (USDOT TS&W Vol-I 2000).

Safety: Safety becomes a major concern when considering changes to truck size and weight. The majority of the general public included in focus groups pertaining to weight regulations expressed negative concerns with allowing heavier trucks on roadways (USDOT TS&W Vol-I 2000). However, when accident reports that include truck length and weight are analyzed, crash rates from long combination vehicles (LCVs) closely resemble those of five-axle semi-trailers with GVW under 80-kip. For vehicles with additional axles above that of the standard five-axle semi-trailer, braking capacity will be enhanced due to advanced technology in the motor vehicle industry. Each additional axle can be equipped with braking mechanisms to help combat against the increased momentum that heavier trucks demonstrate.

One factor directly related to safety that can be measured is the vehicles' stability and control. Vehicle rollover is a leading concern to safety when discussing the allowance of heavier trucks on the National Network (NN). Rollover is a function of speed, GVW, axle length, suspension type, and tire properties. It occurs in two basic scenarios. The first is caused by high speeds when negotiating a steady-state turn. Every vehicle has a static roll stability (SRS) threshold which decreases with an increasing center-of-gravity. If the SRS value is exceeded, the vehicle will overturn. The second rollover scenario entails high speeds where evasive maneuvers have taken place much like the phenomena of cracking a whip. Factors that play a key role in these situations involve the number of articulation points and the dynamic roll stability (DRS) of each vehicle. Semi-trailers have one articulation point while double and triple trailer combinations usually have three and five points respectively. Susceptibility to rollover magnifies with the addition of articulation points as the DRS is lowered.

In order to sustain safety, several issues need to be addressed. It has been recommended that operators of heavier motor vehicles extend their training with certified programs and receive monetary incentives to ensure operations are carried out at superior safety levels. Subpar roadway conditions and geometrics should be rehabilitated as well as dated equipment that do not meet safety standards (USDOT TS&W Vol-I 2000).

Highway Infrastructure Costs: An increase in GVW will have substantial effects on highway infrastructure with roadway and bridge improvement costs. In past circumstances observed, specifically focusing on modifications to vehicle configurations, annual repair costs to roadways remain quite stationary if not being reduced. Since the federal government has capped single axle (20-kip) and tandem axle (34-kip) weight limits, innovative configurations maintain this limit and frequently suggest slightly lowering it. Pavement wear is directly related to individual axle loadings rather than gross vehicle weight. Referring to the study involving the Turner-Double, it was determined that a 50% reduction in equivalent single axle loads (ESALs) will result when compared to the standard five-axle 80-kip semitrailer. This would prevent 15-billion ESAL miles per year. At the time of this study, pavement repair costs averaged 1.6 cents per ESAL mile producing a cumulative annual savings of $250 million for state departments of transportation (DOT) (Cohen, Godwin, Morris, and Skinner 1987).

The Comprehensive Truck Size &Weight Study of 2000 sponsored by the USDOT compared two tractor-semitrailers both with a 12-kip load on the steering axle. A five-axle truck had two tandem axles of 34-kip with a GVW of 80-kip. The second truck was configured with six-axles including one tandem axle with the same axle weight as the first vehicle but a rear tridem axle of 44-kip resulting in a GVW of 90-kip. According to the study in regards to flexible pavement surfaces, the five-axle truck will cause 18% more roadway damage per VMT than the six-axle combination, despite having an 11% reduction in gross weight (USDOT TS&W Study Vol-II 2000).

On the other hand, state DOTs will see an increase in the funding required for bridge rehabilitation if GVW limits are increased. Previous studies conducted by the United States Department of Transportation (USDOT) Federal Highway Administration (FHWA), Transportation Research Board (TRB), and others have determined that repair cost from bridge damage will be the greatest single highway infrastructure cost due to heavier trucks. Estimating the net cost for bridge repair is a detailed and complicated process because a degree of uncertainty is always present. It requires the composite sum of several cost factors only reasonably estimated at a global level. These main factors can be summarized as: cost of construction, cost due to diminished service life, and user costs.

Construction costs include the price of building new bridges and/or rehabilitation to those existing. Reducing the service life of a bridge adds additional costs that are not accounted for during the design phase. Every interstate bridge is designed with a service life under a notional design loading. Allowing applied loads above that of the design load negatively affects the service life expectancy. Construction costs are directly representative of the increased shear, moment, and fatigue effects felt by bridge elements due to these increased loadings.

Short-term effects result from overstressing bridge elements. Overstressing a bridge can cause cracks in its girders and deck, diminishing the load-carrying capacity and eventually resulting in closure or failure. Once signs of overstressing are apparent, the bridge owner has three options: replace the bridge, strengthen the bridge, or post weight limits. Bridge type typically governs the capability of being strengthened. Studies show that the cost of strengthening reinforced concrete (RC) bridges and prestressed bridges can equal the cost of replacing them.

Long-term effects of overstressing are seen in gradual fatigue damage. After numerous loading cycles, bridges show signs of fatigue witnessed by the cracking of the superstructure at locations of high stress. Greater fatigue directly results in a shorter life span of a bridge and the cost effects of fatigue are entangled in the bridge's reduced life. Steel bridges are at a greater risk of experiencing fatigue but studies show that prestressed concrete bridges and RC decks can exhibit fatigue symptoms if continually overloaded (TRB 1990 and Weissmann and Harrison 1998).

Increased user cost is a result felt by the daily traffic. Essentially it is a function of time delay caused by bridge repair. During this time bridges will either be closed and the traffic rerouted or partially closed causing traffic to merge into single lanes. In either case, traffic flow will be affected. A tangential part of user costs is also found in additional vehicle maintenance and fuel consumption stemming from rerouting and traffic congestion (Weissmann and Harrison 1998).

History of Truck Size and Weight Regulations: In the early 1900s truck size and weight limitations were governed on a per state basis with the focal point of protecting state highways and bridges. However, only a small percentage of states adopted any regulations at all. In 1932 the American Association of State Highway Officials (AASHO) suggested guidelines for single and tandem axle weight limits and by 1933 all states had truck size and weight regulations of some kind. The AASHO policy of 1946 reformed the guidelines of 1932 and proposed that state agencies limit single axles to 18-kip and tandem axles to 32-kip. A maximum gross vehicle weight of 73.28-kip was also suggested for "vehicles having a maximum length of 57-ft between the extremes of the axles" (TXDOT 2009). This was the first instance that related the notion of GVW to axle spacing. The contents of the Federal-Aid Highway Act of 1956 established that all interstate highway improvements were to be funded with a 90/10 split between federal and state governments respectively. Due to the sizeable investment from the federal government, the regulation recommendations by AASHO in 1946 became federal policy. If states accepted higher weights prior to the adoption of the 1956 Act, they were allowed to continue to operate under a "grandfather clause". Vehicle width was also set to a maximum of 96-in. but height and length restrictions were still left to state declarations. To increase carrying capacity and fuel efficiency, the Federal-Aid Highway Act Amendments of 1974 increased the GVW restriction to

80-kip as well as the single and tandem axle weights to 20- and 34-kip respectively. "As in the 1956 Act, these limits were permissive and States could adopt lower limits if they chose," (USDOT TS&W Study Vol-I 2000). The maximum weight two or more axle groupings for any vehicle could possess was determined by a bridge formula that utilizes a vehicle's weight-to-length ratio. This formula is currently in use and was created to provide safety and sustain the service life of bridges. The basic concept of the formula is to prevent overstressing HS-20 bridges by more than 5% and HS-15 bridges by more than 30% (USDOT TS&W Study—Working Paper 4 1995).

$$W = 500\left[\frac{LN}{N-1} + 12N + 36\right]$$

Federal Bridge Formula (a.k.a. "Formula B")

- W = overall gross weight on any group of two or more consecutive axles to the nearest 500 pounds
- L = distance in feet between the outer axels of any group of two or more consecutive axles
- N = number of axles in the group under consideration

Due to the lack of several states adopting the 80-kip weight limit, hence hindering carriers of states that adopted the 1974 limit, Congress enacted the Surface Transportation Assistance Act (STAA) of 1982 which mandated all states to practice and uphold the federal limits set in 1974 on interstate highways and other parts of the National Network (NN). STAA trucks are primarily classified as semitrailers with a minimum length of 48-ft and 28-ft (minimum) twin-trailers (USDOT TS&W Study Vol-I 2000).

The Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) along with the Transportation Efficiency Act of the 21st Century (TEA-21) put a freeze on state allowances of longer combination vehicles. This limitation restricted the use of LCVs in states that had not adopted the use of LCVs and additionally prevented those currently in use from expanding LCV routes as well as LCV weights and dimensions. In contrast, state exemptions and grandfather rights regarding federal GVW limits can still be issued depending on certain criteria such as transporting goods that promote a state's economy (USDOT TS&W Study Vol-I 2000).

Previous Bridge Study-*Impact of 44,000-kg (97,000-lb) Six-Axle Semitrailer Trucks on Bridges on Rural and Urban U.S. Interstate System:* This 1998 study investigates the cost impacts that a proposed 97-kip six-axle truck would have on interstate bridges in the U.S. Over 37,500 simple and continuous span bridges were analyzed that were adequate for handling a typical five-axle semitrailer with a GVW of 80-kip (CS5). The effects are demonstrated by pairing the currently efficient bridges that become structurally deficient per the 97-kip configuration with the replacement costs and user costs. The replacement costs contain any cost accrued in raising bridge capacity to greater standards while user costs entail traffic congestion due to work zones. All bridge data were taken from the National Bridge Inventory (NBI). Using the previously developed technique from a FHWA project, the computerized "moment model" was used for analysis. By comparing the maximum positive/negative moments due to the live-load with those produced from the inventory ratings given in the NBI, the functionality of the bridges became apparent. Bridges were declared deficient only if the live load moment surpassed the inventory moment by 5%. The deck area of all deficient bridges were then quantified by state and multiplied by an average cost per deck surface area, depending on state location, determining partial strengthening costs. Since these costs per deck area varied widely from state to state and other factors suggesting that strengthening a bridge will ultimately cost more than replacing the bridge, this study negated strengthening costs to replacement costs. User costs were quantified by time lost in work-zone congestion as well as additional fueling costs acquired in the traffic. By using the moment model and the work-zone analysis model, the following results and conclusions were determined.

- 38% of the bridges were declared deficient
- Deficient bridges were 56% rural and 44% urban
- Total replacement cost – $13.85 billion
- Rural replacement cost – $4.36 billion (31%)
- Urban replacement cost – $9.49 billion (69%)
- Total user cost – $56.07 billion
- Rural user cost – $6.55 billion (12%)
- Urban user cost – $49.51 billion (88%)

The 97-kip commercial vehicle will not be acceptable on almost 40% of the bridges on the U.S. Interstate Highway System that are currently equipped with the load carrying capacity that allows passage of the CS5 (80-kip) truck. On top of that, the replacement costs will increase above the value shown as bridges that are currently structurally deficient for the legal GVW of 80-kip must be replaced as well. A portion of this replacement cost should be added to the replacement cost for the six-axle truck. As replacement and user costs were the only variable cost associated with the impact heavier trucks have on bridges, additional impacts will be felt. The effect that vehicle emissions have on the environment during traffic congestion is an example. Due to the lack of specific data obtained from the NBI, detailed and complex models were not suitable for this study (Weissmann and Harrison 1998).

3.0 Methodology

Vehicular Live Loads: The shear and moment effects included herein are the maximum internal bending moments and shear forces developed in each bridge due to the vehicular loadings from the two proposed trucks and the three base models. When considering the longitudinal force effects, the shear and moment are direct results from single vehicular live loads and do not contain any modification or design factors unless otherwise noted. However, additional factors are applied to the transverse force effects when ALDOT's Standard Bridge Slab design specifications are checked against the proposed vehicles using LRFD design criteria.

Proposed 97,000-lb Trucks: The selection of the steering-to-rear axle length of each proposed vehicle is geared to provide a wide range of force effects between the two truck types. The extreme lengths selected can aid state agencies in determining the best alternative for increasing truck weight. For this reason, the overall lengths of the two 97-kip trucks are set at 40-ft and 65-ft (see Figure 3-1). Each truck has six axles that make up the tractor-trailer combination. The shorter truck is denoted the "97-S" while "97-TRB" refers to the other. The 97-TRB was used in the previous study for the Transportation Research Board conducted by Weissmann and Harrison (1998). Since the study did not include the actual force effects generated by the 97-TRB, it is used in this report as one of the two 97-kip configurations.

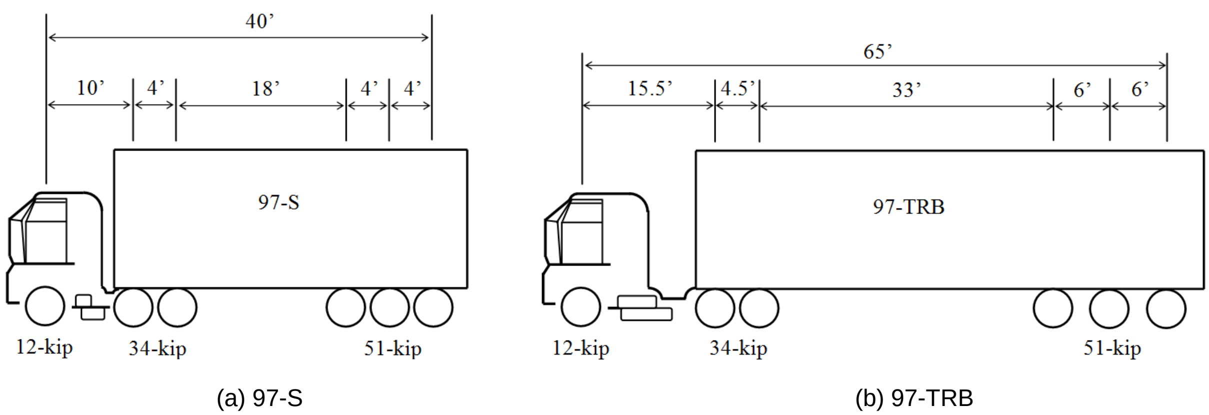


Figure 3-1. Proposed 97,000-lb Vehicles (a) 97-S (b) 97-TRB

AASHTO Standard Specifications: The Standard Specifications utilize three highway live load scenarios for determining the critical design force effects (Figure 3-2). In general they are the notional HS20-44 design truck and design lane loading and an Alternate Military Loading (AML). The HS20-44 design lane load includes a 640-lb/ft uniform load with a single concentrated load of 26-kip or 18-kip applied to the location causing the maximum force effect for shear or moment, respectively. The AML represents a tandem axle with a spread of 4-ft and a GVW of 48-kip. The maximum force effect produced from one of the three loadings is taken as the design load (Article 3.7 AASHTO Standard Specifications 1996).

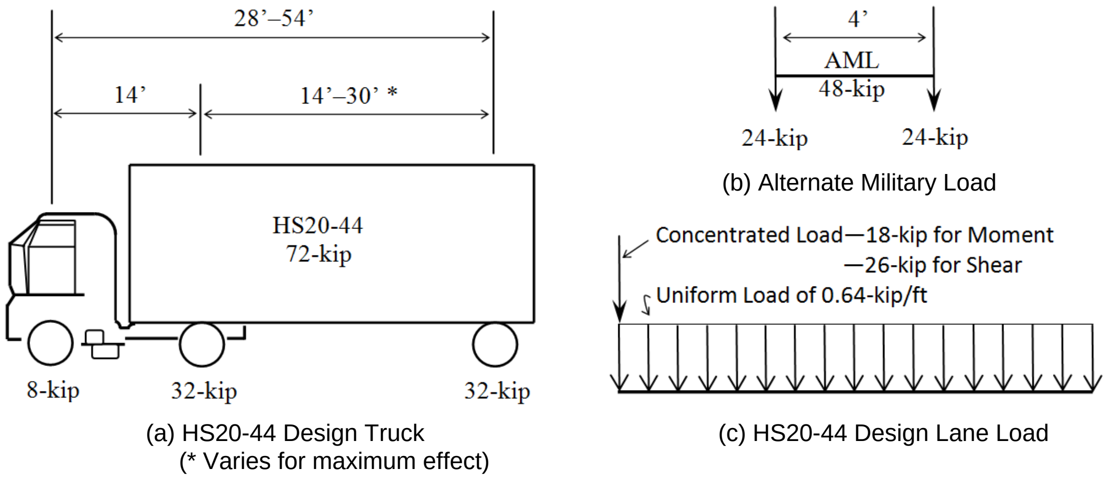


Figure 3-2. Design Live Loadings from AASHTO Standard Specifications

AASHTO LRFD Specifications: The LRFD Specifications use a vehicular live loading denoted HL-93, made up of three loadings: design truck, design tandem, and design lane load. Even though many similarities exist between both specifications, a few vital alterations are made. The design truck remains as the HS20-44 design truck. The AML is replaced with the design tandem axle loading which has a 2-kip increase in gross weight to that of the AML. The design lane load remains as 640-lb/ft but the additional concentrated loads are removed. The biggest difference from the design loadings of the Standard Specifications to LRFD is that the maximum force effect is the largest cumulative result of the design truck + design lane or design tandem + design lane loadings (Article 3.6.1.2 AASHTO LRFD Specifications 2010). It is shown in later figures that the force effects from LRFD loadings are comparatively greater than those of the standard specification.

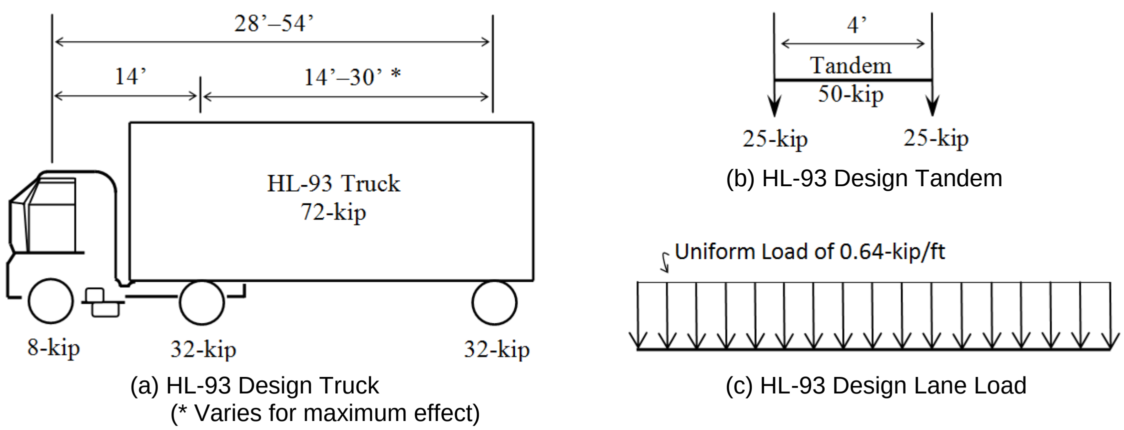
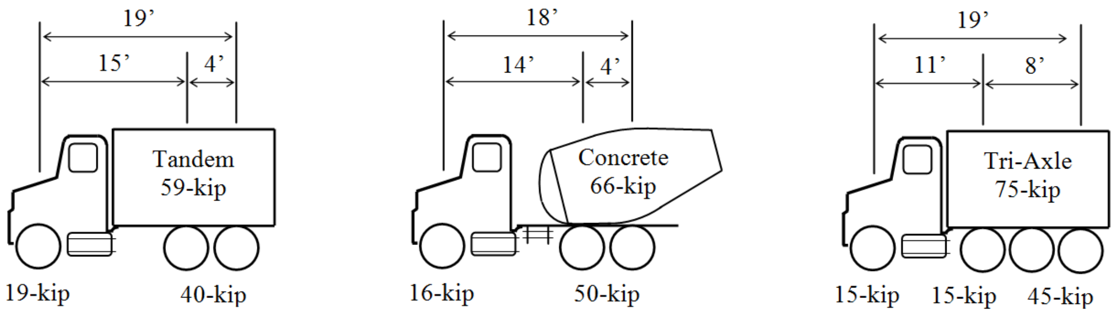
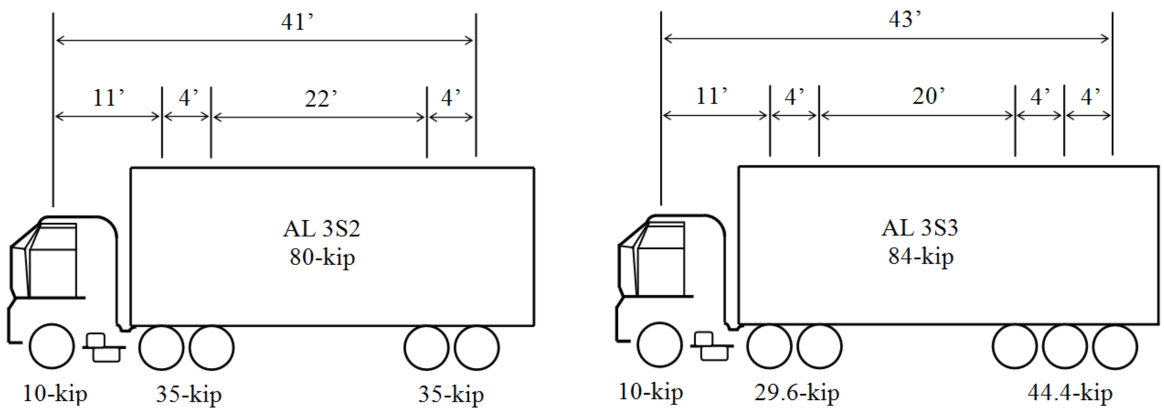


Figure 3-3. Design Live Loading from AASHTO LRFD Specifications

Alabama Legal Loads: The final base model includes selective legal loads that are specific to the State of Alabama. The five Alabama legal loads investigate are: Alabama Tandem-Axle, Alabama Concrete truck, Alabama Tri-Axle, Alabama 3S2, and the Alabama 3S3. As seen in Figure 3-4, the minimum steering-to-rear-axle spacing of the five vehicles is 18-ft while the maximum spacing is 43-ft. The minimum and maximum GVW of the Alabama legal loads ranges from 59-kip to 84-kip respectively. These vehicle configurations represent transport trucks that do not require permits to operate on Alabama highways



(a) Alabama Tandem (b) Alabama Concrete (c) Alabama Tri-Axle



(d) Alabama 3S2 (e) Alabama 3S3

Figure 3-4. Alabama Legal Loads

Longitudinal Bridge Models: All simply supported bridges are treated as determinant two-dimensional structures. A single rigid pin, preventing vertical and axial translation, and a rigid roller, only preventing vertical translation, make up the support conditions for all simple spans. Each bridge is modeled with a pin at one end and a roller support at the other. The span lengths of the simply supported bridges range from 20 to 300 feet in five foot increments. All continuous span bridges consist of two equal spans with pin-roller-roller supports at their respective ends. The continuous spans range from 20x20 feet up to 150x150 feet.

Simply Supported Bridges: For simple spans, the maximum shear effect due to vehicular loads will always occur at a minuscule distance along the bridge span from one of the two supported ends and will have the greatest axle load bearing down on the support in question. The direction of vehicular travel is irrelevant as the truck is always positioned for its critical loading, creating maximum shear. By visualizing the influence line of arbitrary span length, one can easily determine the influence area ordinate under each adjacent axle by the use of similar triangles. This task is quite simple for determinant structures, as influence lines maintain constant linear slopes.

For determining the maximum moment in simple spans, the critical loading of a vehicle is at a position where the vehicle's resultant force and the adjacent concentrated axle load mirror the spans centerline. The axle closest to the resulting force usually dictates the maximum moment, but both adjacent axels should be checked (Hibbeler 2006). The location of the maximum moment will always be located directly under the governing axle load. Therefore, the maximum moment effects from all configurations are simple functions of vehicle geometry and span length. Referring to engineering terminology, this maximum moment is classified as positive since the extreme upper and lower fibers of the span's cross-section will be in compression and tension respectively. In LRFD design, the location of the maximum moment due to the vehicle and the uniform lane load will differ so both locations must be checked for each load. The cumulative design moment is then recorded for the single location that experiences the maximum effect.

Continuous Span Bridges: All continuous span bridges are analyzed using CSI's SAP2000 V15. Using the moving load feature, each bridge model is effectively loaded and analyzed under linear static load conditions. The maximum discretization length for the vehicle loadings is set at one-foot or one-one-hundredth of the span length with the smallest value controlling the discretization length.

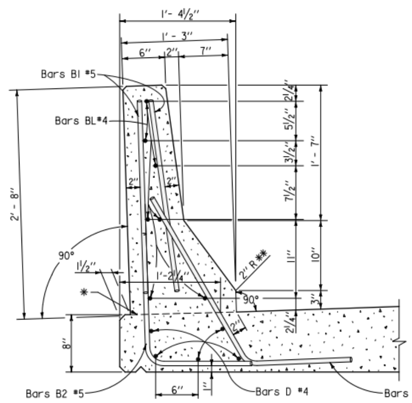
For continuous spans, the critical locations of maximum shearing force occur at the three bridge supports. For two-span continuous bridges with a span ratio of 1:1, the center support experiences the greatest shearing effect.

Two internal moment effects are vital in continuous spans: maximum positive and maximum negative moment. The location of maximum positive moments varies depending on the span length of the bridge and the vehicle's load configuration but usually occurs at a distance within 35%-45% of the span length. The location of the maximum negative moment for two-span continuous bridges is always about the center support of the bridge. Referring to the HS20-44 design lane load in the AASHTO Standard Specifications, the maximum negative design moment for continuous spans is determined by modifying the lane load to include an additional 18-kip concentrated load (one in each span) to produce the maximum effect per Article 3.11.3. In the LRFD Specifications, the maximum negative moment in continuous spans is described in Article 3.6.1.3, where the negative moment is determined using 90% of the effect of two design trucks spaced 50-ft apart combined with 90% of the lane load.

Transverse Deck Analysis Overview: Using LRFD techniques, the strength limit state of the transverse reinforcement provided in ALDOT's standard deck slab is checked under the critical axle group of the 97-kip trucks and the dead load from the deck slab. Decks supported by longitudinal girders having aspect ratios of 1.5 or greater can be considered one-way slab systems. The aspect ratio is defined as the longitudinal span distance between supports divided by the transverse girder spacing (Barker and Puckett 2007). With all bridge models meeting this minimum criterion, it is justified to treat each deck as a continuous two-dimensional beam.

Moment effects are critical over shearing forces in deck design. This is due to the limited flexural stiffness in the reinforced concrete deck sections spanning between girders. For this reason, only positive and negative moment conditions are recorded. Using LRFD techniques, the ultimate factored design moment is checked against the nominal moment capacity or resistance supplied by the reinforced concrete deck slabs outlined in ALDOT's standard slab detail. All referenced articles within this section refer to the AASHTO LRFD Bridge Design Specifications.

ALDOT Standard Bridge Slab Design: ALDOT design specifications use the current edition of "AASHTO Standard Specifications for Highway Bridges" under the HS 20-44 design live load in compliance with the Service Load Design Method (Allowable Stress Design). To ensure safety and uniformity in design, the State Bridge Engineer provides bridge designers with the ALDOT Standard Bridge Slab details for reinforced concrete (RC) decks supported by girder type: steel girders, AASHTO girders, RC deck girders (T-beams), and Bulb-Tee girders (see Figure 3-2). Slab thickness and deck reinforcement requirements have been predetermined based on girder type and girder spacing (ALDOT Bridge Bureau 2008). Concrete decks require a 28-day compressive strength, fc', of 4.0 ksi with reinforcement steel of ASTM A615, Grade 60 billet steel. The typical barrier configuration for non skewed bridges is presented in Figure 3-5. This barrier has a 15-in.base width and extends the entire bridge span on opposite sides of the deck.

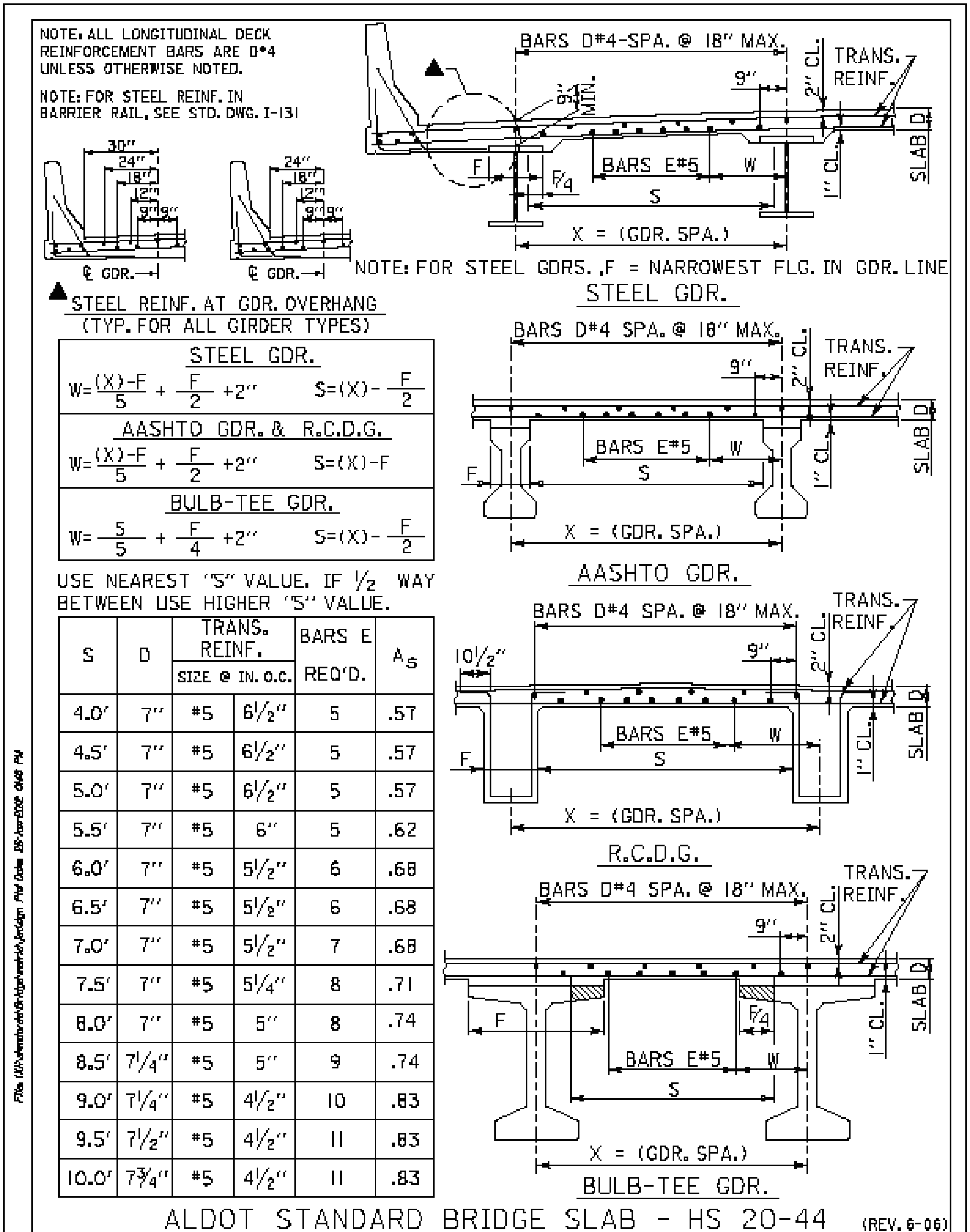


Note: From "Standard Barrier Rail for Non Skewed Bridges" by Alabama Department of Transportation, 2012, *ALDOT Bridge Bureau Standard Drawings*, I-131, Sheet 3 of 8. Copyright 2012 by Alabama Department of Transportation. Reprinted with permission.

Figure 3-5. ALDOT Barrier Rail for Non Skewed Bridges

Transverse Deck Models: The design parameters used are in accordance with RC decks supported by typical AASHTO girders and RC Deck-Girder combinations. A deck-girder combination involves the flanges of the girders acting as part of the deck system. Two cases are investigated: (1) deck sections consisting of four girders and (2) sections with six girders. The moment results of both cases are approximately equal if not exact with the largest differential being less than half a percent. For this reason, the results of case-2 are not discussed.

Constants used for all deck models are barrier widths, overhang length, and girder stem width. The overhang deck length from centerline of each exterior girder is 3-¾-ft. This length is used per ALDOT deck standards from Figure 3-6. The stem width of each T-beam girder is considered as 12-in. Center-to-center girder spacing varied from 5-ft to 11-ft increasing at ½-ft increments resulting in thirteen cross-section deck models. The depth, D, of the concrete deck varied from 7-in to 7¾-in, increasing as the clear span (S) reaches 8 ½-ft or more. "S" is defined as the clear span distance between two adjacent girders (e.g. for a girder to girder spacing of 5-ft, the clear spacing is the girder-to-girder spacing minus the girder stem width or 5-ft minus 12-in resulting in a clear spacing of 4-ft). The cross-section of a typical deck-girder bridge model is presented in Figure 3.7.



S	D	TRANS. REINF. SIZE	TRANS. REINF. @ IN. O.C.	BARS E REQ'D.	A_S
4.0'	7"	#5	6½"	5	.57
4.5'	7"	#5	6½"	5	.57
5.0'	7"	#5	6½"	5	.57
5.5'	7"	#5	6"	5	.62
6.0'	7"	#5	5½"	6	.68
6.5'	7"	#5	5½"	6	.68
7.0'	7"	#5	5½"	7	.68
7.5'	7"	#5	5¼"	8	.71
8.0'	7"	#5	5"	8	.74
8.5'	7¼"	#5	5"	9	.74
9.0'	7¼"	#5	4½"	10	.83
9.5'	7½"	#5	4½"	11	.83
10.0'	7¾"	#5	4½"	11	.83

Note: From "ALDOT Bridge Bureau Structures Design and Detail Manual" by Alabama Department of Transportation, 2008, p. 29. Copyright 2008 by Alabama Department of Transportation. Reprinted with permission.

Figure 3-6. ALDOT Standard Bridge Slab – HS20-44 Chart

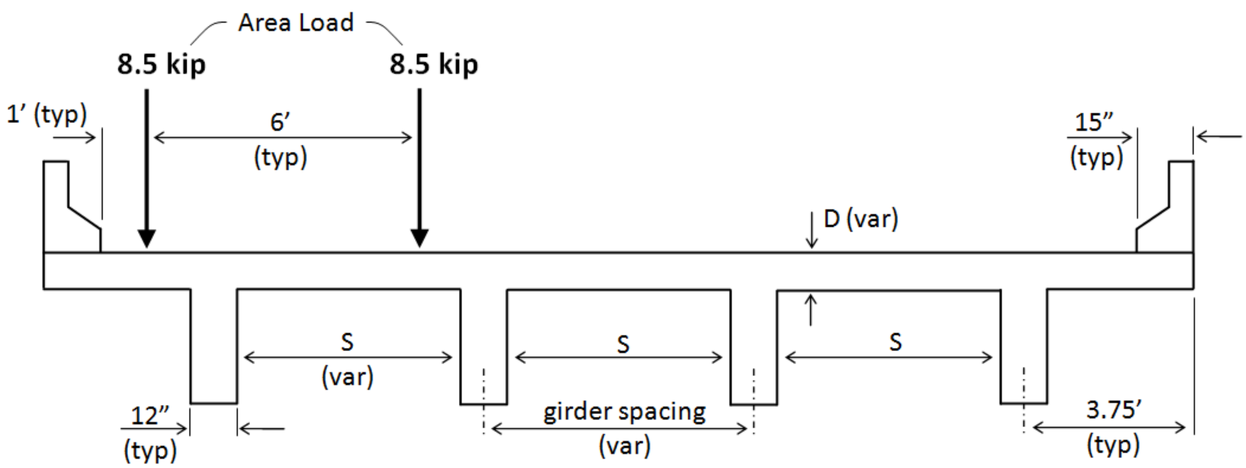


Figure 3-7. Typical Bridge Cross-Section

To ensure bridge safety, all engineering design specifications are geared toward the general principle of supplying member resistance that is greater than or equal to the force effects caused by applied loads. Load and Resistance Factor Design makes use of statistically determined load and resistance factors to achieve this. The general LRFD equation is:

$$\Phi R_n \geq \sum \eta_i \gamma_i Q_i \qquad \textit{Equation 3-1}$$

Where,

Φ = Resistance factor dependent on limit state
R_n = Nominal resistance supplied by member
η_i = Load modification factor dependent on ductility, redundancy, importance
γ_i = Load factor dependent on load type
Q_i = Load/force effect dependent on load type

The deck analysis is checked at the strength limit state with the following factors and variables:

- Φ = 0.9 (for tension controlled sections) [A5.5.4.2.1]
- $R_n = M_n$ = Nominal Moment capacity of ALDOT deck slabs
- $\eta_i = \eta_D \text{ x } \eta_R \text{ x } \eta_I = 1.0$ [A1.3.3–A1.3.5]
- $\gamma_i = \gamma_{LL} = 1.75$; $\gamma_P = 1.25$ (max) & 0.9 (min) [Table A3.4.1-2]
  - γ_{LL} - Live Load factor
  - γ_P - Permanent Load factor
- $Q_i = M_{LL}$ and M_{DC}
  - M_{LL} - Maximum Moment of Live Load
  - M_{DC} - Dead Load Moment of slab and barrier at $M_{LL\text{-}Max}$ location

To account for dynamic load effects of moving vehicles over decks and deck components, LRFD imposes a 33% impact factor that is added to the static load effect when designing for the strength limit state. This empirical factor represents the dynamic allowance applied to the static wheel loads when vehicles are in motion [A3.6.2].

The dynamic load effect is given by the following:

$$Q_{LL+IM} = Q_{LL}(1 + IM) = 1.33Q_{LL} \qquad \textit{Equation 3-2}$$

Where,

Q_{LL+IM} = Static live load effect plus the dynamic load allowance
Q_{LL} = Live load force effect
IM = Impact factor of 33%

Loadings: The dead loads acting on the deck include the uniform load of the concrete (ksf) and the two point loads from the ALDOT standard barriers (kip/ft). The barrier loads are positioned according to their center of gravity, 5-in from the free edge of each overhang. The dead loads used for the thirteen deck models are summarized in Appendix C: Table C-1. The maximum dead load moment results are gathered using line-girder methods. Loads from future wearing surfaces are not included in the deck analyses per ALDOT design standards.

The critical live load/force that bridge decks must withstand is from the rear tri-axle group of the 97-S truck. This tri-axle is a combination of three 17-kip axles with the center axle separated by 4-ft from the two adjacent axles (Figure 3-1.a). Using SAP 2000, three-dimensional finite element bridge models are created and the tire loads acting over their contact area are applied. Each area load consists of the 8.5-kip tire load distributed over an 18-in (width) by 6-in (length) contact area. The initial placement of the outside tire load is located on the deck overhang 1.5-ft from the centerline of the exterior girder which is 1-ft from the barrier curb face [A3.6.1.3.1]. Six feet across the deck marks the initial position of the inside 8.5-kip tire load. The six area loads representing the tri-axle loading are moved along the transverse width of the deck in increments of 6-in, creating a moment envelope of the live loads. A typical cross section with the live loading is shown in Figure 3.7.

The AASHTO LRFD Bridge Design Specifications use multiple presence factors, *m*, for adequately adjusting force effects due to multiple heavy vehicles being present in adjacent lanes [A3.6.1.1.2]. This factor is based on probability of occurrence so the conservative design uses a factor of 1.2 for the presence of a single truck.

ALDOT Standard Bridge Slab Reinforcement Evaluation: Using the LRFD maximum moment effects, the transverse deck reinforcement provided in the ALDOT Standard Bridge Slab chart (Figure 3.6) is evaluated using LRFD methods at the strength limit state. This task is achieved by checking both the positive and negative moment reinforcement for adequate ductility and tensile strength. Member capacity is based on compatibility and equilibrium for the reinforced concrete section as shown in Figure 3-8.

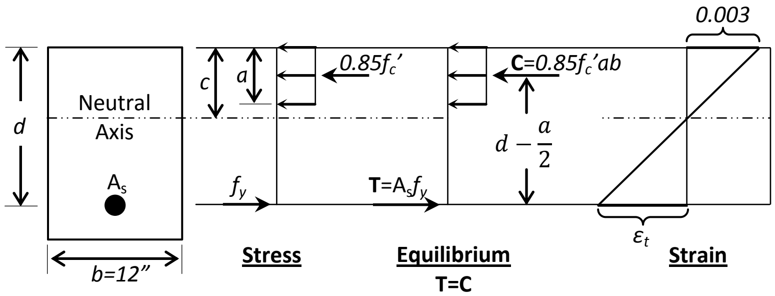


Figure 3-8. Stress-Strain Diagram of Typical RC Section

Where,

d – Effective depth of extreme compression fiber to centroid of tensile reinforcement (in)
c – Depth from extreme compression fiber to neutral axis of section (in)
a – Depth of equivalent compression stress block (in)

$a = \beta_1 c$ *Equation 3-3*

β_1 – Factor relating depth of equivalent compressive stress block to depth of neutral axis

- for f_c' of 4000-psi → $\beta_1 = 0.85$

b – 1-ft longitudinal width of deck section (in)
f_y – Yield strength of steel (ksi)
f_c' – Compressive strength of concrete (ksi)
ε_t – Net tensile strain of reinforcement (in/in)
Neutral Axis – Separates the tension and compression zones of a section

Positive Reinforcement: The function of the positive reinforcement is to provide tensile strength against the flexural phenomena of positive moments. The location of this steel is in the tension region of the deck which is below the neutral axis when positive moment is present. In order to carry out the desired strength checks, the material properties of the deck must be known. Table C-2 in Appendix C lists the required slab properties as described in the ALDOT Bridge Bureau Structures Design and Detailing Manual.

Resistance Factors [A5.5.4.2.1]: If an excessive amount of steel is present in deck slabs it can have adverse affects on safety that can lead to brittle failure. Prior to 2005 a maximum allowable reinforcement check was used to verify ductility in flexural members [A5.7.3.3.1]. Ductile members will experience large visible deformations and cracking before failure occurs. This reinforcement limitation is based on the ratio of the compression zone depth to the total compression and tension zone depth in a section, *c/d*. The maximum reinforcement check has since been altered and is now governed by the resistance factor, Φ, which varies according to the reinforcement strain type: tension-controlled, compression-controlled, or in a transitional state between the two. Table 3-1 shows the resistance factors for use at the strength limit state.

Table 3-1: Resistance Factors

ε_t	Φ
Tension-controlled $\varepsilon_t \geq 0.005$	0.9
Transitional $0.002 < \varepsilon_t < 0.005$	$0.65+0.15(d/c-1)$
Compression-controlled $\varepsilon_t \leq 0.002$	0.75

From Figure 3.8, the net tensile strain is explained as:

$$\varepsilon_t = \frac{(d-c)}{c}0.003 \qquad \textit{Equation 3-4}$$

To determine the location of the neutral axis from the extreme compression fiber, *c*, the equivalent compressive stress block depth, *a,* is determined and input into *Equation 3-3*.

$$a = \frac{A_s f_y}{0.85 f_c' b} \qquad \textit{Equation 3-5}$$

Utilizing *Equations 3-3* thru *3*-5 with the material properties specified in Appendix C, the appropriate resistance factor for the nominal moment capacity is determined (see Table 4-7).

<u>Minimum Positive Reinforcement [A5.7.3.3.2]:</u> The minimum reinforcement limitation for components in flexure requires that enough reinforcement be present so that the factored nominal strength, $\Phi M_n = M_u$, is no smaller than the minimum value of:

(1) $1.2M_{cr}$
(2) $1.33M_u$

M_{cr} – Cracking moment of concrete
$M_{cr} = S_{nc} f_r$ *Equation 3-6*
S_{nc} – Section modulus for the extreme fiber of the noncomposite section where tensile stress is caused by external loads

$$S_{nc} = \frac{bh^2}{6} \quad (\text{in}^3) \qquad \textit{Equation 3-7}$$

b – Longitudinal base length = 12-in
h – Height of slab/Depth
f_r – Rupture modulus of concrete (ksi) [A5.4.2.6]

$$f_r = 0.37\sqrt{f_c'} \quad (\text{ksi}) \qquad [\text{for normal concrete}] \qquad \textit{Equation 3-8}$$
$$\Phi M_n = \Phi A_s f_y (d - \frac{a}{2}) \qquad \textit{Equation 3-9}$$

LRFD check of flexural strength for the ALDOT specified reinforcement is listed in Table 4-8.

Negative Reinforcement: Negative reinforcement provides strength and ductility in the same manner as the positive reinforcement. The difference between the two types stems from the placement of the negative reinforcement. Since negative bending moment causes tension in the upper region of a typical section, the location of transverse reinforcement is in the top portion of the slab with enough clear cover to be considered fully bonded with the concrete.

<u>Resistance Factors [A5.5.4.2.1]:</u> The amount and bar type of primary reinforcement suggested by ALDOT applies to both positive and negative moment regions for a specified clear spacing (Figure 3.6). Therefore, the neutral axis depth to the extreme compressive fiber, *c*, remains the same as calculated for the positive reinforcement. However, the tensile strain in the reinforcement of *Equation 3-4* will change because the effective depth, d_{neg}, is altered due to the top cover requirements differing from the bottom requirements (Table C-2).

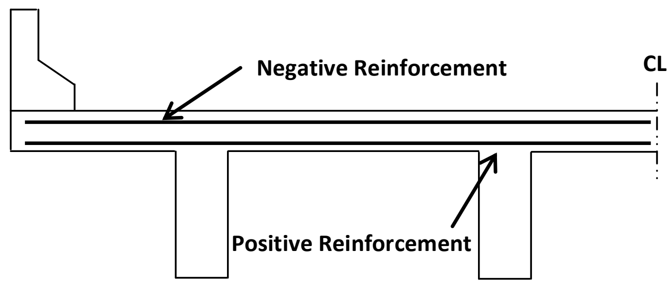


Figure 3-9. Primary Reinforcement

<u>Minimum Negative Reinforcement [A5.7.3.3.2]:</u> The strength requirement for negative reinforcement is checked in the same manner as the positive reinforcement.

4.0 Results

Longitudinal Bridge Analysis: To understand the impact that 97,000-lb vehicles have on bridges, the effects from the proposed and base live load models are displayed as ratios. In the following figures, the proposed-to-base shear ratio is less than 1.0 if the maximum resulting force effect from the 97-kip vehicle falls below that of the base model and above 1.0 if not.

Simply Supported:

Maximum Shear at Supports: Shear results for each load model are listed in Appendix A: Table A-1.

Maximum shear from the 97-S exceeds the AASHTO Standard Specification in bridge spans ranging from approximately 40-ft to 200-ft (Figure 4-1). A 20% increase in shear is seen for span lengths from 80-ft to 140-ft. The sharp increase in shear beginning at 40-ft spans is due to the 72-kip HS20-44 design truck controlling the design shear which is outweighed by the 97-S. At simple span lengths of 130-ft and greater, the design lane load becomes critical resulting in the decreasing slope of the shear curve.

Maximum shear from the 65-ft 97-TRB exceeds the AASHTO Standard Specifications in bridge spans ranging from approximately 80-ft to 170-ft. A 10% increase in shear results for spans lengths of 110-ft to 140-ft. The critical span length for both the 97-S and the 97-TRB is 125-ft, where the shear surges are 26% and 13% respectively.

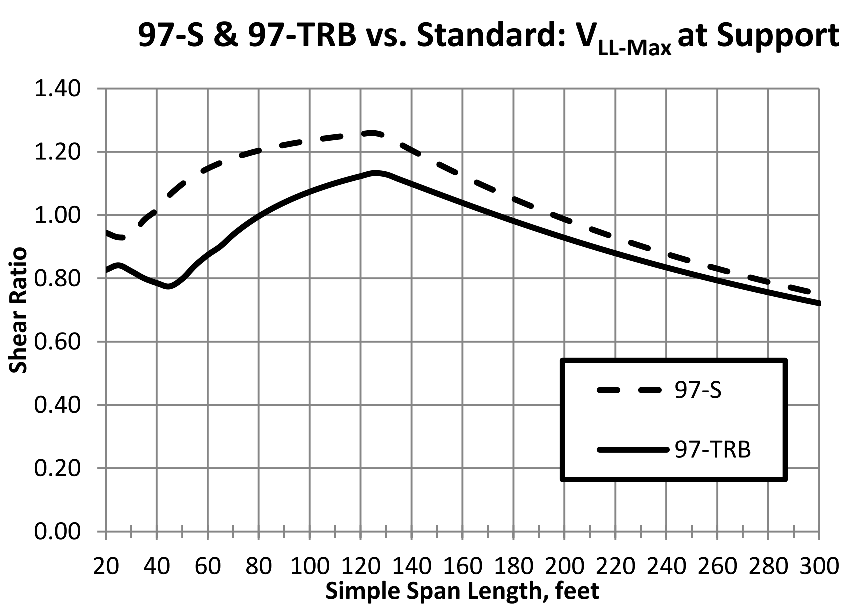


Figure 4-1. Shear Ratio of 97-S & 97-TRB to AASHTO Standard Specifications

The maximum shear from the both proposed trucks is completely enveloped by the LRFD design shear (Figure 4-2). The maximum value of the 97-S is less than 90% of the design shear for 60' spans and the shear effect decreases as simple span length increases. This is due to the combination loading of the HL-93 design truck with the lane load. The maximum shear effect of the 97-TRB is only 72% of the LRFD design. Comparing the ratios of the two proposed vehicles, it is seen that the maximum shear of the 97-S is greater than the 97-TRB for all simply supported spans. This is due to the shorter, more concentrated steering-to-real axle length of the 97-S.

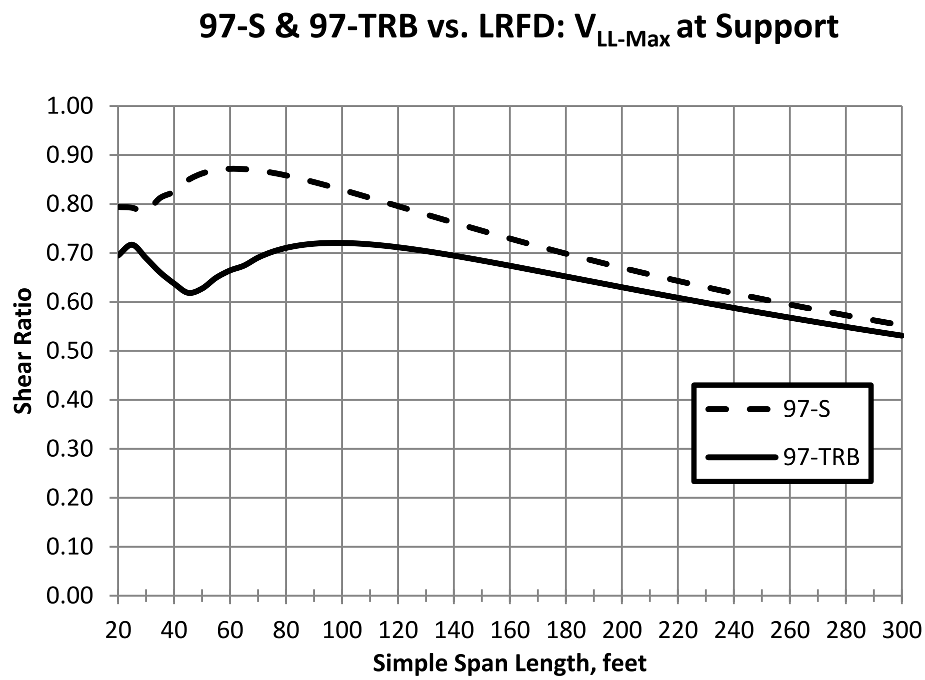


Figure 4-2. Shear Ratio of 97-S & 97-TRB to LRFD Specifications

In comparison to the five Alabama Legal Loads (Figure 4-3), the 97-S generates greater shear for all span lengths above 50-ft. For all spans 110-ft or longer a relatively constant shear ratio exists at 1.16. This can be explained by the 97-S's resemblance in axle grouping and overall length to the controlling legal load, AL 3S3. In addition, the GVW of the 97-S is 115% of the 84-kip AL 3S3.

Similar to the 97-S, as span length increases the 97-TRB causes more shear effects than the Alabama Legal Loads. However, the 97-S begins to exhibit greater shear at simple spans of 100-ft while the 97-S generates more shear beginning with spans of only 50-ft. The maximum shear increase in the 97-TRB is 11% while it is 17% for the 97-S loading. Comparing both proposed vehicles it is evident that shear effects not only increase with increasing truck weight but they are also dependent upon axle spacing. If axle spacing is minimized, shear effects will increase. The maximum shear at supports also approaches the gross vehicle weight as simple span length is increased.

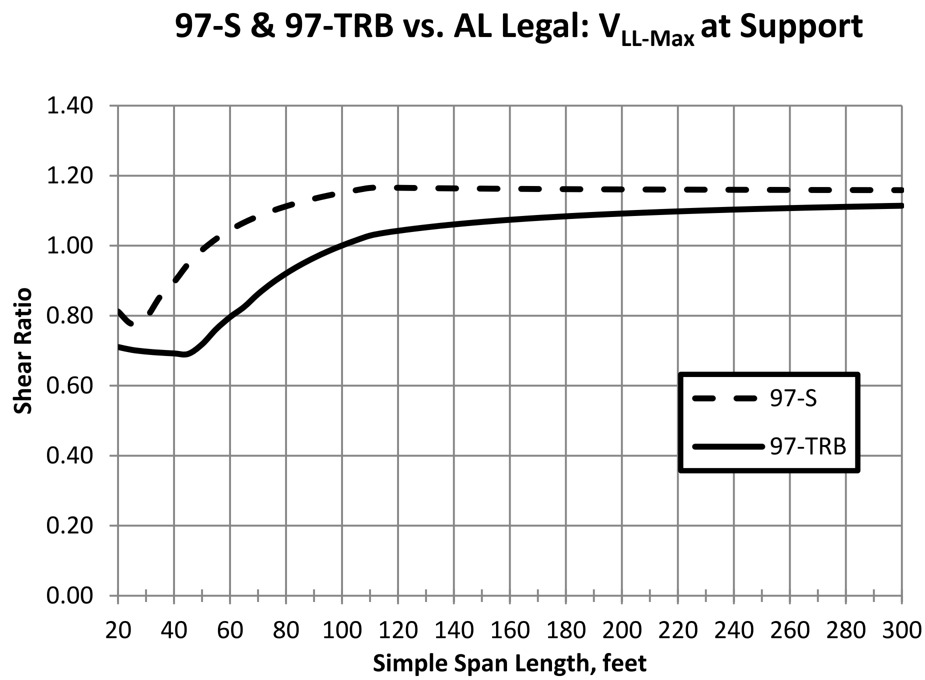


Figure 4-3. Shear Ratio of 97-S & 97-TRB to AL Legal Loads

Maximum Positive Moment: The maximum moment resulting from the proposed and base load models for simple spans of 20-ft to 300-ft is listed in Appendix A: Table A-2. Recall that all moment values are positive due to the position of compression and tension zones of each member's hypothetical cross-section.

Compared to the unfactored design moments from the AASHTO Standard Specifications, the flexural effects of the 97-S loading create greater moment in simply supported spans ranging from 50-ft to 205-ft in length (Figure 4-4). In spans 105-ft to 155-ft long, a moment growth of 20% or more is shown. Regarding the Standard Specifications, the AML loading controls the design moment for simple spans up to 35-ft. The HS20-44 design truck then reins for spans extending to 140-ft where the design lane load governs for the remaining spans.

The 97-TRB creates more moment than the Standard Specifications in simple spans from 125-ft to 165-ft long. However, a 5% or more moment increase is only felt by spans from 140-ft to 150-ft long. The critical simple span length for both proposed vehicles is 145-ft, but the moment increase due to the 97-S is 24% over the positive moment effect from the Standard Specifications while the 97-TRB is 7%. Since the dominant variable of the proposed vehicles is the overall axle length, the maximum moment values begin to converge as the span length becomes infinite. The reasoning behind this can be explained by the ratio of moment arms of each configuration. With increasing span length the distance from the support to each vehicle's center of gravity becomes relatively comparable, treating the gross weight as a single point load.

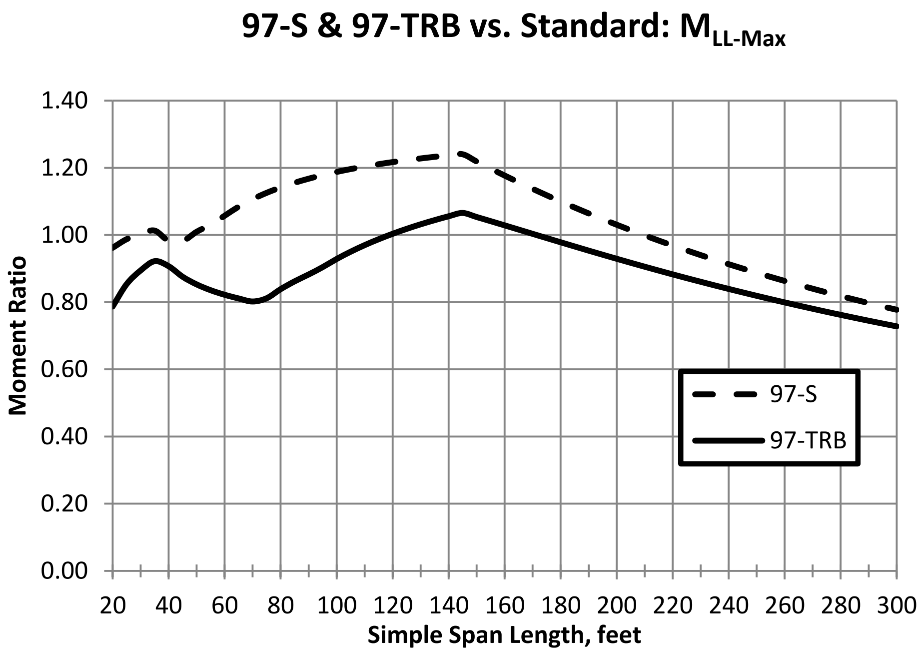


Figure 4-4. Moment Ratio of 97-S & 97-TRB to AASHTO Standard Specifications

As shown in Figure 4-5, LRFD design moments completely envelop those of the 97-S and 97-TRB for all simply supported bridges. The maximum moment effect of the proposed vehicles is only 80% and 71% of the value from the base LRFD model, respectively. Compared to the 97-S, the 97-TRB truck causes less moment effects on all simple spans.

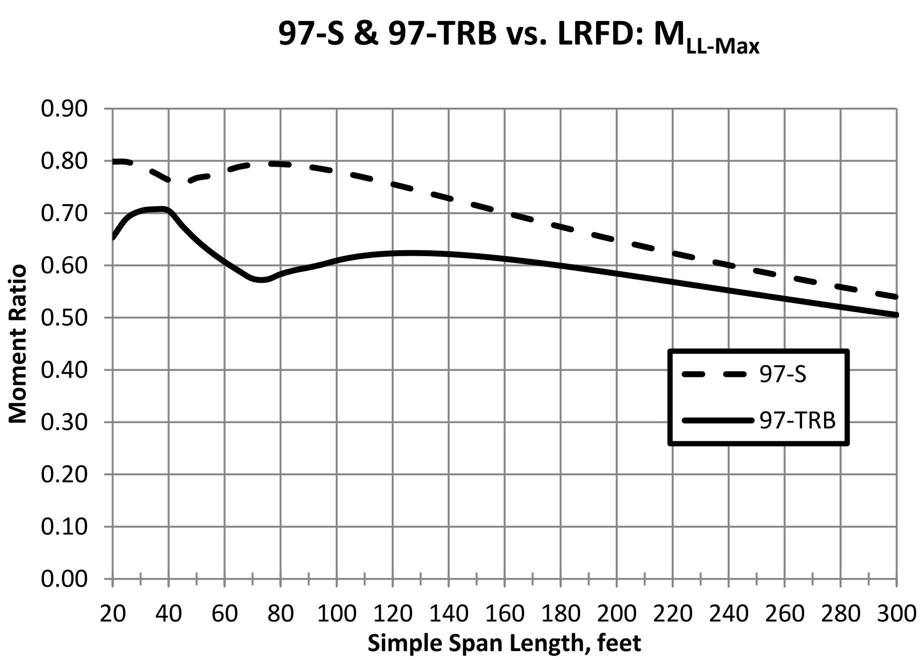


Figure 4-5. Moment Ratio of 97-S & 97-TRB to LRFD Specifications

The 97-S exceeds the maximum bending moments caused by the Alabama Legal Loads on all simple spans of 80-ft and greater (Figure 4-6). At simple spans around 180-ft, the moment increase becomes stable at 17% due to the AL 3S3 similarities previously discussed. At span lengths of 160-ft the 97-TRB will develop positive bending moments greater than the AL Legal Loads, specifically the Alabama Tri-Axle.

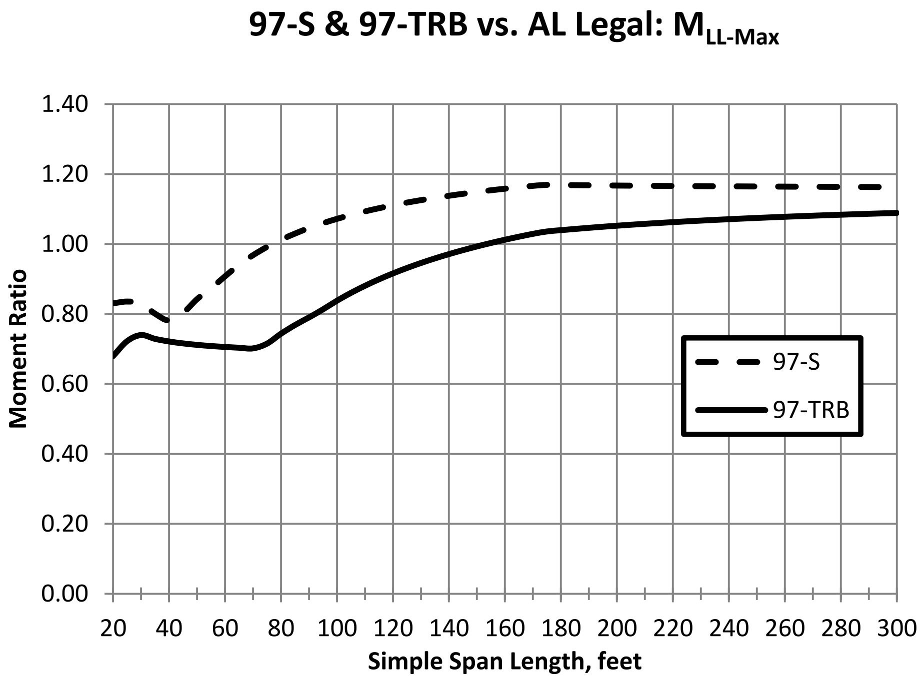


Figure 4-6. Moment Ratio of 97-S & 97-TRB to AL Legal Loads

<u>HL-93 vs. HS20-44/AML:</u> This study focused on comparing the 97-kip truck with the notional loads of the AASHTO Standard Specifications and separately with notional loads of the AASHTO LRFD Specifications. In both these comparisons, the effect of all other parameters influencing the design force effects, such as distribution factors, impact, load factors, and resistance factors, are assumed to be applied identically to each truck, within each specification. Given these constraints, it is clear that a direct comparison of the resulting force effects resulting from the differing notional loads alone is not fully representative of total difference in design effect between the specifications; however, illustrating this comparison, Figure 4-7 for simple spans, still highlights why the LRFD Specifications better envelope the effects of the heavier trucks. Although the notional models of the two specifications are similar (Figures 3-2 and 3-3), the unfactored shear and moment from each vary considerably, with HL-93 of LRFD providing larger internal shear and moment compared to HS20-44/AML from the Standard Specifications.

The notional loads from the LRFD Specifications result in a direct, unfactored increase in shear greater than 50% when compared to the Standard Specifications for simple spans ranging from 80-ft to 180-ft. Similarly, a 50% increase in direct, unfactored moment occurs for simple spans ranging from 100-ft to 250-ft long.

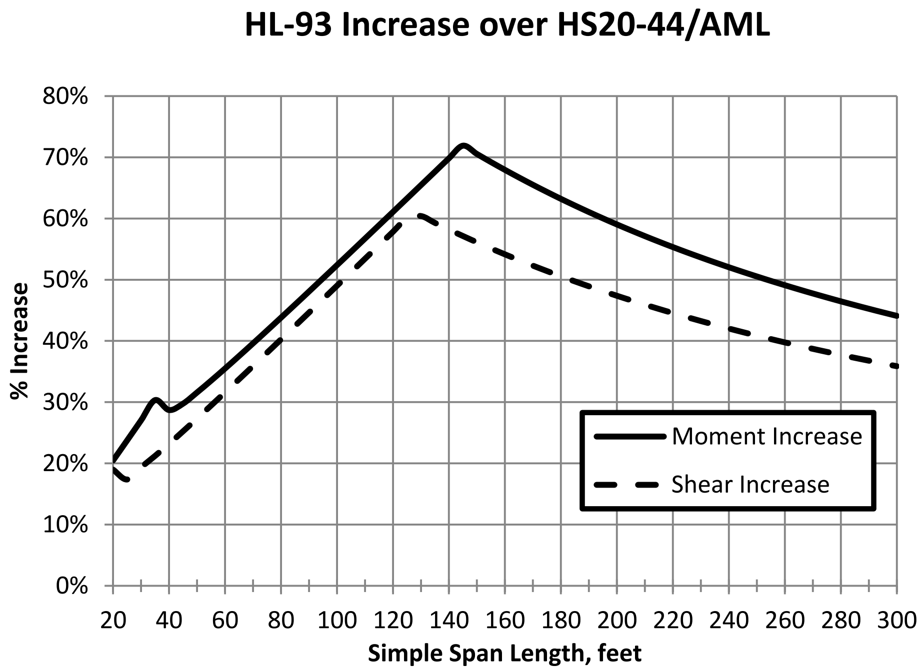


Figure 4-7. Force Effect Increase from Design Loads of LRFD to Standard Specifications

Continuous Span:

Shear at End Supports: The maximum shear in the end supports due to the proposed and base load models is given in Appendix B: Table B-1.

The 97-S causes more shear than the AASHTO Standard Specifications at the end supports for continuous spans greater than 40x40-ft. 90x90-ft spans and longer experience shear increases of 20% or greater (Figure 4-8). The maximum increase of 25% will be felt for 1:1 span ratios of 145-ft. However, the 97-TRB only exceeds shear values of the Standard Specifications for spans of 95-ft or greater and the maximum increase of 12% is less than half the increase of the 97-S. Span lengths ranging from 130-ft to 150-ft will result in shear growth slightly above 10% when traveled by the 97-TRB. As both curves show, the maximum increase in shear slightly drops at 145-ft spans, suggesting that this is the critical span length for the shear initiated by both 97-kip loads.

The maximum shear from the LRFD design loads envelopes both proposed 97-kip models (Figure 4-9). At the bridge abutment/span end interface, the 97-S will produce shear values less than 90% of the LRFD shears while the shears from the 97-TRB are considerably lower at 71%. As noted in the shear analysis of the simple span bridges, the shorter overall axle spacing of the 97-S of 40-ft proves to be the critical parameter for the proposed vehicles. The maximum shear values from the LRFD design loads are increasing at a greater proportion than the 97-kip loadings as seen by the negative slopes in the longer spans in each plot. This is expected since the LRFD loads include both the design truck and the uniform lane load.

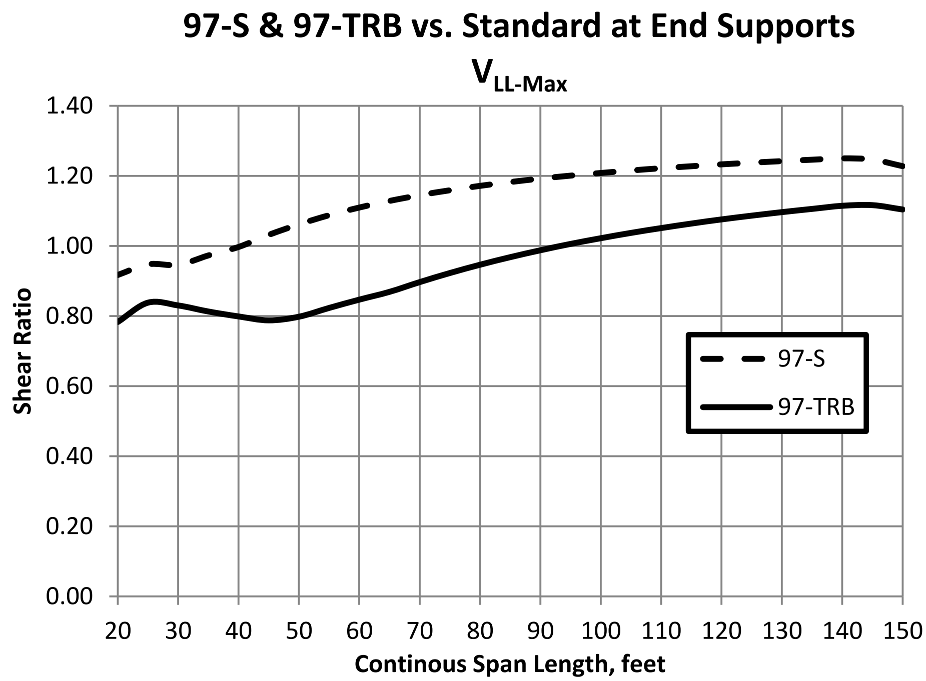


Figure 4-8. Shear Ratio at End Supports of 97-S & 97-TRB to AASHTO Standard Specifications

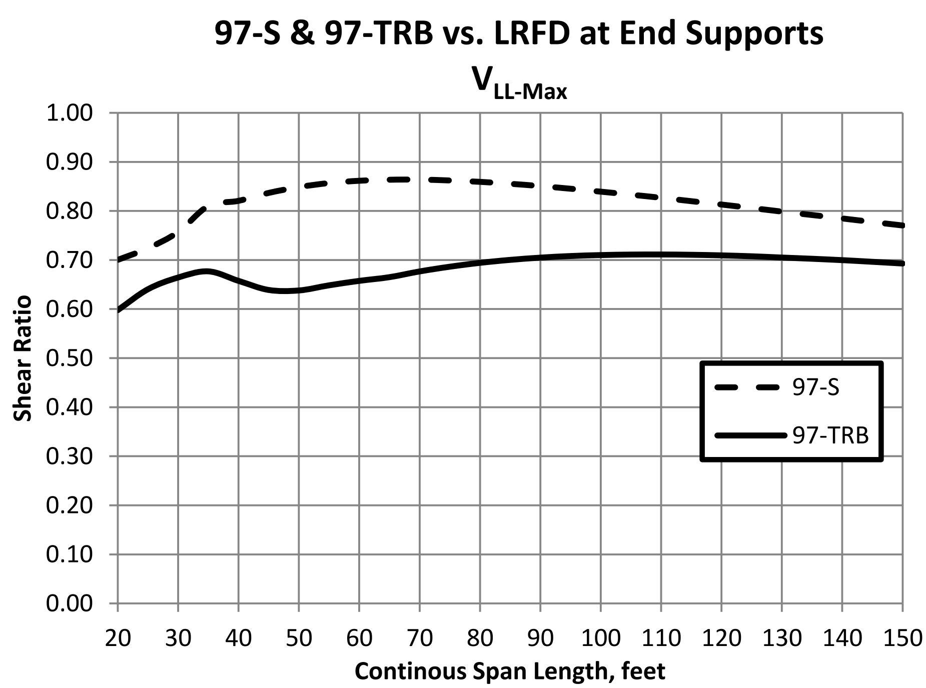


Figure 4-9. Shear Ratio at End Supports of 97-S & 97-TRB to LRFD Specifications

The shear from the 97-S is greater than those of the AL Legal Loads for continuous spans greater than 60-ft (Figure 4-10). The maximum shear increase becomes constant at 17% for spans 140-ft and greater. This 17% increase can be explained by the difference in gross vehicle weight of the 97-S and the 84-kip AL 3S3. Due to the greater 65-ft length of the 97-TRB, the shear effects are lower than the 97-S because the axle loads are not as compressed and are distributed to all supports at greater proportions. The 97-TRB begins to exhibit greater shear effects than the AL Legal Loads at spans above 120-ft. The constant shear increase approaches 5% as bridge spans lengthen.

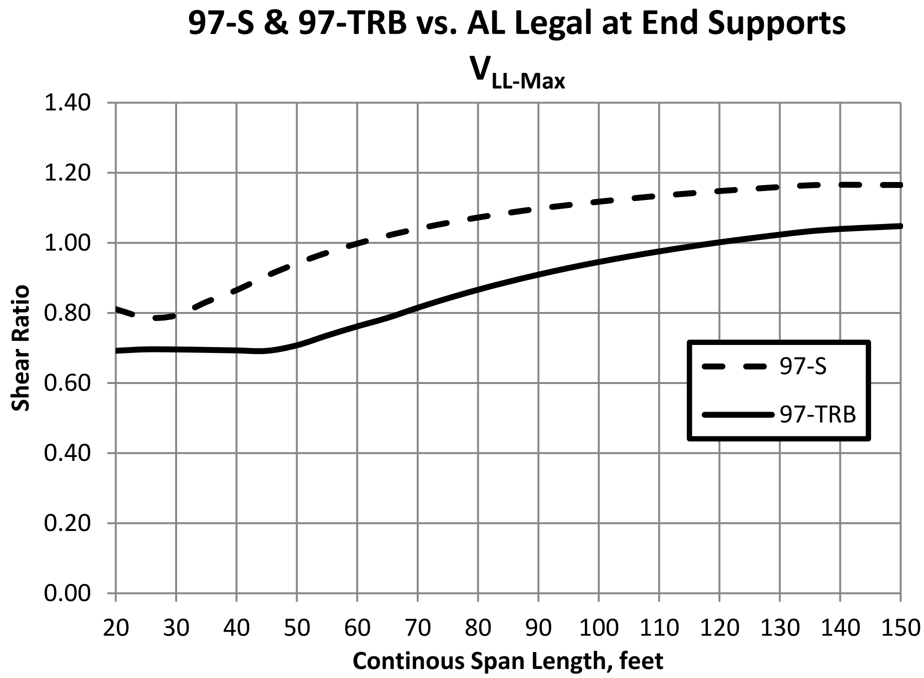


Figure 4-10 Shear Ratio at End Supports of 97-S & 97-TRB to AL Legal Loads

Shear at Center Support: The shear values generated by each live load model at the center support of the continuous spans are shown in Appendix B: Table B-2.

The 97-S demonstrates more shear force than the design loads from the AASHTO Standard Specifications produce at the center support for bridge spans 40-ft to 150-ft long (Figure 4-11). Continuous spans of 70-ft to 115-ft have a 20% increase or more in shear values compared to the Standard Specifications. The critical span length is 105-ft where the shear increase is 27%. The maximum shear values resulting from the 97-TRB are greater for continuous spans of 80-ft to 140-ft in length. At span lengths of 95-ft to 115-ft, the shear increase is at or above 10% of the Standard Specifications. The critical span length is 105-ft where the shear increase is 13%.

The maximum shear from the LRFD design loads envelopes both proposed 97-kip models (Figure 4-12). At locations about the center support, the 97-S will produce shear values less than 90% of the LRFD shears while the shears from the 97-TRB are considerably lower at a maximum of 70%. The shear ratios also decline as the bridge span increases; suggesting LRFD design loads produce conservative/higher design effects as for longer span bridges.

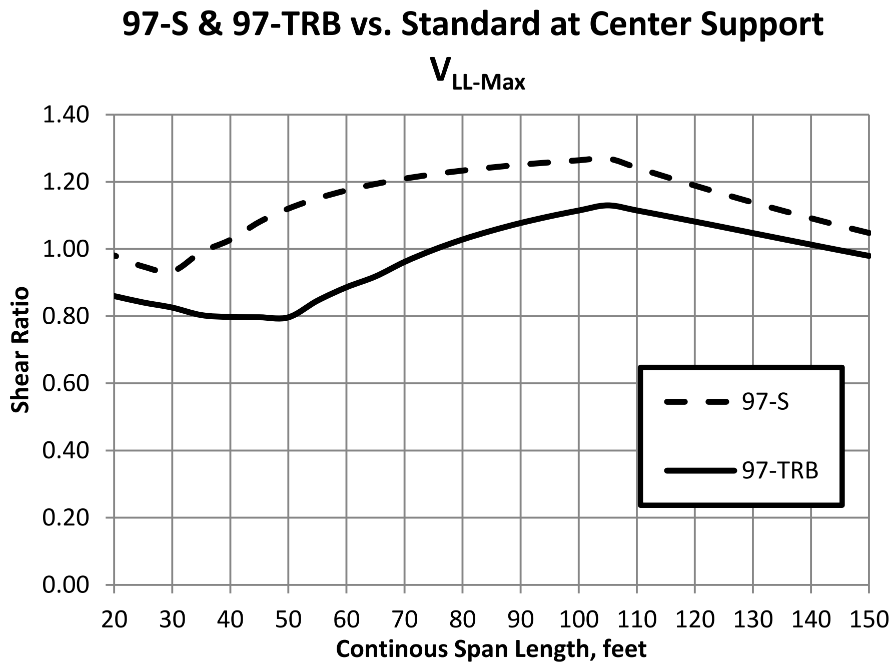


Figure 4-11. Shear Ratio at Center Support of 97-S & 97-TRB to AASHTO Standard Specifications

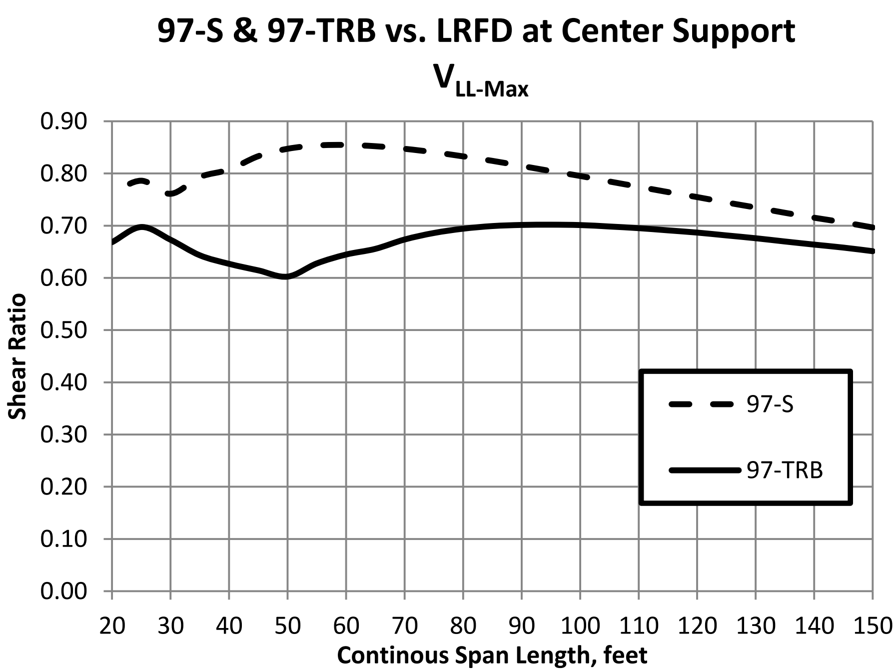


Figure 4-12. Shear Ratio at Center Support of 97-S & 97-TRB to LRFD Specifications

The controlling shear from the 97-S is greater than those of the AL Legal Loads for continuous spans of 50-ft or greater (Figure 4-13). The maximum shear increase becomes nearly constant at 16-17% for spans 85-ft and greater. The 97-TRB begins to exhibit shear forces greater than the AL Legal Loads at continuous spans of 90-ft and longer. The constant shear increase approaches 10% as bridge spans lengthen.

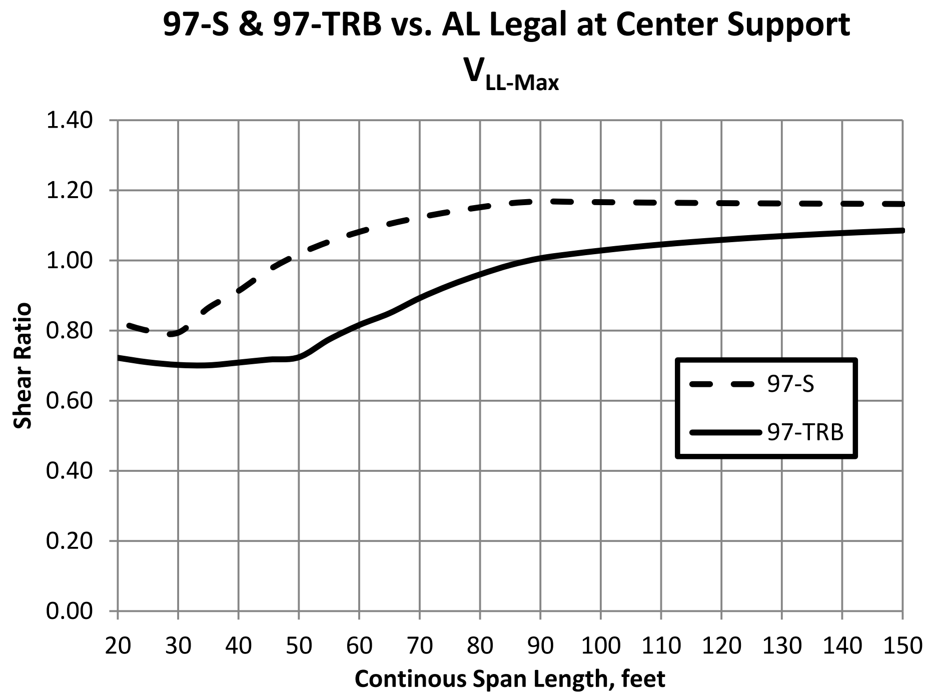


Figure 4-13. Shear Ratio at Center Support of 97-S & 97-TRB to AL Legal Loads

<u>**Maximum Positive Moment:**</u> The positive bending moment values formed in the continuous spans due to each load model are given in Appendix B: Table B-3.

As shown in Figure 4-14, the maximum bending moment of the 97-S exceeds the moment from the Standard Specifications for continuous spans of 40-ft and greater. The increase in moment is proportional to span lengths up to 150-ft. At span lengths of 115-ft and longer, the increase is greater than or equal to 20%. Effects of the 97-TRB are not as drastic, but bridges 130-ft and greater in span length will begin to experience moments greater than those from the Standard Specifications. The critical continuous span length for both proposed trucks is 150x150-ft where the 97-S and the 97-TRB experience positive moment increases of 23% and 5% respectively. However, the ratio curve from both 97-kip trucks appears to be increasing at 150x150-ft spans, so spans exceeding 150-ft in length should be checked to determine the actual critical span length.

Once again the LRFD design loads produce force effects that envelop all effects from the 97-S and the 97-TRB. Seen in Figure 4-15, the maximum positive moment of the 97-S is only 80% of the design value where the maximum value of the 97-TRB is only 63% of that established by LRFD loads.

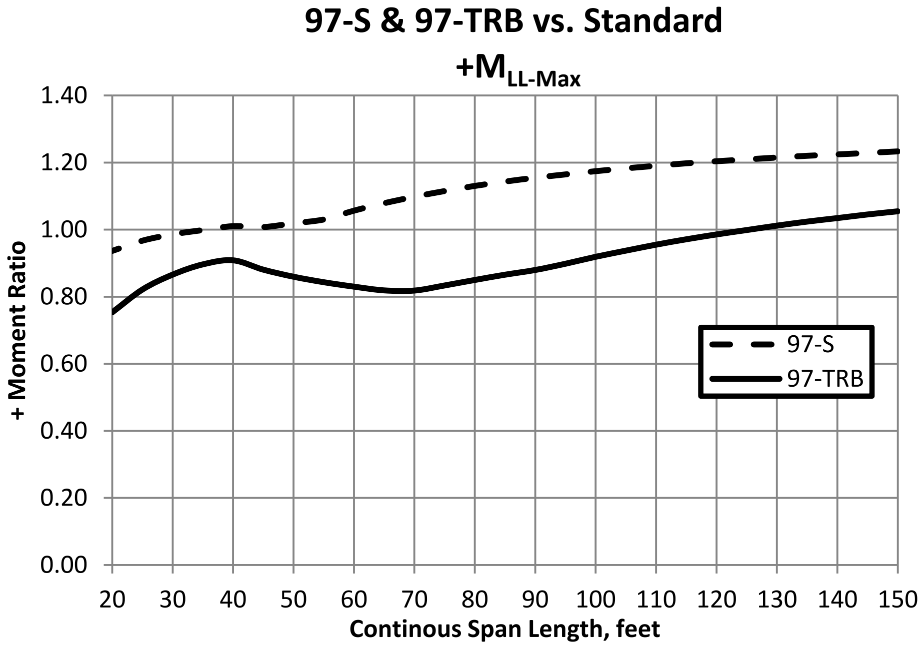


Figure 4-14. Positive Moment Ratio of 97-S & 97-TRB to AASHTO Standard Specifications

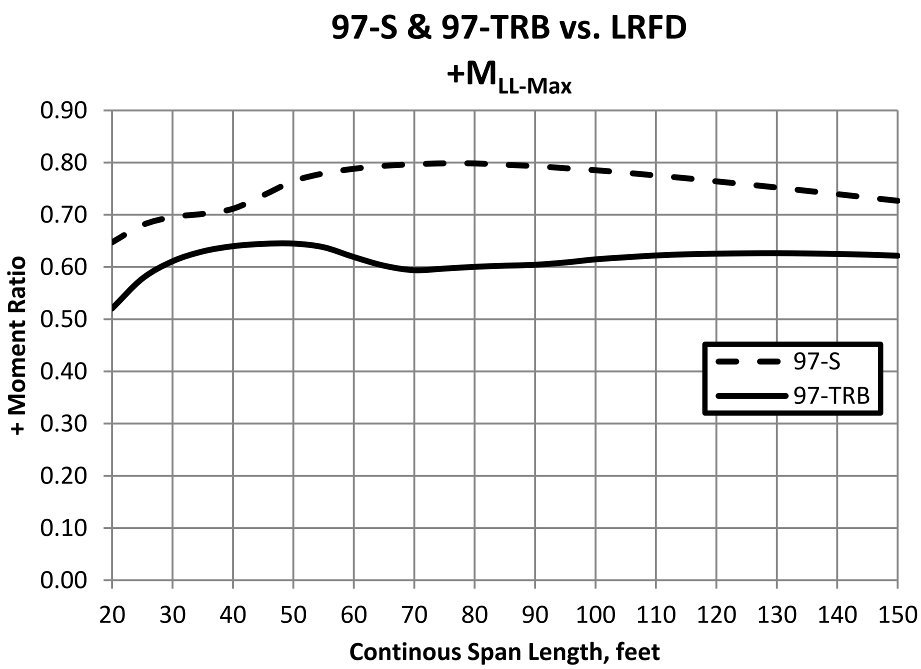


Figure 4-15. Positive Moment Ratio of 97-S & 97-TRB to LRFD Specifications

At spans of 85-ft, the 97-S causes an increase in bending moment over that of the AL Legal Loads (Figure 4-16). The maximum positive moment increase due to the 97-S is 13% at the longest continuous span analyzed of 150'x150'. In regards to the 97-TRB, the AL Legal Loads produce greater positive moments for every continuous span length included in this study. The critical vehicle configuration from the Alabama Legal Loads is the 75-kip Alabama tri-axle due to its short overall axle length of 19-ft combined with the 60-kip tri-axle. As noted previously, axle spacing has a significant impact on the magnitude of the force effect developed in bridge members.

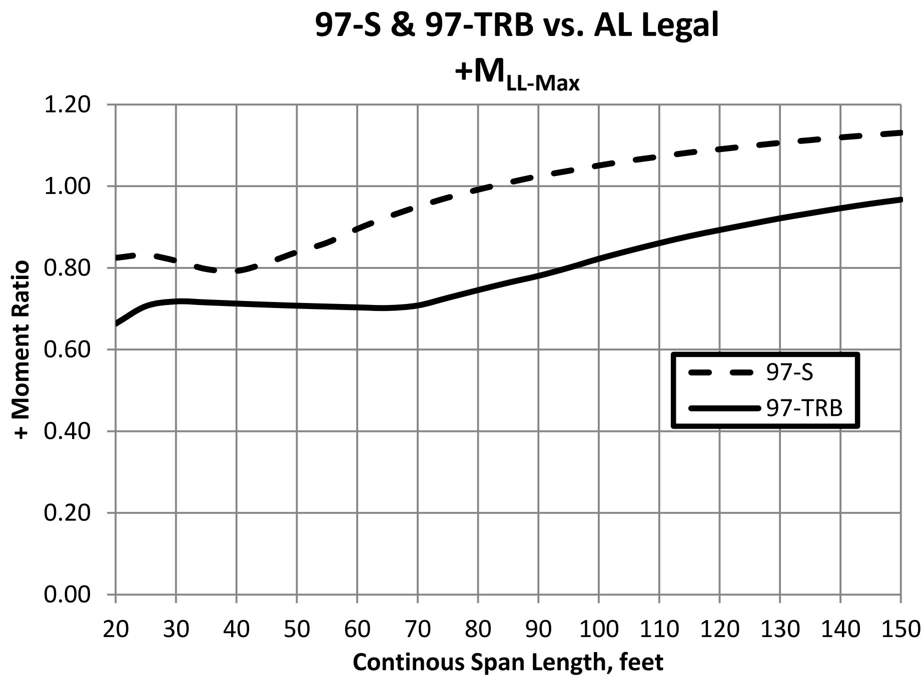


Figure 4-16 Positive Moment Ratio of 97-S & 97-TRB to AL Legal Loads

<u>Maximum Negative Moment:</u> In two-span continuous bridges, the critical negative moment is formed about the center support. These moments, for each live load model, are shown in Appendix B: Table B-4.

The 97-S results in negative bending moments above those from the Standard Specification for continuous span lengths shorter than 80x80-ft (Figure 4-17). The maximum negative moment increase is 31% at the critical span length of 30x30-ft. Continuous spans of 25-ft to 55-ft have a 20% increase or more in shear values compared to the Standard Specifications. The 97-TRB vehicle produces values above the design values for span ranges of 35-ft to 75-ft. The maximum negative moment increase is 41% at the critical span length of 55x55-ft. Continuous spans from 45-ft to 65-ft in length exhibit a 20% increase or more in shear values compared to the Standard Specifications. This is also the first plot that demonstrates force effects from the 97-TRB above those of the 97-S. This takes place in continuous span lengths ranging from 45-ft to 80-ft. Both proposed vehicle models generate negative moments appreciably lower than the Standard Specifications as span length increases.

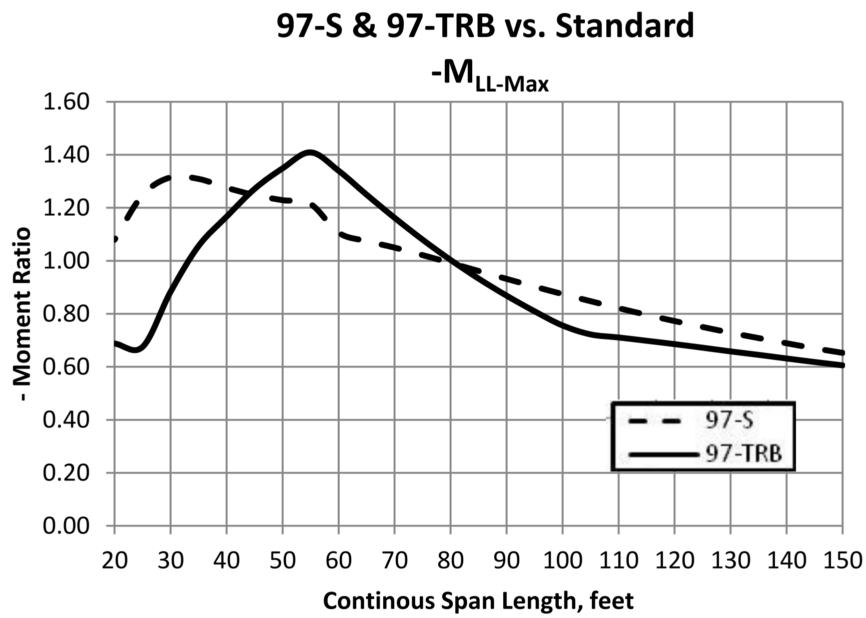


Figure 4-17. Negative Moment Ratio of 97-S & 97-TRB to AASHTO Standard Specifications

Keeping consistent with previous results, the negative moment developed by the LRFD design loadings envelop both proposed 97-kip vehicles (Figure 4-18). At the critical continuous span length of 30x30-ft, the moment caused by the 97-S is only 96% of the effect from maximum design loading of the AASHTO LRFD Specifications. For the 97-TRB, the critical moment is only 82% for a 50x50-ft bridge. As span length increases the proposed models establish negative moments that approach constant ratios near 40% of the LRFD design moments.

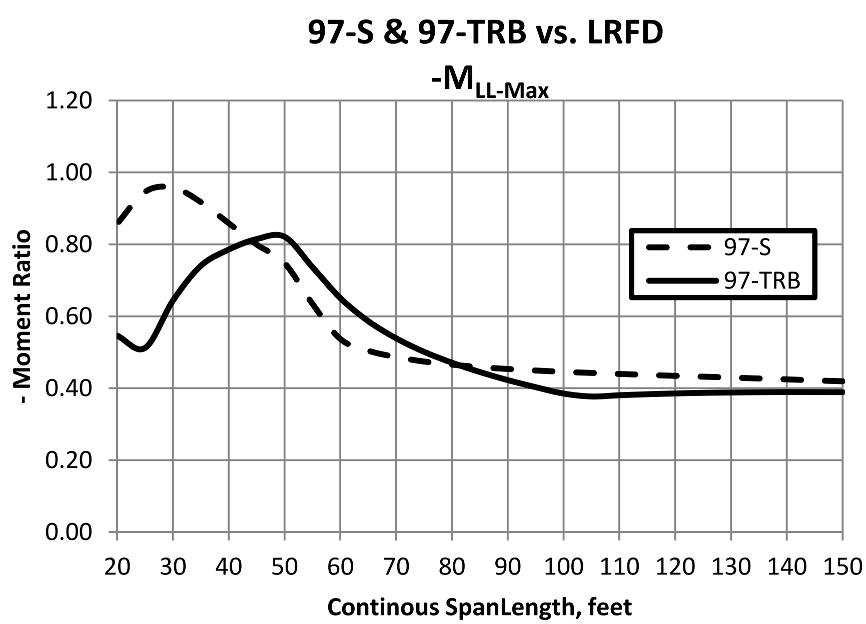


Figure 4-18. Negative Moment Ratio of 97-S & 97-TRB to LRFD Specifications

The 97-S will cause critical negative moments above those of the AL Legal Loads for all continuous span bridges included in this study (Figure 4-19). At span lengths above 80-ft, the variability in the negative moment ratio diminishes as a constant increase of 17% forms. Moments developed from the 97-TRB are greater than those from the AL Legal Loads on continuous spans 40x40-ft and greater.

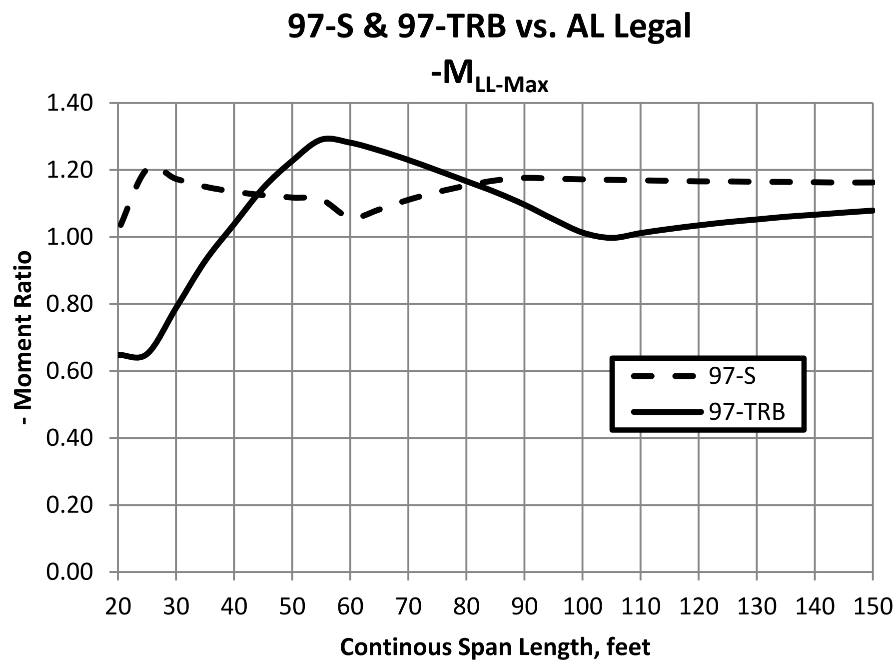


Figure 4-19. Negative Moment Ratio of 97-S & 97-TRB to AL Legal Loads

Transverse Deck Analysis:

SAP 2000 Analysis: After the thirteen deck models are analyzed in SAP 2000, the maximum positive and negative moments are extracted from the results. Since each deck consisted of four girders, they are treated as a continuous span having both positive and negative moment effects. The maximum moment locations are dependent upon the live load positioning. The locations of maximum positive moments varied slightly depending on clear span but are around 35%-45% of the exterior span length with respect to the exterior support.

For continuous beams, the maximum negative live load moment occurs at either the exterior support/overhang or the first interior support/girder. Per Article 4.6.2.1.6, maximum moments can be reduced to the respective values at the support face when reinforcement is being selected. With a 12-in girder base, the negative moments are taken at locations 6-in from the girder centerline. Each face of the exterior and interior supports had to be checked because the maximum values are dependent on their respective strip widths.

The LRFD design moment effects due to the 51-kip tri-axle are determined by multiplying product of the maximum moment generated from SAP 2000, $M_{LL\text{-}SAP}$, by the multiple presence factor of 1.2. These positive and negative moments denoted "$M_{LL\text{-}Max}$", are displayed in Table 4-1 and Table 4-2. The locations along the deck that these moments act are listed according to the nomenclature used by Barker and Puckett (2007). Each deck model has three equal spans between the four supports and two cantilevered overhangs totaling five beam-like members. In chronological order, members (1) and (5) are overhangs, members (2) and (4) are exterior spans, and member (3) is the interior span. The first numeral after the "M" represents which member the maximum moment is located. The numeral directly preceding the decimal, and all numerals following, signifies the precise location the moment acts, represented as a percentage of the member's span length. For the deck with a 5-ft girder spacing in Table 4-1, $M_{204.5}$ corresponds to the moment effect acting on the exterior span [member (2)] at a location 45% of the girder spacing (5-ft) measured from the shared support of the previous member, member (1). $M_{204.5}$ occurs on member (2) located 2.25' from the exterior support.

Table 4-1: Maximum Positive Moment from Live Loads

S/GS* (ft)	$+M_{LL\text{-}Max}$ Location	m	$+ M_{LL\text{-}SAP}$ (kip-ft/ft)	$+ M_{LL\text{-}Max}$ (kip-ft/ft)
4/5	$M_{204.50}$	1.2	1.854	2.225
4.5/5.5	$M_{204.09}$	1.2	1.948	2.338
5/6	$M_{203.75}$	1.2	2.088	2.506
5.5/6.5	$M_{203.85}$	1.2	2.240	2.688
6/7	$M_{203.57}$	1.2	2.399	2.879
6.5/7.5	$M_{203.67}$	1.2	2.580	3.096
7/8	$M_{203.44}$	1.2	2.776	3.331
7.5/8.5	$M_{203.53}$	1.2	2.970	3.564
8/9	$M_{203.61}$	1.2	3.185	3.822
8.5/9.5	$M_{203.42}$	1.2	3.386	4.063
9/10	$M_{203.50}$	1.2	3.589	4.307
9.5/10.5	$M_{203.57}$	1.2	3.796	4.555
10/11	$M_{203.41}$	1.2	3.984	4.781

* Clear Span and Girder Spacing

Table 4-2: Maximum Negative Moment from Live Loads

S/GS* (ft)	$-M_{LL\text{-}Max}$ Location	m	$+M_{LL\text{-}SAP}$ (kip-ft/ft)	$-M_{LL\text{-}Max}$ (kip-ft/ft)
4/5	$M_{201.00}$	1.2	-2.384	-2.861
4.5/5.5	$M_{200.91}$	1.2	-2.386	-2.863
5/6	$M_{200.83}$	1.2	-2.392	-2.870
5.5/6.5	$M_{200.77}$	1.2	-2.395	-2.874
6/7	$M_{200.71}$	1.2	-2.417	-2.900
6.5/7.5	$M_{200.67}$	1.2	-2.426	-2.911
7/8	$M_{200.63}$	1.2	-2.556	-3.067
7.5/8.5	$M_{200.59}$	1.2	-2.464	-2.957
8/9	$M_{200.56}$	1.2	-2.485	-2.982
8.5/9.5	$M_{200.53}$	1.2	-2.502	-3.002
9/10	$M_{200.50}$	1.2	-2.523	-3.028
9.5/10.5	$M_{200.48}$	1.2	-2.599	-3.119
10/11	$M_{200.45}$	1.2	-2.668	-3.202

* Clear Span and Girder Spacing

Realizing the live loads govern the maximum moment location for each deck model, moments from the dead loads are determined at the corresponding locations. These dead load moments, M_{DC}, which include the effects from the slab and barrier weight are shown in Tables 4-3 & 4-4. Once all extreme moment effects from the live and dead loads are determined, the ultimate factored force effect is calculated per the right side of *Equation 3-1*. The specific equation for the ultimate factored moment, M_u, is:

$$M_u = \sum \eta_i \gamma_i M_i = 1.0\gamma_p M_{DC} + 1.0\gamma_{LL} 1.33\, M_{LL} \qquad \textit{Equation 4-1}$$

The permanent load factors are assigned to produce the extreme force effect. If the additive force effects from the dead loads do achieve this, the permanent load factor, γ_p, is taken as the maximum factor (1.25). However, if the additive values lessen the total force effect, the minimum load factor shall be used (0.9) [A3.4.1]. The ultimate factored force effects and corresponding load factors are given in Tables 4-5 and 4-6.

Table 4-3: Moment from Dead Loads at +$M_{LL\text{-}Max}$ Location

S/S_G* ft	$+M_{LL\text{-}Max}$ Location	M_{Slab} kip-ft/ft	$M_{Barrier}$ kip-ft/ft	M_{DC} kip-ft/ft
4/5	$M_{204.50}$	-0.1208	-0.4904	-0.6112
4.5/5.5	$M_{204.09}$	-0.1084	-0.5342	-0.6426
5/6	$M_{203.75}$	-0.092	-0.5717	-0.6637
5.5/6.5	$M_{203.85}$	-0.0395	-0.5583	-0.5978
6/7	$M_{203.57}$	-0.0148	-0.5894	-0.6042
6.5/7.5	$M_{203.67}$	0.0449	-0.5768	-0.5319
7/8	$M_{203.44}$	0.0770	-0.603	-0.5260
7.5/8.5	$M_{203.53}$	0.1439	-0.5912	-0.4473
8/9	$M_{203.61}$	0.2131	-0.5808	-0.3677
8.5/9.5	$M_{203.42}$	0.2655	-0.603	-0.3375
9/10	$M_{203.50}$	0.3449	-0.593	-0.2481
9.5/10.5	$M_{203.57}$	0.4421	-0.5841	-0.1420
10/11	$M_{203.41}$	0.5165	-0.6031	-0.0866

* Clear Span and Girder Spacing

Table 4-4: Moment from Dead Loads at –$M_{LL\text{-}Max}$ Location

S/S_G* ft	$-M_{LL\text{-}Max}$ Location	M_{Slab} kip-ft/ft	$M_{Barrier}$ kip-ft/ft	M_{DC} kip-ft/ft
4/5	$M_{201.00}$	-0.4672	-0.8983	-1.3655
4.5/5.5	$M_{200.91}$	-0.4644	-0.908	-1.3724
5/6	$M_{200.83}$	-0.4608	-0.9164	-1.3772
5.5/6.5	$M_{200.77}$	-0.4565	-0.9235	-1.3800
6/7	$M_{200.71}$	-0.4515	-0.9297	-1.3812
6.5/7.5	$M_{200.67}$	-0.4461	-0.9351	-1.3812
7/8	$M_{200.63}$	-0.4403	-0.9399	-1.3802
7.5/8.5	$M_{200.59}$	-0.4342	-0.9442	-1.3784
8/9	$M_{200.56}$	-0.4278	-0.9480	-1.3758
8.5/9.5	$M_{200.53}$	-0.4360	-0.9514	-1.3874
9/10	$M_{200.50}$	-0.4289	-0.9545	-1.3834
9.5/10.5	$M_{200.48}$	-0.4366	-0.9573	-1.3939
10/11	$M_{200.45}$	-0.4430	-0.9599	-1.4029

* Clear Span and Girder Spacing

Table 4-5: Positive Ultimate Factored Design Moment

S/S_G ft	γ_p	M_{DC} k-ft/ft	γ_{LL}	(1+IM)	+ M_{LL} k-ft/ft	**+ M_u k-ft/ft**
4/5	1.25	-0.611	1.75	1.33	2.225	**4.41**
4.5/5.5	1.25	-0.643	1.75	1.33	2.338	**4.64**
5/6	1.25	-0.664	1.75	1.33	2.506	**5.00**
5.5/6.5	1.25	-0.598	1.75	1.33	2.688	**5.51**
6/7	1.25	-0.604	1.75	1.33	2.879	**5.95**
6.5/7.5	0.9	-0.532	1.75	1.33	3.096	**6.73**
7/8	0.9	-0.526	1.75	1.33	3.331	**7.28**
7.5/8.5	0.9	-0.447	1.75	1.33	3.564	**7.89**
8/9	0.9	-0.368	1.75	1.33	3.822	**8.56**
8.5/9.5	0.9	-0.338	1.75	1.33	4.063	**9.15**
9/10	0.9	-0.248	1.75	1.33	4.307	**9.80**
9.5/10.5	0.9	-0.142	1.75	1.33	4.555	**10.47**
10/11	0.9	-0.087	1.75	1.33	4.781	**11.05**

Table 4-6: Negative Ultimate Factored Design Moment

S/S_G ft	γ_p	M_{DC} k-ft/ft	γ_{LL}	(1 + IM)	- M_{LL} k-ft/ft	**- M_u k-ft/ft**
4/5	1.25	-1.37	1.75	1.33	-2.861	**-8.37**
4.5/5.5	1.25	-1.37	1.75	1.33	-2.863	**-8.38**
5/6	1.25	-1.38	1.75	1.33	-2.870	**-8.40**
5.5/6.5	1.25	-1.38	1.75	1.33	-2.874	**-8.41**
6/7	1.25	-1.38	1.75	1.33	-2.900	**-8.48**
6.5/7.5	1.25	-1.38	1.75	1.33	-2.911	**-8.50**
7/8	1.25	-1.38	1.75	1.33	-3.067	**-8.86**
7.5/8.5	1.25	-1.38	1.75	1.33	-2.957	**-8.60**
8/9	1.25	-1.38	1.75	1.33	-2.982	**-8.66**
8.5/9.5	1.25	-1.39	1.75	1.33	-3.002	**-8.72**
9/10	1.25	-1.38	1.75	1.33	-3.028	**-8.78**
9.5/10.5	1.25	-1.394	1.75	1.33	-3.119	**-9.00**
10/11	1.25	-1.40	1.75	1.33	-3.202	**-9.21**

ALDOT Standard Bridge Slab Reinforcement Evaluation:

Positive Reinforcement:

Resistance Factors: Shown in Table 4-7, the net tensile strain in the thirteen deck members reveals that each section is tension-controlled as the primary reinforcement yields to a point that expresses ductile behavior. Large deflections and cracking will occur before deck failure.

Table 4-7: Resistance Factors for Positive Reinforcement

S ft	D in	d_{pos} in	c in	ε_t in/in	> 0.005	Φ
4.0	7	5.69	0.986	0.014	0.005	0.9
4.5	7	5.69	0.986	0.014	0.005	0.9
5.0	7	5.69	0.986	0.014	0.005	0.9
5.5	7	5.69	1.073	0.013	0.005	0.9
6.0	7	5.69	1.176	0.012	0.005	0.9
6.5	7	5.69	1.176	0.012	0.005	0.9
7.0	7	5.69	1.176	0.012	0.005	0.9
7.5	7	5.69	1.228	0.011	0.005	0.9
8.0	7	5.69	1.280	0.010	0.005	0.9
8.5	7 ¼	5.94	1.280	0.011	0.005	0.9
9.0	7 ¼	5.94	1.436	0.009	0.005	0.9
9.5	7 ½	6.19	1.436	0.010	0.005	0.9
10.0	7 ¾	6.44	1.436	0.010	0.005	0.9

Positive Reinforcement Supplied [A5.7.3.3.2]: Positive reinforcement supplied by ALDOT contains adequate strength for the 51-kip tri-axle of the 97-S for all clear spacing (Table 4-8).

Table 4-8: Minimum Positive Reinforcement Check

S ft	D in	S_{nc} in^3	f_r ksi	$1.2M_{cr}$ k-ft/ft	$1.33M_u$ k-ft/ft	$+M_{u\text{-}Design}$ k-ft/ft	≤	ΦM_n k-ft/ft	CHECK
4.0	7	98	0.74	7.25	5.87	5.87	≤	13.51	✓
4.5	7	98	0.74	7.25	6.17	6.17	≤	13.51	✓
5.0	7	98	0.74	7.25	6.65	6.65	≤	13.51	✓
5.5	7	98	0.74	7.25	7.33	7.25	≤	14.60	✓
6.0	7	98	0.74	7.25	7.91	7.25	≤	15.87	✓
6.5	7	98	0.74	7.25	8.95	7.25	≤	15.87	✓
7.0	7	98	0.74	7.25	9.68	7.28	≤	15.87	✓
7.5	7	98	0.74	7.25	10.50	7.89	≤	16.50	✓
8.0	7	98	0.74	7.25	11.39	8.56	≤	17.13	✓
8.5	7 ¼	105	0.74	7.78	12.17	9.15	≤	17.96	✓
9.0	7 ¼	105	0.74	7.78	13.04	9.80	≤	19.90	✓
9.5	7 ½	113	0.74	8.33	13.93	10.47	≤	20.83	✓
10	7 ¾	120	0.74	8.89	14.70	11.05	≤	21.76	✓

Negative Reinforcement:

<u>Resistance Factors:</u> Table 4-9 shows the resistance factors required for the maximum limit of negative reinforcement. The reinforcement all for members experiencing negative moment is tension-controlled. Therefore sufficient ductility exists in every deck member.

Table 4-9: Resistance Factors for Negative Reinforcement

S ft	D in	d_{neg} in	c in	ε_t in/in	> 0.005	Φ
4.0	7	4.69	0.986	0.011	0.005	0.9
4.5	7	4.69	0.986	0.011	0.005	0.9
5.0	7	4.69	0.986	0.011	0.005	0.9
5.5	7	4.69	1.073	0.010	0.005	0.9
6.0	7	4.69	1.176	0.009	0.005	0.9
6.5	7	4.69	1.176	0.009	0.005	0.9
7.0	7	4.69	1.176	0.009	0.005	0.9
7.5	7	4.69	1.228	0.008	0.005	0.9
8.0	7	4.69	1.280	0.008	0.005	0.9
8.5	7 ¼	4.94	1.280	0.009	0.005	0.9
9.0	7 ¼	4.94	1.436	0.007	0.005	0.9
9.5	7 ½	5.19	1.436	0.008	0.005	0.9
10.0	7 ¾	5.44	1.436	0.008	0.005	0.9

<u>Negative Reinforcement Supplied [A5.7.3.3.2]:</u> The design check is shown in Table 4-10. The negative reinforcement given in the ALDOT Standard Bridge Slab chart does meet the strength requirements of the LRFD specifications.

Table 4-10: Minimum Negative Reinforcement Check

S ft	D in	S_{nc} in^3	f_r ksi	$1.2M_{cr}$ k-ft/ft	$1.33M_u$ k-ft/ft	$-M_{u\text{-}Design}$ k-ft/ft	≤	ΦM_n k-ft/ft	CHECK
4.0	7	98	0.74	7.25	11.13	8.37	≤	10.9	✓
4.5	7	98	0.74	7.25	11.14	8.38	≤	10.9	✓
5.0	7	98	0.74	7.25	11.18	8.40	≤	10.9	✓
5.5	7	98	0.74	7.25	11.19	8.41	≤	11.8	✓
6.0	7	98	0.74	7.25	11.27	8.48	≤	12.8	✓
6.5	7	98	0.74	7.25	11.31	8.50	≤	12.8	✓
7.0	7	98	0.74	7.25	11.79	8.86	≤	12.8	✓
7.5	7	98	0.74	7.25	11.44	8.60	≤	13.3	✓
8.0	7	98	0.74	7.25	11.52	8.66	≤	13.8	✓
8.5	7 ¼	105	0.74	7.78	11.60	8.72	≤	14.6	✓
9.0	7 ¼	105	0.74	7.78	11.67	8.78	≤	16.2	✓
9.5	7 ½	113	0.74	8.33	11.97	9.00	≤	17.1	✓
10.	7 ¾	120	0.74	8.89	12.24	9.21	≤	18.0	✓

5.0 Conclusion, Recommendations, & Future Research

Conclusions

It is recommended that 97-kip trucks be limited to longer truck lengths were the kingpin-to-rear axle spacing is at or near the maximum allowed (41 ft in Alabama). This would allow tractor-semitrailer combinations with van-type, 53-ft trailers equipped with six axles (similar to the 97-TRB) to operate with a GVW of 97-kip, but would preclude the more impactful, shorter trucks (similar to the 97-S) from operating due to the greater increase in bridge shear and moment that the shorter trucks would produce. The figures presented in Chapter 4 show that for many short and medium span bridges (less than 100 feet), the effect of the longer 97-kip truck would not exceed the HS20 design load by more than 5%, which is the same limit on which the current Federal Bridge Formula is based. Specific summaries and conclusions regarding the specific force effects for simple and two-span, symmetric, continuous bridges are discussed below.

Simple Spans: Minimum span length of 20-ft & maximum of 300-ft

- The 97-S induces greater shear and moment effects than the longer 97-TRB. Axle spacing is a critical factor when determining force effects on bridges. For trucks of the same GVW, decreasing axle spacing will increase the magnitude of the force effect.

- Maximum force effects (i.e. shear, moment, or both) from the 97-S truck exceed the effects produced by the design loadings from the AASHTO Standard Specifications by over 20% on simple span bridges from 80-ft to 155-ft in length. Maximum force effects (i.e. shear, moment, or both) from the 97-TRB truck exceed the effects produced by the design loadings from the AASHTO Standard Specifications by over 5% only for simple span bridges from 110-ft to 150-ft in length but the greatest effect (shear) increase is not more than 13%.

- The critical shear and moment developed from the design loads of the AASHTO LRFD Specifications completely envelope the force effects from both 97-kip trucks

- Compared to the five Alabama Legal loads investigated, the 97-TRB initiates greater effects only for spans of 100-ft or longer while the 97-S produces higher effects on spans of 55-ft and longer.

- HL-93 loadings from the AASHTO LRFD Specifications generate direct, unfactored effects that are 17% to over 70% above the effects resulting from the HS20-44 design loading and the AML from the AASHTO Standard Specifications. This equates to greater potential for bridges designed for the LRFD loading to accommodate trucks with increased gross vehicle weight.

Continuous Spans: Span ratio of 1:1 with minimum span length of 20-ft & maximum of 300-ft

- The 97-S generally produces greater shear and moments than the 97-TRB. The only instance when this is not valid is the negative moment effect for bridge spans of 45-ft to 80-ft in length.

- HL-93 loading provides unfactored design shear that exceed both proposed truck models. The shear effect from the 97-S and 97-TRB is a maximum 86% and 71% of the design shear, respectively.

- All critical moments from HL-93 loading envelope both 97-kip trucks. All of the positive moments from the 97-TRB are exceeded by at least 50% and all negative moments by 22%. The moment effect from the 97-S and 97-TRB is a maximum 96% and 82% of the design moment, respectively.

- The HS20-44 design loading and AML of the Standard Specifications do not fully envelope the critical effects from the proposed 97-kip models. At the end supports, the 97-S causes a shear increase of 20% or greater on span lengths of 90-ft and longer while the 97-TRB induces a 10% increase on spans 130-ft and longer. At the center support, the maximum increase is for continuous spans of 105-ft, where the increase in shear from the 97-S and 97-TRB are 27% and 13% respectively. The maximum positive moment created by the 97-S loading exceeds the moment from HS20-44/AML for all spans above 35-ft while the 97-TRB moment is higher for spans 130-ft and longer. The negative moment of the 97-S exceeds the moment effect from the design loading by as much as 31% and the 97-TRB exceeds this effect by as much as 41%.

- When considering positive moment effects, the critical load of the Alabama Legal Loads is the 19-ft 75-kip Alabama Tri-Axle. For all continuous spans up to 150-ft, the tri-axle causes greater positive bending moment than the 97-TRB. However, the moment produced from the 97-S exceeds the Alabama Tri-Axle on continuous spans greater than 80x80-ft long.

Transverse Deck Reinforcement:

- The positive and negative reinforcement currently supplied in the ALDOT Standard Bridge Slab chart satisfies LRFD strength requirements for the critical axle grouping of the 97-S for all reinforced concrete deck girders.

- RC deck sections experiencing positive moment have at least a 94% increase in strength capacity over the ultimate design moment. Sections influenced by negative moment have at least a 30% increase over the ultimate design moment

Recommendations

Simple and Continuous Span Bridges:

- The 97-S produces greater force effects on all simple span and most continuous span bridges than the longer 97-TRB. More analysis is needed to evaluate bridge safety and the associated costs before the 97-S configuration is considered a viable option for 97-kip trucks.

- LRFD methods should be adopted by state agencies because the design force effects from the notional HL-93 loading effectively envelope heavier trucks. GVW increases of 20% or more are expected in the future so bridges must be designed to withstand the greater force effects. The HS20-44 loading under-predicts the force effects that heavier vehicles demonstrate.

- In terms of the force effects created, the 97-TRB is the better alternative compared to the 97-S. This should equate to less construction cost required to strengthen or build new bridges. However, the overall cost depends on several complicated factors that have only been reasonably estimated at the global level. Additional research is required in order to justify heavier trucks on the IHS.

Transverse Deck Reinforcement:

- Since the reinforcement called out in ALDOT's design standards meets LRFD strength limit requirements, the 51-kip tri-axle of the 97-S should be considered as potential axle configuration for heavier trucks on the Alabama bridge network. ALDOT Bridge Bureau should not have to implement drastic reform to their standard deck slabs to accommodate this axle grouping. However, adopting LRFD for the design of all bridge elements will likely require additional reinforcement in the deck slab to accommodate the barrier collision extreme event load cases of the LRFD Specifications.

Future Research: In order to effectively quantify the effects that increasing truck weight will have on bridges, more extensive and detailed analysis is required. The force effects determined from this sensitivity analysis only consider two hypothetical 97,000-lb truck configurations with six-axles and constant axle spacings. Additional vehicles with variable axle spacing and overall length need to be examined to produce optimum configurations of increased GVW while limiting the impacts on the bridge network.

When heavier vehicles are allowed to operate, certain bridges will likely be overstressed. Overstressing bridge elements can result in decreased service life and more rapid accumulation of damage, but the extent of damage will be dependent on bridge type, bridge age, construction method utilized, geographical location, average daily truck traffic, etc. Fatigue damage is not addressed in this analysis but plays a role regarding the impacts on bridges and should be considered in future studies.

State agencies should expect an increase in total bridge cost and a standard methodology for evaluating bridges is needed to arrive at this value. All factors listed herein should be included, but it is essential to verify that the increased benefits outweigh additional costs. What percentage of the annual commercial truck traffic will switch to heavier vehicles? Will uniformity in weight limits exist between states? Restructuring the bridge network will be gradual at best but should begin on routes that are expected to have the highest demand for increased GVW, supplying the greatest return on the investment. Once the facts have been collected and sorted, the cost impacts of heavier trucks can be fully measured.

6.0 References

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Appendix A: Force Effects in Simply Supported Bridges

Table A-1: Simple Span–Maximum Shear due to Vehicular Loads

Span ft	97-S kip	97-TRB kip	Standard kip	LRFD kip	AL Legal kip
20	40.80	35.70	43.20	51.40	50.25
25	42.84	38.76	46.08	54.08	55.20
30	46.47	40.80	49.60	59.20	58.50
35	51.97	42.26	52.80	64.00	60.86
40	56.10	43.35	55.20	68.00	62.63
45	60.64	44.20	57.07	71.47	64.00
50	64.28	46.75	58.56	74.56	65.10
55	67.25	50.23	59.78	77.38	66.00
60	69.73	53.13	60.80	80.00	66.75
65	71.83	55.58	61.66	82.46	67.38
70	73.63	58.54	62.40	84.80	67.93
75	75.19	61.10	63.04	87.04	68.40
80	76.55	63.34	63.60	89.20	68.81
85	77.75	65.32	64.09	91.29	69.18
90	78.82	67.08	64.53	93.33	69.50
95	79.78	68.66	64.93	95.33	69.79
100	80.64	70.08	65.28	97.28	70.05
105	81.42	71.36	65.60	99.20	70.29
110	82.13	72.52	65.89	101.1	70.50
115	82.77	73.59	66.16	103.0	70.99
120	83.37	74.56	66.40	104.8	71.54
125	83.91	75.46	66.62	106.6	72.04
130	84.42	76.29	67.60	108.4	72.50
135	84.88	77.06	69.20	110.2	72.92
140	85.31	77.77	70.80	112.0	73.32
145	85.72	78.43	72.40	113.8	73.69
150	86.09	79.05	74.00	115.5	74.03

Table A-2: Simple Span–Maximum Moment due to Vehicular Loads

Span ft	97-S kip-ft	97-TRB kip-ft	Standard kip-ft	LRFD kip-ft	AL Legal kip-ft
20	187.0	153.0	194.4	234.2	225.2
25	250.8	216.8	253.9	314.2	300.2
30	314.5	280.5	313.6	398.3	379.2
35	378.3	344.3	373.4	486.6	472.4
40	442.0	408.0	449.8	578.9	565.7
45	529.1	471.8	538.7	699.0	659.1
50	633.8	535.5	627.8	826.1	752.5
55	738.9	599.3	717.1	957.4	846.1
60	852.8	663.0	806.5	1093	939.6
65	971.6	726.8	896.0	1232	1033
70	1091	790.5	985.6	1376	1127
75	1210	872.7	1075	1523	1220
80	1330	977.0	1165	1675	1314
85	1450	1082	1255	1831	1408
90	1570	1186	1344	1991	1501
95	1690	1297	1434	2154	1595
100	1810	1415	1524	2322	1689
105	1930	1533	1614	2494	1782
110	2051	1652	1704	2670	1876
115	2171	1771	1793	2850	1970
120	2292	1890	1883	3034	2064
125	2412	2009	1973	3221	2157
130	2533	2129	2063	3413	2251
135	2654	2248	2153	3609	2345
140	2775	2368	2243	3809	2438
145	2895	2488	2335	4013	2532
150	3016	2607	2475	4221	2626

Table A-3: Simple Span–LRFD Shear Increase to AASHTO Standard Specifications

Span ft	Standard kip	LRFD kip	**LRFD Increase**	Span ft	Standard kip	LRFD kip	**LRFD Increase**
20	43.20	51.40	**19%**	165	78.80	120.7	**53%**
25	46.08	54.08	**17%**	170	80.40	122.4	**52%**
30	49.60	59.20	**19%**	175	82.00	124.2	**51%**
35	52.80	64.00	**21%**	180	83.60	125.9	**51%**
40	55.20	68.00	**23%**	185	85.20	127.6	**50%**
45	57.07	71.47	**25%**	190	86.80	129.3	**49%**
50	58.56	74.56	**27%**	195	88.40	131.0	**48%**
55	59.78	77.38	**29%**	200	90.00	132.6	**47%**
60	60.80	80.00	**32%**	205	91.60	134.3	**47%**
65	61.66	82.46	**34%**	210	93.20	136.0	**46%**
70	62.40	84.80	**36%**	215	94.80	137.7	**45%**
75	63.04	87.04	**38%**	220	96.40	139.3	**45%**
80	63.60	89.20	**40%**	225	98.00	141.0	**44%**
85	64.09	91.29	**42%**	230	99.60	142.7	**43%**
90	64.53	93.33	**45%**	235	101.20	144.3	**43%**
95	64.93	95.33	**47%**	240	102.80	146.0	**42%**
100	65.28	97.28	**49%**	245	104.40	147.7	**41%**
105	65.60	99.20	**51%**	250	106.00	149.3	**41%**
110	65.89	101.1	**53%**	255	107.60	151.0	**40%**
115	66.16	103.0	**56%**	260	109.20	152.6	**40%**
120	66.40	104.8	**58%**	265	110.80	154.3	**39%**
125	66.62	106.6	**60%**	270	112.40	155.9	**39%**
130	67.60	108.4	**60%**	275	114.00	157.6	**38%**
135	69.20	110.2	**59%**	280	115.60	159.2	**38%**
140	70.80	112.0	**58%**	285	117.20	160.8	**37%**
145	72.40	113.8	**57%**	290	118.80	162.5	**37%**
150	74.00	115.5	**56%**	295	120.40	164.1	**36%**
155	75.60	117.3	**55%**	300	122.00	165.8	**36%**
160	77.20	119.0	**54%**				

Table A-4: Simple Span–LRFD Moment Increase to AASHTO Standard Specifications

Span ft	Standard kip-ft	LRFD kip-ft	**LRFD Increase**	Span ft	Standard kip-ft	LRFD kip-ft	**LRFD Increase**
20	194.4	234.2	**20%**	165	2921	4869	**67%**
25	253.9	314.2	**24%**	170	3077	5093	**66%**
30	313.6	398.3	**27%**	175	3238	5320	**64%**
35	373.4	486.6	**30%**	180	3402	5552	**63%**
40	449.8	578.9	**29%**	185	3571	5788	**62%**
45	538.7	699.0	**30%**	190	3743	6028	**61%**
50	627.8	826.1	**32%**	195	3920	6272	**60%**
55	717.1	957.4	**34%**	200	4100	6520	**59%**
60	806.5	1093	**35%**	205	4285	6772	**58%**
65	896.0	1232	**38%**	210	4473	7028	**57%**
70	985.6	1376	**40%**	215	4666	7288	**56%**
75	1075	1523	**42%**	220	4862	7552	**55%**
80	1165	1675	**44%**	225	5063	7820	**54%**
85	1255	1831	**46%**	230	5267	8092	**54%**
90	1344	1991	**48%**	235	5476	8368	**53%**
95	1434	2154	**50%**	240	5688	8648	**52%**
100	1524	2322	**52%**	245	5905	8932	**51%**
105	1614	2494	**55%**	250	6125	9220	**51%**
110	1704	2670	**57%**	255	6350	9512	**50%**
115	1793	2850	**59%**	260	6578	9808	**49%**
120	1883	3034	**61%**	265	6811	10108	**48%**
125	1973	3221	**63%**	270	7047	10412	**48%**
130	2063	3413	**65%**	275	7288	10720	**47%**
135	2153	3609	**68%**	280	7532	11032	**46%**
140	2243	3809	**70%**	285	7781	11348	**46%**
145	2335	4013	**72%**	290	8033	11668	**45%**
150	2475	4221	**71%**	295	8290	11992	**45%**
155	2620	4433	**69%**	300	8550	12320	**44%**
160	2768	4649	**68%**				

Appendix B: Force Effects in Continuous Span Bridges

Table B-1: Continuous Span–Maximum Shear at End Supports

Span ft	97-S kip	97-TRB kip	Standard kip	LRFD kip	AL Legal kip
20x20	38.55	32.91	42.03	55.05	47.55
25x25	40.95	36.22	43.21	56.56	52.03
30x30	43.93	38.55	46.43	58.03	55.40
35x35	48.18	40.26	49.52	59.48	57.96
40x40	51.87	41.56	52.02	63.21	59.96
45x45	55.76	42.58	54.05	66.64	61.55
50x50	59.16	44.47	55.72	69.71	62.84
55x55	62.12	47.01	57.11	72.50	63.91
60x60	64.69	49.37	58.30	75.09	64.82
65x65	66.93	51.54	59.31	77.50	65.58
70x70	68.91	53.98	60.19	79.77	66.25
75x75	70.64	56.24	60.95	81.94	66.82
80x80	72.19	58.32	61.62	84.01	67.32
85x85	73.57	60.23	62.22	86.01	67.77
90x90	74.81	61.98	62.75	87.94	68.16
95x95	75.92	63.60	63.23	89.82	68.52
100x100	76.94	65.09	63.68	91.67	68.85
105x105	77.86	66.45	64.06	93.45	69.14
110x110	78.71	67.71	64.42	95.20	69.40
115x115	79.48	68.89	64.75	96.90	69.65
120x120	80.19	69.97	65.04	98.6	69.86
125x125	80.84	70.97	65.32	100.3	70.07
130x130	81.46	71.91	65.58	102.0	70.26
135x135	82.02	72.78	65.82	103.6	70.44
140x140	82.55	73.61	66.04	105.2	70.81
145x145	83.03	74.37	66.61	106.8	71.25
150x150	83.51	75.10	68.02	108.4	71.67

Table B-2: Continuous Span–Maximum Shear at Center Support

Span ft	97-S kip	97-TRB kip	Standard kip	LRFD kip	AL Legal kip
20x20	44.04	38.61	44.90	57.76	53.45
25x25	47.01	41.71	49.60	59.80	58.77
30x30	49.35	43.64	52.86	64.84	62.15
35x35	55.70	45.17	56.23	70.22	64.43
40x40	60.33	46.84	58.73	74.71	66.08
45x45	65.47	48.28	60.59	78.58	67.30
50x50	69.49	49.41	62.03	82.01	68.24
55x55	72.68	53.45	63.17	85.14	68.99
60x60	75.26	56.78	64.08	88.06	69.59
65x65	77.39	59.54	64.84	90.81	70.09
70x70	79.17	62.94	65.46	93.44	70.51
75x75	80.68	65.84	66.00	95.97	70.86
80x80	81.96	68.33	66.45	98.42	71.17
85x85	83.08	70.49	66.85	100.8	71.43
90x90	84.05	72.39	67.19	103.2	71.94
95x95	84.89	74.05	67.49	105.5	72.73
100x100	85.65	75.52	67.76	107.7	73.44
105x105	86.31	76.83	68.00	110.0	74.06
110x110	86.91	78.01	69.98	112.2	74.62
115x115	87.45	79.06	71.98	114.4	75.12
120x120	87.93	80.01	73.97	116.5	75.58
125x125	88.37	80.88	75.97	118.7	75.99
130x130	88.77	81.66	77.97	120.8	76.36
135x135	89.14	82.38	79.97	123.0	76.71
140x140	89.48	83.04	81.97	125.1	77.02
145x145	89.79	83.64	83.97	127.1	77.31
150x150	90.07	84.20	85.96	129.3	77.58

Table B-3: Continuous Span–Maximum Positive Moment

Span ft	97-S kip-ft	97-TRB kip-ft	Standard kip-ft	LRFD kip-ft	AL Legal kip-ft
20x20	147.7	118.8	157.7	228.2	179.1
25x25	199.8	169.7	206.6	293.7	240.3
30x30	252.1	221.4	255.7	362.4	308.4
35x35	304.6	273.4	305.0	434.1	382.1
40x40	362.1	325.6	358.4	508.9	456.9
45x45	432.5	378.0	429.2	586.7	532.5
50x50	510.3	430.5	500.8	667.6	608.5
55x55	590.2	483.1	572.9	757.4	684.9
60x60	681.9	535.7	645.4	865.3	761.6
65x65	775.0	588.3	718.4	976.4	838.3
70x70	869.3	647.9	791.7	1091	915.2
75x75	964.6	721.1	865.1	1208	992.3
80x80	1061	797.8	938.7	1329	1070
85x85	1157	876.0	1012	1454	1147
90x90	1254	955.3	1086	1581	1224
95x95	1351	1042	1160	1712	1302
100x100	1449	1134	1234	1845	1379
105x105	1547	1226	1308	1982	1456
110x110	1645	1320	1382	2122	1534
115x115	1744	1414	1456	2265	1611
120x120	1842	1508	1530	2411	1689
125x125	1941	1603	1605	2560	1767
130x130	2040	1699	1679	2712	1844
135x135	2139	1795	1753	2868	1922
140x140	2238	1891	1828	3026	1999
145x145	2337	1988	1902	3188	2077
150x150	2437	2084	1976	3353	2155

Table B-4: Continuous Span–Maximum Negative Moment

Span ft	97-S kip-ft	97-TRB kip-ft	Standard kip-ft	LRFD kip-ft	AL Legal kip-ft
20x20	132.0	84.14	122.3	154.0	129.8
25x25	194.4	105.2	155.8	205.5	161.7
30x30	253.1	170.0	192.5	264.2	215.7
35x35	299.6	241.7	228.9	326.5	260.5
40x40	337.1	308.3	264.3	392.5	297.0
45x45	367.6	374.5	294.6	459.7	326.8
50x50	393.2	431.6	320.0	525.6	351.8
55x55	414.6	481.2	341.5	653.1	372.9
60x60	432.9	524.5	391.5	806.1	409.3
65x65	483.7	562.5	450.2	958.9	447.1
70x70	538.3	596.0	512.8	1106	484.6
75x75	591.9	625.7	579.5	1248	522.0
80x80	644.8	652.3	650.2	1385	559.2
85x85	697.0	676.2	724.9	1519	596.4
90x90	748.7	697.8	803.5	1651	636.5
95x95	799.9	717.4	886.2	1780	681.4
100x100	850.6	735.4	972.8	1909	725.8
105x105	901.1	767.8	1063	2036	769.9
110x110	951.1	823.1	1158	2163	813.7
115x115	1001	877.8	1257	2290	857.3
120x120	1050	931.8	1359	2417	900.6
125x125	1100	985.4	1466	2545	943.6
130x130	1149	1038	1577	2674	986.5
135x135	1198	1091	1691	2805	1029
140x140	1247	1143	1810	2936	1072
145x145	1295	1195	1933	3070	1114
150x150	1344	1247	2059	3205	1156

Appendix C: Transverse Deck Parameters

Table C-1: Dead Load by Clear Spacing & Slab Depth

S	D	$\rho_{concrete}$	w_{slab} *	$A_{barrier}$	$P_{barrier}$ **
ft	in	kcf	ksf	in^2	kip/ft
4	7	0.150	0.0875	293	0.305
4.5	7	0.150	0.0875	293	0.305
5	7	0.150	0.0875	293	0.305
5.5	7	0.150	0.0875	293	0.305
6	7	0.150	0.0875	293	0.305
6.5	7	0.150	0.0875	293	0.305
7	7	0.150	0.0875	293	0.305
7.5	7	0.150	0.0875	293	0.305
8	7	0.150	0.0875	293	0.305
8.5	7 ¼	0.150	0.0906	293	0.305
9	7 ¼	0.150	0.0906	293	0.305
9.5	7 ½	0.150	0.0938	293	0.305
10	7 ¾	0.150	0.0969	293	0.305

* distributed load throughout full length of cross-section

** concentrated load positioned 5" from free end of each overhang

Table C-2: ALDOT Bridge Slab Properties

CONCRETE	REINFORCEMENT
Normal Weight	ASTM A615, Gr 60 Billet Steel
f_c' - 4000 psi	f_y - 60 ksi
<u>Clear Cover:</u>	Bar Size - #5
Top & Side - 2"	Dia. bar, d_b - 0.625"
Bottom - 1"	Area steel, A_s - 0.31 in^2