Comprehensive Truck Size and Weight (TS&W) Study

Phase 1-Synthesis

Bridges

and

Truck Size and Weight Regulations

Working Paper 4

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By

Battelle Team
505 King Avenue
Columbus, Ohio 43201-2693

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Phase 1—Synthesis

Working Paper 4—Bridges and TS&W Regulations

1.0 Technical Relationships of Policy Consequence Concerning Bridges

1.1 Bridge Design Considerations[1]

Most highway bridges in the United States were designed according to the provisions of the American Association of State Highway and Transportation Officials (AASHTO). The AASHTO bridge specifications 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.

A key task of the bridge designer is the selection of 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), wind, seismic, and thermal forces. The relative importance of these loads depends upon the types of materials used in construction, anticipated traffic, climate, and environmental conditions. For a short 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 while the remaining 30 percent supports the weight of the bridge itself. For a long bridge (for example, span length of 1,000 feet), only about 25 percent of the load-bearing capacity of the main structural members may be required to support the live load while the remaining 75 percent supports the weight of the bridge itself.

In evaluating the effects of changes in truck size and weight limits on bridges, both overstress and fatigue should be considered. Overstress creates the possibility of severe damage and possible collapse caused by a single extreme loading event. Fatigue produces the cumulative damage caused by thousands and even millions of load passages, which can damage key elements of a bridge.

For overstress, the loading event that governs bridge capacity in most instances is two or more heavy trucks on the bridge simultaneously. The probability of occurrence of a multiple-presence phenomenon can be evaluated by simulation and

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[1]Much of this background discussion is drawn from TRB Special Report 225, *Truck Weight Limits: Issues and Options*, and from DOT Section 161 Report, *An Investigation of Truck Size and Weight Limits*.

depends on the frequency of occurrence of heavy vehicles. As the number of heavy vehicles increases, there is a higher likelihood of a "critical" load event in which several heavy vehicles are on the bridge simultaneously. Dynamic impact, which varies with speed and roadway roughness, and the distribution of loads, which varies with the position of the truck on the bridge, also affect bridge response. Overall, the critical loading event is usually two or more heavy vehicles present on the bridge with unfavorable dynamic and load distribution conditions. Typically, bridge engineers plan for the rare loading event by taking a load model similar to the legal loading and magnifying it to represent a rare combination of multiple presence of overloads, impacts, and load distribution. This magnification of the legal loading is reflected in the safety factor, which is selected so that there is only a very small probability that a loading will be reached within the design life of a bridge that exceeds its load capacity.

The methods used by bridge engineers to calculate stresses in bridges caused by a given loading also are necessarily conservative, and therefore the actual measured stresses are generally much less than the calculated stresses. A margin of safety is necessary 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

- Users of the highway on occasion violate truck size and weight laws.

Some of the added margins of safety used by bridge engineers in the past have been eroded in recent constructions. Use of new design procedures and computer-aided engineering and design has enabled more precise analysis of load effects and the selection of lowest size bridge members and configurations. The competition between steel and concrete has led each group to foster lower costs for their own material. For example, many designs now proposed for steel reduce the conservativeness by reducing the number of members and increasing the girder spacings. This suggests that we must be prepared to settle on good load models and regulations now because we cannot rely in the future on large margins of safety to cover more load increases.

Bridge engineers must be concerned not only with overstress due to a single extreme loading event, but also with fatigue life considerations caused by 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 occurrence of a fatigue failure is signaled by cracks developing at points of high stress concentration. The magnitude of stress depends on vehicle weight and the size of the bridge component. Generally, only steel bridges are susceptible to fatigue, although some studies suggest that commonly used prestressed concrete spans, if overloaded, are also susceptible to fatigue damage. 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 (Fisher 1977). The consequences of a fatigue failure in steel bridges depend on whether there are multiple load paths.

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. AASHTO specifications give different allowable fatigue stresses for different categories of detail. Moses (1989) notes that these fatigue rules were only initiated in the mid-1960's, so many older bridges were never checked during their original design for fatigue life. He notes further that the AASHTO fatigue rules apply to welded and bolted details with stresses induced directly by load passages. 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 this category of distortion induced. Some of the worst detailing can be removed by repair and retrofit.

The literature includes somewhat conflicting assessments regarding the effects of truck traffic on bridge decks. James (1987) found that the effects of overloads on decks are the most significant manifestation of truck-related damage to concrete bridges, that "the most important deck damage mechanisms are transverse and longitudinal cracking", that "reinforced concrete decks on steel I-beams are more susceptible to damage of this type than are decks on prestressed girders", and that "corrosion of reinforcement is intensified by the increased cracking caused by overloaded vehicles, and spalling of concrete cover resulting from reinforcing steel corrosion is certainly accelerated by traffic".

James' conclusions regarding the significance of damage to bridge decks are not accepted by many bridge engineers. In discussing bridge life in relation to cyclic traffic loadings, Moses (1989) notes that "there is a considerable factor of safety in decks", that "the multi-billion dollar deck replacement program is mostly related to environmental damage (i.e., salt) which corrodes deck steel", and that "the schedule for deck replacement is usually not affected by deck loading". Several agencies (including Ontario and New York) are in fact reducing the amount of steel reinforcing in the deck to improve durability. This indicates that environmental factors including salt are a major problem in bridge decks. Procedures for correcting deck problems include using less steel, better protection for the steel such as epoxy coating or galvanizing, waterproof membranes, denser concrete, and better construction control. Thus, while traffic loadings may have some effect on bridge deck durability, environmental factors are also of concern.

1.2 Truck Characteristics Affecting Bridges

Bridge stresses caused by vehicles depend on both the gross weight of the vehicle and the length over which this weight is distributed. Highly concentrated loads generally result in greater stresses. The length of a truck relative to the length of bridge spans is also important. For relatively short spans (20 to 40 feet), not all axles of a combination will be on the bridge at the same time.

Exhibit 1 shows maximum bending moments (which determines stresses in the main load-carrying members of simple-span bridges) by span length 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.

Dynamic effects can also be important, particularly for bridges carrying trucks operating at higher speeds. In bridge design, the static weight of design loadings are adjusted upward to account for dynamic effects such as a vehicle bouncing on it's springs because it is traveling on rough pavement or a vertical curve. When extra-heavy loads are carried across bridges under special indivisible load permits, a frequent condition of such permits is that the truck cross the bridge at crawl speed to minimize dynamic effects.

1.3 Bridge Formula

A 1964 study by the Secretary of Commerce on the "Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid System" recommended a table of maximum weights for axle groups to protect bridges (Exhibit 2). The values in the table can be derived from the following formula, which is known as Bridge Formula B:

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

where:

- W is the maximum weight in pounds carried on any group of two or more consecutive axles
- L is the distance in feet between the extremes of the axle group
- N is the number of axles in the axle group

Exhibit 1 Maximum Bending Moments on a Simple Span For 50,000 lb. Single Unit Truck and 80,000 lb. Combination Truck

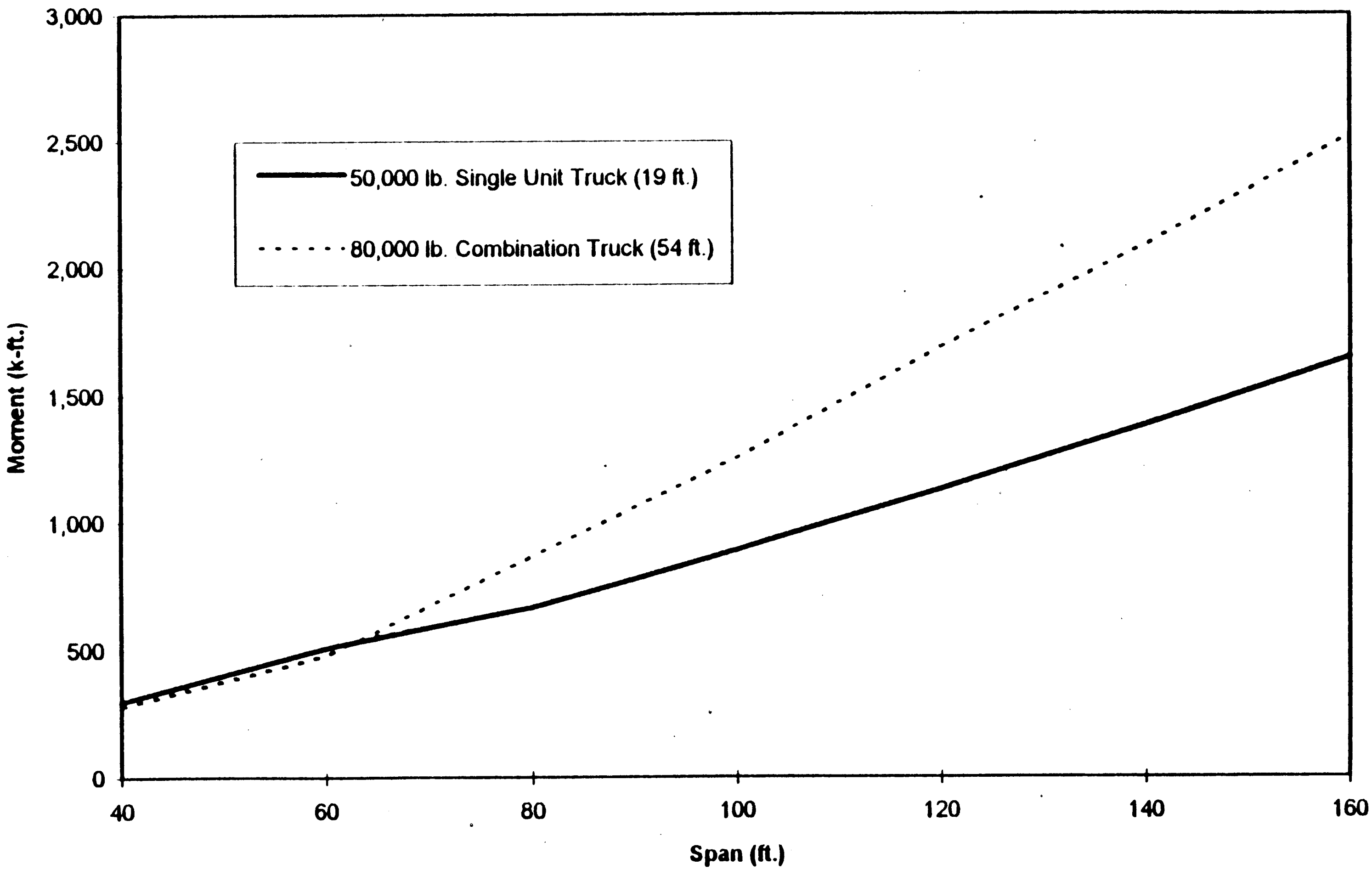

Exhibit 2: Permissible Gross Loads Recommended by Secretary of Commerce For Vehicles in Regular Operation (U.S. Secretary of Commerce 1964)

Distance Between Extremes of Any Group of 2 or More Consecutive Axles (ft)	Maximum Load (lb) by No. of Axle Groups *							
	2	3	4	5	6	7	8	9
4	34,000							
5	34,000							
6	34,000							
7	34,000							
8	34,000	42,000						
9	39,000	42,500						
10	40,000	43,500						
11		44,000						
12		45,000	50,000					
13		45,500	50,500					
14		46,500	51,500					
15		47,000	52,000					
16		48,000	52,500	58,000				
17		48,500	53,500	58,500				
18		49,500	54,000	59,000				
19		50,000	54,500	60,000				
20		51,000	55,500	60,500	66,000			
21		51,500	56,000	61,000	66,500			
22		52,500	56,500	61,500	67,000			
23		53,000	57,500	62,500	68,000			
24		54,000	58,000	63,000	68,500	74,000		
25		54,500	58,500	64,500	69,000	74,500		
26		55,500	59,500	65,000	69,500	75,000		
27		56,000	60,000	65,000	7,000	75,500		
28		57,000	60,500	65,500	71,000	76,500	82,000	
29		57,500	61,500	66,000	71,500	77,000	92,500	
30		58,500	62,000	66,500	72,000	77,500	83,000	
31		59,000	62,500	67,500	72,500	78,000	83,500	
32		60,000	63,500	68,000	73,000	78,500	84,500	90,000
33			64,000	68,500	74,000	79,000	85,000	90,500
34			64,500	69,000	74,500	80,000	85,500	91,000
35			65,500	70,000	75,000	80,500	86,000	91,500
36			66,000	70,500	75,500	81,000	86,500	92,000
37			66,500	71,000	76,000	81,500	87,000	93,000
38			67,500	72,000	77,000	82,000	87,500	93,500
39			68,000	72,500	77,500	82,500	88,500	94,000
40			68,500	73,000	78,000	83,500	89,000	94,500
41			69,500	73,500	78,500	84,000	89,500	95,000
42			70,000	74,000	79,000	84,500	90,000	95,500
43			70,500	75,000	80,000	85,000	90,500	96,000
44			71,500	7,500	80,500	85,500	91,000	96,500
45			72,000	76,000	81,000	86,000	91,500	97,500
46			72,500	76,500	81,500	87,000	92,500	98,000
47			73,500	77,500	82,000	87,500	93,000	98,500
48			74,000	78,000	83,000	88,000	93,500	99,000
49			74,500	78,500	83,500	88,500	94,000	99,500
50			75,500	79,000	84,000	89,000	94,500	100,000
51			76,000	80,000	84,500	89,500	95,000	100,500
52			76,500	80,500	85,000	90,500	95,500	101,000
53			77,500	82,000	86,000	91,000	96,500	102,000
54			78,000	81,500	86,500	91,500	97,000	102,500
55			78,500	82,500	87,500	92,000	97,500	103,000
56			79,500	83,000	87,500	92,500	98,000	103,500
57			80,000	83,500	88,000	93,000	98,500	104,000
58				84,000	89,000	94,000	99,000	104,500
59				85,000	89,500	94,500	99,500	105,000
60				85,500	90,000	95,000	100,500	105,500

Note: The weights in this table are based on tthe formula $W = 500(LN/(N-1)+12N+36)$, modified. Ther permisssible loads are computed to the nearest 500 lb. The modification consists in limiting the maximum load on any single axle to 20,000 lb.

* The following loaded vehicles must not operate over H15-44 bridges: 3-S2 (five axles) with wheelbase less than 38ft; 2-S1-2 (five axles) with wheelbase less than 45 ft; 3-3 (six axles) with wheelbase less than 45 ft; and seven-, eight-, and nine-axle vehicles regardless of wheelbase.

In 1974, when Congress increased the limit on gross weight to 80,000 pounds and the limits on single and tandem axles to 20,000 and 34,000 pounds, it also adopted Formula B.

Formula B is based on assumptions about the amount by which the design loading can be safely exceeded for different types of bridges. Specifically, this formula was designed to avoid overstressing HS-20[1] bridges by more than 5 percent and H-15[2] bridges by more than 30 percent. According to FHWA, overstressing an H-15 bridge in good condition by up to 30 percent should be safe, although the fatigue lives of these structures may be shortened by repeated loadings at this level. However, FHWA has taken the position that because of the nation's large investment in HS-20 bridges, and because these bridges carry high volumes of truck traffic, design stresses for these bridges should not be exceeded by more than 5 percent.

Formula B reflects the fact that increasing the spacing between axles generally results in less concentrated loads and lower stresses in bridge members. For example, the bridge formula would allow a three-axle truck with a wheelbase of 20 feet to operate at 51,000 pounds. If the wheelbase of this truck is increased to 24 feet, then the maximum weight allowed under the bridge formula would increase to 54,000 pounds. The bridge formula also allows more weight to be carried if the number of axles is increased. For example, if a fourth axle is added to a three-axle truck with a wheelbase of 20 feet, the maximum weight allowed under the bridge formula is increased from 51,000 to 55,500 pounds. Notwithstanding the presence of the variable N in Formula B, increasing the number of axles in an axle group without increasing the overall length of the group has very little effect on bridge stresses. In fact, to the extent that increasing the number of axles does affect bridge stresses, it is likely to increase them slightly by making the load more concentrated. However, increasing the number of axles on a truck does provide substantial benefits to pavements.

A less conservative bridge formula would reduce the margin of safety, thereby increasing the likelihood of bridge damage due to overstress. Since illegal overloads are often associated with damage due to overstress, some of the

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[1]HS-20 is the minimum design load recommended by AASHTO for bridges on Interstate highways. This loading is based on a hypothetical vehicle with one 8,000 pound axle and two 32,000 pound axles.

[2]The H-15 bridge is based on a hypothetical vehicle with one 6,000 pound axle and one 24,000 pound axle. This design load has been used for many non-Interstate bridges.

reduction in the margin of safety for bridges could be offset by better enforcement of truck weight limits.

In 1985, the Texas Transportation Institute (TTI) completed a bridge formula study for FHWA (Noel 1985). TTI had been asked to develop a new bridge formula based on the same overstress criteria as the current formula, which meant not overstressing HS-20 bridges by more than 5 percent and H-15 bridges by more than 30 percent. The primary motivation for this study was the fact that when the current formula is applied to certain vehicles, the overstress criteria on which it is nominally based can be exceeded.

The formula recommended by the TTI study was as follows:

$$W = 1{,}000\ (L + 34) \qquad \text{for } L < 56 \text{ feet}$$

$$W = 1{,}000\ (L / 2 + 62) \qquad \text{for } L > 56 \text{ feet}$$

where:

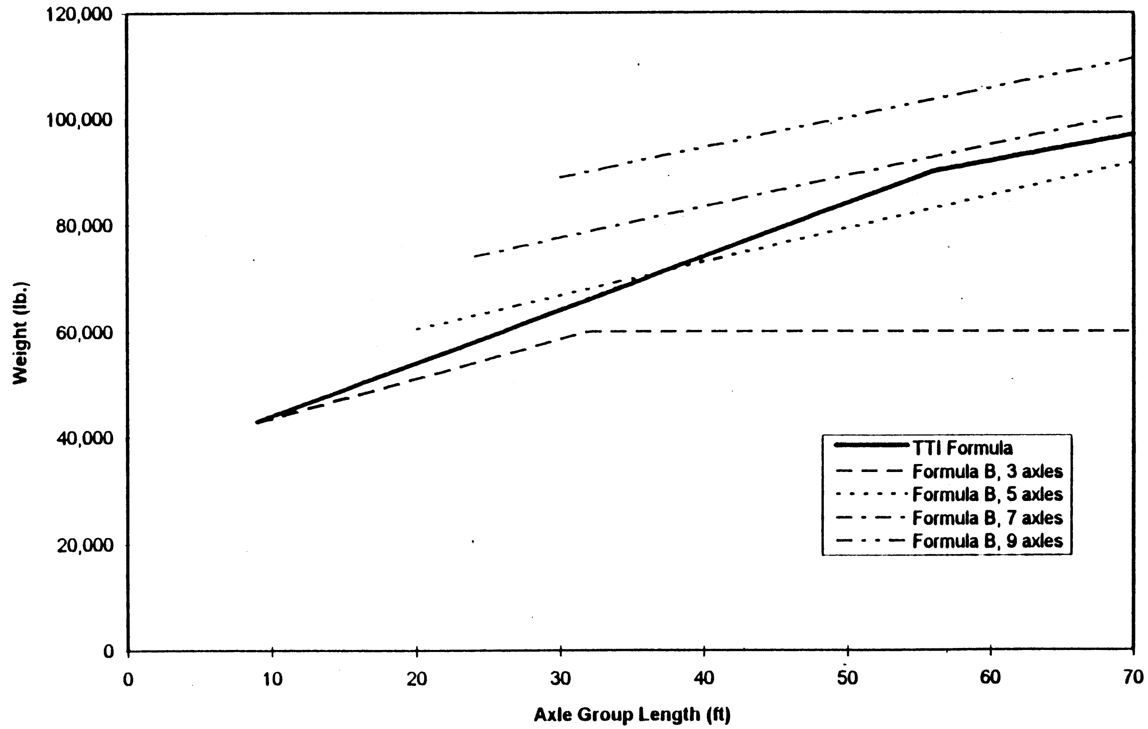
- W is the maximum weight (in pounds) permitted on a group of two or more consecutive axles

- L is the length of the axle group (in feet)

The TTI formula and Formula B are compared in Exhibit 3. The TTI formula generally allows slightly higher weights on single-unit trucks and short combinations. When applied to vehicles with more than six axles, however, the TTI formula is less permissive than the current formula. Both the TTI formula and Formula B considered simple span bridges only. Continuous spans with critical loadings at intermediate supports will behave and respond differently to increases in truck weights. Most designs on Interstates and most recent construction use continuous spans.

Comments from the trucking industry on the original TTI formula were generally negative. Operators of short-wheelbase trucks opposed the TTI formula because, although it generally allowed them slightly higher weights, they believed it to be still overly cautious. The TTI formula was also opposed by truckers in western states who currently operate longer combinations over 80,000 pounds under grandfather exemptions, because for these longer combinations, the TTI formula is more restrictive than the current formula. For example, the TTI formula would restrict combinations with a wheelbase of 65 feet to 94,500 pounds or less. Under Bridge Formula B, combinations with a 65-foot wheelbase can operate at 98,000 pounds with seven axles, 103,000 pounds with eight axles, and 108,500 pounds with nine axles.

Exhibit 3 TTI Bridge Formula Versus Formula B: Maximum Weight for Axle Groups



TTI also developed a modified version of their recommended formula (James 1986). The modified formula, referred to as the TTI HS-20 formula, keeps the 5 percent criterion for HS-20 bridges but drops the 30 percent criterion for H-15 bridges. The TTI HS-20 formula is as follows:

$$W = 1{,}000\ (\ 2\ L + 26\ ) \qquad \text{for } L < 24 \text{ feet}$$

$$W = 1{,}000\ (\ L / 2 + 62\ ) \qquad \text{for } L > 24 \text{ feet}$$

where W and L are as defined above. The current formula and the TTI HS-20 formula are compared in Exhibit 4. The TTI HS-20 formula allows much higher weights on shorter trucks than either the current bridge formula or the original TTI formula. However, since it is identical to the original TTI formula for lengths over 56 feet, it would reduce maximum weights permitted on combinations with more than six axles.

TRB Special Report 225 (1990) developed a table of maximum axle group weights under which the TTI HS-20 formula would be applied to vehicles weighing 80,000 pounds or less, and the current bridge formula would be applied to vehicles weighing over 80,000 pounds (Exhibit 5 and 6). These limits would increase maximum weights for shorter vehicles, but would leave unchanged the maximum weights for longer combinations.

The bridge formula has been criticized as overly conservative and critics often cite the experience of the province of Ontario, the state of Michigan, etc. that allow higher loads than permitted by the current federal legislation for bridges designed based on the same AASHTO code. The use of the arbitrary overstress ratios in the original and proposed formulas has also been widely criticized (Ghosn and Moses). These ratios do not seem to consider increased damage due to repeated load applications, the state of deterioration of existing bridges, or the likelihood of overloads and simultaneous truck presence. Ghosn and Moses argue for an approach based on structural reliability theory. The aim of such an approach is to obtain the overstress ratios using statistical data on bridge safety. The steps involved in such an analysis would be based on determining acceptable safety levels using statistics on the safety margins of typical bridges including the likelihood of overloads, simultaneous truck presence, impact allowance, girder distribution and component deterioration or loss of serviceability. New safety criteria could then be developed on the basis of limiting the number of posted bridges based on traffic and funds available for rehabilitation.

Exhibit 4 TTI HS-20 Bridge Formula Versus Formula B: Maximum Weight for Axle Groups

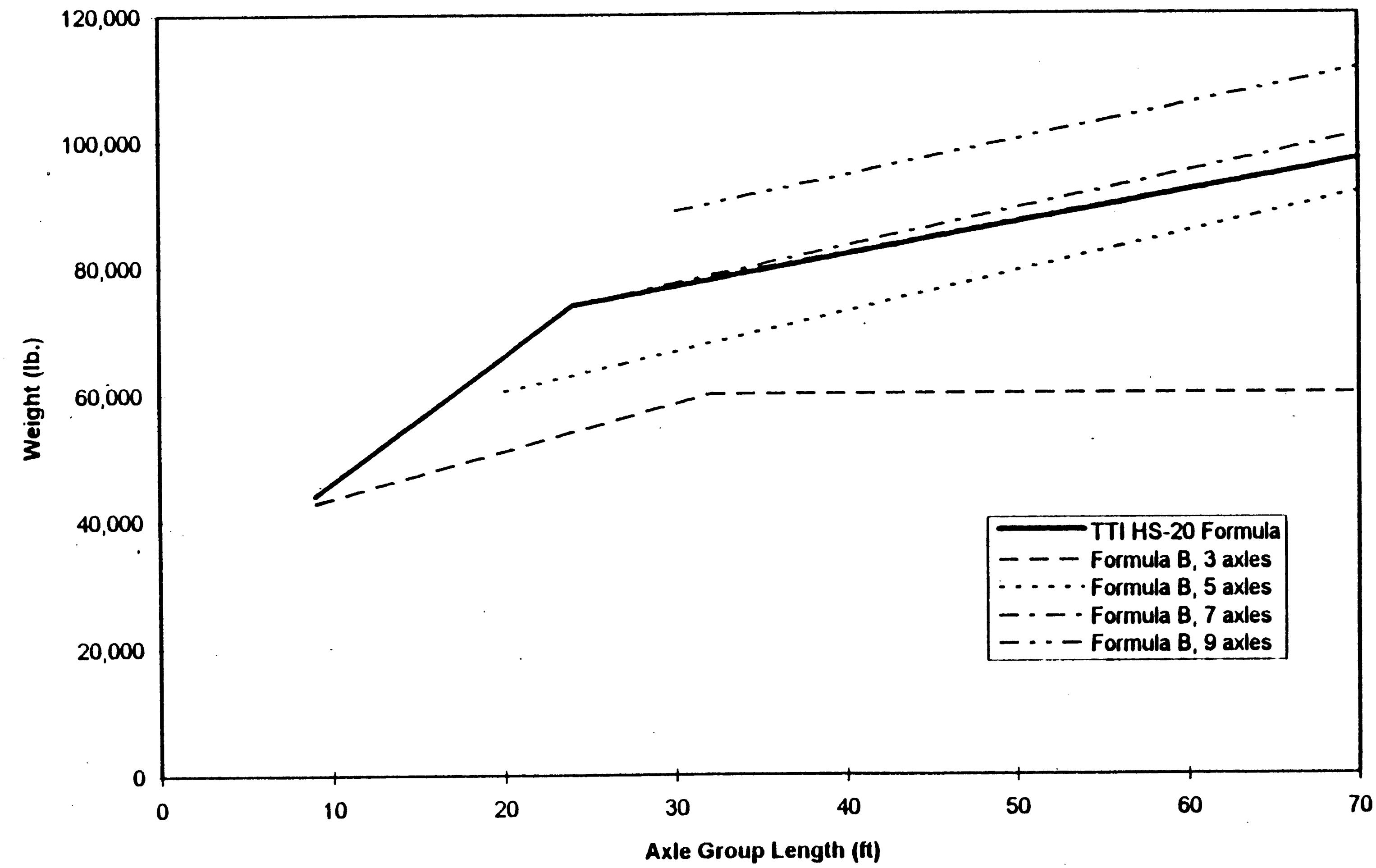


Exhibit 5 Combined TTI HS-20/Formula B: Maximum Weight for Axle Groups

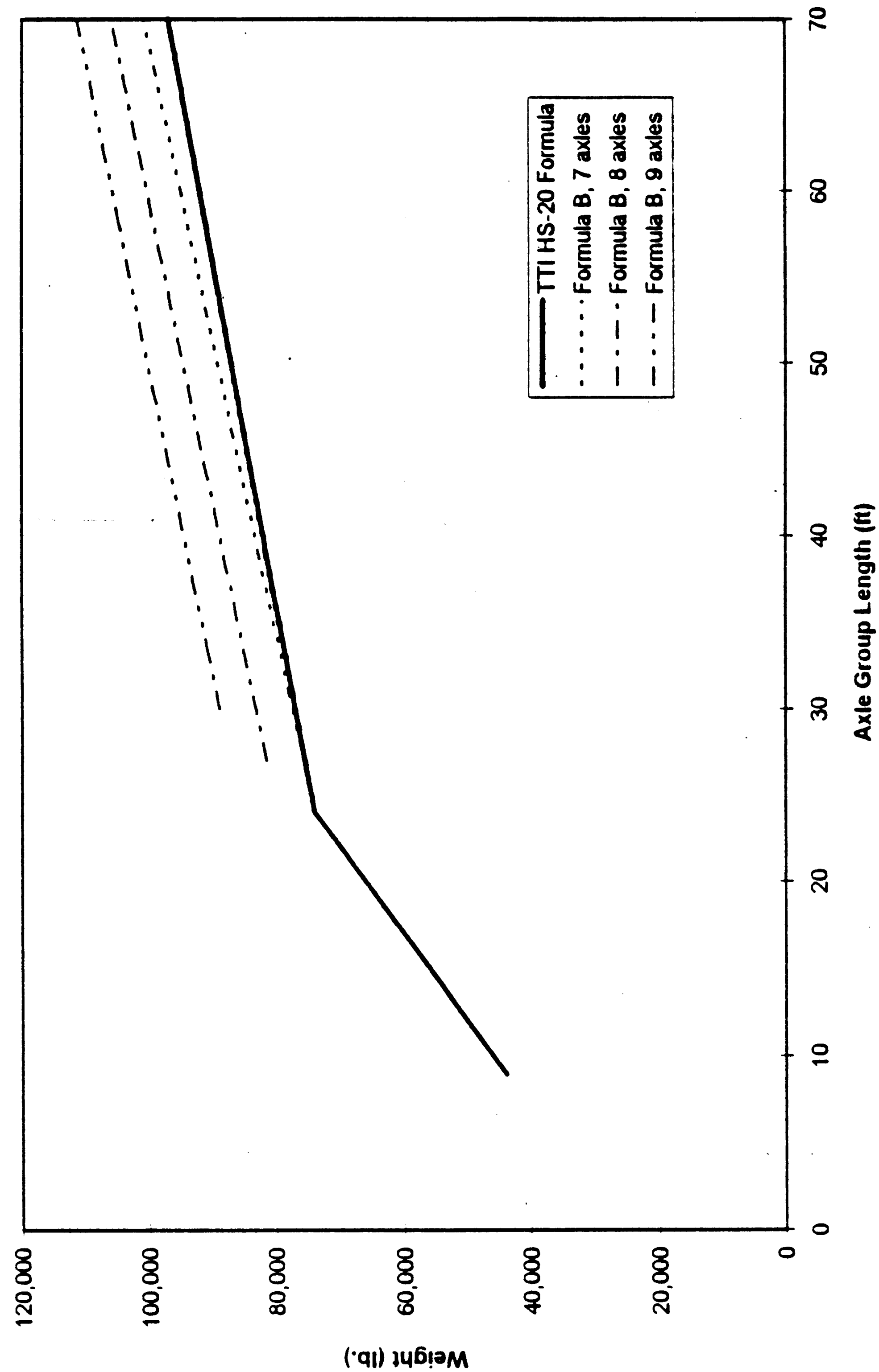


Exhibit 6 Combined TTI HS-20/Formula B Scenario: Maximum Weight For Axle Groups

Axle-Group Length (ft.)	Maximum Weight (kip 3 or More Axles	Axle-Group Length (ft.)	Maximum Weight (kips) 5-6 Axles	7 Axles	8 Axles	9 or More Axles	Axle-Group Length (ft.)	Maximum Weight (kips) 5-6 Axles	7 Axles	8 Axles	9 or More Axles
8	42.0	37	80.5	81.5	87.0	93.0	72	98.0	102.0	107.0	112.5
9	44.0	38	81.0	82.0	87.5	93.5	73	98.5	102.5	107.5	113.0
10	46.0	39	81.5	83.0	88.5	94.0	74	99.0	103.0	108.5	113.5
11	48.0	40	82.0	83.5	89.0	94.5	75	99.5	104.0	109.0	114.0
12	50.0	41	82.5	84.0	89.5	95.0	76	100.0	104.5	109.5	115.0
13	52.0	42	83.0	84.5	90.0	95.5	77	100.5	105.0	110.0	115.5
14	54.0	43	83.5	85.0	90.5	96.0	78	101.0	105.5	110.5	116.0
15	56.0	44	84.0	85.5	91.0	97.0	79	101.5	106.0	111.0	116.5
16	58.0	45	84.5	86.5	91.5	97.5	80	102.0	106.5	111.5	117.0
17	60.0	46	85.0	87.0	92.5	98.0	81	102.5	107.5	112.5	117.5
18	62.0	47	85.5	87.5	93.0	98.5	82	103.0	108.0	113.0	118.0
19	64.0	48	86.0	88.0	93.5	99.0	83	103.5	108.5	113.5	118.5
20	66.0	49	86.5	88.5	94.0	99.5	84	104.0	109.0	114.0	119.5
21	68.0	50	87.0	89.0	94.5	100.0	85	104.5	109.5	114.5	120.0
22	70.0	51	87.5	90.0	95.0	100.5	86	105.0	110.0	115.0	120.5
23	72.0	52	88.0	90.5	95.5	101.5	87	105.5	111.0	115.5	121.0
24	74.0	53	88.5	91.0	96.5	102.0	88	106.0	111.5	116.5	121.5
25	74.5	54	89.0	91.5	97.0	102.5	89	106.5	112.0	117.0	122.0
26	75.0	55	89.5	92.0	97.5	103.0	90	107.0	112.5	117.5	122.5
27	75.5	56	90.0	92.5	98.0	103.5	91	106.5	113.0	118.0	123.0
28	76.0	57	90.5	93.5	98.5	104.0	92	108.0	113.5	118.5	124.0
29	76.5	58	91.0	94.0	99.0	104.5	93	108.5	114.5	119.0	124.5
30	77.0	59	91.5	94.5	99.5	105.0	94	109.0	115.0	119.5	125.0
31	77.5	60	92.0	95.0	100.5	106.0	95	109.5	115.5	120.5	125.5
32	78.0	61	92.5	95.5	101.0	106.5	96	110.0	116.0	121.0	126.0
33	78.5	62	93.0	96.0	101.5	107.0	97	110.5	116.5	121.5	126.5
34	79.0	63	93.5	97.0	102.0	107.5	98	111.0	117.0	122.0	127.0
35	79.5	64	94.0	97.5	102.5	108.0	99	111.5	118.0	122.5	127.5
36	80.0	65	94.5	98.0	103.0	108.5	100	112.0	118.5	123.0	128.5
		66	95.0	98.5	103.5	109.0	101	112.5	119.0	123.5	129.0
		67	95.5	99.0	104.5	109.5	102	113.0	119.5	124.5	129.5
		68	96.0	99.5	105.0	110.5	103	113.5	120.0	125.0	130.0
		69	96.5	100.5	105.5	111.0	104	114.0	120.5	125.5	130.5
		70	97.0	101.0	106.0	111.5	105	114.5	121.5	126.0	131.0
		71	97.5	101.5	106.5	112.0					

Note: Axle Group Length is the distance between the extremes of any group of three or more consecuttive axles. Maximum weights over 80,000 bl. are permitted only under special permits on disignated routes. All vehicles are also subject to a single-axle limit of 20 kips and a tandem-axle limit of 34 kips.

The steps involved in applying reliability procedures to obtain a bridge formula were summarized by Ghosn and Moses as follows:

1. "Choose suitable safety criteria. The safety index widely used in structural reliability theory as a measure of structural safety is used in this study as the basis for the determination of the safety of bridge members.

2. Select an acceptable reliability level. For example, a safety index of 2.5 for redundant bridges seems to provide a reasonable safety target based on the performance of existing bridge members. This safety index target of 2.5 corresponds to a probability of about 0.6 percent that the safety and serviceability criteria will be satisfied in any member of the bridge.

3. Choose a range of typical bridges with different design criteria, span lengths, and configurations giving a preventative sample of the nation's bridges. These bridges should include simple as well as continuous spans, both steel and concrete bridges should be considered. In this study, simple span steel bridges were used to obtain a bridge formula. The implications in terms of safety and cost of other types of bridges are studied separately as part of the cost analysis.

4. Use statistics on the safety margins of these typical bridges including the likelihood of overloads and simultaneous truck occurrences to obtain the live load envelope that will produce the target safety index. It will be assumed that the uncertainties (C.O.V.) of the live load random variables will remain the same as currently observed. The live load envelope as defined herein is the mean total bridge live load required to produce the target safety index for each span length.

5. Calibrate a bridge formula that will produce the load envelope obtained in step 4.

6. Verify that the bridge formula will lead to an acceptable number of bridges that will need upgrading to support the proposed additional load.

7. Review the implications of adopting the suggested bridge formula in terms of safety of typical steel and concrete bridges of simple and continuous spans. This should include strength requirements and fatigue and serviceability.

8. Study the costs required to maintain the bridge infrastructure under the proposed bridge formula."

Ghosn and Moses applied this approach to develop a new bridge formula such that a simply-supported steel bridge designed to satisfy AASHTO's WSD criteria for HS-20 loading will have a safety index beta = 2.5 when subjected to the loads expected over the next 50 years if the new bridge formula is implemented. The projections of the loads is based on the assumption that a lateral shift of the gross weight histograms accompanies any shift in the legal limit. With this assumption, the number of illegal and overloaded vehicles as a percentage of the total traffic remains unchanged from the present situation. Also, to account for future increases in truck traffic and the more frequent number of multiple truck occurrences caused by that, a traffic growth factor of 1.15 is included in the maximum load model. The proposed formula obtained by Ghosn and Moses under the assumptions outlined above was as follows:

$$W = (1.64\ L + 30)\ 1000 \qquad \text{for } L < 50 \text{ ft}$$

$$W = (0.80\ L + 72)\ 1000 \qquad \text{for } L > 50 \text{ ft}$$

The proposed formula is considerably more permissive than Bridge Formula B when applied to longer combination vehicles. For example, a nine-axle double with a wheelbase of 65 feet is limited to 108,500 pounds under Bridge Formula B; however, this vehicle could operate at 124,000 pounds under the proposed formula.

Ghosn and Moses estimated the number of existing bridges that would have to be replaced if the proposed formula were implemented. These estimates, and related cost implications, are discussed in the next section of this paper, which deals with bridge costs.

In 1988, the Freightliner Corporation made a proposal that would exempt steering axles when applying the bridge formula to combinations. The Freightliner Proposal would make setback steering axles more practical by eliminating the distance from the steering axle to the second axle of a combination as a consideration in how much weight the combination could carry. Benefits of tractors with setback steering axles include better aerodynamics, improved maneuverability due to a shorter wheelbase and sharper steering angles, more load-carrying capacity, easier cab entrance and exit for cab-over-engine configurations, and improved frontal energy absorption in collisions because of the added space between the front bumper and steering axles.

Steering-axle weights for most combinations are limited by practical considerations to about 10,000 pounds. Under the Freightliner Proposal, steering-axle weights could exceed 14,000 pounds and, at least theoretically, approach the 20,000-pound limit for single axles. A large increase in steering axle weights would adversely affect pavements, particularly because steering axles have single tires and are more damaging to pavements than axles with dual tires that carry the same weight. Also, large increases in steering axle weights might increase bridge costs, unless there are compensatory reductions in the weights of other axles. TRB Special Report 225 (1990) suggested that the problem of overloaded steering axles might be circumvented by adding a special weight limit for steering axles to the Freightliner Proposal. A 12,000-pound steering axle limit together with the 34,000 pounds allowed on each of two sets of tandem axles could allow five-axle combinations to reach 80,000 pounds. However, certain operators (notably automobile haulers) currently operate with front-axle loads of 14,000 pounds, and adding a 12,000 pound steering axle limit would hurt these operators.

1.4 Estimates of Bridge Costs

This section provides estimates from recent studies of changes in bridge costs associated with changes in truck size and weight limits.

TRB Special Report 225 (1990) developed estimates of additional bridge costs for the following truck weight limit increase scenarios:

- Uncapped Formula B: Elimination of the 80,000-pound cap on gross vehicle weight, so that it is controlled only by the current federal bridge formula

- NTWAC Proposal[1]: Permit program that would allow significantly higher weights for specialized hauling vehicles (SHVs) with short wheelbases. Under this scenario, SHVs could travel under special permit at weights and configurations up to those that would exceed the operating rating of an HS-20 bridge. Examples include: (1) a three-axle truck with a wheel base of 16 feet weighing 80,000 pounds, (2) a four-axle truck with a wheelbase of 22 feet weighing 85,000 pounds, and (3) a five-axle tractor-semitrailer with a wheelbase of 36 feet weighing 110,000 pounds. These weights are far in excess of the weights that would be allowed for these configurations under current axle weight limits and Bridge Formula B.

[1]The National Truck Weight Advisory Council (NTWAC) is an organization representing industries involved in hauling of heavy items such as construction materials, solid waste, forest products, scrap iron, paper products, bulk materials, and building materials and supplies.

- Canadian Interprovincial Limits: Proposal calling for higher gross weights and minimum axle spacings instead of a bridge formula. This scenario assumed that current U.S. axle limits (20,000 pounds for single axles and 34,000 pounds for tandems) would remain in effect, but that the 80,000 cap on gross vehicle weight would be eliminated, and that Bridge Formula B would be replaced by Canadian rules regarding minimum axle spacings. The Canadian rules are much more permissive than Bridge Formula B. For example, an eight-axle double with a wheelbase of 75 feet, which is limited to 109,000 pounds under Bridge Formula B, could operate at 131,000 pounds under the Canadian Interprovincial Limits Scenario.

- TTI HS-20 Bridge Formula: Modified version of the TTI formula that would allow higher weights on single-unit trucks and shorter combination vehicles

- Uncapped TTI HS-20 Bridge Formula: Same as the preceding scenario except that the 80,000 cap on GVW is also eliminated

- Combined Uncapped TTI HS-20/Formula B: Scenario under which shorter vehicles could take advantage of the TTI HS-20 formula while longer vehicles could take advantage of the higher weights allowed to them under Formula B.

Vehicle loadings used to estimate bridge costs for the scenarios are shown in Exhibit 7.

Bridge costs for the scenarios were developed at the national level in three categories:

- Upgraded design loads for new bridges: An estimate was made of how states would be likely to change the loadings they use in design if weight limits are changed. The percentage change in costs for prototype bridges under current and alternative design loads was then calculated and applied to estimates of annual expenditures for new bridges under the assumption of no changes in truck weight limits.

Exhibit 7 Vehicle Loadings for Truck Weight Study Scenarios

		1st Axle	2nd Axle	3rd Axle	4th Axle	5th Axle	6th Axle	7th Axle	8th Axle	9th Axle	Total Length (ft)	Total Weight (Kips)
NTWAC												
1A)	Weight	20.8	29.6	29.6	-	-	-	-	-	-	16'	50
	Spacing		(11')	(5')								
1B)	Weight	20.95	21.35	21.35	21.35	-	-	-	-	-	22'	82
	Spacing		(12')	(5')	(5')							
1C)	Weight	14	25.1	25.1	22.9	22.9	-	-	-	-	36'	110
	Spacing		(11')	(5)	(15')	(5')						
Canadian Interprovincial Limits												
2A)	Weight	20	17	17	-	-	-	-	-	-	16'	54
	Spacing		(12')	(5')								
2B)	Weight	20	17	17	17	-	-	-	-	-	22'	71
	Spacing		(12')	(5')	(5')							
2C)	Weight	12	17	17	17	17	-	-	-	-	36'	80
	Spacing		(11')	(5')	(15')	(5')						
2D)	Weight	12	17	17	17	17	17	17	17	-	65'	131
	Spacing		(13')	(4')	(18')	(4')	(4')	(18')	(4')			
Uncapped Formula B												
3A)	Weight	12	14	14	12	12	11	11	11	11	65'	108
	Spacing		(13')	(4')	(12')	(4')	(8')	(4')	(14')	(4')		

Exhibit 7 Vehicle Loadings for Truck Weight Study Scenarios (Continued)

		1st Axle	2nd Axle	3rd Axle	4th Axle	5th Axle	6th Axle	7th Axle	8th Axle	9th Axle	Total Length (ft)	Total Weight (Kips)
Modified TTI Formula with 80 Kip Limit												
4A)	Weight	20	17	17	-	-	-	-	-	-	16'	54
	Spacing		(11')	(5')								
4B)	Weight	20	13.3	13.3	15.3	-	-	-	-	-	22'	66
	Spacing		(12')	(5')	(5')							
4C)	Weight	12	17	17	17	17	-	-	-	-	36'	80
	Spacing		(11')	(5)	(15')	(5')						
Modified TTI Formula without 80 Kip Limit												
5A)	Weight	20	17	17	-	-	-	-	-	-	16'	54
	Spacing		(11')	(5')								
5B)	Weight	20	15.3	15.3	15.3	-	-	-	-	-	22'	66
	Spacing		(12')	(5')	(5')							
5C)	Weight	12	17	17	17	17	-	-	-	-	36'	80
	Spacing		(11')	(5')	(15')	(5')						
5D)	Weight	11	14	14	12	12	16	16	-	-	66'	95
	Spacing		(13')	(4')	(15')	(4')	(8')	(22')				

Exhibit 7 Vehicle Loadings for Truck Weight Study Scenarios (Continued)

		1st Axle		2nd Axle		3rd Axle		4th Axle		5th Axle		6th Axle		7th Axle		8th Axle		9th Axle	Total Length (ft)	Total Weight (Kips)
Combined TTI HS-20 Formula B																				
6A)	Weight	20		17		17		-		-		-		-		-		-	16'	54
	Spacing		(11')		(5')															
6B)	Weight	20		15.3		15.3		15.3		-		-		-		-		-	22'	66
	Spacing		(12')		(5')		(5')													
6C)	Weight	12		17		17		17		17		-		-		-		-	36'	80
	Spacing		(11')		(5')		(15')		(5')											
6D)	Weight	11		14		14		12		12		16		16		-		-	66'	95
	Spacing		(13')		(4')		(15')		(4')		(8')		(22')							
6E)	Weight	12		14		14		12		12		11		11		11		11	65'	108
	Spacing		(13')		(4')		(12')		(4')		(8')		(4')		(14')		(4')			

- Replacement of existing bridges that could not accommodate the heavier loadings: Replacement costs for existing bridges on primary[1] and nonprimary highways were estimated by: (1) specifying worst-case legal loadings under the base case (no changes in truck weight limits) and each alternative scenario, (2) compiling information on the load-carrying capacities of existing bridges from the National Bridge Inventory maintained by FHWA, (3) identifying load-deficient bridges under the base case and each alternative scenario, and (4) calculating the cost to replace load-deficient bridges. To estimate bridge cost impacts, the bridge replacement costs for the base case were subtracted from the bridge replacement costs for each scenario. A bridge was considered deficient in this study if legal loadings cause stresses that exceed the operating rating (discussed below) plus a 5 percent tolerance on the rating load. The 5 percent tolerance was added because many agencies do not post bridges for loads that are within 5 percent of legal weights.

- Fatigue costs: Fatigue costs for existing bridges were estimated using projections of truck traffic by vehicle type for the base case (no change in truck weight limits) and the alternative scenario. A fatigue damage cost model, which accounts for the relative impact of different truck types on fatigue, was then applied.

Exhibit 8 presents annual bridge costs in 1988 dollars for each of the scenarios. A seven percent discount rate was assumed in annualizing one-time costs. The costs shown in Exhibit 8 are bridge-related costs to highway agencies. TRB Special Report 225 noted that there would also be costs to highway users as a result of delays during reconstruction. However, numerical estimates of these costs were not presented in the report. Delay costs during bridge replacement can be considerable, particularly for bridges that carry high traffic volumes or bridges that must be completely closed during replacement. If practical, numerical estimates of delay costs due to bridge replacements should be included in future analyses of the effects of changes in truck size and weight limits on bridges.

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[1]The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 eliminated the "Federal Aid Primary System" and created the "National Highway System". According to *Highway Statistics 1991*, the Primary System (including Interstates) was 305,200 miles. According to *Highway Statistics 1992*, the Interim National Highway System (also including Interstates) is 200,900 miles, nearly all of which were on the now-eliminated Primary System.

Exhibit 8 National Bridge Costs for Truck Weight Study Scenarios

	Combined TTI HS-20 Formula B Scenario	Uncapped TTI HS-20 Bridge Form. Scenario	TTI HS-20 Bridge Form. Scenario	Canadian Interprovincial Limit Scenario	NTWAC Scenario	Uncapped Formula B Scenario
Bridge Costs Per Year ($ Millions)						
Upgraded Design Loads for New Bridges	150	90	60	310	280	150
Replacement of Load-deficient Bridges						
Interstates and Other Primary Highways	300	100	70	960	1,240	270
Nonprimary Highways	430	230	200	1,070	1,510	240
Fatigue Related	20	20	20	70	10	20
Total	900	440	350	2,410	3,040	680

Source: Transportation Research Board, (1990) *Truck Weight Limits: Issues and Options*.

Estimates of bridge cost impacts of truck size and weight limit changes are very sensitive to assumptions regarding acceptable levels of stress in bridges. The majority of states use an "operating rating" criterion under which the stress in bridge members is not allowed to exceed 75 percent of the level of stress at which the member would undergo permanent deformation or "yield". However, some states use the conservative "inventory rating" criterion under which stress is not allowed to exceed 55 percent of yield stress. Also, a few states use criteria which falls between the inventory and operating ratings.

The Association of American Railroads sponsored a 1991 study using the inventory rating criterion to project the impact of longer combination vehicles (LCVs) on Interstate System bridges. Also in 1991, FHWA used an intermediate criterion (65 percent of yield stress) to estimate bridge costs due to LCVs. Both the AAR and FHWA analyses used data on the load-carrying capacity of bridges from the National Bridge Inventory. These data are collected by states and then compiled into a single national data set by FHWA.

In 1993, the General Accounting Office (GAO) asked FHWA to update its analysis and to also estimate bridge costs for current trucks and LCVs using the inventory and operating rating criteria. FHWA was able to develop results for the inventory rating and for the intermediate criterion, but not for the operating rating.[1] These results, which are shown in Exhibit 9, demonstrate the sensitivity of replacement cost estimates to bridge rating criteria. The costs provided in this exhibit are total costs to replace load-deficient bridges (not annual costs).

Mohammedi (1991) examined fatigue-related effects on bridges in Illinois due to an increase in maximum gross vehicle weights from 73,280 to 80,000 pounds. They examined 15 sample bridges on a designated truck route system. These bridges represent 1,059 older bridges in Illinois with limited load-carrying capacity. The 15 sample bridges were all made of steel girders with reinforced concrete deck slabs. Estimates of shortened fatigue life for the 15 sample bridges were used to predict future bridge costs. The report provides cost estimates ranging from $6.7 to $30.0 million per year (however, the latter figure is based on the assumption that there would be traffic growth with the 80,000- pound limit but no growth with the 73,280 limit). Also, the report contains conceptual errors in the application of a discount rate to calculate annual cost.

[1]GAO (1993) states that "Unfortunately, FHWA could not use the operating ratings in the database because states had not reported them consistently." No explanation is provided as to how it was possible to develop results for the inventory rating and for an intermediate case, but not for the operating rating.

Exhibit 9 How Bridge Cost Estimates Vary With Different Ratings
(Dollars in Millions)

Bridge Rating Criteria[a]	Bridge Replacement Costs For Current Trucks[b]	Additional Costs for Turnpike Doubles and Triples[b]
Intermediate Rating		
Rural Interstates	428	248
Urban Interstates	2,125	1,078
Total	**2,553**	**1,326**
Inventory Rating		
Rural Interstates	819	5,095
Urban Interstates	3,444	13,234
Total	**4,263**	**18,324**

[a] The inventory rating is 55 percent of yield stress, the intermediate rating is approximately 65 percent and the operating rating (not used in the analysis) is 75 percent.

[b] Bridge replacement costs were calculated from the unit costs furnished by the states in 1993. FHWA's results are for a turnpike double weighing 129,000 pounds and a triple weighing 115,000.

Source: FHWA analysis of National Bridge Inventory for GAO (1993).

Based on a rough analysis, we estimate that correcting these errors would result in cost of about $3 million per year due to the increased weight limit.

Moses (1992) estimated the effects of changes in truck weights on Ohio bridges. The analysis used the same general methodology that was used in TRB Special Report 225, and included costs for new bridges, replacement of existing bridges, and fatigue. The report provides cost estimates for a large number of vehicles. To illustrate his findings, Moses used the example of increasing the weight of a conventional five-axle tractor-semitrailer from 80,000 to 100,000 pounds. He estimated that this increase would increase Ohio's costs for new bridges by $5.7 million per year, and costs for replacement of existing bridges by $28 million per year, assuming a discount factor of 7 percent.

A recent study by Transtec (1993) examined how costs for new bridges vary with the traffic loadings assumed in design. The primary application of this work is in highway cost allocation, since it provides estimates of how bridge costs could be reduced if bridges had to accommodate only lighter vehicles. However, the work does have some implications for analyzing the effects of increasing truck size and weight limits, since it provides estimates of the added costs for new bridges if the minimum design load for Interstate System bridges is increased by 25 percent (from HS-20 to HS-25). Cost estimates were developed for 960 sample bridges. The study indicated considerable economies of scale with respect to bridge design loadings: the 25 percent increase in design loadings resulted in construction cost increases in the 1 to 10 percent range.

As discussed in Section 1.3, Ghosn and Moses applied reliability procedures to develop a new bridge formula such that a simply-supported steel bridge designed to satisfy AASHTO's WSD criteria for HS-20 loading will have a safety index beta of 2.5 when subjected to the loads expected over the next 50 years if the new bridge formula is implemented. Ghosn and Moses estimated the number of bridges and bridge length that are currently deficient and would become deficient under the proposed formula for three common bridge types: steel, reinforced concrete, and prestressed concrete. These estimates, which are shown in Exhibit 10, can be used to develop a rough estimate of bridge replacement costs under the proposed formula. Ghosn and Moses noted that, for the nation as a whole, some 130,000 bridges were then rated structurally deficient with an estimated $53 billion replacement or upgrading cost. Using figures from Exhibit 10, the length of deficient bridges (which is directly related to replacement costs) would increase by 65 percent under the proposed formula. Applying this percentage to the $53 billion current backlog implies that implementing the proposed formula would increase the backlog by $34 billion. Applying a discount rate of 7 percent, this corresponds to an annual cost of $2.4 billion.

Exhibit 10 Current Defiencies and Added Deficiencies Under Bridge Formula Proposed by Ghosn and Moses

	Total No. of Bridges Analyzed	Current Deficiencies		Added Deficiencies	
		Number	Length (000's of ft)	Number	Length (000's of ft)
Primary Highways					
Steel	41,140	1,738	119.3	7,248	679.5
Reinf. Conc.	25,510	1,174	43.7	2,346	124.4
Prestr. Conc.	26,474	1,264	95.6	2,896	249.7
Non-Primary Highways					
Steel	154,401	80,243	3,759.1	18,742	1,234.5
Reinf. Conc.	81,671	21,510	652.8	11,957	521.0
Prestr. Conc.	48,568	5,503	247.5	6,079	378.9
Total	377,764	111,432	4,918.0	49,268	3,188.0

Source: Ghosn and Moses

2.0 Knowledge Gaps and Research Needs

1. Estimates of bridge costs were developed in past studies based on assumptions about permissible stress levels and information on the load-bearing capacity of bridges from the bridge inventory. It might be desirable to examine a sample of bridges in more detail, and to assess whether the assumptions and analytical procedures used in past studies might be understating or overstating bridge cost impacts.

2. Estimates of bridge costs should include delays to highway users during bridge replacement.

3. Bridge cost models should be developed that account explicitly for critical loading events on bridges, such as simultaneous presence of two or more heavy vehicles, likelihood of illegal overloads, dynamic effects, and the positions of trucks on bridges.

4. A key finding of TRB Special Report 225 was that new truck weight regulations should be evaluated on the basis of overall costs rather than arbitrary overstress criteria. Arbitrary assessments such as 5 percent overstress on HS-20 have no meaning in terms of either consistent reliability or impact costs.

5. More information is needed about the criteria used by states for determining bridge postings and bridge replacement practices. Surveys have indicated drastic differences among states, far beyond what would be expected due to state-to-state variations in bridge condition and traffic loadings.

3.0 References for Bridges Working Paper

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