CHAPTER 6

HIGHWAY INFRASTRUCTURE

INTRODUCTION

Highway infrastructure protection historically has been the primary consideration in determining TS&W limits as the weights and dimensions of trucks in particular determine the costs that highway agencies must bear to construct and maintain a highway system to serve present traffic and that anticipated in the near future. This Chapter is intended to acquaint the reader with the technical and practical side of TS&W interaction with the infrastructure elements. Pavement deterioration increases with axle weight, the number of axle loadings, and the spacing within axle groups. The axle loads and spacing on trucks also affects the design and fatigue life of bridges. Truck dimensions influence roadway design -- truck width affects lane widths, trailer or load height affects bridge and other overhead clearances, and length affects intersection and curve design. And conversely, truck designs are determined by existing pavement and bridge strength and roadway geometry.

Pavement types analyzed in this Study include flexible, asphaltic concrete; and rigid, portland cement concrete. Bridge features included in the analysis are span length and type of member support -- simple or continuous. The list of roadway geometry features analyzed includes interchange ramps, intersections, and mainline curves. Alternative truck configurations analyzed, in terms of their interaction with highway infrastructure features, include single-unit or straight trucks and single- and multitrailer truck combinations.

OVERVIEW OF INFRASTRUCTURE IMPACTS

The TS&W characteristics -- axle weights, GVW, truck length, width, and height -- affect pavements, bridges, and roadway geometry in different ways, as shown in Table VI-1.

Table VI-1
Highway Infrastructure Elements Affected by TS&W Limits

Highway Infrastructure Element		Axle Weight	GVW	Axle Spacing	Truck Length	Truck Width	Truck Height
Pavement	Flexible	E		E			
	Rigid	E		e			
Bridge Features	Short-Span	E		E	E		
	Long-Span		E	e	E		
	Clearance					e	E
Roadway Geometric Features	Interchange Ramps		e		E	e	
	Intersections				E	e	
	Climbing Lanes		E				
	Horizontal Curvature		e		e		
	Vertical Curve Length		E				
	Intersection Clearance Time		E		E		
	Passing Sight Distance				e		

Key: E = Significant Effect
e = Some Effect

IMPACT OF WEIGHT

There are two aspects of truck weight that are interdependent and that interact with the highway infrastructure -- axle weight (loading) and GVW. As shown in Table VI-1, the effect of axle weight is more significant to pavements and short-span bridges, whereas GVW is of more significance to long-span bridges.

Generally, highway pavements are stressed by axle and axle group loads directly in contact with the pavement rather than by GVW. The GVW, taking into account the number and types of axles and the spacing between axles, is distributed among the axles and determines axle loads. Over time, the accumulated strains (the pavement deformation from all the axle loads) deteriorate pavement condition, eventually resulting in cracking of both rigid and flexible pavements and permanent deformation or rutting in flexible pavements. If the pavement is not routinely maintained, the axle loads, in combination with environmental effects, will accelerate the cracking and deformation. Proper pavement design relative to loading is a significant factor in pavement life, and varies by highway system and the number of trucks in the traffic stream.

Axle groups, such as tandems or tridems, distribute the load along the pavement, allowing greater weights to be carried and resulting in the same or less pavement distress than that occasioned by a single axle at a lower weight. The spread between two consecutive axles also affects pavement life or performance; the greater the spread, the more each axle in a group acts as a single axle. For example, a spread of 9 to 10 feet results in no apparent interaction of 1-axle with another, and each axle is considered a separate loading for pavement impact analysis or design purposes. Conversely, the closer the axles in a group are, the greater the weight they may carry without increasing pavement deterioration beyond that occasioned by a single axle, dependent on the number of axles in the group. This benefit to pavements of adding axles to a group decreases rapidly beyond 4-axles.

Axle loads also have a beneficial effect on short-span bridges -- that is, bridge spans that are shorter than the truck, thereby resulting in only 1-axle group, front or rear, being on the span at any time. While spreading the axles in an axle group is beneficial to short-span bridges, it is detrimental to pavement. It is not GVW but the distribution of the GVW over axles that impacts pavements.

However, GVW *is* a factor for the life of long-span bridges -- that is, bridge spans longer than the wheelbase of the truck. Bridge bending stress is more sensitive to the spread of axles than to the number of axles. The FBF takes into account both the number of axles and axle spreads in determining allowable GVW.

In the context of roadway geometrics, increasing GVW affects a truck's ability to accelerate from a stop, to enter a freeway, or to maintain speed on a long grade. Acceleration from a stop influences the time required to clear an intersection. Acceleration into a freeway affects the determination of acceleration lane length requirements. Inability to maintain speed on a long grade requires the construction of truck climbing lanes. Some of these effects can be ameliorated by changes in truck design, primarily to engine and drive train components. The GVW also has a second order effect on offtracking -- that is, on how the rear axle of a trailer tracks relative to the steering axle of the truck. Other truck characteristics affected by roadway geometrics are discussed in more detail later in this Chapter.

IMPACT OF DIMENSIONS

The dimensions of trucks and truck combinations have various effects on the three elements of highway infrastructure. The most significant effects relate to *length*, particularly when combined with GVW. *Width* has a limited effect on swept path -- the combination of offtracking and vehicle width. Swept path affects highway geometrics in terms of interchange ramp or roadway intersection design which is based on mapping a maximum swept path that the truck encroaches on the shoulder, over the curb, or into another lane of traffic. *Height* regulations are intended to ensure that trucks will clear overhead bridges, bridge members, overhead wires, traffic signals, and other obstructions.

In general, truck length -- or more specifically wheelbase -- has a strong effect on bridge stress for long-span bridges. The longer the wheelbase the shorter the distance from the support member to where the load is being applied (the moment arm) when the truck is in the middle of the span. The shorter the truck the greater the concentration of load at the middle of the span, and the longer the distance (moment arm) to the support member for the bridge span member. A truck at mid-span is the loading condition for the maximum stress in a simple supported span. This is not the case for some continuous supported spans: when a truck is straddling the center pier of a continuous span, increasing the truck length can increase the stress in the span at the pier.

The effect of truck wheelbase on offtracking is reduced considerably if the combination is articulated, especially in a multitrailer combination. Low-speed offtracking affects interchange and intersection design, and high-speed offtracking affects lane width.

BRIDGES

Bridges are critical to the safe and efficient movement of people and freight on the Nation's highways. This section discusses the important considerations that have influenced the decision making and investments of Federal and State transportation officials for bridges.

BRIDGE DESIGN[1]

Most highway bridges in the United States were designed according to the design guidelines of the AASHTO. These guidelines provide traffic-related loadings to be used in the development and testing of bridge designs, as well as other detailed requirements for bridge design and construction.

Dynamic effects (vibration resulting in bridge loads that vary above and below that load resulting trucks operating at higher speeds. In bridge design, design loadings (in the static condition) are adjusted upward to account for dynamic effects. To minimize the dynamic effects of extra-heavy nondivisible loads on some bridges, permits often require the truck to cross at a very slow speed, depending on its GVW.

A key task in bridge design is to select bridge members that are sufficiently sized to support the various loading combinations the structure may carry during its service life. These include dead load (the weight of the bridge itself); live load (the weights of vehicles using the bridge); and wind, seismic, and thermal forces. The relative importance of these loads is directly related to the type of materials used in construction, anticipated traffic, climate, and environmental conditions. For a short-span bridge (for example, span length of 40 feet), about 70 percent of the load-bearing capacity of the main structural members may be required to support the traffic-related live load, with the remaining 30 percent of capacity supporting the weight of the bridge itself. For a long bridge (for example, span length of 1,000 feet), as much as 75 percent

[1] A substantial amount of the background material is drawn from the TRB Special Report 225, *Truck Weight Limits: Issues and Options*, 1990 and from the 1981 U.S. DOT Report to Congress under Section 161, *An Investigation of Truck Size and Weight Limits.*

of the load-bearing capacity of the main structural members may be required to support the weight of the bridge.

In most instances, the loading event that governs bridge capacity is a design vehicle placed at the critical location on the bridge. In certain cases, a lane loading simulating the presence of multiple trucks on a bridge is the governing factor. Bridges are also affected by the dynamic impact and lateral distribution of weight of trucks; dynamic impact is determined by speed and roadway roughness, and the lateral distribution of loads varies with the position of the truck(s) on the bridge.

The methods used to calculate stresses in bridges caused by a given loading are necessarily conservative; therefore, the actual measured stresses are generally much less than calculated stresses. Providing for a margin of safety is necessary to bridge design because:

- The materials used in construction are not always completely consistent in size, shape, and quality;
- The effects of weather and the environment are not always predictable;
- Highway users on occasion violate vehicle weight laws;
- Legally allowed loads may increase during the design life of a structure; and
- Overweight loading is occasionally allowed by permit.

The adjustment of the nominal legal loading is reflected in the safety factors, which are selected so that there is only a very small probability that a loading condition that exceeds load capacity will be reached within the bridge's design life.

The margins of safety used by bridge designers in the past have been reduced in recent bridge design procedures. Use of new design procedures and computer-aided engineering and design has enabled more precise analysis of load effects and the selection of smaller bridge members. Also, the competition between the steel and concrete industries has led each group to foster lower costs for their own material. For example, many designs now proposed for steel bridges reduce the safety factor by reducing the number of girders, which increases their spacing.

Design and construction of highway bridges in the United States has been governed by the AASHTO's *Standard Specifications for Highway Bridges* since 1931, with subsequent revisions. In the early 1990s AASHTO decided to develop an entirely new bridge code to incorporate state-of-the-art bridge engineering that is based on the load and resistance factor design (LRFD) approach.[2] In 1993, AASHTO adopted LRFD bridge design specifications on a trial basis, as an

[2] FHWA http://www.ota.fhwa.dot.gov/tech/struct/dp99lr.html, February 19, 1998.

alternative to standard bridge design specifications. In 1996, interim LRFD specifications were made available by AASHTO and conversion to this method was encouraged wherever practical.[3]

The LRFD method applies statistically determined factors to bridge design parameters, using a series of load and resistance factors to account for variabilities in loads and material resistance. The specifications use statistical methods and probability theory to define the variations in loading and material properties and the likelihood that various load combinations will occur simultaneously.[4]

BRIDGE IMPACT

Past studies of the impact of truck weight limit changes on bridges were based on various percentages of the yield stress for steel girder bridges, such as 55 percent or 75 percent. The yield stress, a property of the particular type of steel, is the stress at the upper limit of the elastic range for bridge strain. The elastic range of a structural member is the set of stresses over which the deformation -- the strain of the member -- is not permanent. In the elastic range, the member returns to its former size and shape when the stress is removed. There is no permanent set in the structural member. For this discussion, strain is the elongation of a steel girder when (1) a portion of the strain becomes permanent at a stress level above the yield stress; and (2) the girder continues to elongate, or stretch, under increasing load until it ruptures or fails. Beyond the elastic range, there is permanent elongation of the bridge girder, that is, for those stresses that are greater than the yield stress. However, in structural steel there is considerable strain before failure occurs.

BRIDGE INVENTORY AND OPERATING RATINGS

States rate bridges, at their discretion, at either an inventory rating (55 percent of the yield stress) or operating rating (75 percent of the yield stress).[5] Bridges are never intentionally loaded to yield stress in order to provide an adequate margin of safety. The design stress level for bridges is the same as the inventory rating, 55 percent of the yield stress. These two ratings are also used for posting bridges; either may be used under AASHTO guidelines, at the option of the State. A sign specifying weight limits is posted on bridges when it is determined that a vehicle above the specified weight would overstress the bridge. This weight could be that which stresses the bridge at either the 55 percent or 75 percent level of the yield stress.

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3 AASHTO http://www2.epix.net/~lrfd/develop.html, February 19, 1998.

4 Ibid.

5 According to the AASHTO *Manual for Maintenance Inspection of Highway Bridges* (1983) an operating rating is defined as RF = 0.75-D/L(1+I) where RF= rating factor arrived at with the equation 0.55R= D + L (1 + I) where R= the limiting stress (often the stress at which steel will undergo permanent deformation, or "yield"), D= stress due to dead load (the effect of gravity on bridge components), L= stress due to live load (vehicles on the bridge), I= an adjustment to the static effect of live loads to account for dynamic effects. An inventory bridge rating is arrived at by selecting the most highly stressed bridge component and inserting the rating factor (RF) into the Equation, RF= 0.55R- D/L(1 + I), as a multiplier on the live load of the rating truck.

As States have the option to use either level for posting purposes, both ratings have been used in past studies to assess the bridge impacts for evaluating TS&W policy scenarios. Significant cost differences result from choice of rating. Use of the lower stress level (inventory rating) results in more bridges being identified as needing to be upgraded to accommodate increased weights or decreased lengths.[6]

Following the reviews of the TRB Special Reports 225 and 227 the FHWA determined that the stress level most representative of all State bridge posting practices was the inventory rating (55 percent of the yield stress) plus 25 percent, which gives a level of 68.8 percent of yield stress. The FHWA used this 68.8 percent of yield to estimate the bridge cost impacts of LCVs. The resulting cost estimate reported by the FHWA in May 1991 was much closer to that based on the 75 percent rating, the TRB findings.

BRIDGE STRESS

Bridge stresses caused by vehicles depend on both GVW and the distances between the axles that act as point loads. Trucks having equal weight but different wheelbases produce different bridge stresses. The shorter the wheelbase, the greater the stress. On a simple-span bridge, the length of a truck relative to the length of bridge span is also important. For relatively short spans (20 feet to 40 feet), all axles of a truck combination will not be on the bridge at the same time. The maximum bending moments determine stresses in the main load-carrying members of simple span bridges.

Figure VI-1 shows the maximum bending moments, by span lengths between 40 and 160 feet, for two trucks: a 50,000-pound single unit truck with a wheelbase of 19 feet, and an 80,000-pound combination with a wheelbase of 54 feet. For shorter bridges, the 50,000-pound single unit truck produces slightly higher stresses than the 80,000-pound combination; however, for longer bridges, the combination produces higher stresses.

TS&W REGULATION RELATED TO BRIDGE PROTECTION

The TS&W regulation to protect bridges generally takes the form of a bridge formula or table. Federal bridge protection regulation, which became effective in 1975, uses a formula. Some States still use bridge tables, which were grandfathered by the 1975 Federal law. Other States use bridge tables for issuing overweight permits. The FBF is based on overstress criteria, the amount of bridge stress above the design stress to be allowed.

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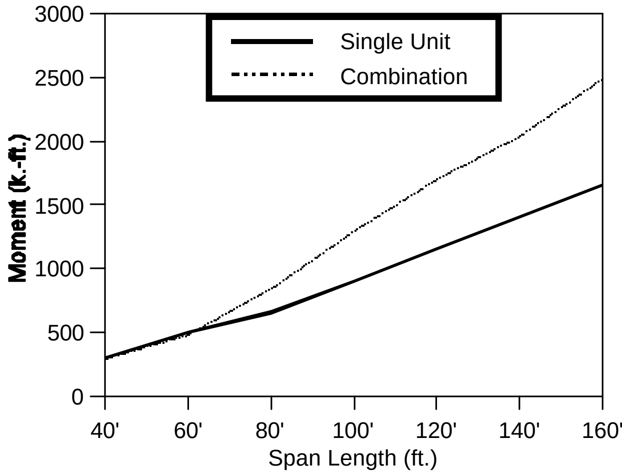
[6] The TRB *Special Reports 225, Truck Weight Limits: Issues and Options* and *227, New Trucks for Greater Productivity and Less Road Wear: an Evaluation of the Turner Proposal* estimated the bridge costs of the TS&W changes under study based on the operating rating of 75 percent of yield stress, whereas reviewers of those reports found much higher bridge costs resulting from the use of the inventory rating of 55 percent of yield stress.

Figure VI-1
Maximum Bending Moments on a Simple Span Bridge:
50,000-pound Single Unit Truck vs. 80,000-pound Truck Combination



OVERSTRESS CRITERIA AND LEVEL OF RISK

The level of risk to accept in determining acceptable loadings for a given bridge, or acceptable bridge design requirements for given loadings, is an element of TS&W regulation. A less conservative bridge formula, one that did not preserve the underlying FBF criteria, would reduce the margin of safety, thereby increasing somewhat the likelihood of bridge damage due to overstress. An overstress sufficient to damage a bridge would necessitate bridge repair and/or replacement sooner than anticipated.

BRIDGE FATIGUE

Another factor to be considered is fatigue life, which is related to repetitive loadings. Each truck crossing produces one or more stress cycles in bridge components, which use up a portion of the components' fatigue lives. The magnitude of stress depends on vehicle weight and the size of the bridge component. The occurrence of a fatigue failure is signaled by cracks developing at points of high stress concentration.

Generally, only steel bridges are susceptible to fatigue, although some studies suggest that commonly used prestressed concrete spans, if overloaded, are similarly susceptible. The governing damage law for steel components has a third-power relationship between stress and damage, so that a doubling of stress causes an eight-fold increase in damage.[7]

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[7] Fisher, 1977.

Bridge details that are particularly susceptible to fatigue include weld connections in tension zones, pin and hanger assemblies, and cover plates on the bottom flanges of steel beams.[8] Many fatigue failures result from stresses induced indirectly by the distortion of the structure due to poor design details or unforeseen restraints. Most steel cracks reported to date probably fall into the category of distortion induced. Some of the worst detailing can be corrected by repair and retrofit.

FEDERAL BRIDGE FORMULA

In 1975 along with axle and maximum GVW limits for Interstate highways, Federal law adopted a bridge formula that restricts the maximum weight allowed on any group of consecutive axles based on the number of axles in the group and the distance from the first to the last axle. The AASHO proposed the formula concept in the 1940s. It was further developed and presented in a 1964 Report to Congress from the Secretary of Commerce.[9] That Study recommended a table of maximum weights for axle groups to protect bridges (see Appendix A). The values in the table are derived from the following formula, that is, FBF:

$$W = 500 [L N / (N - 1) + 12 N + 36]$$

where:

W = maximum weight in pounds carried on any group of two or more consecutive axles

L = distance in feet between the extremes of the axle group

N = number of axles in the axle group

Current Federal law specifies exceptions to the results given by the above formula: 68,000 pounds may be carried on two sets of tandem axles spaced at least 36 feet apart, and a single set of tandem axles spread no more than 8 feet is limited to 34,000 pounds.

The FBF is based on assumptions about the amount by which the design loading can be exceeded for different bridge designs. Specifically, this formula was designed to avoid overstressing HS-20 bridges by more than 5 percent and H-15 bridges by more than 30 percent. The FHWA established a bridge stress level of not more than 5 percent over the design stress for HS-20 bridges to preserve the significantly large investment in these bridges by Federal, State, and local governments, and because these bridges carry high volumes of truck traffic.

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8 AASHTO specifications give different allowable fatigue stresses for different categories of detail. These fatigue rules were initiated in the mid-1970s, therefore many older bridges were never checked during their original design for fatigue life. Further, the AASHTO fatigue rules apply to welded and bolted details with stresses induced directly by load passages (Moses, 1989).

9 *Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid System,* 1964 Study Report to Congress, U.S. Department of Commerce.

Although a level of up to 30 percent is considered a safe level for overstressing an H-15 bridge in good condition, the fatigue lives of these structures may be shortened by repeated loadings at this level.

The FBF reflects the fact that increasing the spacing between axles generally results in less concentrated loadings and lower stresses in bridge members. For example, the bridge formula would allow a 3-axle single-unit truck with a wheelbase of 20 feet to operate at 51,000 pounds. If the wheelbase of this truck is increased to 24 feet, the maximum weight allowed under FBF would increase to 54,000 pounds as shown in Table VI-2.

Table VI-2
FBF 3-axle, 4-axle, And 5-axle Single-unit Truck Limit

Distance[10] (Feet)	GVW (Pounds)		
	3-Axles	4-Axles	5-Axles
20	51,000	55,500	60,500
24	54,000	58,000	63,000
28	57,000	60,500	65,500
32	60,000	63,500	68,000
36		66,000	70,500
40		68,500	73,000

As noted, there is a greater gain in allowable load by adding an axle than by increasing the distance between axles. For instance, at 30 feet a 3-axle vehicle is allowed a maximum GVW of 58,500 pounds and by adding 2 feet can gain only 1,500 pounds. If the same 3-axle vehicle at 30 feet adds an axle there is a gain of 3,500 pounds -- or 2,000 pounds more than by increasing distance by 2 feet. Increasing the number of axles in an axle group without increasing the overall length of the group has very little effect in reducing bridge stress. However, more axles do provide substantial benefits to pavements.

POTENTIAL ALTERNATIVES TO FBF

Actually, the FBF is not just one formula but a series of formulas with the appropriate one chosen by a parameter, N, the number of axles in the group in question. However, bridge stress is affected more by the total amount of load than by the number of axles. Thus the FBF is not effective in modeling the actual physical phenomenon, and it results in loads, especially for long combinations, that overstress bridges more than intended. More importantly, it encourages the addition of axles to obtain more payload even though one or both bridge stress criteria are exceeded. At other times, the equation restricts allowable loads for some short trucks below that

[10] Between the outside axles of any group of 2 or more axles.

allowed by the stress criteria themselves. In summary, the FBF actually results in overstressing some of the bridges it is intended to protect.

Since 1975, there have been a number of proposals to revise the FBF and reduce its shortcomings. However, significant areas of concern have been identified with respect to the alternatives as well. Three alternative formulas proposed in recent years are discussed here: a TRB (a combination of the Texas Transportation Institute (TTI) and FBFs) alternative, an AASHTO alternative, and a Goshen alternative.

TRB ALTERNATIVE

In 1990, the TRB recommended adoption of the formula developed by the TTI which would allow a 5 percent overstress for HS-20 bridges, in conjunction with existing Federal axle limits for vehicles with GVWs of 80,000 pounds or less.[11] The TRB Report further recommended the FBF continue to be applied to vehicles weighing more than 80,000 pounds. The effect of this proposal would be an increase in maximum weights allowed for shorter vehicles, while the maximum weight limits for the longer wheelbase trucks would remain unchanged. It was asserted that the TTI formula was overly conservative at heavier weights.

The TTI formula is in the form of two equations for straight lines that meet at a wheelbase length of 56 feet. For wheelbases less than 56 feet, it is:

$$W = 1{,}000\ (L + 34)$$

For wheelbases equal to or greater than 56 feet, it is:

$$W = 1{,}000\ (L/2 + 62)$$

where:

W = allowable weight

L = wheelbase for truck configuration

[11] TRB Special Report 225.

AASHTO ALTERNATIVE

In 1993, AASHTO issued a report which recommended that its member committees (1) evaluate nationwide adoption of the TTI bridge formula as a replacement for FBF; (2) consider a limit on maximum extreme axle spacing of 73 feet in the short term; (3) retain existing single- and tandem-axle limits; (4) control tridem-axle weights -- and the special permitting of vehicles with GVWs more than 80,000 pounds -- using the original TTI bridge formula which protects both H-15 and HS-20 bridges, as opposed to the TTI formula mentioned above, which protects only HS-20 bridges. The recommendation was reviewed by the AASHTO Highway Subcommittees on Bridges and Structures and Highway Transport, accepted in resolution form, and approved by the Standing Committee on Highways. The AASHTO Board of Directors considered the recommendations at its 1996 Fall Meeting. The board expressed concern that the impact on pavements was not adequately addressed and remanded it for further consideration to the Subcommittees on Design and on Bridges and Structures.

GHOSN ALTERNATIVE

In 1995 a research study by Ghosn and others for FHWA, proposed a new formula based on structural reliability theory as a replacement for the FBF.[12] Structural reliability theory more explicitly accounts for the uncertainties associated with bridge design and load evaluation. The proposed formula, however, is considerably more permissive than the FBF when applied to long vehicles. It results in bridge stresses well above the criteria selected for this Study. Therefore, it was not considered.

ALLOWABLE WEIGHTS BASED ON FBF STRESS CRITERIA

Original research conducted for this Study suggests that a series of look-up tables may be developed based on the underlying the FBF stress criteria -- that is, a maximum overstress of 5 percent for HS-20 bridges, and 30 percent for H-15 bridges. These stresses were computed for both simple and continuous spans for the most critical span lengths for truck configurations. The following discussion illustrates how this approach might be applied to three vehicles: (1) a tractor-semitrailer combination vehicle with a 3-axle tractor and 2-axle semitrailer, (2) a tractor-semitrailer combination vehicle with a 3-axle tractor and a semitrailer with a tridem-axle group, and (3) a RMD. The GVWs for each configuration with varying semitrailer lengths were calculated based on axle spacing.

Table VI-3 presents the weight values for the first vehicle combination under the FBF, TTI, and FBF stress criteria; and Figure VI-2 graphically displays maximum GVW from the Table, for semitrailers of varying lengths.

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[12] *Bridge Overstress Criteria*, Michael Ghosn, Charles G. Schilling, Fred Moses, and Gary Runco, Report by the City College of the City University of New York for the FHWA (Washington, D.C., FHWA, 1995).

Table VI-3
Maximum GVW For 5-axle Semitrailer Combination Applying Federal And TTI Bridge Formulas And FBF Stress Criteria

Semitrailer Length (Feet)	Maximum GVW (1,000 Pounds)			Semitrailer Length (Feet)	Maximum GVW (1,000 Pounds)		
	FBF	TTI	FBF Stress Criteria		FBF	TTI	FBF Stress Criteria
28.0	70.0	70.1	78.4	**45.0**	80.0	80.0	80.0
35.0	74.5	77.1	80.0	**48.0**	80.0	80.0	80.0
40.0	78.0	80.0	80.0	**53.0**	80.0	80.0	80.0

NOTE: GVWs specific to 22.5-foot tractor wheelbase, 52-inch tractor tandem spread, and trailer 48-inch tandem spread. The distance from the first drive axle (on the tractor to the last trailer axle is the trailer length minus 6 feet.

Figure VI-2
Maximum GVW For 5-axle Semitrailer Combination Applying Federal and TTI Bridge Formulas And FBF Stress Criteria

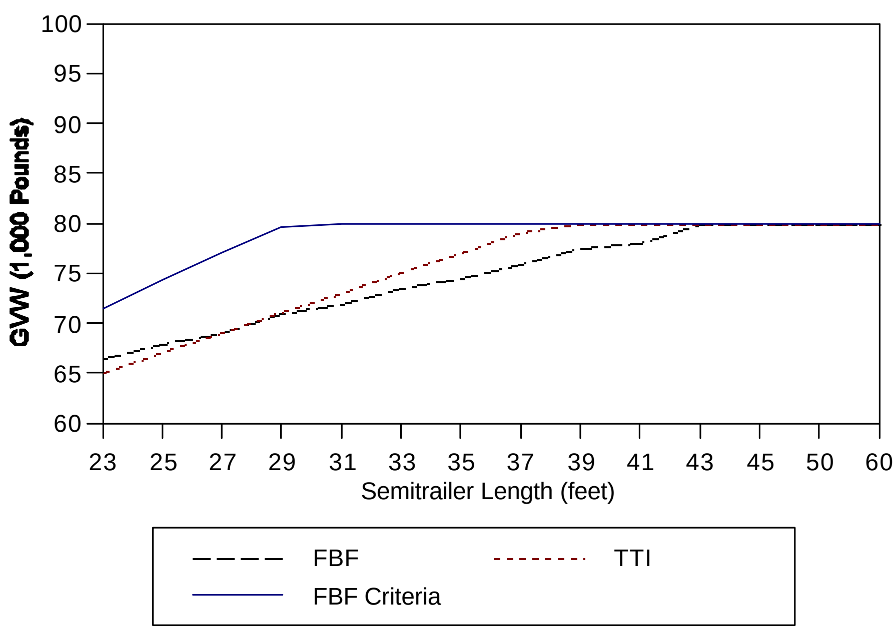

Table VI-4 presents weight values and maximum GVWs for the 6-axle semitrailer combination with the semitrailer supported at the rear by a tridem-axle group. In this case, both the tractor wheelbase and semitrailer length are varied (common descriptive dimensions). The allowable GVW for varying semitrailer lengths is shown in Figure VI-3.

Table VI-4
Maximum GVW For 6-axle Semitrailer Combination Applying Federal And TTI Bridge Formulas And FBF Stress Criteria
Tractor Wheelbase = 22.5 Feet

Semitrailer Length (Feet)	Maximum GVW (1,000 Pounds)			Semitrailer Length (Feet)	Maximum GVW (1,000 Pounds)		
	FBF	TTI	FBF Stress Criteria		FBF	TTI	FBF Stress Criteria
28.0	75.0	70.1	73.4	**45.0**	85.5	87.1	88.6
35.0	79.5	77.1	84.5	**48.0**	87.5	90.1	90.0
40.0	82.5	82.1	88.7	**53.0**	90.5	92.0	94.2

Figure VI-3
Maximum GVW For 6-axle Semitrailer Combination
Applying Federal And TTI Bridge Formulas And FBF Stress Criteria
Tractor Wheelbase = 22.5 Feet

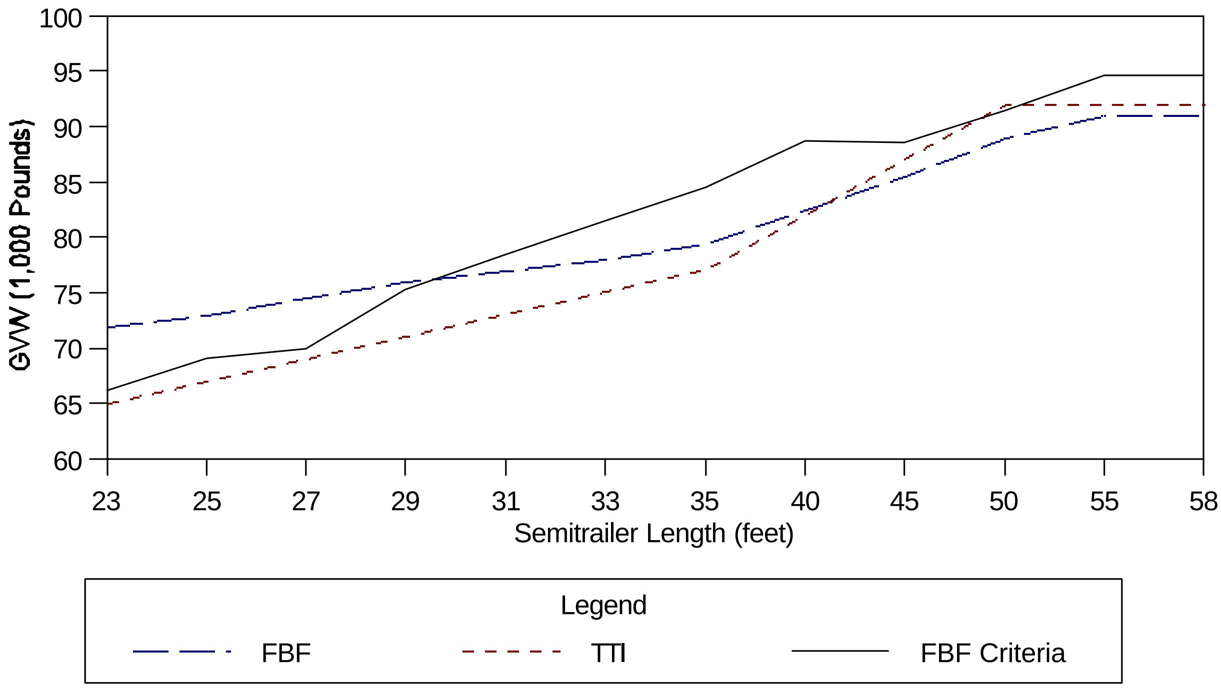

Table VI-5 presents the values and maximum GVWs for the RMD combination, a tractor-semitrailer combination with a 3-axle tractor pulling a 2-axle semitrailer and a 2-axle full trailer. The tractor and semitrailer length of this double are varied, with the trailer remaining constant at 28 feet. The limiting axle loads and maximum GVW for the entire vehicle are easily read from a table. This approach negates the need to compute the many axle group combinations inherent in the use of the existing and proposed formulas (which can amount to as many as 36 different combinations in the case of a 9-axle vehicle). The GVW for varying semitrailer lengths is shown in Table VI-5.

Table VI-5
Maximum GVW for RMD with Semitrailer of Variable Length And 28' Trailer Applying Federal and TTI Bridge Formulas And FBF Stress Criteria
Tractor a = 18.2 Feet, Tractor B = 22.5 Feet

Semitrailer Length (Feet)	GVW (1,000 Pounds)					
	FBF		TTI		FBF Stress Criteria	
	Tractor A	Tractor B	Tractor A	Tractor B	Tractor A	Tractor B
45	109.5	109.5	105.16	107.3	111.4	112
48	111	111	106.6	108.8	112.8	113.4
53	111	111	109.1	111.3	115.2	116

In summary, there is significant variation in the results derived from the three formulaic approaches by vehicle configuration. In general, the TTI formula is better matched than the FBF for bridges, and there is a significant amount of load capacity available before limits are exceeded for the 5- and 6-axle semitrailer and 7-axle RMD configurations. This is not the case, however, for larger vehicles such as the 9-axle turnpike doubles -- FBF allows too much weight for these in terms of the stress criteria. The TTI curve for that vehicle is on the low side of the FBF stress criteria curve. Also, FBF is conservative for multiaxle short straight trucks.

There are benefits to adhering to the criteria on which the FBF is based and incorporating the consideration of continuous beams into the control. Tools such as user-friendly computer software programs can be designed to assess allowable loading configurations for any vehicle, and standard (bridge formula) tables for the more common vehicles can be generated. The use of the FBF stress criteria described in this section addresses the documented drawbacks of FBF and provides a basis for truck weight control that conforms to the criteria upon which both FBF and TTI are based -- but to which they do not always adhere.

It should be noted that the FBF, by design, incorporates a degree of control for pavement damage by explicitly including the number of axles in the formula. The TTI formula and FBF stress criteria indirectly control for pavement damage by adhering to axle weight limits -- the higher GVW limits, such as for LCVs, require more axles to avoid exceeding axle limits.

PAVEMENTS

The condition and performance of highway pavements depend on many factors, including the thickness of the various pavement layers, quality of construction materials and practices, maintenance, properties of the roadbed soil, environmental conditions (most importantly rainfall and temperature), and the number and weights of axle loads to which the pavements are subjected.[13]

WEIGHT

While pavement engineers traditionally have used ESAL factors estimated from the AASHO Road Test (started in 1956 and completed in 1962) as the basis for designing pavements, there is increasing recognition that better relationships between axle load and pavement deterioration are needed. Pavement distress models used in both the 1982 and 1997 Federal HCA Studies (HCAS) abandoned the use of ESALs to relate axle loading to pavement deterioration, and AASHTO will be replacing its ESAL-based pavement design formula with one that more directly relates axle loads to factors that determine pavement life. While ESALs were not used as the basis for estimating pavement costs for this Study, they are widely understood by highway administrators, pavement engineers, and others concerned with the pavement impacts of TS&W scenarios. Therefore, they are used here as a benchmark for comparing relative pavement impacts of various truck configurations with different numbers and types of axles.

Pavement deterioration increases sharply with increases in axle load. On both flexible and rigid pavements, the load equivalence factor for a 20,000-pound single axle is about 1.5. Thus, 100 passes across a pavement by a 20,000-pound axle would have the same effect on pavement life as 150 passes by an 18,000-pound axle.

The number of axles is also important in estimating pavement impact, other things being equal, as a vehicle with more axles has less effect on pavements. For example, a 9-axle combination vehicle carrying 80,000 pounds has less effect on pavements than a 5-axle combination vehicle carrying 80,000 pounds. A significant amount of additional weight can be carried by the 9-axle vehicle without causing greater pavement consumption relative to the 5-axle vehicle. Comparing vehicles in terms of ESALs provides information on load-related pavement impact, but it does not include an offsetting benefit gained by a reduction in the number of trips required to transport

[13] TRB *Special Report 225, Truck Weight Limits: Issues and Options,* 1990.

the same amount of freight. Vehicles are often compared in terms of ESALs per unit of freight carried as a means of including the reduction in pavement deterioration from fewer trips.

The increase in pavement costs per added ESAL mile can vary by several orders of magnitude depending upon pavement thickness, quality of construction, and season of the year. Thinner pavements are much more vulnerable to traffic loadings than thicker pavements.[14] Additionally, pavements are much more vulnerable to traffic loadings during spring thaw in areas subject to freeze-thaw cycles.

AXLE SPACING

The primary load effect of axle spacing on flexible pavement performance is fatigue. Axle spacing is a major concern for fatigue. When widely separated loads are brought closer together, the stresses they impart to the pavement structure begin to overlap, and they cease to act as separate entities. While the maximum deflection of the pavement surface continues to increase as axle spacing is reduced, maximum tensile stress at the underside of the surface layer (considered to be a primary cause of fatigue cracking) can actually decrease as axle spacing is reduced. However, effects of the overlapping stress contours also include increasing the duration of the loading period. Thus, the beneficial effects of stress reduction are offset to an unknown degree by an increase in the time or duration of loading. The net effect of changes in axle spacing on pavement deterioration is complex and highly dependent on the nature of the pavement structure.[15]

TIRE CHARACTERISTICS

In recent years, several studies on the impact of tire characteristics on pavement have raised concern over the possibility of accelerated pavement deterioration, particularly rutting, caused by increasing tire pressures. The tires of the AASHO Road Test trucks of the 1950s were bias-ply construction with inflation pressures between 75 pounds and 80 pounds per square inch (psi). The replacement of bias-ply tires with radial tires and higher inflation pressures, averaging 100 psi, result in a smaller size tire "footprint" on the pavement and, consequently, a concentration of weight over a smaller area.[16] These changes hasten the wear of flexible pavements, increasing both the rate of rutting and the rate of cracking.

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[14] Results of a study by Hutchinson and Haas compare the average and marginal costs per ESAL on highways with 500,000 ESALs per year and 2 million ESALs per year. The cost per ESAL for highways with 500,00 ESALs is almost four times as great as the cost per ESAL on highways designed for 2 million ESALs. One important implication of this finding is that a policy that encourages heavy trucks to shift from highways with thicker pavements, such as the Interstate or NHS, to highways with thinner pavement can have a significant impact on pavement costs.

[15] TRB *Special Report 225*.

[16] A study by Bartholomew (1989) summarized surveys of tire pressure conducted in seven States between 1984 and 1986 and found that 70 to 80 percent of the truck tires used were radials and that average tire pressures were about 100 psi.

The AASHTO load equivalency factors apply only to axles supported at each end by *dual* tires. Recent increases in steering axle loadings and more extensive use of single tires on load-bearing axles have precipitated efforts to examine the effect on pavement deterioration of substituting single for dual tires. Both standard and wide-based tires have been considered. Past investigations of the pavement deterioration effects of single versus dual tires have found that single tires induce more pavement deterioration than dual, but that the differential wear effect diminishes with increases in pavement stiffness, in the width of the single tire, and in tire load.[17]

A general finding from the studies is that wide-base single tires appear to cause about 1.5 times more rutting than dual tires on flexible pavements (the most common type of pavement) as they do not have good rut resistance. Another finding is that one of the wheels in a dual tire assembly is frequently overloaded due to variability in the roadway cross-section and that the average overload causes an increase in rutting similar to that caused by wide-based single and dual tire assemblies.

Based upon past studies, single tires have more adverse effects on pavements than dual tires,[18] it appears likely, however, that past investigations have overstated the adverse effects of single tires by neglecting two potentially important effects: (1) unbalanced loads between the two tires of a dual set, and (2) the effect of randomness in the lateral placement of the truck on the highway. Unbalanced loads between the tires of a dual set can occur as a result of unequal tire pressures, uneven tire wear, and pavement crown. As with unequal loads on axles within a multiaxle group, pavement deterioration increases as the loads on the two dual tires become more unbalanced.

The second neglected factor, sometimes termed "wander," is the effect of randomness in the lateral placement of trucks within and sometimes beyond lane boundaries. Less than perfect tracking is beneficial to pavement deterioration, as the fatiguing effect is diminished because the repetitive traffic loads are distributed over wider areas of the pavement surface. The greater overall width of dual tires naturally subjects a greater width of pavement to destructive stresses, therefore, wander is expected to have a smaller beneficial effect for dual than for single tires. Once rutting

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[17] Gillespie (1993) found that a steering axle carrying 12,000 pounds with conventional single tires is more damaging to flexible pavements than a 20,000-pound axle with conventional dual tires. Gillespie proposed that road damage from an 80,000-pound vehicle combination would be decreased by approximately 10 percent if a mandated load distribution of 10,000 pounds on the steering axle and 35,000 pounds on tandems. Since the operating weight distribution of a 5-axle tractor-semitrailer at 80,000 pounds GVW generally has less than 11,000 pounds on the steering axle, the practical effect of the proposal would be to increase tandem axle weights without a compensating decrease in steering axle weights.

[18] Bauer (1994) summarized several recent studies on the effects of single versus dual tires: "Smith (1989), in a synthesis of several studies . . . evaluated at 1.5 on average the relationship of the damage caused by wide base single assemblies and that caused by traditional dual tire assemblies with identical loading at the axle. Sebaaly and Tabataee (1992) found rutting damage ratios between wide base and dual tire assemblies varying between 1.4 and 1.6 . . . Bonaquist (1992), reporting on results obtained from a study . . . on two types of roadway, using a dual tire assembly with 11 R 22.5 and a wide base with 425/65 R 22.5, indicates rutting damage ratios varying from 1.1 to 1.5, depending on the layers of the roadway."

begins, however, tires -- especially radial tires -- tend to remain in the rut, thereby greatly reducing the beneficial effects of wander for both single and dual tires.[19]

Another consideration in evaluating wide-base single versus dual tires is dynamic loadings that arise from the vertical movement of the truck caused by surface roughness. Thus, peak loads are applied to the pavement that are greater than the average static load.[20] Signs of pavement damage from dynamic loadings are typically localized, at least initially. Because of the localized nature of the dynamic loading, its severity is much greater than previously thought.[21] A further note on wide-base single tires is that those having only two sidewalls are much more flexible than a pair of dual tires with four sidewalls. This means the tire absorbs more of the dynamic bouncing of the truck, and less of the dynamic load is transmitted to the pavement.

SUSPENSION SYSTEMS

The subject of road-friendly suspensions -- within the context of the broader subject of vehicle-pavement interaction -- was researched as an Organization for Economic Cooperation and Development (OECD) Project -- the Dynamic Interaction between Vehicles and Infrastructure Experiment (DIVINE) Project -- involving the United States and 16 other countries.[22] The work focused on (1) how well different suspension systems distribute load among axles in a group (the more evenly, the better); (2) how well different suspension systems dampen vertical dynamic

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[19] The TRB *Special Report 225* examined the importance of loading imbalance and wander. The TRB Study examined two types of pavement deterioration: surface cracking due to fatigue and permanent deformation or rutting in the wheel tracks. Fatigue was found to be more sensitive to the differences between single and dual tires than rutting. Both balanced and unbalanced dual-tire loads were considered in analyzing the affect on wander. The analysis indicated that the adverse effects of single tires on pavement deterioration were reduced when wander was taken into account, although the effects were still significant.

[20] From research summarized by the Midwest Research Institute (MRI) that suggests dynamic loadings are a consideration in assessing the relative merits of wide base single versus dual tires. Gyenes and Mitchell report that the magnitude of the added dynamic components was earlier thought to increase road damage over that of the static loading alone between 13 and 38 percent, according to research reported by Eisenmann. The MRI research noted that many recent studies have pointed out the fallacy in the earlier work, which assumed that the dynamic component of loading was distributed uniformly over the pavement in the direction of travel. The research found, however that the dynamic component is very localized, arising out of pavement surface irregularities and therefore is spatially correlated with these irregularities.

[21] Gillespie, et. al. estimate that damage due to the combination of static and dynamic loading can be two to four times that due to static loading locally. Von Becker estimates the combined loading produces a “shock factor” between 1.3 and 1.55, depending upon suspension characteristics. Applying the fourth power law would translate these figures into relative damage estimates ranging from 2.8 to 4.8 times the static loading damage. Gyenes and Mitchell suggest impact factors in the range of 1.3 to 1.5 for relative damage estimates of 2.8 to 5.1.

[22] TRB *Special Report 225* noted that a heavy truck travels along the highway, axle loads applied to the pavement surface fluctuate above and below their average values. The degree of fluctuation depends on factors such as pavement roughness, speed, radial stiffness of the tires, mechanical properties of the suspension system, and overall configuration of the vehicle. On the assumption that the pavement deterioration effects of dynamic loads are similar to those of static loads and follow a fourth-power relationship, increases in the degrees of fluctuation increase pavement deterioration.

loads (the more, the better); and (3) spatial repeatability of dynamic loads. The research also examines how road and bridge characteristics act to excite a truck, and in turn influence the loads received by the road and bridge.

The findings of the DIVINE research primarily relate to the physical interaction between heavy vehicles and the highway infrastructure -- pavements and bridges. The research breaks new ground, providing scientific evidence of the effects of heavy vehicles. Conclusions that relate to vehicle and pavement interaction are summarized from the final report.

Pavement wear -- the gradual loss of functional condition -- is expressed in permanent deformations to the longitudinal profile of the pavement surface. Whereas, pavement damage results from an accumulation of rutting and cracking distress from repeated applications of vehicle loads. "Road research . . . has historically tended to over-emphasize pavement damage, and the true importance and nature of pavement wear has not yet been recognized."[23] The DIVINE research focused primarily on examining pavement wear rather than damage.

Two scientific breakthroughs resulted from the DIVINE accelerated pavement tests: "the effects of dynamic loading were measured for the first time, and a detailed statistical analysis of both the pavement and vehicle variables was undertaken."[24] Conclusions reached are:

- Changes in pavement profile under dynamically-active steel suspensions relate to: local structural compliance (the opposite of strength), and local dynamic wheel load.

- Changes in pavement profile under dynamically-quiet air suspensions are mainly related to the local structural compliance of the pavement.

- The relationship between tensile strain at the bottom of the pavement surfacing layer and dynamic wheel loading appears to depend on the pavement thickness. For thick pavement, strain is directly related to dynamic wheel loading. For thin pavement, strain directly related to dynamic wheel loading is weaker. This difference in pavement behavior is believed to be related to changes in tire contact conditions occurring from variances in the dynamic wheel load.

- Air suspension would increase pavement life by 60 percent for thick pavement and 15 percent for thin pavement (based on two types of implied assumptions: selected pavement response parameter measured and analyzed, and the "damage law" applied).

- Spatial repeatability on a relatively smooth road would increase total wheel loading at certain locations by approximately 10 percent, reducing pavement life at those locations by approximately 35 percent to 50 percent.

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[23] OECD DIVINE Programme, Final Report "Dynamic Interaction of Heavy Vehicles with Roads and Bridges," May 1997, p. 145.

[24] Ibid.

The findings indicate that "pavement wear is the key concept to be used in the scientific consideration of the effect of heavy vehicles on highway pavements."[25]

Additionally, recent research outside the DIVINE Program evaluated the role of suspension damping in enhancing the road friendliness of a heavy vehicle. The findings indicated an increase in linear suspension damping tends to reduce the dynamic load coefficient and the dynamic tire forces -- factors related to road wear. The research concluded that linear and air spring suspensions with light linear damping offer significant potentials to enhance the road friendliness of the vehicle with a slight deterioration in ride quality.[26] It is worth noting that approximately 90 percent of all truck-tractors and 70 percent of all van trailers sold in the United States are equipped with air suspensions. Additional studies on various types of axle suspension systems include studies on: torsion suspensions, four-leaf suspensions, and walking-beam suspensions.[27]

The research has yet to produce any compelling argument to incorporate a suspension system determinant into U.S. regulations, although some countries have done so. Mexico is in the final stages of preparing regulations that will allow up to 2,200 pounds of additional weight for each trailer axle equipped with air suspension or its equivalent. For a drive axle, Mexico may allow up to an additional 3,300 pounds. The impacts of different suspension systems on pavement deterioration are of secondary importance compared to the static axle load levels themselves. Use of road-friendly suspensions is beneficial, particularly for large trucking operations with well-controlled axle loadings.

LIFT AXLES

The widespread use of lift axles in Canada and the United States raises concern for resulting pavement deterioration when a driver, attempting to improve fuel consumption, fails to lower the axle when loaded. A 1988 and 1989 survey conducted in Ontario and Quebec found that approximately 17 percent and 21 percent, respectively, of trucks on highways in those Provinces had lift axles.[28] Lift axles have been adopted in response to GVW limits governed by the number

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[25] Ibid, p. 147.

[26] In the Rakheja and Woodroofe model suspension effects are represented using a sprung mass, an unsprung mass, and restoring and dissipative effects due to suspension and tire. The tire is modeled assuming linear spring rate, viscous damping, and point contact with the road.

[27] Sousa, Lysmer and Monismith investigated the influence of dynamic effects on pavement life for different types of axle suspension systems. They calculated a Reduction of Pavement Life (RPL) index of 19 percent for torsion suspensions (an ideal suspension would have RPL of 0). Similar results were found by Peterson in a study for RTAC: under rough roads at 50 mph, air bag suspensions exhibited dynamic loading coefficients (DLC) of 16 percent, spring suspensions had a DLC of 24 percent, and rubber spring walking beam suspensions had a DLC of 39 percent. Problems with walking-beam suspensions were also noted by Gillespie, et. al. who state that on rough and moderately rough roads, walking-beam suspensions without shock absorbers are typically 50 percent more damaging than other suspension types.

[28] Billing, et. al.

of axles (such as the FBF), and because trucks with multiple widely spaced axles have difficulty turning on dry roads and the lift axles can be raised by the driver prior to turns.

Lift axles make compliance with and enforcement of axle weight limits difficult. Improperly adjusted lift axles can damage pavements. The lift axle can be adjusted to any level by the driver. If the lift axle load is too high, the lift axle is overloaded. If it is too low, other axles may be overloaded. For example, under current Federal limits, a 4-axle single unit truck with a wheelbase of 30 feet can carry 62,000 pounds: 20,000 pounds on the steering axle and 42,000 pounds on the rear tridem. This vehicle would produce approximately 2.1 ESALs on flexible pavements. However, if the first axle of the tridem is a lift axle carrying little or no weight, this vehicle would produce approximately 4.0 ESALs.

PAVEMENT COST

Unit pavement costs and pavement costs per unit of payload-mile by configuration are shown in Tables VI-6 and VI-7. They illustrate how the addition of axles allows for increased payloads and at the same time reduces pavement deterioration. Particularly striking, are comparisons between the 3- and 4-axle single unit trucks, the 5- and 6-axle semitrailer combinations, and the 5- and 8-axle doubles. As shown in Table VI-7, the 4-axle truck has costs per payload ton-mile about 75 percent of that for the 3-axle truck even though its gross weight is 10,000 pounds more than the 3-axle truck. The comparison of the 6-axle semitrailer with the 5-axle is very similar on non-Interstate highways. The costs for the 8-axle double-trailer are less than half those for the 5-axle double-trailer. Triples do not compare well with doubles. Generally, truck owners would be opposed to adding axles because this increases the tare weight of the vehicle and reduces payload capacity.

TS&W REGULATION RELATED TO PAVEMENT PRESERVATION

TIRE REGULATIONS

Federal law and most State laws, do not address truck tire pressure. Tire pressure may have a large effect on fatigue of flexible pavements as discussed earlier (albeit a small to moderate effect on rigid pavements), and today's tire pressures are higher than in the 1950s -- primarily the consequence of a change from bias to radial ply tires. Concern has been raised about accelerated pavement rutting as a result of increased tire pressures. Recent research gives conflicting views as to whether or not pressures should be regulated.[29]

Federal, and most State, laws do not discourage or prohibit the use of wide-base tires. The consensus of United States and international research is that these tires have substantially more

[29] TRB Special Report 225 (1990) suggested regulation could be warranted if the more pessimistic analyses proved to be correct. NCHRP Study (1993) suggested limiting tire pressure to the recommended cold setting plus 15-psi; AASHTO (1993) suggested more research is required to answer all questions regarding the relationship of tire size, contact pressure, and contact area to pavement damage.

adverse effects on pavements than dual tires because current designs employ smaller, overall tire-road contact patch sizes than equivalent dual tire sizes. Future tire designs could address this issue. Wide-base tires -- which are widely used in Europe -- are being increasingly adopted by U.S. trucking operations. The benefits of wide-base tires are reduced energy use, emissions, tire weights, and truck operating costs. The trade off between changes in Federal pavement costs and operating benefits that would result from permitting or prohibiting extensive adoption of wide-base tires in the United States has not been analyzed.

Table VI-6
Unit Pavement Cost For Various Truck Types
$/1,000 MILES

		Truck Type								
		Single-Unit		Semitrailer		Double-Trailer			Triple	
		3-Axles	4-Axles	5-Axles	6-Axles	5-Axles	7-Axles	8-Axles	7-Axles	
	GVW (Pounds)	54,000	64,000	80,000	90,000	80,000	100,000	105,000	100,000	115,000
Area Type	Functional Class									
Rural	Interstate	0.09	0.07	0.05	0.05	0.03	0.10	0.05	0.04	0.08
	Prin. Art.	0.17	0.16	0.12	0.11	0.07	0.15	0.10	0.17	0.31
	Min. Art.	0.37	0.33	0.29	0.22	0.32	0.41	0.21	0.39	0.75
	Maj. Col.	1.38	1.35	0.90	0.80	1.17	1.03	0.65	1.46	2.95
	Min. Col.	2.27	2.08	1.49	1.24	1.92	1.69	1.07	2.42	4.87
	Locals	5.90	5.63	3.87	3.23	4.99	4.40	2.79	6.27	12.60
Urban	Interstate	0.06	0.04	0.04	0.04	0.03	0.04	0.02	0.03	0.05
	Freeway & Expressway	0.09	0.06	0.06	0.05	0.04	0.07	0.04	0.09	0.18
	Prin. Art.	0.13	0.12	0.10	0.09	0.11	0.09	0.06	0.13	0.26
	Min. Art.	0.30	0.24	0.22	0.17	0.19	0.18	0.12	0.34	0.70
	Collectors	0.66	0.70	0.54	0.49	0.46	0.34	0.25	0.86	1.82
	Locals	2.34	2.53	1.91	1.75	1.64	1.19	0.88	3.06	6.45

Historically, many States specified some form of tire load regulation for safety. In recent years, additional States have adopted tire load regulations to control the damage effect of wide-base tires. They restrict the weight that can be carried on a tire based on its width. The limits range from 550 pounds per inch (in Alaska, Mississippi, and North Dakota) to 800 pounds per inch (in Indiana, Massachusetts, New Jersey, New York, and Pennsylvania). Such restrictions result in lower pavement costs; however, the size of the pavement cost savings (either in absolute terms or in relation to the increase in goods movement costs also resulting from these restrictions) have not been estimated.

Table VI-7
Unit Cost per Payload-mile for Various Truck Types
$/1,000 Ton-miles

		Truck Type								
		Single-Unit		**Semitrailer**		**Double-Trailer**			**Triple**	
	Weights (Pounds)	**3-Axles**	**4-Axles**	**5-Axles**	**6-Axles**	**5-Axles**	**7-Axles**	**8-Axles**	**7-Axles**	
	GVW	54,000	64,000	80,000	90,000	80,000	100,000	105,000	100,000	115,000
	Tare	22,600	26,400	30.490	31,530	29,320	38,600	33,470	41,700	41,700
	Payload	31,400	37,600	49,510	58,470	50,680	61,400	71,530	58,300	73,300
Area Type	**Functional Class**									
Rural	Interstate	0.006	0.004	0.002	0.002	0.001	0.003	0.001	0.001	0.002
	Prin. Art.	0.011	0.009	0.005	0.004	0.003	0.005	0.003	0.006	0.008
	Min. Art.	0.024	0.018	0.012	0.008	0.013	0.013	0.006	0.013	0.020
	Maj. Col.	0.088	0.072	0.036	0.027	0.046	0.034	0.018	0.050	0.080
	Min. Col.	0.145	0.111	0.060	0.042	0.076	0.055	0.030	0.083	0.133
	Locals	0.376	0.299	0.156	0.110	0.197	0.143	0.078	0.215	0.344
Urban	Interstate	0.004	0.002	0.002	0.001	0.001	0.001	0.001	0.001	0.001
	Freeway & Expressway	0.006	0.003	0.002	0.002	0.002	0.002	0.001	0.003	0.005
	Prin. Art.	0.008	0.006	0.004	0.003	0.004	0.003	0.002	0.004	0.007
	Min. Art.	0.019	0.013	0.009	0.006	0.007	0.006	0.003	0.011	0.019
	Collectors	0.042	0.037	0.022	0.017	0.018	0.011	0.007	0.030	0.050
	Locals	0.149	0.136	0.077	0.060	0.065	0.039	0.024	0.105	0.176

SPLIT-TANDEM VERSUS TRIDEM-AXLE LOAD LIMITS

There is increasing use of split tandem axle groups with spreads up to 10 feet, particularly in flatbed heavy haul operations. These axles are allowed to be loaded at single axle limits -- 20,000 limits -- 20,000 pounds on each of the 2 axles -- as opposed to 34,000 pounds on a closed tandem when they are split more than 8 feet. They offer two key benefits to 5-axle tractor-semitrailer usage: (1) flexibility in load distribution; and (2) full achievement of the 80,000-pound GVW cap, which is limited by the ability to distribute up to 12,000 pounds on the steering axle of a combination. But they do so at a significant cost to pavement life.

In the United States, the allowable load on a group of three axles connected by a common suspension system (tridem) is determined by the Federal bridge formula rather than a limit set by law (or regulation). In Europe, Canada, Mexico, and most other jurisdictions, tridem axles are given a specific load limit in the same way the United States specifies single and tandem axle limits without direct reference to a bridge formula. This is not to say that these tridem limits are not bridge-related. For example, the tridem limits prescribed by the RTAC, which vary as a function of spacing, are based on bridge loading limitations -- not pavement limitations.

THE GVW LIMIT

The existing legal Federal maximum GVW (cap) limit for the Interstate System is 80,000 pounds, although some States allow truck combination weights above this cap under Federal grandfathering provisions. Axle weight limits and the FBF are designed to protect pavements and bridges, respectively. As such, the cap may not be providing any additional protection to pavements and bridges. Nevertheless, it is important to consider such factors as bridge design loads and criteria, structural evaluation procedures, the age of the existing bridges, and the extent to which increased GVWs would affect the fatigue life of bridges in the United States.

44,000-POUND TRIDEM-AXLE WEIGHT LIMIT

Original research done for this Study on the pavement and bridge impacts of tridem axles showed how bridge stresses decrease as the axles in the tridem group are spread apart. This allows more weight to be carried on the tridem group as the axles are spread. The opposite is true for pavement damage. The more the axles are spread, the greater the damage. Therefore, as the axles are spread within the group, the allowable weight must be reduced to hold pavement damage constant.

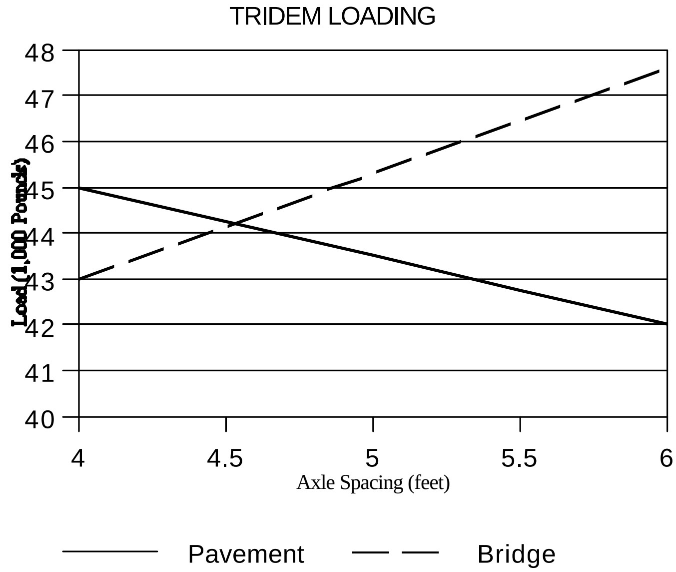
The tridem-axle weight limit of 44,000 pounds was determined by observing where the curve of the increasing bridge allowable load function crosses the curve of the decreasing pavement load equivalency function (see Figure VI-4). The two curves cross at a spread of 9 feet between the two outer axles which gives 44,000 pounds for both functions. To stop short of 9 feet would require a lower load limit as bridge damage would be greater than at 44,000 pounds. To go beyond 9 feet would increase pavement damage over that at 44,000 pounds.

A 6-axle semitrailer combination is more effective in reducing pavement damage than a 5-axle semitrailer combination with a split tandem (two trailer axles spread apart), which is allowed under the current FBF. Table VI-8 provides the weight limits for a tridem axle between 8 and 16 feet and Figure VI-4 illustrates the impact on pavement and bridges.

Table VI-8
Tridem-axle Weight Limits

Axle Spreads (Feet)	Distance Between Adjacent Axles (Feet)	Load at LEF=1	Allowable Bridge Load (1,000 Pounds)
8	4	45	43
12	6	42	48.6
16	8	40	------

Figure VI-4
Pavement and Bridge Impact of Tridem-axle



USE OF TRIDEMS

The use of tridem axles could increase truck load capacity while reducing pavement damage.[30] Many heavy bulk haulers have already switched from 3-axle to 4-axle single unit trucks, and as noted above, significant pavement cost savings may be possible. The 80,000-pound GVW limit poses a constraint on adding axles to 5-axle combinations because the extra axle would reduce the payload.

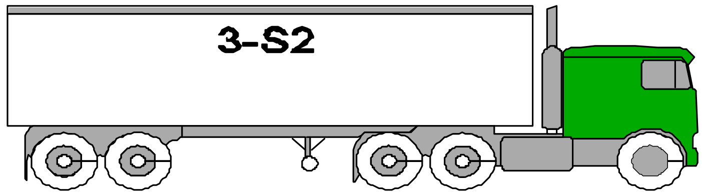
When viewed using the AASHTO load equivalence factors, combinations with tridem axles generally have much lower pavement costs per ton of freight carried than conventional 5-axle combinations. To illustrate this, as shown in Figur VI-5, a 6-axle tractor-semitrailer loaded to 90,000 pounds with a rear tridem carrying 44,000 pounds produces 2.00 ESALs on flexible pavements and 3.83 ESALs on rigid pavements. The corresponding ESAL values for a conventional 5-axle tractor-semitrailer carrying 80,000 pounds are 2.37 (flexible) and 3.94 (rigid).

Assuming tare weights of 28,000 and 29,500 pounds for the 5- and 6-axle combinations, respectively, and using the AASHTO load equivalence factors, the ESALs per million pounds of payload for the trucks shown in Figure VI-5 are shown in Table VI-9.

[30] Both the TRB *Special Report 225* and the AASHTO TS&W Subcommittee suggest consideration of the TTI bridge formula which could allow about 90,000 pounds for a 6-axle tractor-semitrailer combination.

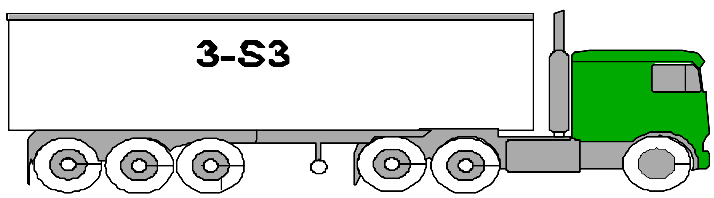
Figure VI-5
ESAL Comparison of 5-axle and 6-axle Combinations on Pavement

5-Axle Tractor-Semitrailer



				TOTAL
Weight (pounds)	34,000	34,000	12,000	80,000
ESALs				
Flexible	1.09	1.09	0.19	2.37
Rigid	1.88	1.88	0.18	3.94

6-Axle Tractor-Semitrailer



				TOTAL
Weight (pounds)	44,000	34,000	12,000	90,000
ESALs				
Flexible	0.72	1.09	0.19	2.00
Rigid	1.77	1.88	0.18	3.83

Table VI-9
ESALS per Million Pounds Payload for 5- and 6-axle Combinations

	Flexible Pavement	Rigid Pavement
5-Axle Tractor-Semitrailer	46	76
6-Axle Tractor-Semitrailer	33	63

ROADWAY GEOMETRY

ELEMENTS OF ROADWAY GEOMETRY AFFECTING TRUCK OPERATIONS

INTERCHANGE RAMPS

Access and exit ramps for controlled access highways are intended to accommodate design vehicles at certain design speeds. Otherwise, trucks heavier than the design vehicle have an increased probability of rolling over, and trucks longer than the design vehicle will have trailer wheels that travel off the pavement to the inside of a curve. The TS&W, configuration, and speed influence the potential for rollover on short loop ramps. The AASHTO policy recommends widening ramps to accommodate combination vehicles. For example, the width of a 1-lane ramp, with no provision for passing a stalled vehicle, would be 15 feet on a tangent section.

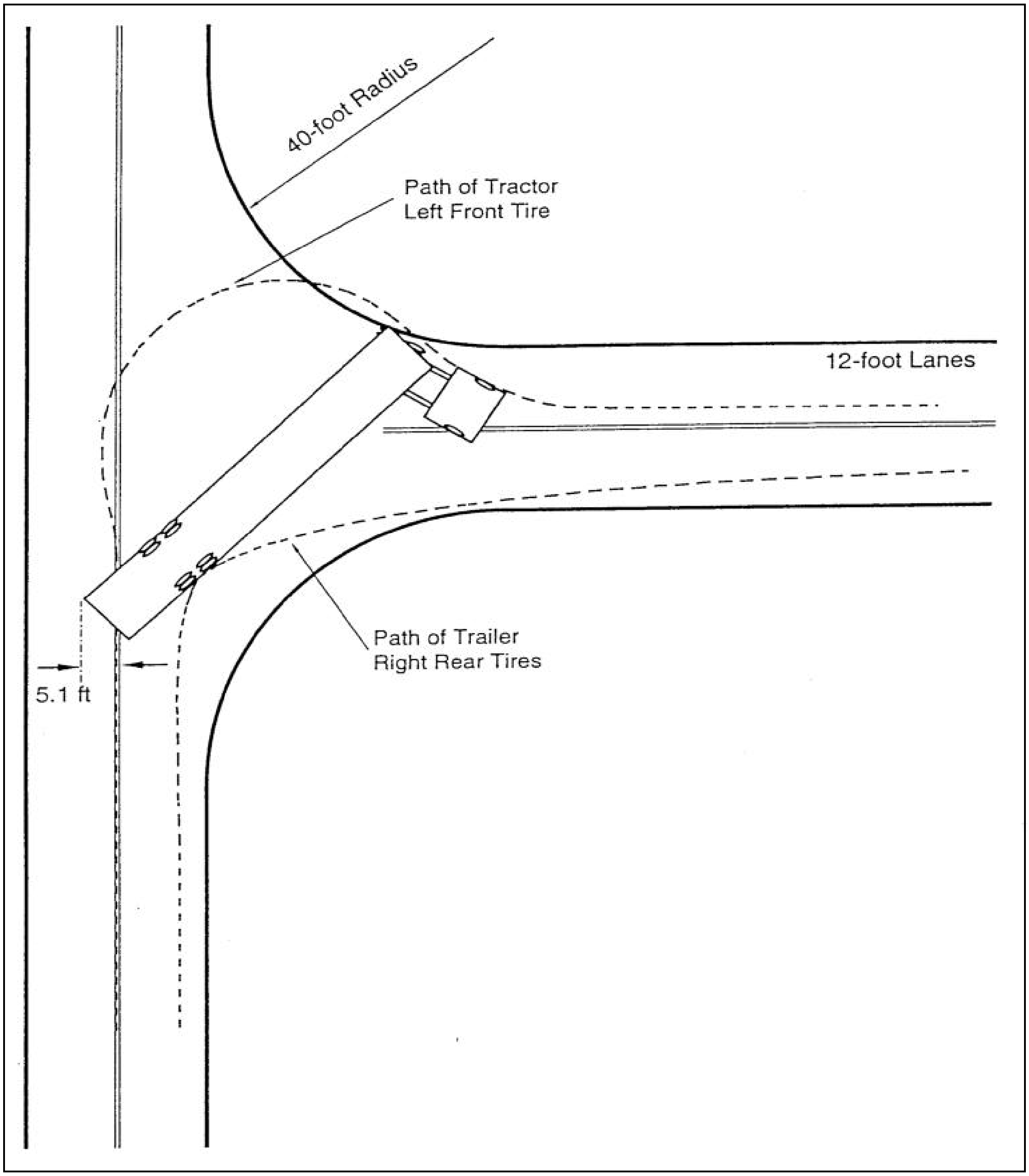
The extreme case for design consideration occurs when traffic is congested and stop-and-go conditions exist. The speed component to the offtracking equation is negligible and maximum offtracking to the inside of the curve occurs. Under this condition, the turnpike doubles analyzed in this study offtrack 20 percent more than a 5-axle 53-foot semitrailer combination and as a result, encroach on adjacent lanes or shoulders and necessitate widening beyond AASHTO standards.

INTERSECTIONS

Most truck combinations turning at intersections encroach on either the roadway shoulder or adjacent lanes. For example, the turning path of a truck making a right turn is generally controlled by the curb return radius, whereas the turning path in left turns is not constrained by roadway curbs, but may be constrained by median curbs and other traffic lanes. Combination vehicles with long semitrailers are critical in determining needed intersection improvements to accommodate offtracking requirements. Additionally, the increased time required for a large truck to complete its turn requires longer traffic signals and affects pedestrian safety and intersection efficiency. Figure VI-6 illustrates the intersection maneuver.

Proper design and operation requires that no incursion into the path of vehicles traveling in opposing directions be allowed. A higher standard is often used in design, especially in urban areas, where no incursion into <u>any</u> adjacent lane is allowed. This is particularly critical at signalized intersections where heavy traffic is a prevailing condition. A substantial number of intersections on the existing highway and street network cannot accommodate even a 5-axle tractor-semitrailer combination with a 48-foot semitrailer. Even more intersections would be inadequate to accommodate vehicles that offtrack more than the standard 48-foot semitrailer combination.

Figure VI-6
Path of Tractor Semitrailer Keeping Tires Within Lanes



NOTE: Distance from kingping to rear axle is 40 feet; distance from rear axle to rear of trailer is 14.5 feet

Currently, there are a substantial number of intersections on the highway and street network where improvements for combinations with semitrailers over 48 feet are not feasible and where controls on vehicles, routing, or travel times are needed. Examples of common constraints to intersection improvements are bridges, buildings and sensitive environmental or historic plots. The use of permits in such cases can provide a desirable level of control. Another option for States might be the provision of staging areas where routes and intersections have prohibitive constraints off Interstate-type highways.

CLIMBING LANES

The ability of a truck to maintain speed on a grade is described by the term "gradeability;" the truck's ability to start on a grade from a standstill is termed "startability." The ability of various trucks to start and to maintain speeds on grades is a complex subject that primarily depends on net engine horsepower, torque, gearing, drive train efficiency, friction, GVW, and minimum allowable speed. Gradeability and startability are discussed in Chapter 5, Safety and Traffic Operations. The AASHTO recommends that separate climbing lanes be provided on grades that have substantial truck traffic or that cause typical trucks to slow by more than 10 miles per hour.[31]

CROSS-SECTION

Cross-section refers to the shape of the surface of the roadway perpendicular to the direction of traffic.[32]   Under normal operating conditions, cross-section is not a dominant factor in increased TS&W, but under extreme icing conditions, a superelevated cross slope can be a significant problem for vehicles with greater offtracking. The presence of cross-slope discontinuities can also be a problem for vehicles more prone to rollover because of the dynamic forces that they tend to introduce.

HORIZONTAL CURVATURE

The rear wheels of trucks and truck combinations traversing horizontal curves generally offtrack to one side or the other of the paths of the wheels on the steering axle. When a truck is traveling at higher speeds the rear wheels can follow a path outside that of the steering wheels. This effect is relatively small and virtually never results in the need to make geometric improvements beyond those normally made in the design process. On the other hand, when offtracking is to the inside of the curve at lower speeds and in stop-and-go traffic, it is usually more substantial and must be accommodated. Truck combinations with longer trailers are often prone to producing relatively large amounts of offtracking beyond that provided for in AASHTO

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[31] Substantial is not defined by AASHTO. There is no universally acceptable standard and it is left to the States to define.

[32] The major determinants of the cross section are the number of lanes, the presence of curbing or shoulders, and cross slope. Generally, a slight cross slope is designed into the cross section to assist in proper drainage of precipitation. Often this slope breaks to a steeper slope at the shoulder line, on a divided multilane highway the grade or elevation is generally highest at the centerline.

standards. For roadways not constructed to AASHTO standards more improvement would be required to accommodate longer combinations where offtracking would exceed normal lane width.

VERTICAL CURVE LENGTH

The height of the truck driver's eye is a distinct advantage of trucks over passenger vehicles for crest vertical curves that are designed to maximize stopping sight distance. Vertical curves are generally designed for passenger cars, as a passenger car driver's eye is lower than is a truck driver's. For a sag vertical curve going from a downgrade to an upgrade, headlight coverage and passenger comfort usually control. The vehicles considered in this study have braking distances similar to vehicles in common use at this time; therefore, no geometric adjustments would be required.

PASSING SIGHT DISTANCES

Distances required for passing trucks can be significantly longer than for automobiles and pickups. Longer trucks increase the distance required for a car or truck to pass and require more care in order do so safely. Drivers of passenger cars passing trucks, and drivers of trucks who desire to pass other vehicles, are expected to follow the rules of the road and exercise discretion, passing only where sight distance is adequate. On multilane highways, passing is not as critical as passing on a 2-lane highway with traffic in opposing directions. Sight distance criteria for marking passing and no-passing zones on 2-lane highways are more appropriate for a passenger car passing another passenger car: they do not consider trucks, even the standard truck-and 48-foot semitrailer combination vehicle at 80,000 pounds.

The additional lengths of LCVs could require as much as 8 percent more passing sight distance for cars passing LCVs on 2-lane roads; longer and/or heavier trucks would require incrementally longer passing sight distances to pass cars safely on 2-lane roads.

DIMENSIONAL LIMITS IMPACTING TRUCK MANEUVERS

LENGTH LIMITS FOR SEMITRAILERS

The STAA of 1982 requires States to allow the operation of a semitrailer of at least 48 feet long on the NN. All States now allow up to 53 feet on at least some highways. The majority of States prohibit semitrailers longer than 53 feet, the exceptions being Alabama, Arizona, Arkansas, Colorado, Florida, Kansas, Louisiana, New Mexico, Oklahoma, Texas, and Wyoming.[33] Most of these States allow trailers in the 57- to 60-foot range to operate.

[33] *Federal Size Regulations for Commercial Motor Vehicles,* U.S. DOT, Publication Number FHWA-MC-96-03.

LENGTH LIMITS FOR DOUBLE TRAILERS IN COMBINATION

The STAA of 1982 also established a requirement for States to allow, at a minimum, the operation of two 28-foot trailers (twins) in combination on the Interstate and NN. About one-fourth of the States prescribe 28 feet as a maximum; the others allow additional length up to 30 feet with 28.5 feet being the most common. Prior to passage of the ISTEA, Federal law allowed States to permit longer trailers in combination (commonly referred to as doubles) but did not require States to do so.

OVERALL LENGTH LIMITS

The STAA of 1982 established a prohibition against State laws specifying a maximum length for semitrailer and STAA double combinations operating on the Interstate and NN. Consequently, most States control total length on the NN by limiting semitrailer and trailer lengths. About two-thirds of the States have some form of control of total combination length for non-NN highways. While there are no proposals that the Federal law prescribe a total length limit at this time, offtracking standards could effectively limit overall lengths for single- and double-trailer combinations.

VEHICLE WIDTH AND HEIGHT LIMITS

Vehicle widths and heights are important from the standpoint of safety and traffic operations. The effect on roadway geometric design relates to lane and shoulder width and vertical clearances. A 1-lane ramp with a narrow shoulder would result in a blockage if a truck were disabled. Many older structures (overpasses) were constructed with minimal vertical clearances. The addition of pavement overlays over the years may have further reduced these clearances. Increases in vehicle height increases the potential for striking these overhead structures as well as vehicle rollover.

ROADWAY GEOMETRY AND TRUCK OPERATING CHARACTERISTICS

When a vehicle makes a turn, its rear wheels do not follow the same path as its front wheels. The magnitude of this difference in path, known as “offtracking,” generally increases with the spacing between the axles of the vehicle and decreases for larger radius turns. Offtracking of passenger cars is minimal because of their relatively short wheel bases; however, many trucks offtrack substantially. The magnitude of the offtracking is often measured by the differences in the paths of the centerlines of the front and subsequent axles. The maximum extent of offtracking for a turn of a given radius and length occurs at the rearmost axle or the center of the rearmost axle group.

Offtracking develops gradually as a vehicle enters a turn and, if the turn is long enough, eventually reaches what is termed as fully-developed offtracking. The offtracking does not continue to increase beyond this point for curves that are any longer. The extent of this fully-developed offtracking is used to determine if the nominal lane width can accommodate the offtracking or how much the lane should be widen through the curve to accommodate the offtracking characteristics of the trucks using the highway.

In contrast, for a short radius 90-degree turn such as a truck would make at an intersection, the turn is too short for fully-developed offtracking to occur. Nevertheless, the maximum extent of offtracking may be readily calculated for designing an intersection that can accommodate the trucks expected to make right turns at the intersection.

LOW-SPEED OFFTRACKING

When a combination vehicle makes a low-speed turn -- for example a 90-degree turn at an intersection -- the wheels of the rearmost trailer axle follows a path several feet inside the path of the tractor steering axle. This is called low-speed offtracking. Excessive low-speed offtracking may make it necessary for the driver to swing wide into adjacent lanes to execute the turn (that is, to avoid climbing inside curbs or striking curbside fixed objects or other vehicles). When negotiating exit ramps, excessive offtracking can result in the truck tracking inboard onto the shoulder or up over inside curbs.

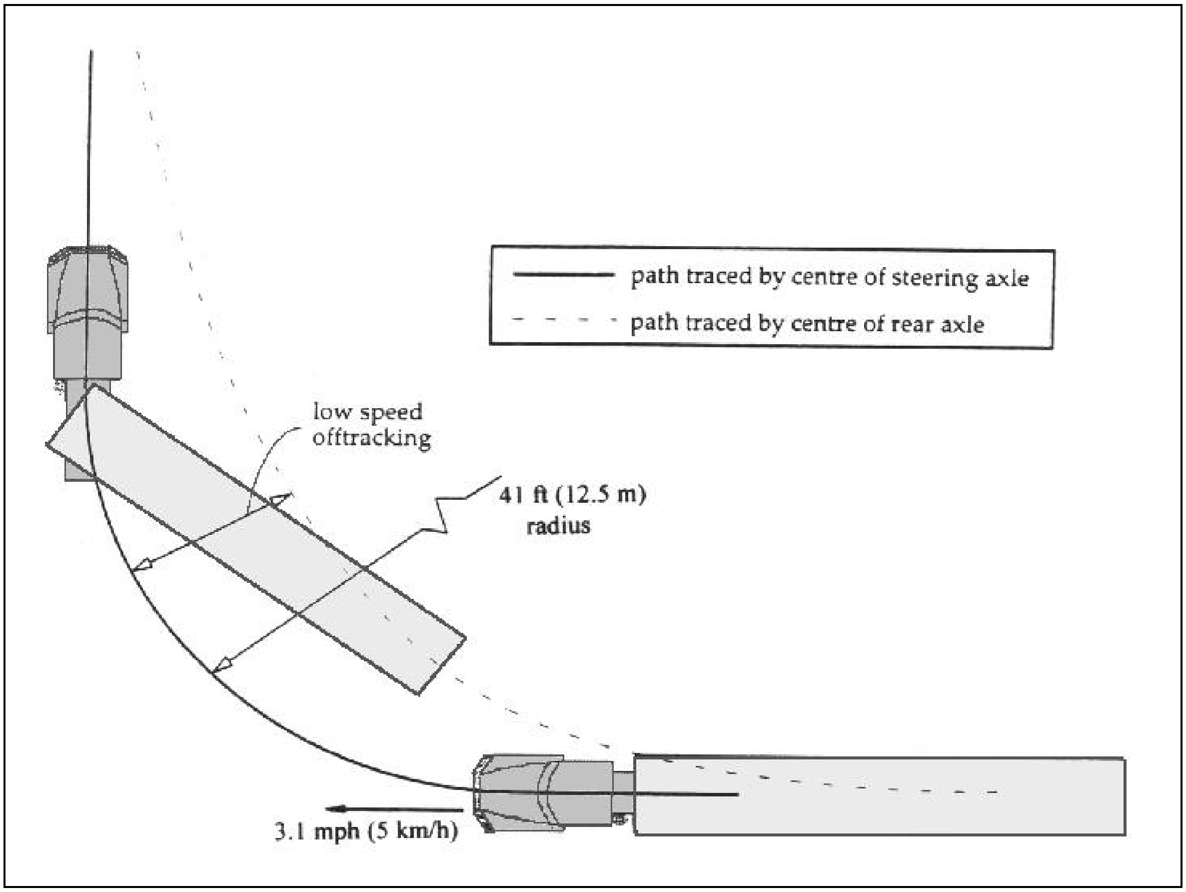
This performance attribute is affected primarily by the distance from the tractor kingpin to the center of the trailer rear axle, or the wheelbase of the semitrailer. In the case of multitrailer combinations, the effective wheelbase(s) of all the trailers in the combination, along with the tracking characteristics of the converter dollies, dictate this property. In general, longer wheelbases worsen low-speed offtracking. However, other factors including the use of tandem or tridem axles, the kingpin offset from the center of the supporting axle group, the cross slope of the roadway, the loads of the axles, and the truck suspension have small, generally negligible, effects on low-speed offtracking. Figure VI-7 illustrates low-speed offtracking in a 90-degree turn for a tractor-semitrailer combination.

The standard double-trailer combination (two 28-foot trailers) and triple combinations (three 28-foot trailers) exhibit better low speed offtracking performance when compared to a standard tractor and 53-foot semitrailer combination. This is because they have more articulation points in the vehicle combination, and use trailers with shorter wheelbases.

HIGH-SPEED OFFTRACKING

High-speed offtracking, on the other hand, is a dynamic, speed-dependent phenomenon. It results from the tendency of the rear of the truck to move outward due to the lateral acceleration of the vehicle as it makes a turn at higher speeds. High-speed offtracking is actually the algebraic combination of the low-speed offtracking toward the inside of the turn and the outward displacement due to the lateral acceleration. As the speed of the truck increases, the total offtracking decreases until, at some particular speed, the rear trailer axles follow exactly the tractor steering axle. At still higher speeds, the rear trailer axles will track outside of the tractor steering axle. The speed-dependent component of offtracking is primarily a function of the spacing between truck axles, the speed of the truck, and the radius of the turn; it is also dependent on the loads carried by the truck axles and the truck suspension characteristics. Figure VI-8 illustrates offtracking maneuver for a standard tractor-semitrailer.

Figure VI-7
Low-speed Offtracking



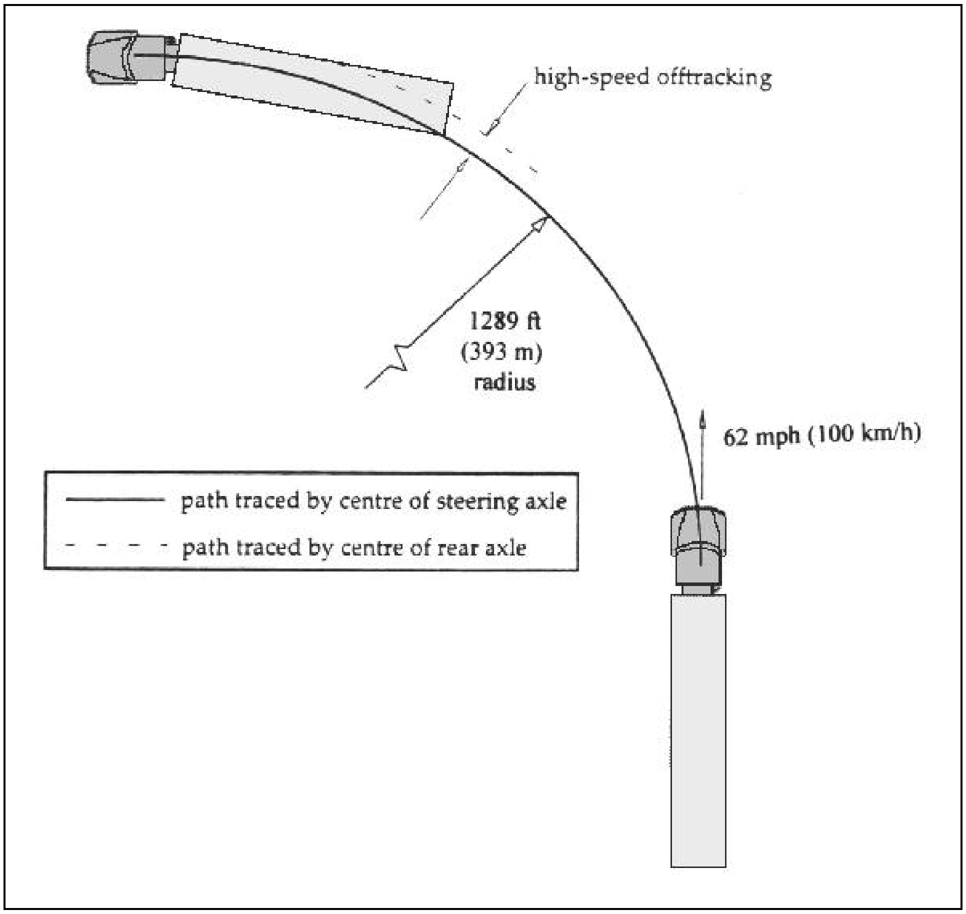
Source: Roaduser Research

OFFTRACKING ON MAINLINE HORIZONTAL CURVE AND INTERCHANGE RAMPS

An analysis of offtracking and swept path width for horizontal curves designed in accordance with AASHTO's high-speed design criteria (1994) was completed for the vehicle configurations considered in this study. Such curves are typically found on mainline roadways and higher speed ramps. Alternative design criteria that permit higher unbalanced lateral acceleration and, thus, tighter radii can be used under AASHTO policies for horizontal curves with design speeds of 40 mph or less, which are typically found on ramps and turning roadways at intersections.

Under AASHTO policy (1994), the minimum radius for a horizontal curve varies with the roadway design speed and the maximum superelevation rate. For horizontal curves with a

Figure VI-8
High-Speed Offtracking



Source: Roaduser Research

maximum superelevation rate of 0.06 feet/foot (the maximum superelevation rate most commonly used by State highway agencies), the minimum radii permitted by the AASHTO high-speed design criteria vary with design speed, as shown in Table VI-10.

Table VI-10
AASHTO High-speed Design Criteria

Design Speed (Mph)	Minimum Radius (Feet)
30	273
50	849
70	2,083

The AASHTO policy for horizontal curve design specifies pavement widening on sharp radius horizontal curves for which truck offtracking is a concern. For the minimum-radius curves listed above on a highway with a lane width of 12 feet on tangent sections, only the 273-foot radius curve (for a 30-mph design speed) would require widening. The AASHTO criteria call for such a curve to be widened from 12 to 14.5 feet.

An analysis was conducted to determine whether minimum-radius curves with the widths described above, designed in accordance with AASHTO policies, would be capable of accommodating each of the vehicle configurations considered in this Study. This analysis was conducted by comparing the lane or ramp width to the swept path width of the truck making a turn with the specified radius. Tables VI-11 and VI-12 present this comparison for selected truck configurations.

The swept path widths in Table VI-11 are based on fully-developed offtracking determined with the Glauz and Harwood Model for a truck traversing the curve with a travel speed equal to the roadway design speed. None of the swept path widths shown in Table VI-11 exceed the corresponding lane width for mainline roadways or the corresponding ramp widths, although the turnpike double with 53-foot trailers does require nearly all of the (widened) 14.5 feet of the 30-mph AASHTO horizontal curve. Thus, there is no indication that any of the Study vehicles, traveling at the roadway design speed, would necessarily offtrack into an adjacent lane or shoulder of the roadway or ramps designed in accordance with AASHTO policies.

Table VI-11
Swept Path Width for Selected Trucks on Horizontal Curves
At AASHTO Design Speed Criteria

		Maximum Swept Path Width (Feet) at the Design Speed on the Sharpest Horizontal Curve Allowed by AASHTO Design Policy		
	Design Speed (Mph)	30	40	60
	Curve Radius (Feet)	273	509	1,348
Truck Configuration	**Length (Feet)**			
3-Axle Single Unit Truck	39.5	8.12	8.00	8.00
5-Axle Tractor Semitrailer	64.3	10.09	8.56	8.50
5-Axle Tractor Semitrailer	76.8	11.88	9.43	8.50
6-Axle Tractor Semitrailer	76.8	11.79	9.48	8.50
7-Axle Truck-Full Trailer	61.3	8.44	8.00	8.00
7-Axle Rocky Mtn Double	99.3	11.62	9.21	8.50
8-Axle B-Train Double	84.3	10.39	8.70	8.50
9-Axle Turnpike Double	124.3	14.29	10.54	8.50
7-Axle Triple	109.0	9.69	8.50	8.50

Table VI-12 presents comparable results when the trucks travel at very slow speeds on these same curves, such as they may be required to do in congested traffic. The swept path widths at low speed in Table VI-12 are generally greater than those in Table VI-11, but except for the turnpike doubles, none of the study vehicles would encroach on adjacent lanes or shoulders. Both turnpike doubles would encroach on adjacent lanes or shoulders on 30-mph design speed horizontal curves; the turnpike double with 53-foot trailers would offtrack at low speeds into adjacent lanes or shoulders on 40-mile per hour design speed horizontal curves and on 30-mile per hour design speed ramps.

Table VI-12
Swept Path Width for Selected Trucks on Horizontal Curves
At AASHTO Design Speed Criteria

		Maximum Swept Path Width (Feet) at Very Low Speed on the Sharpest Horizontal Curve Allowed by AASHTO Design Policy		
	Design Speed (Mph)	30	40	60
	Curve Radius (Feet)	273	509	1,348
Truck Configuration	**Length (Feet)**			
3-Axle Single Unit Truck	39.5	8.80	8.26	8.00
5-Axle Tractor Semitrailer	64.3	11.54	9.95	8.80
5-Axle Tractor Semitrailer	76.8	13.65	11.12	9.30
6-Axle Tractor Semitrailer	76.8	13.22	10.85	9.14
7-Axle Truck-Full Trailer	61.3	8.98	8.34	8.00
7-Axle RMD	99.3	13.65	11.15	9.35
8-Axle B-Train Double	84.3	11.92	10.16	8.89
9-Axle Turnpike Double	124.3	16.69	12.83	10.05
7-Axle Triple	109.0	12.15	10.40	9.14

The analyses assume that the turn is made at the intersection of two 2-lane or two 4-lane streets and that the truck making the turn positions itself as far to the left as possible on the approach to the intersection without encroaching on the opposing lanes, and completes the turn as far to the left as possible without encroaching on the opposing lanes. In other words, the truck does encroach on adjacent lanes for traffic moving in the same direction (on 4-lane roads), but does not encroach on lanes used by traffic moving in the opposing direction. The maneuver specified above requires a turning radius for the truck tractor which is 8 feet longer than the curb return radius on a 2-lane road and 20 feet longer than the curb return radius on a 4-lane road, if all lanes are 12 feet wide.

Table VI-13 presents estimates of encroachment on the curb return for selected trucks for right turns at corners with curb return radii of 30, 60, and 100 feet. The data in these exhibits are based on the maximum value of the partially developed offtracking because, in most cases, offtracking will not develop fully as a large truck proceeds through an intersection turning maneuver.

Table VI-13
Curb Encroachment for 90-degree Right-turn Maneuvers
At Intersection of 4-lane Roads

		Encroachment on Curb Return		
Truck Configuration	**Length (Feet)**	**30-Foot Curb Return Radius**	**60-Foot Curb Return Radius**	**100-Foot Curb Return Radius**
3-Axle Single Unit Truck	39.5	-9.97	-12.07	-13.37
5-Axle Tractor Semitrailer	64.3	-0.09	-4.47	-7.88
5-Axle Tractor Semitrailer	76.8	6.42	1.11	-3.49
6-Axle Tractor Semitrailer	76.8	5.34	0.16	-4.25
7-Axle Truck-Full Trailer	61.3	-8.10	-10.82	-12.54
7-Axle RMD	99.3	6.73	1.23	-3.48
8-Axle B-Train Double	84.3	1.58	-3.23	-7.02
9-Axle Turnpike Double	124.3	15.38	8.83	2.69
7-Axle Triple	109.0	1.97	-2.97	-6.87

The encroachment columns in Table VI-13 indicates the amount of encroachment on the curbline by the rear axles of the turning truck. A negative value indicates that the truck does not encroach on the curbline. A positive value indicates that encroachment does occur, and the magnitude of the value indicates the maximum encroachment distance. Where a positive value is shown for the encroachment distance, that particular truck could make the turn without encroaching on the curbline only if it encroached on an opposing lane(s) instead.

The turn from a 4-lane street to another 4-lane street was chosen as the case of interest because none of the trucks considered -- baseline or study vehicles -- are capable of making a short-radius turn from one 2-lane street to another without encroaching on either the curbline or an opposing lane, unless the curb return radius is very large (say, 100 feet), and then only by short trucks.

With a 30-foot curb return radius, many of the truck configurations will encroach on the curb return, with a few exceptions. The single unit trucks, the tractors with a 45-foot semitrailer, the truck-full trailers, and the Western twins can successfully negotiate these turns. The encroachment of the 5-axle semitrailer configuration with a 45-foot trailer is very marginal, however, as is the triple with 28-foot trailers.

By expanding the curb return radius to 60 feet, nearly all configurations examined can negotiate the turn without encroaching on the curb return. The exceptions that cannot successfully complete the turn are the tractors with 57.5-foot semitrailers, the longer RMD, and (especially) the turnpike doubles. At an even larger curb return radius of 100 feet, all but the turnpike double with 53-foot trailers can properly negotiate the turn.

TS&W REGULATION RELATED TO ROADWAY GEOMETRY

CURRENT REGULATIONS ON OFFTRACKING

Federal law does not address offtracking-related characteristics of trucks and combinations. In particular, it specifies no requirements for kingpin setting, kingpin setback, and rear overhang. In nearly half of the States, regulations require a kingpin setting for semitrailers over 48 feet in length. Although there is no one uniform standard, the most common setting is 41 feet.

REGULATORY APPROACHES

Control of offtracking can be accomplished in either of two ways. The first requires specifying the length limit(s) of the combination units within the context of overall combination length, restrictions on the kingpin setback, wheelbase, and effective rear overhang, as in Canadian regulations. The second approach is a performance specification requiring that a truck be able to turn through a given angle, at a given speed, within a defined swept path as in European regulations. Such a regulation would require matching truck equipment with trailer equipment for operation based on knowledge of specific system characteristics, which would require extensive documentation and signage to implement and enforce.