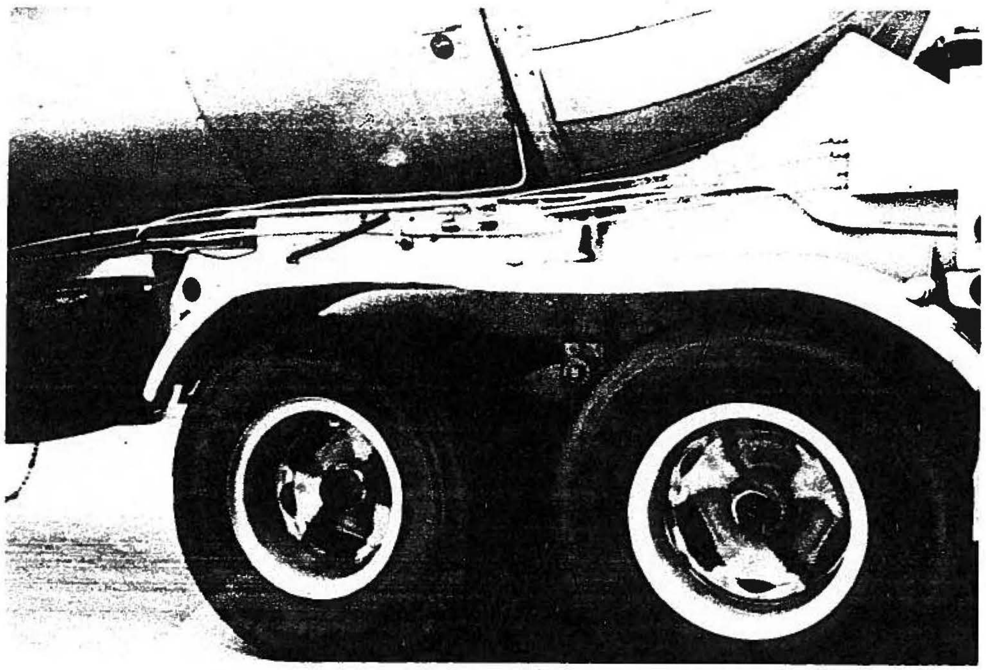
REPORT NO. FHWA/RD-85-088

BRIDGE FORMULA DEVELOPMENT

FINAL REPORT



JUNE 1985

FINAL REPORT

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PREPARED FOR

FEDERAL HIGHWAY ADMINISTRATION

OFFICES OF RESEARCH & DEVELOPMENT

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16. Abstract

The objective of this research was to review the existing bridge formula to find if modifications could be suggested to make it more rational. The intent was to more fully utilize the capacity of existing bridges without significantly shortening the service life of any.

A formula, independent of the number of included axles, was developed to accomplish the objective. As with the current formula, it is applicable both to the overall wheelbase and to all included subgroups of axles. The maximum weights for single and tandem axles were assumed to be unchanged. If enforced, the proposed formula assures that HS20 bridges will not be loaded to more than 1.05 times the design stress nor will H15 bridges be loaded to more than 1.30 times the design stress. The formula reduced the maximum weight allowed on four or more closely spaced axles. However, for most practical lengths, the formula is less restrictive than the current law.

A brief study of the influence the proposed formula would have on pavement fatigue was accomplished. For most practical heavy vehicles, the formula would result in a greater number of equivalent axle loads per vehicle. One equivalent axle load causes the same pavement fatigue damage as a single 18,000 lb (80.06 kN) axle. The number of equivalent axle loads is commonly used as a measure of the fatigue damage a heavy vehicle imposes on the pavement.

A detailed study of the effect the adoption of the proposed bridge formula would have on pavements is recommended. Such a study should consider costs, benefits, and potential formula modifications.

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PREFACE

This work was accomplished, and this report prepared by a team of engineers and researchers including James S. Noel, Ray W. James, Howard L. Furr, Francisco E. Bonilla, and Norris D. Stubbs.

Early in the program a technical advisory committee was invited to assist and advise in the conduct of the work. These prominent highway engineers and transportation authorities were especially helpful with their thoughtful reviews and suggestions. They included Frank D. Sears, Federal Highway Administration; Veldo M. Goins, Oklahoma Department of Highways; Edward V. Hourigan, New York State Department of Transportation; Robert Cassano, California Department of Transportation; Clellon Loveall, Tennessee Department of Transportation; W. Jack Wilkes, Figg and Mueller Engineers, Inc.; and John P. Rutter, G. A. and F. C. Wagnan, Inc. This committee met on two separate occasions with the staff, listened patiently to our ideas, and then made their detailed and professional critiques to us.

Our grateful acknowledgment is given to the many industries, trade associations, and professional organizations that gave freely of their data and time to help with this project.

A special word of appreciation is due Mr. Eldon Klein of the California Department of Transportation, who spent a great deal of effort to help with this endeavor. He understands, as well as anyone, the complex relationships between highways and highway users.

Support for this research was provided by the Federal Highway Administration, Offices of Research and Development, Contract No. DTFH61-84-C-00022. We are especially grateful for the valuable technical coordination and leadership provided by Dr. Lloyd R. Cayes, FHWA Contract Manager.

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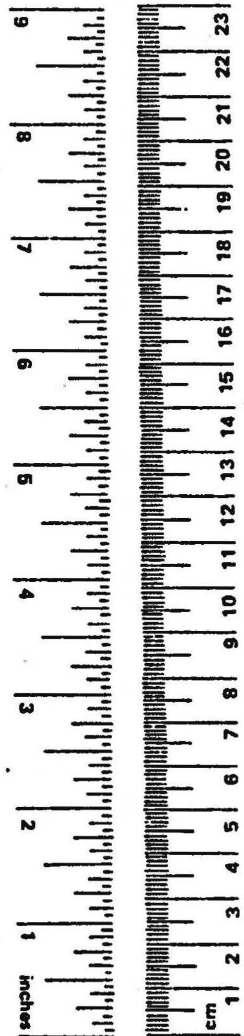
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

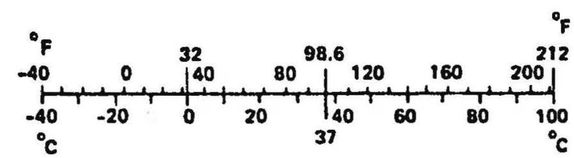
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



INTRODUCTION

During the past several years there has been increasing activity oriented toward reevaluating the Federal truck weight laws. The primary motive for this reevaluation seems to be to change the truck bridge formula so that the stress producing potential in highway bridges is more uniform across the range of both truck and bridge configurations. The bridge formula currently in effect, the so-called Table B (or Formula B), does not always follow the rationale that the weight of the heavier vehicles should be systematically distributed to guarantee that specified stress levels are never exceeded in bridge members.

The levels chosen for this report were 1.05 times the design stress for HS20 bridges and 1.30 times the design stress for H15 bridges. These ratios reflect an expectation that HS20 bridges will perform satisfactorily for a full length design service life while a foreshortened service life can be tolerated of H15 bridges where earlier replacement is typically anticipated. These specific maximum stress levels have traditionally been used by highway structures engineers as a middle ground between the demand for heavier vehicles and the need to protect the bridges from premature failures due to fatigue.

Consequently a new formula, designed to regulate maximum truck weights as well as the allowable weights on all possible axle subgroupings and based on a rational consideration of the conflicting interests of both the trucking industry and the highway engineers and managers, is being proposed. The intention for introducing the new formula is to allow every vehicle to have the maximum possible gross weight while simultaneously assuring that the deterioration of the highway pavements and bridges is not accelerated due to excessive stress levels. However, the implementation of the new, in many instances more liberal, bridge weight formula should be accompanied by an increased resolve to enforce the law in all its aspects, including single axle, tandem axle, intermediate axle groupings, and gross weights.

HISTORICAL COMMENTS

The use of trucks for intercity transportation and the construction of highways and highway structures has grown and improved at near geometric rates since 1900. Currently trucks handle near 60 percent of the manufactured products, 80 percent of the fruits and vegetables, and 100 percent of the livestock that are transported cross-country.(1) During most of this period the individual states built the roads and regulated the trucks, but from the early 1930's the American Association of State Highway Officials (AASHO) began showing concern for the nationwide regulation of truck weights and dimensions to both protect the pavements and bridges and to expedite interstate transport. This concern culminated in the 1946 AASHO policy that single axles not weigh more than 18,000 lb (80.06 kN), tandem axles (under 8 ft (2.44 m) spacing) not weigh more than 32,000 lb (142.3 kN), and that the gross weight nor the weight of any interior group of axles exceed

$$W = 1025(L + 24) - 3L^2 \tag{1}$$

where W is the weight in pounds and L is the out-to-out dimension of the extreme axles in feet.(2)

The first significant Federal legislation of truck weight came with the Federal Aid Highway Act of 1956, the act which initially provided for the planning, financing, and construction of the National System of Interstate and Defense Highways. This legislation stated that no funds would be used for the Interstate System within any State that allowed single axles heavier than 18,000 lb (80.06 kN), tandem axles heavier than 32,000 lb (142.3 kN), or an overall gross weight greater than 73,280 lb (321.9 kN). However, "Grandfather Clauses" provided that any vehicle that operated legally within a State could continue to operate, legally, within that State after the passage of the law.

In the 1950's, under the leadership of H. K. Stephenson, a formula having a format very similar to the current Federal law was advanced.(3,4) His work led directly to the formulas recommended in the House Document No.

354.[5] In that document, representing the views of many responsible organizations and the findings of the recently completed AASHO Road Test, the Highway Research Board recommended that bridge Formula A, see table 1, be immediately adopted for the Interstate System.[6] The document further recommended that after July 1, 1967 that bridge Formula A be replaced by bridge Formula B, reproduced here in table 2. In conjunction with this latter recommendation the document suggested increasing the maximum single axle weight to 20,000 lb (88.96 kN) and the maximum tandem axle weight to 34,000 lb (151.2 kN).

One important aspect of the tables is the second footnote under each. This footnote flatly prohibits the operation of certain short wheelbase, multiaxial trucks over H15 bridges. The point was clearly made in that document that such vehicles would overstress the H15 bridges more than 30 percent; an intolerable situation.

Very little happened in response to these recommendations, however, until 1975, at which time the U.S. Congress enacted legislation permitting the states to increase the weight limits on the Interstate System to essentially those of Formula B. A maximum gross weight of 80,000 lb (355.8 kN), irrespective of the formula, was also imposed. This legislation was passed shortly after the 55-mph (88 km/hr) speed limit was adopted in December 1973 and is generally believed to be a concession to the trucking industry to allow them to regain some of the productivity lost due to the slower speeds.

The most recent legislation is referred to as the Surface Transportation Assistance Act of 1982. The Vehicle Weight Limitations section of the Act is reproduced verbatim below.

VEHICLE WEIGHT, LENGTH, AND WIDTH LIMITATIONS

Sec. 133. (a) Section 127 of title 23 of the United States Code is amended to read:

"127. Vehicle weight limitations-Interstate System

"(a) No funds authorized to be appropriated for any fiscal year under provisions of the Federal-Aid Highway

Table 1. Permissible Gross Loads for Vehicles in Regular Operation. Bridge Table A Taken From Reference (5)

$$W = 500\left(\frac{LN}{N - 1} + 12\ N + 32\right) \qquad \begin{array}{l} 1\ \text{ft} = 0.3048\ \text{m} \\ 1\ \text{lb} = 4.448\ \text{N} \end{array}$$

Distance in feet between the extremes of any group of 2 or more consecutive axles	Maximum load in pounds carried on any group of 2 or more consecutive axles [1]							
	2 axles	3 axles	4 axles	5 axles	6 axles	7 axles	8 axles	9 axles
4	32,000							
5	32,000							
6	32,000							
7	32,000							
8	32,000	40,000						
9	(37,000)	41,000						
10	(38,000)	41,500						
11	(39,000)	42,000						
12	(40,000)	43,000	48,000					
13		44,000	49,000					
14		44,500	49,500					
15		45,000	50,000					
16		46,000	50,500	56,000				
17		47,000	51,500	56,500				
18		47,500	52,000	57,000				
19		48,000	52,500	58,000				
20		49,000	53,500	58,500	64,000			
21		50,000	54,000	59,000	64,500			
22		50,500	54,500	60,000	65,000			
23		51,000	55,500	60,500	66,000			
24		52,000	56,000	61,000	66,500	72,000		
25		53,000	56,500	61,500	67,000	72,500		
26		53,500	57,500	62,000	67,500	73,000		
27		54,000	58,000	63,000	68,000	74,000		
28		(55,000)	58,500	63,500	69,000	74,500	80,000	
29		(56,000)	59,500	64,000	69,500	75,000	80,500	
30		(56,500)	60,000	65,000	70,000	75,500	81,000	
31		(57,000)	60,500	65,500	70,500	76,000	81,500	
32		(58,000)	61,500	66,000	71,000	76,500	82,500	88,000
33		(59,000)	62,000	66,500	72,000	77,000	83,000	88,500
34		(59,500)	62,500	67,000	72,500	78,000	83,500	89,000
35		(60,000)	63,500	68,000	73,000	78,500	84,000	89,500
36			64,000	68,500	73,500	79,000	84,500	90,000
37			64,500	69,000	74,000	79,500	85,000	91,000
38			65,500	70,000	75,000	80,000	85,500	91,500
39			66,000	70,500	75,500	81,000	86,500	92,000
40			66,500	71,000	76,000	81,500	87,000	92,500
41			67,500	71,500	76,500	82,000	87,500	93,000
42			68,000	72,000	77,000	82,500	88,000	93,500
43			68,500	73,000	78,000	83,000	88,500	94,000
44			69,500	73,500	78,500	83,500	89,000	95,000
45			70,000	74,000	79,000	84,000	89,500	95,500
46			70,500	75,000	79,500	85,000	90,500	96,000
47			71,500	75,500	80,000	85,500	91,000	96,500
48			72,000	76,000	81,000	86,000	91,500	97,000
49			(72,500)	76,500	81,500	86,500	92,000	97,500
50			(73,500)	77,000	82,000	87,000	92,500	98,000
51			(74,000)	78,000	82,500	88,000	93,000	98,500
52			(74,500)	78,500	83,000	88,500	93,500	99,000
53			(75,500)	79,000	84,000	89,000	94,500	100,000
54			(76,000)	80,000	84,500	89,500	95,000	100,500
55			(76,500)	80,500	85,000	90,000	95,500	101,000
56			(77,500)	81,000	85,500	90,500	96,000	101,500
57			(78,000)	81,500	86,000	91,000	96,500	102,000
58			(78,500)	82,000	87,000	92,000	97,000	102,500
59			(79,500)	83,000	87,500	92,500	97,500	103,000
60			(80,000)	83,500	88,000	93,000	98,500	104,000

[1] The permissible loads are computed to the nearest 500 pounds. The modification consists of limiting the maximum load on any single axle to 18,000 pounds (values in parentheses are for 20,000-pound axle loads).

[2] The following loaded vehicles must not operate over H15–44 bridges: 3-S2 (5 axle) with wheelbase less than 36 feet; 2-S1-2 (5 axle) with wheelbase less than 42 feet; 3-3 (6 axle) with wheelbase less than 44 feet; and 7-, 8-, and 9-axle vehicles regardless of wheelbase.

Table 2. Permissible Gross Loads for Vehicles in Regular Operation. Bridge Table B Taken From Reference (5)

$$W = 500\left(\frac{LN}{N-1} + 12N + 36\right) \qquad \begin{array}{l} 1 \text{ ft} = 0.3048 \text{ m} \\ 1 \text{ lb} = 4.448 \text{ N} \end{array}$$

Distance in feet between the extremes of any group of 2 or more consecutive axles	Maximum load in pounds carried on any group of 2 or more consecutive axles [1]							
	2 axles	3 axles	4 axles	5 axles	6 axles	7 axles	8 axles	9 axles
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	70,000	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	82,500	--
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	75,500	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	81,000	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

[1] The permissible loads are computed to the nearest 500 pounds. The modification consists in limiting the maximum load on any single axle to 20,000 pounds.

[2] The following loaded vehicles must not operate over H15-44 bridges: 3-S2 (5 axles) with wheelbase less than 38 feet; 2-S1-2 (5 axle) with wheelbase less than 45 feet; 3-3 (6 axle) with wheelbase less than 45 feet; and 7-, 8-, and 9-axle vehicles regardless of wheelbase.

Act of 1956 shall be apportioned to any State which does not permit the use of the National System of Interstate and Defense Highways within its boundaries by vehicles with a weight of twenty thousand pounds carried on any one axle, including enforcement tolerances, or with a tandem axle weight of thirty-four thousand pounds, including enforcement tolerances, or a gross weight of at least eighty thousand pounds for vehicle combinations of five axles or more. However, the maximum gross weight to be allowed by any State for vehicles using the National System of Interstate and Defense Highways shall be twenty thousand pounds carried on one axle, including enforcement tolerances, and a tandem axle weight of thirty-four thousand pounds, including enforcement tolerances, on a group of two or more consecutive axles produced by application of the following formula:

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

where W equals overall gross weight on any group of two or more consecutive axles to the nearest five hundred pounds, L equals distance in feet between the extreme of any group of two or more consecutive axles, and N equals number of axles in group under consideration, except that two consecutive sets of tandem axles may carry a gross load of thirty-four thousand pounds each providing the overall distance between the first and last axles of such consecutive sets of tandem axles is thirty-six feet or more: Provided, That such overall gross weight may not exceed eighty thousand pounds, including all enforcement tolerances, except for those vehicles and loads which cannot be easily dismantled or divided and which have been issued special permits in accordance with applicable State laws, or the corresponding maximum weights permitted for vehicles using the public highways of such State under laws or regulations established by appropriate State authority in effect on July 1, 1956, except in the case of the overall gross weight of any group of two or more consecutive axles, on the date of enactment of the Federal-Aid Highway Amendments of 1974, whichever is the greater. Any amount which is withheld from apportionment to any State pursuant to the foregoing provisions shall lapse. This section shall not be construed to deny apportionment to any State allowing the operation within such State of any vehicles or combinations thereof which the State determines could be lawfully operated within such States on July 1, 1956, except in the case of the overall gross weight of any group of two or more consecutive axles, on the date of enactment of the Federal-Aid Highway Amendments of 1974. With respect to the State of Hawaii, laws or regulations in effect on February 1, 1960, shall be applicable for the purposes of this section in lieu of those in effect on July 1, 1956. With respect to the State of Michigan, laws or regulations in effect on May 1, 1982, shall be applicable for the purposes of this subsection.

"(b) No State may enact or enforce any law denying reasonable access to motor vehicles subject to this title to and from the Interstate Highway System to terminals and facilities for food, fuel, repairs, and rest."

These limitations are exactly the same as Table B, with the 80,000 lb (355.8 kN) gross weight cap, except for the relaxation on the maximum weight of tandems spaced 36 ft (10.97 m) or more.

It is interesting to note that in the legislation of 1975 and 1982 no mention is made of the footnote restricting short, multiaxled vehicles on H15 bridges.

FATIGUE IN BRIDGES AND PAVEMENTS

The service lives of bridges and pavements are greatly affected by the ranges and number of applications of stresses to which they are subjected. A large body of information exists on the influences of these factors, and much of it comes from the AASHO Road Test conducted at Ottawa, Illinois in the period 1958-1962.(6) This knowledge is reflected in the AASHO Interim Guide for Design of Pavement Structures and the AASHTO Specifications for Highway Bridges.(8,9)

In the sections that follow, the reports of the AASHO Road Test and the AASHO guide for pavement design form the bases for the discussion of pavements.(6,7,8) The same road test reports and other research reports, referred to as they appear below, form the bases for the discussion of bridges.

PAVEMENTS

The AASHO Road Test evaluated the performances of a number of designs of flexible and rigid pavements under repeated loads from single- and tandem-axle trucks of various weights. Data collected at periodic inspections were used to compare the performance of a pavement system under one load type to that of another type. From the mathematical relationships developed, the ratio of the number of passes of a standard loading to the number of passes of another loading to produce the same serviceability condition on the same pavement was determined. An 18-kip (80.06 kN) single axle was selected as the standard, and the ratio was called the equivalent axle load factor (EAL). That factor is widely used in pavement design.

The number of axles in a set and the spacing of axles in that set influence pavement performance, but the AASHO test equations referred to above accounts for only single and tandem axles with no consideration for variable spacing. Work by Finney in 1973 indicated that pavement damage is minimized if tandem axles are spaced between 4 and 7 ft (1.219 and 2.134 m), and that the most common spacing at that time was 42 to 54 in (1067 to 1372 cm).(11)

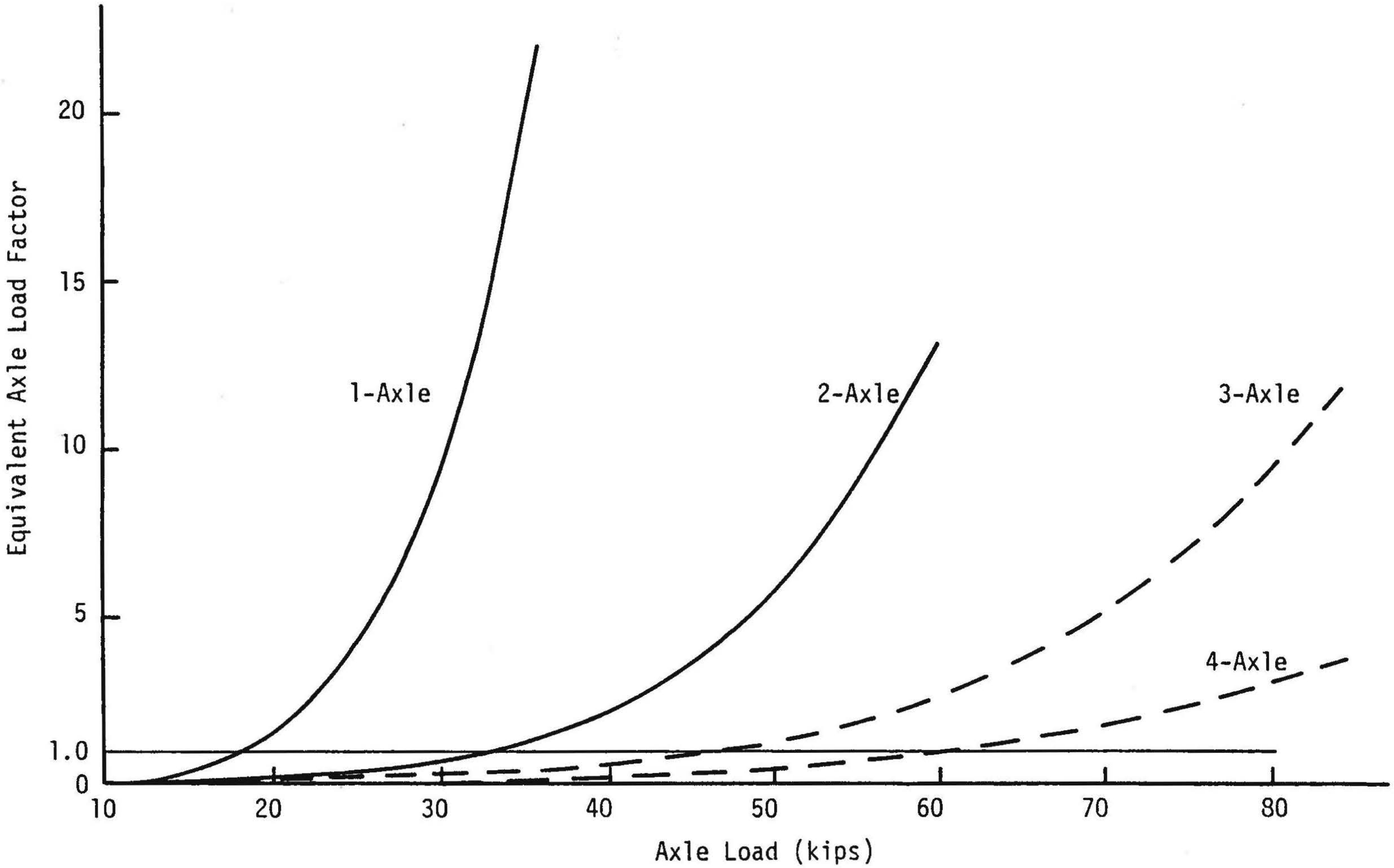
Although the AASHO equations are not developed for more than a two-axle set, the equations are extrapolated here to three- and four-axle bogies for purposes of comparing effects of multiple-axle loads. Figures 1 and 2 show the relationship between the EAL factor, axle load, and number of axles for one condition each of a flexible pavement and a rigid pavement.

Under the assumption that the extrapolation to multiple-axle sets is valid, figures 1 and 2 are used to determine how the equivalence factor, and hence pavement deterioration, is affected by increasing the loading by 30%. This percentage is selected to agree with the 30% overstress of H15 bridges referred to earlier in this report. Table 3 shows that the equivalence factor is increased in both the particular flexible and rigid pavement treated. The destructive effect of a 30 percent increase in load is slightly greater on the flexible pavement than on the rigid pavement. The effct is a little greater for the tandem axle load than for any other, but the difference is small. Although the actual percentages might differ considerably from these because of the type and weight spectrum of truck traffic and the type and maintenance of the pavement structure, it is clear that pavement life would be decreased by heavier truck axles.

BRIDGES

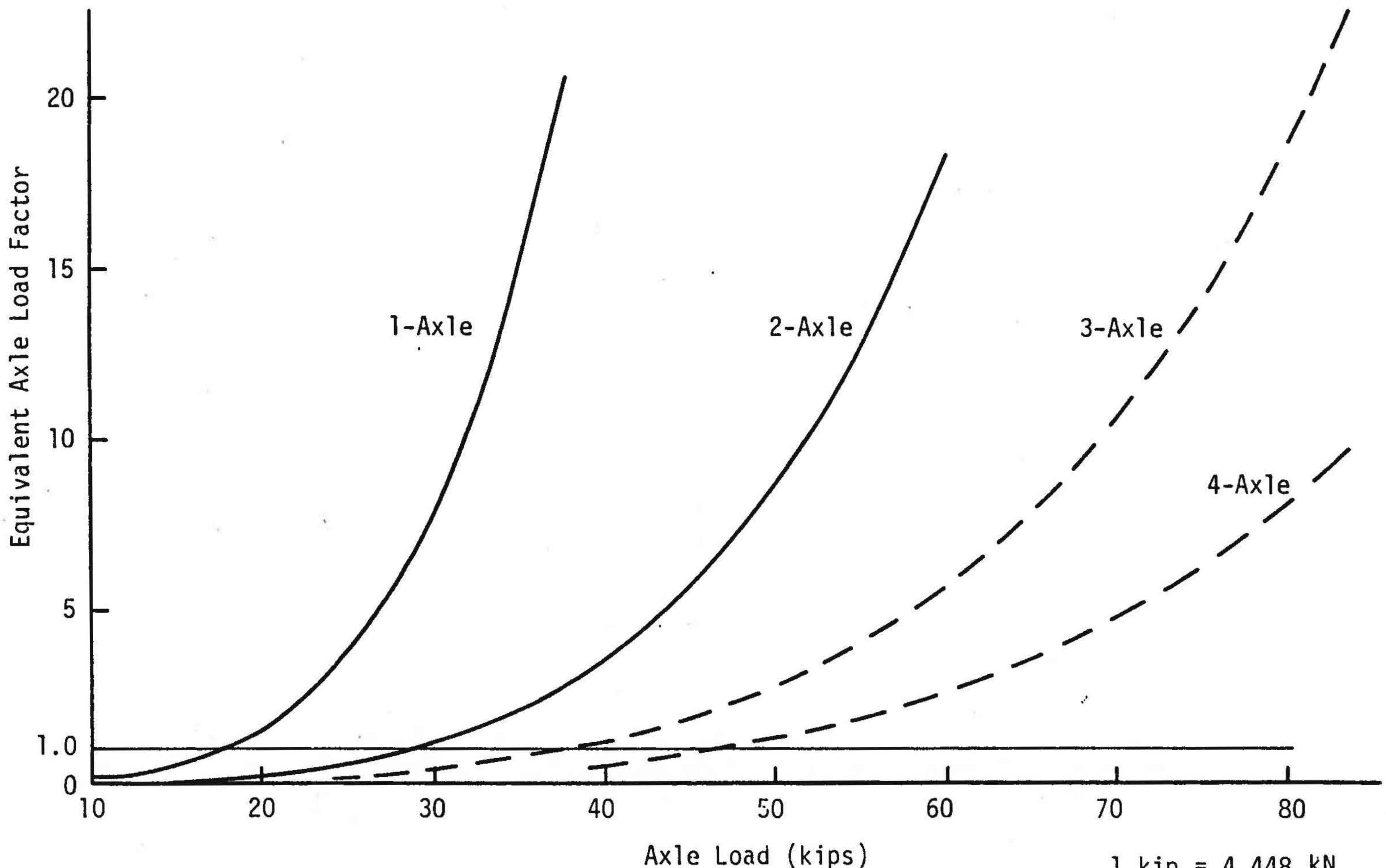
The most common types of beams used in the interstate highway system are steel I-beam and plate girder, reinforced concrete, and prestressed concrete. The deck slabs are almost all of reinforced concrete. The AASHO Road Test included all of these bridge types in test runs, and supplementary tests on steel and concrete articles were made in laboratories. Numerous tests, not associated with the AASHO Road Test, on the relationship of stress range and repetitions have been made and reported. The data gathered in these field and laboratory tests has expanded the knowledge and understanding of fatigue in materials, and has influenced AASHTO to include fatigue design in the highway bridge design specifications.(9,10)

The sections that follow give information on the general behavior of steel and concrete under repeated stresses, and how the life of a bridge might be affected by that behavior.



1 kip = 4.448 kN

Figure 1. Equivalent Axle Load Factor vs. Axle Load, Flexible Pavement, SN = 6, p = 2.5[7]



1 kip = 4.448 kN
1 in = 25.4 mm

Figure 2. Equivalent Axle Load Factor vs. Load, Rigid Pavement, D = 8 in, p = 2.7[7]

Table 3. Effect of a Thirty Percent Increase in Axle Load on Equivalent Axle Load Factors of Two Pavements

Line									
1	Pavement Type	Flexible				Rigid			
2	Number of Axles	1	2	3	4	1	2	3	4
3	Total W(kips) for EAL=1	18	34	48	62	18	29	39	47
4	1.3 x Line 3	23	44	62	81	23	38	50	61
5	EAL for Line 4	2.9	3.3	3.0	3.1	2.7	3.0	2.8	2.8
6	Percentage Decrease in Pavement Life Due to 30% Increase in W $(1 - \frac{1}{\text{Line 5}}) \times 100$	65	70	67	68	63	67	64	64
7	Current Formula	20	34	42	50	20	34	42	50
8	Proposed Formula	20	34	42	46	20	34	42	46

W = Load in kips on the Axle Set

EAL = Equivalent Axle Load Factor

Data from figures 1 and 2

1 kip = 4.448 kN

Concrete Bridges

Concrete is able to undergo an unlimited number of stress repetitions provided that the stress does not exceed 50% of its static strength.[11] This applies, so far as it is currently known, for both tension and compression. Compressive strength is of major concern in reinforced and prestressed concrete bridges, and tension (modulus of rupture) is of major concern in rigid pavements. Figure 3 shows the relationship of modulus of rupture to number of stress cycles to failure, S-N curve, developed from flexural tests on plain concrete.

The tensile stresses in concrete bridge elements are carried by reinforcing or prestressing steel, and fatigue of these steels is of concern. S-N curves shown in figures 4 and 5 indicate that an unlimited number of stress cycles can be carried by reinforcing steel with a stress range (SR) not exceeding about 24 ksi (165.5 MPa), and by prestressing steel with a stress range not exceeding some 9.7 ksi (66.88 MPa) for 270 ksi (1861 MPa) prestressing steel. The AASHTO bridge specifications [9] permit 24 ksi (165.5 MPa) tensile stress in grade 60 reinforcing steel,

$$f_{(LL+I)} + f_{DL} = 24 \text{ ksi } (165.5 \text{ MPa}) \qquad (2)$$

The ratio of $f_{(LL+I)}$ to f_{DL} will vary with the bridge type, span, and live load, but if (LL+I) accounts for 50 percent of the total stress for purposes of illustration, then

$$f_{(LL+I)} = f_{DL} = 24 \quad 2 = 12 \text{ ksi } (82.74 \text{ MPa})$$

Now, if the live load is increased to make $f_{(LL+I)}$ 30 percent greater, then

$$1.3\, f_{(LL+I)} + f_{DL} = 12(1.3+1) = 27.6 \text{ ksi } (190.3 \text{ MPa})$$

and $$SR = 27.6 - 12 = 15.6 \text{ ksi } (93.77 \text{ MPa})$$

This 15.6 ksi (93.77 MPa) range is much less than the fatigue limit of

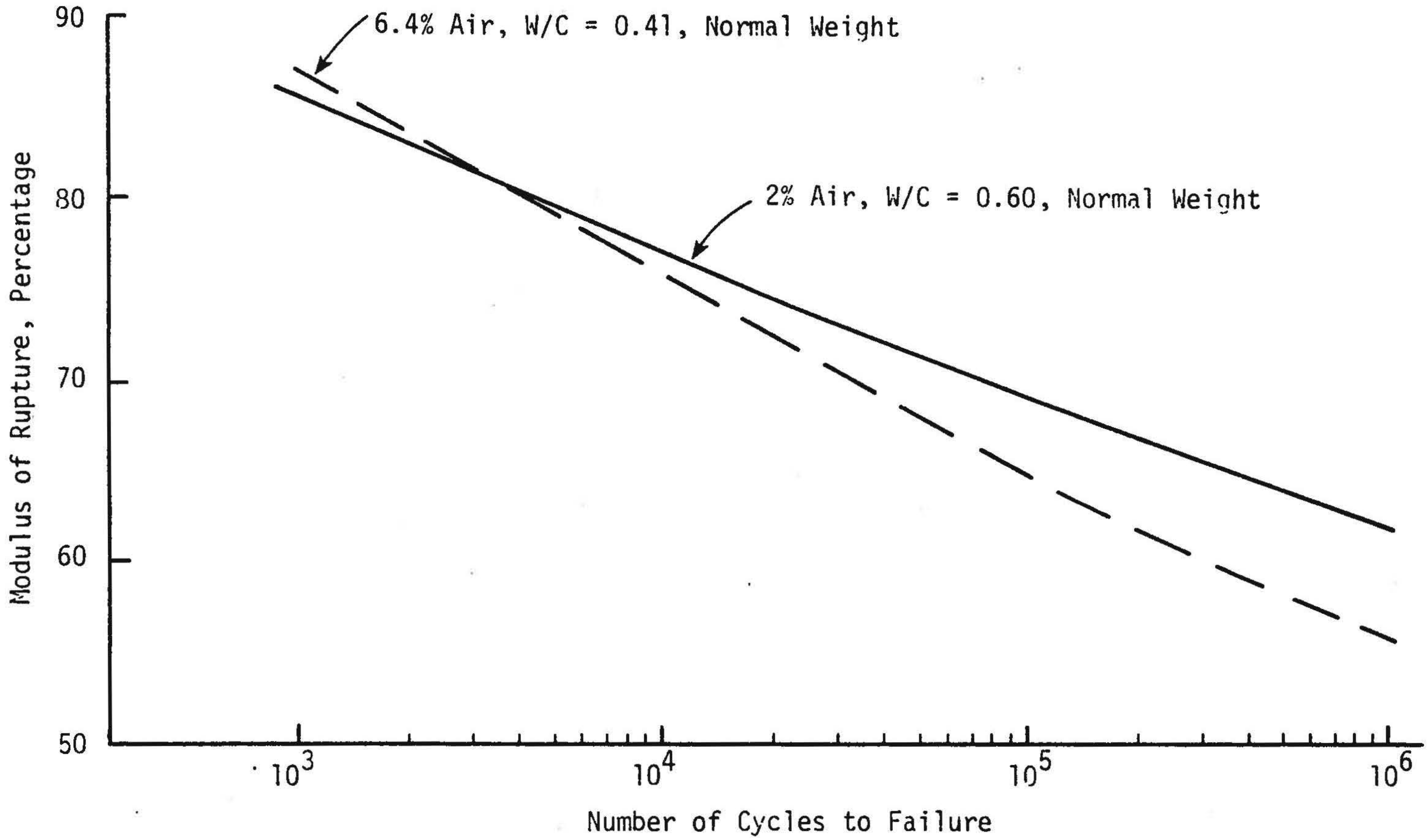


Figure 3. S-N Curves for Plain Concrete Beams (12)

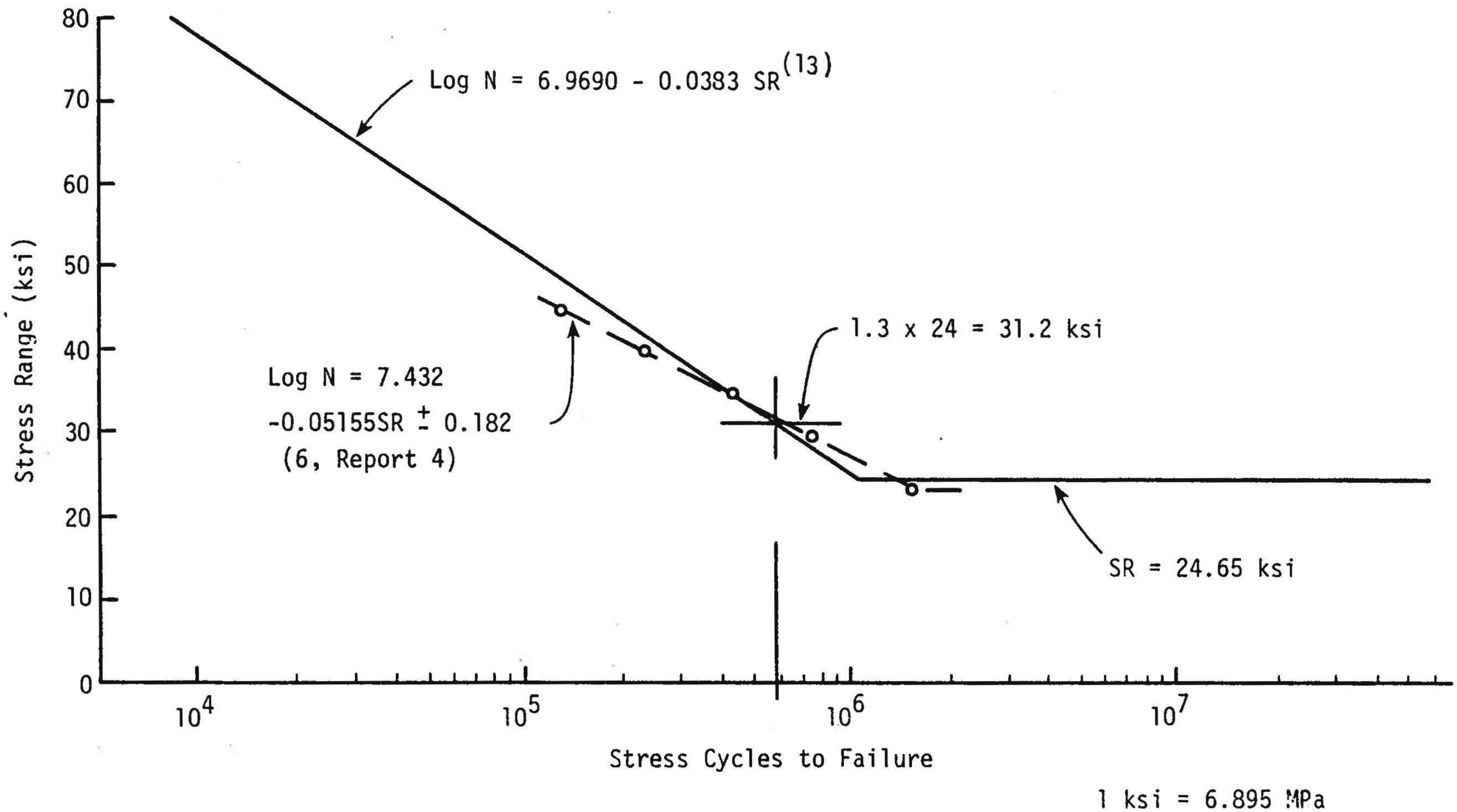


Figure 4. S-N Curves for Concrete Reinforcing Steel[6,13]

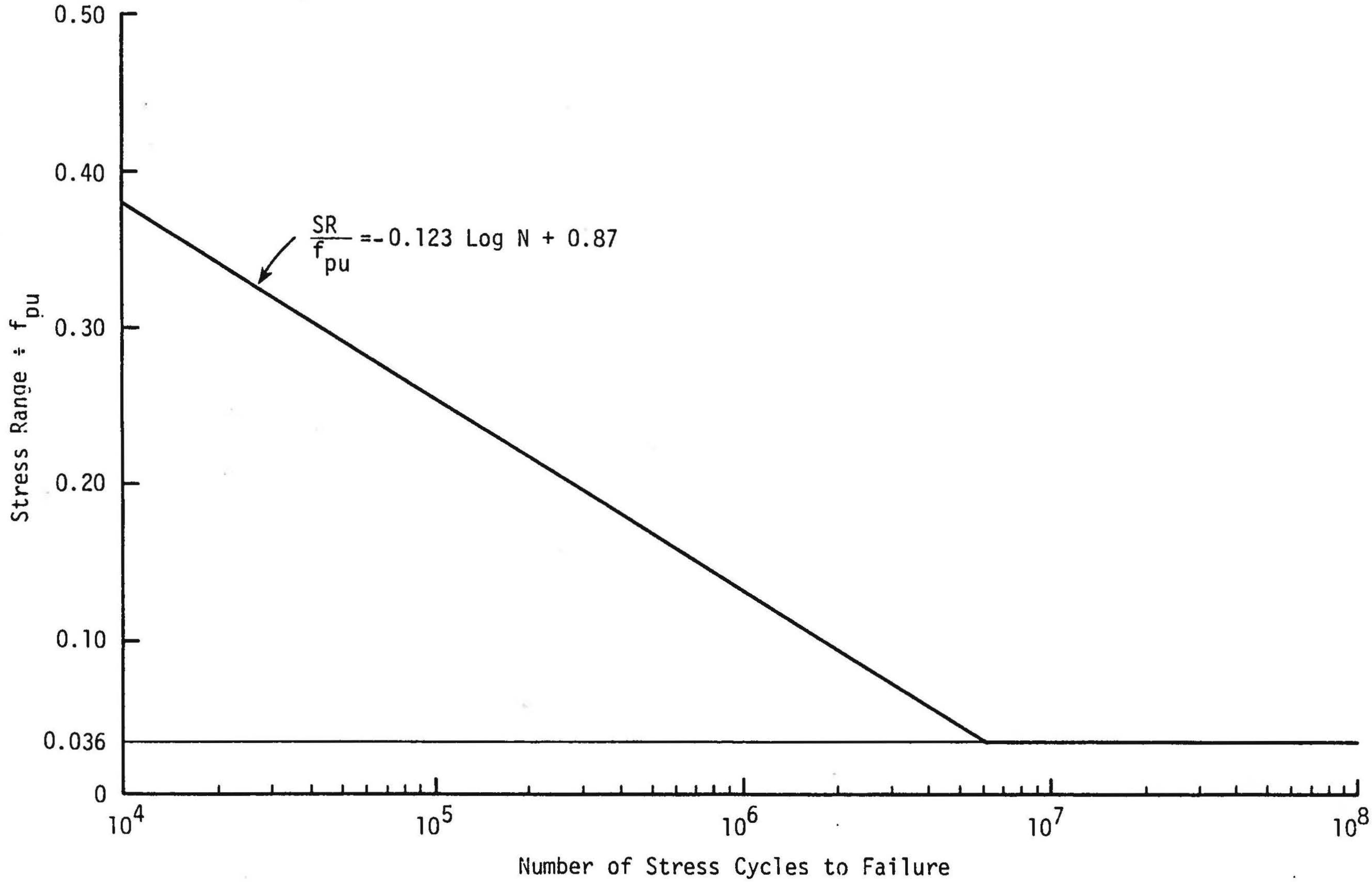


Figure 5. Derived S-N Curve for 7-Wire Strand Prestressing Steel(14)

figure 4, and there would be no measurable reduction in the total number of axle loads caused by the 30 percent increase in live load plus impact. The bridge specifications do not permit a design to a stress range of 24 ksi (165.5 MPa) since the maximum allowable total stress is 24 ksi (165.5 MPa).

Bridges with no floor system, beam and girder bridges, have reinforced concrete deck slabs reinforced transverse to traffic, and the bridge specifications do not require a fatigue analysis of these slabs. The deck slab of a bridge with a floor system is reinforced parallel to traffic and fatigue analysis is required. There are no fatigue problems in a properly reinforced slab on either of these two deck systems. Punch-through failures have occurred in bridge decks, but if the deck concrete is of good quality and in good condition there is no danger of such a failure.

There were no failures in either the pretensioned or the posttensioned concrete beams tested through some 556,000 vehicle passes in the AASHO Road Test.(7) Using the S-N curve for 7-wire prestressing strand tested in air shown in figure 5(14), 270 ksi (1861 MPa) as ultimate strength, f_{pu}, and live load plus impact design stress of 4 ksi (27.58 MPa), then the stress range divided by the ultimate strength, SR/f_{pu} equals 4/270 = 0.015. This ratio is far less than SR/f_{pu} of 0.036 at which an unlimited number of cycles could be applied without failure. An increase of 30 percent in $f_{(LL+I)}$ gives an SR/f_{pu} ratio of 1.3 x 4/270 = 0.019, still far less than the fatigue limit. From this it can be said that there is little, if any, danger of fatigue failure of steel in pretensioned beams. It is, however, almost certain that flexural cracks will develop in the bottom of the beams, and the state of stress at a crack is complicated. These cracks in the AASHO Road Test bridges were small in the beams stressed below half of tensile strength of concrete.

Steel Beam Bridges

I-beam bridges without cover plates developed no beam damage in the AASHO Road Test, but weld cracks developed at about one-half million vehicle passes in beams with partial cover plates.(7) In most of the beams, $f_{(LL+I)}$ was about the same as f_{DL}, some 14 ksi (96.53 MPa), and the range of stresses varied between 12 to 15 ksi (82.74 to 103.4 MPa), approximately.

An S-N curve, developed from laboratory tests which supplemented the AASHO field tests, is shown in figure 6. At N = 2,000,000 and minimum stress of 14 ksi (96.53 MPa), it is found that the stress range is approximately 9.4 ksi (64.81 MPa) -- some 75 percent of the range in the field test. With minimum stress = 14 ksi (96.53 MPa) = f_{DL}, and an increase of 30 percent in $f_{(LL+I)}$, then the stress range = 1.3 x 9.4 = 12.2 ksi (84.12 MPa), and the number of truck passages from figure 6 is 1,600,000. This is 80 percent of the life at the 9 ksi (62.06 MPa) stress range, a 20 percent reduction.

Average daily truck traffic, ADTT, often exceeds 10 percent of the total traffic, ADT, and this could amount to more than 1000 trucks per day. At this rate, the life at SR = 9 ksi (62.05 MPa) would be about 5 1/2 years, and at SR = 1.3 x 9 ksi (62.05 MPa), the life would be about 4 years, assuming that the same path were used by each vehicle. A long life can be designed into a bridge in the planning stage, but once a bridge is built, it is not possible to meet higher demands without major revisions.

CORROSION FATIGUE

Weldments, reinforcing bar deformations, and corrosion damage provide discontinuities that concentrate and amplify stresses. Aggressive environments such as deicing salt runoff and industrial gasses promote corrosion which sometimes creates stress raisers. Very high stresses can cause cracks to develop even under static load conditions, but the development is accelerated by cycled stress. Once started, such a crack will grow relatively slowly until a critical condition is reached, at which time rapid growth sets in and failure eventually develops.[15]

Laboratory tests on concrete beams reinforced with hot-rolled steel were made in air and partially submerged in salt water. The S-N curves of the tests are shown in figure 7. All of the bars in the beams tested in air fatigued at the roots of deformations -- geometric stress raisers -- while at least some of those tested in the corrosive environment of salt water failed

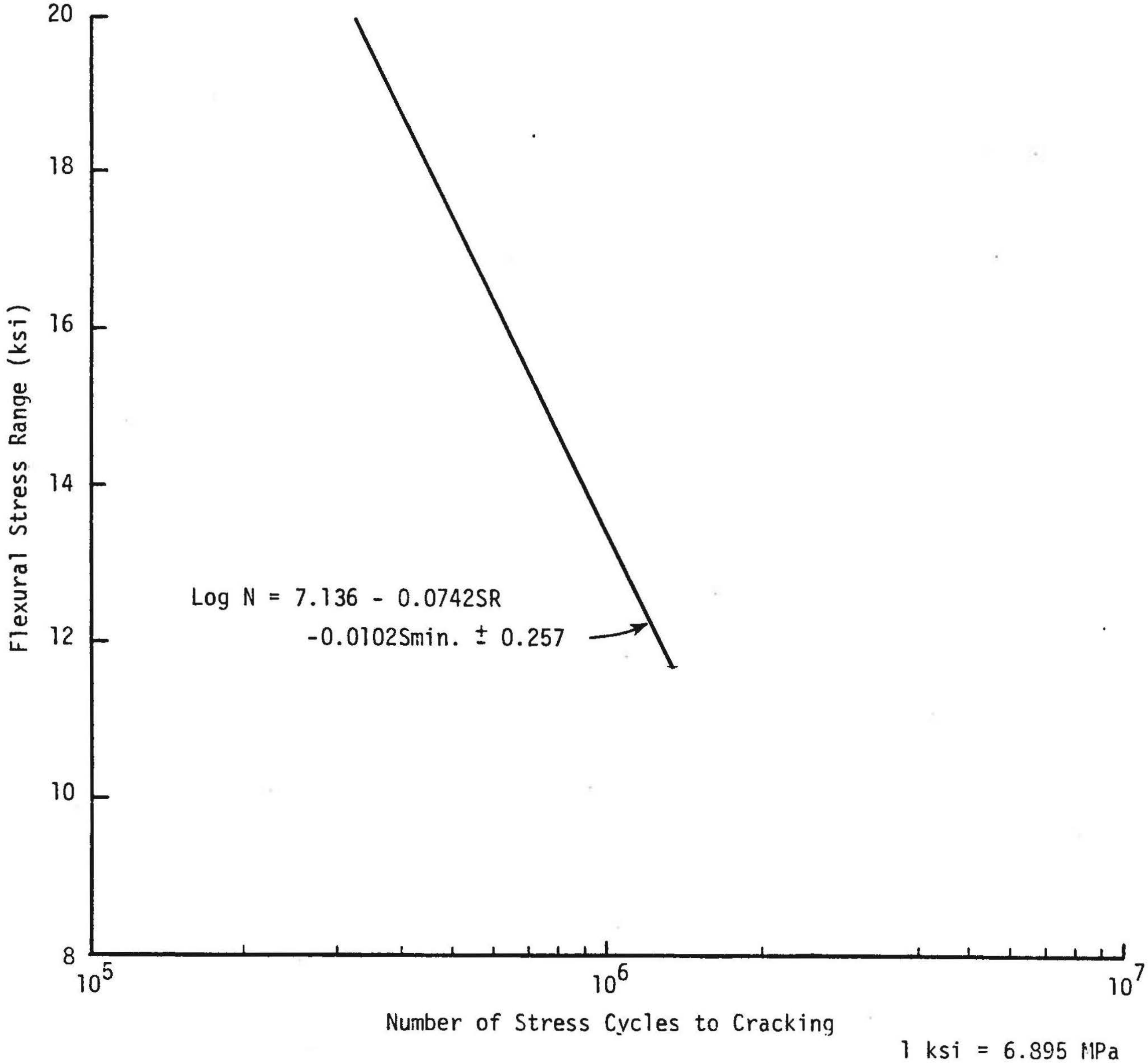
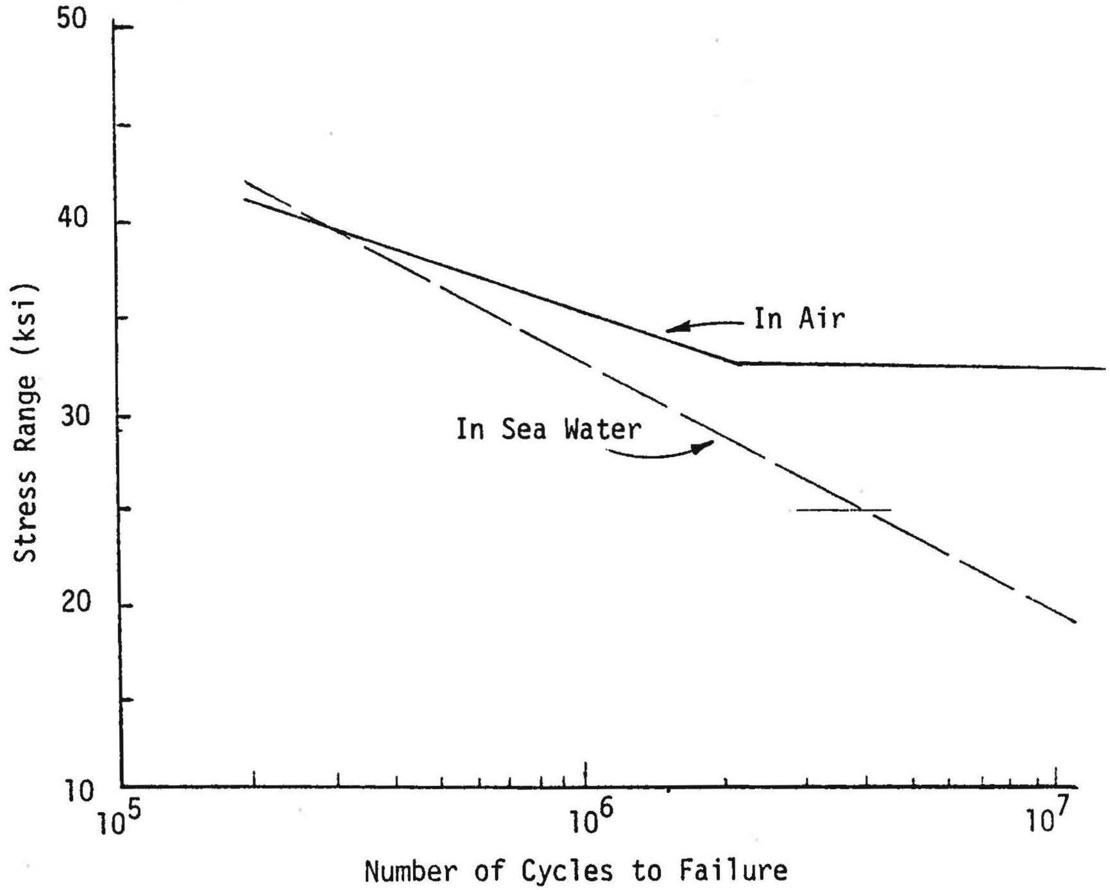


Figure 6. S-N Curve for Steel Beam with Partial Cover Plate (6, Report 4)



1 ksi = 6.895 MPa

Figure 7. S-N Curves for Beams Reinforced with Hot Rolled Bar[16]

from cracks initiated in other regions.[18] This indicates that corrosion was responsible for the initiation of some of the cracks.

In order to get some idea of the influence of stress magnitude on the life of the beams tested in the corrosive environment, the 30 percent increase in live load plus impact used in preceding sections is applied here. In the figure, when $f_{(LL+I)}$ at 10 million cycle life is increased by 30 percent, the life is reduced to approximately 3.6 million cycles, a 64 percent decrease in the 10 million cycle life. As in the earlier example, a 1000 ADTT would have the life reduced from 27 years to about 10 years.

SUMMARY

The service lives of bridges and pavements are greatly affected by the types, weights, and numbers of vehicles that are carried over them. Various research findings give quantitative information that enables one to estimate the service life of these highway structures in terms of the traffic carried. In the design phase, the projected life can be increased by decreasing certain design stresses, but an existing structure cannot be easily changed to reduce fatigue damage from heavier traffic.

A 30 percent increase in load was selected in the illustrations to demonstrate how pavement and bridge lives are changed by the increase. For the pavement structures selected, the life of both the flexible and rigid pavements was reduced by approximately 2/3 by this load increase. The life of AASHO Road Test concrete bridges was not reduced measurably by the stress increase, but it was shown that corrosion fatigue might cause a considerable reduction in service life. Cover plated steel beam bridges might suffer about 20% reduction in traffic life by the stress increase.

On the basis of these studies of bridge elements and pavement structures, the fatigue life of pavements places a greater restriction on load limits than does that of the bridge elements. From the point of view of fatigue, pavements should control the permissible weight of single axles, tandem axles, and other very short axle groups.

LIVE LOADS TO CAUSE SPECIFIED STRESS RATIOS

DEAD LOAD RATIOS FOR VARIOUS BRIDGE DESIGNS

Data for dead load stress ratios has been collected from various sources. NCHRP Report 141 presents(17) these ratios for simple span AASHTO bridges designed for H15, HS15, H20, and HS20 vehicles. Both shear and moment ratios are presented. Data was also collected from the Texas SDHPT for moment stress ratios for noncomposite simple spans. Also, the FHWA has provided data from which moment ratios have been calculated.

Not so much information was found for dead load ratios of steel-concrete composite bridges. Data from U.S. Steel's Highway Structures Design Handbook was obtained and is reported.(18)

Tables 4 to 7 present the data for dead load ratios of noncomposite bridges in the form of DL/(LL+I). This data is presented graphically in figures 8 to 11. For purposes of calculation of critical vehicle weights to cause specified overstresses, the critical values of the reported dead load ratios are the minimum values, or the lower bound of the reported data. Approximate lower bounds, in the form of piecewise linear functions of span length, of the reported data were determined and are plotted in figures 12 to 15. These curves are reported also in figures 16 and 17 in the form of DL/TL = DL/(DL+LL+I).

Table 8 and figures 18 and 19 present the limited data for the dead load ratios, in the form of DL/(LL+I), for steel-concrete composite bridges. The data represent continuous multispan structures of various span lengths and is based on the length of the span within which the critical, or lowest, dead load ratio occurs. All data is for HS20 or greater design loadings. This data is presented for information only and has not been used in the calculations which follow.

Table 4. Dead Load Moment Ratios DL/(LL+I) -- HS20 Bridges

Span (ft)	Texas SDHPT Simple Spans RC Slab	Panform	Steel I	PC Beams	NCHRP 141 Stl Gird	FHWA All
10	0.111					0.125
15	0.221					
20	0.375					0.256
25	0.558					
30	0.712	0.460			0.380	0.313
35	0.895				0.415	
40	1.204	0.630	0.430	0.617	0.450	0.402
45			0.463	0.655	0.490	
50			0.499	0.698	0.530	0.472
55			0.540	0.744	0.575	
60			0.586	0.792	0.620	0.562
65			0.635	0.842	0.670	
70				0.892	0.720	0.667
75				0.944	0.785	
80				0.996	0.850	0.729
85				1.049	0.925	
90				1.102	1.000	0.827
95				1.156	1.070	
100				1.211	1.140	0.930
105					1.200	
110					1.260	1.049
115					1.315	
120					1.370	1.175
125					1.410	
130					1.450	1.273
135					1.495	
140					1.540	1.410
150						1.508

Table 5. Dead Load Shear Ratios DL/(LL+I) -- HS20 Bridges

Span (ft)	NCHRP Report 141 Conc Slab	Conc Box	Conc Tee	Stl Gird	PC Gird
10	0.15				
20	0.33			0.22	
30	0.61	0.30	0.32	0.27	0.45
40	1.17	0.55	0.50	0.34	0.52
50		0.80	0.72	0.43	0.67
60		1.03	0.95	0.52	0.87
70		1.26	1.19	0.62	1.17
80		1.53	1.46	0.73	1.48
90		1.78	1.79	0.83	1.85
100		2.04	2.09	0.94	
110		2.24	2.42	1.09	
120		2.52		1.20	
130		2.80		1.34	

1 ft = 0.3048 m

Table 6. Dead Load Moment Ratios DL/(LL+I) -- H 15 Bridges

Span (ft)	Texas SDHPT Simple Spans RC Slab	Texas SDHPT Simple Spans Panform	Texas SDHPT Simple Spans Steel I	Texas SDHPT Simple Spans PC Beams	NCHRP 141 Stl Gird	FHWA All
10	0.147					0.863
15	0.294					
20	0.500					0.342
25	0.771					
30	1.085	0.700			0.500	0.515
35	1.455					
40	2.087	1.090	0.651	0.811	0.670	0.723
45			0.741	0.997		
50			0.827	1.101	0.840	0.855
55			0.921	1.389		
60			1.006	1.476	1.010	1.024
65			1.068	1.542		
70				1.604	1.150	1.158
75				1.661		
80				1.715	1.280	1.229
85				1.766		
90				1.814	1.400	1.333
95				1.860		
100				1.903	1.500	1.443
105						
110					1.590	1.557
115						
120					1.670	1.669
125						
130					1.740	
135						
140					1.800	1.860
150						

Table 7. Dead Load Shear Ratios DL/(LL+I) -- H 15 Bridges

Span (ft)	NCHRP Report 141 Conc Slab	NCHRP Report 141 Conc Box	NCHRP Report 141 Conc Tee	NCHRP Report 141 Stl Gird	NCHRP Report 141 PC Gird
10				0.43	
20	0.50		0.60	0.48	
30	1.07	0.78	0.81	0.55	0.60
40	1.82	1.03	1.10	0.63	0.73
50		1.30	1.40	0.72	0.94
60		1.55	1.55	0.82	1.25
70		1.82	1.82	0.91	1.60
80		2.09	2.09	1.02	2.00
90		2.32	2.32	1.13	2.40
100		2.62	2.62	1.28	
110		2.88	2.88	1.37	
120		3.15	3.15	1.51	
130		3.41	3.41	1.63	
140		3.57	3.57	1.76	

1 ft = 0.3048 m

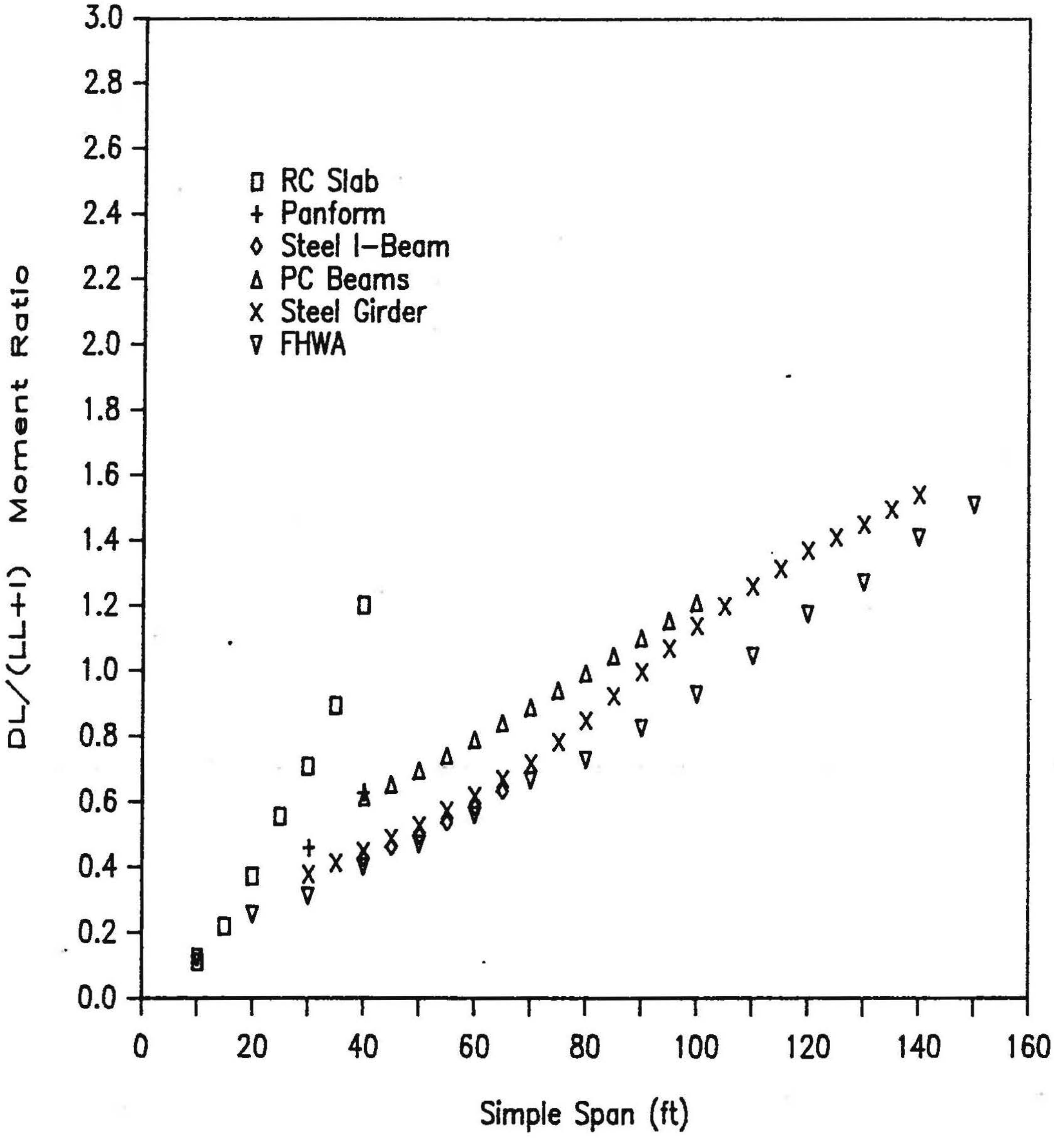


Figure 8. Dead Load Moment Ratios DL/(LL+I) for Simple Span, Noncomposite HS20 Bridges

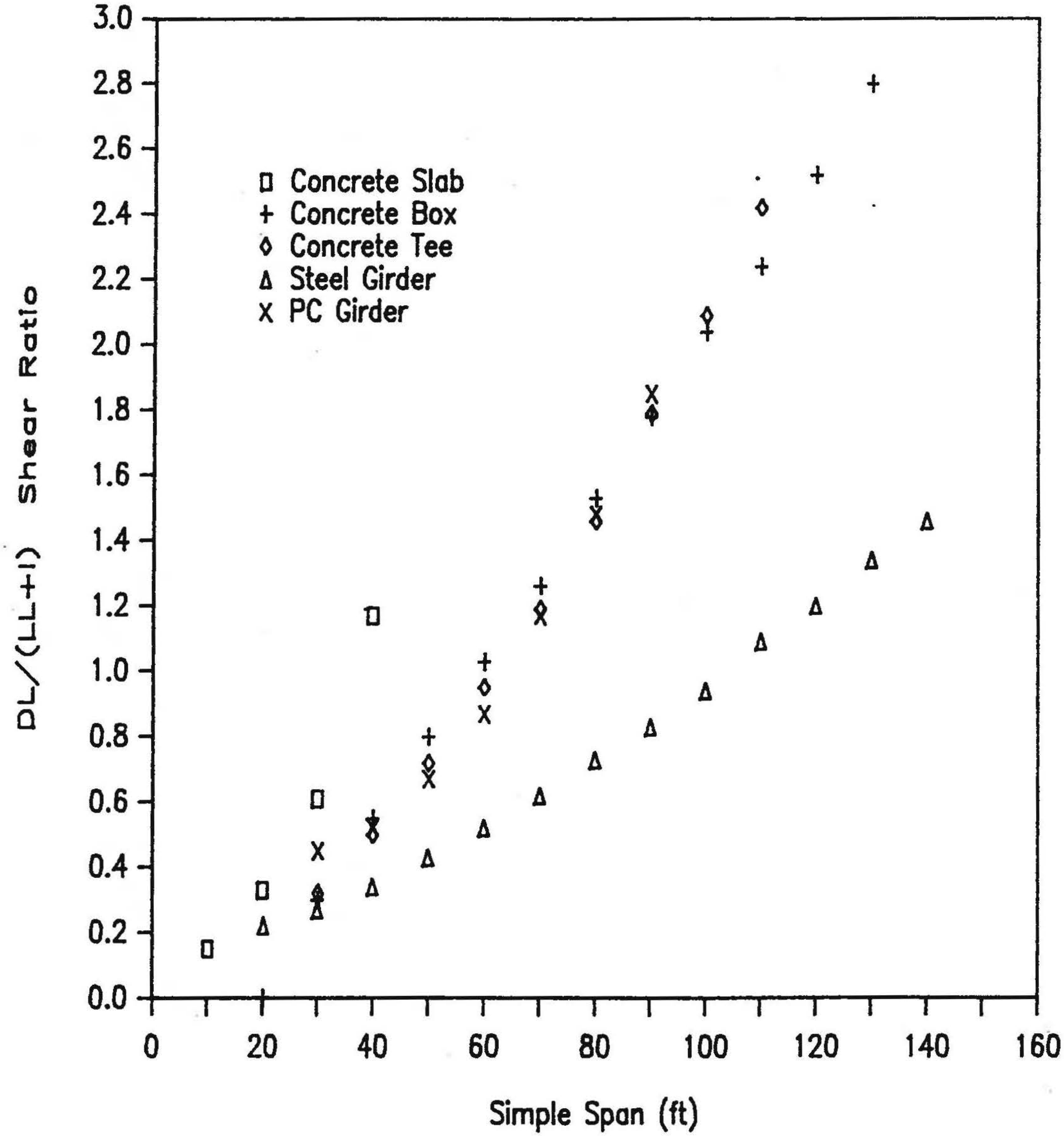


Figure 9. Dead Load Shear Ratios DL/(LL+I) for Simple Span, Noncomposite HS20 Bridges

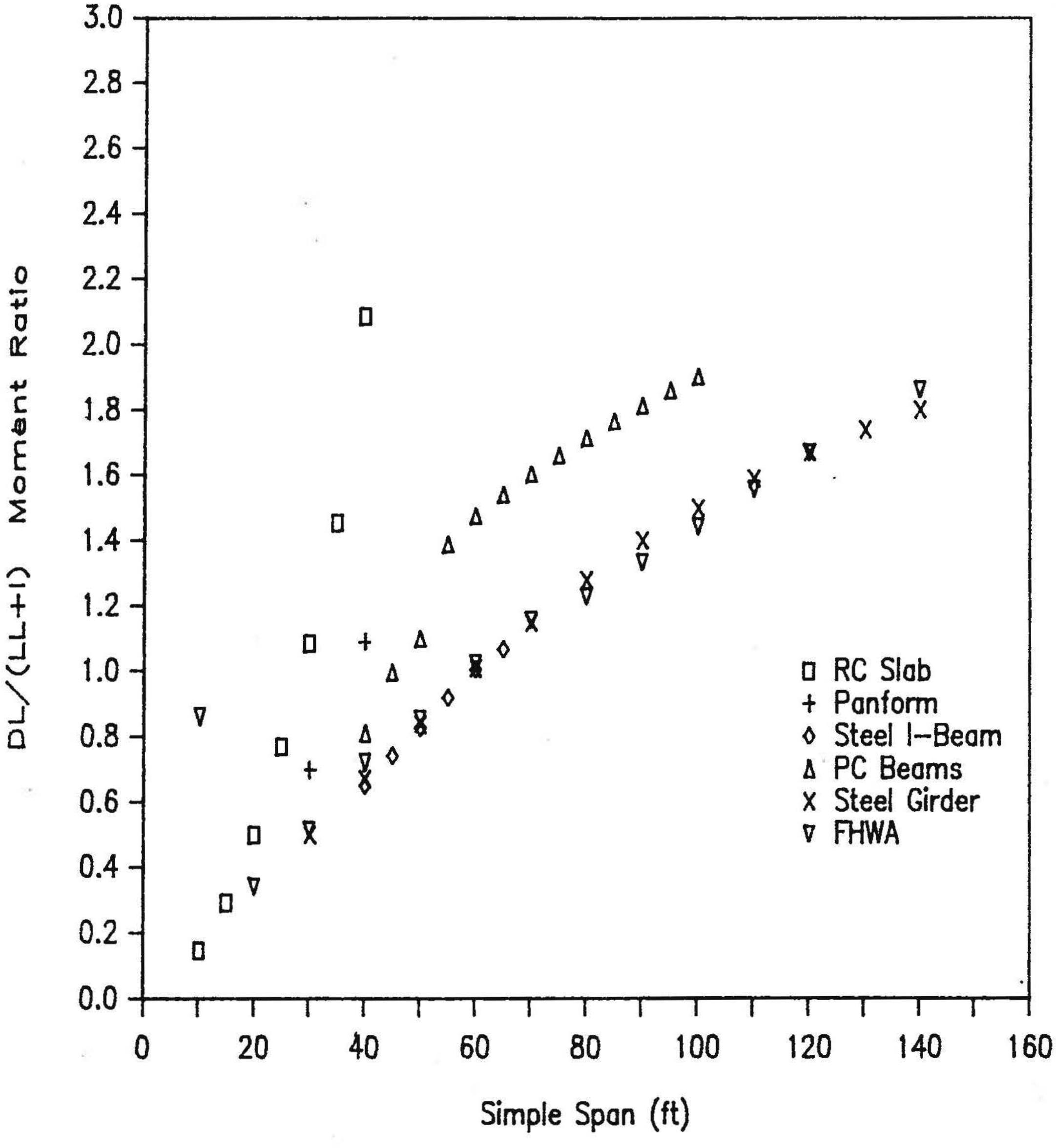


Figure 10. Dead Load Moment Ratios DL/(LL+I) for Simple Span, Noncomposite H15 Bridges

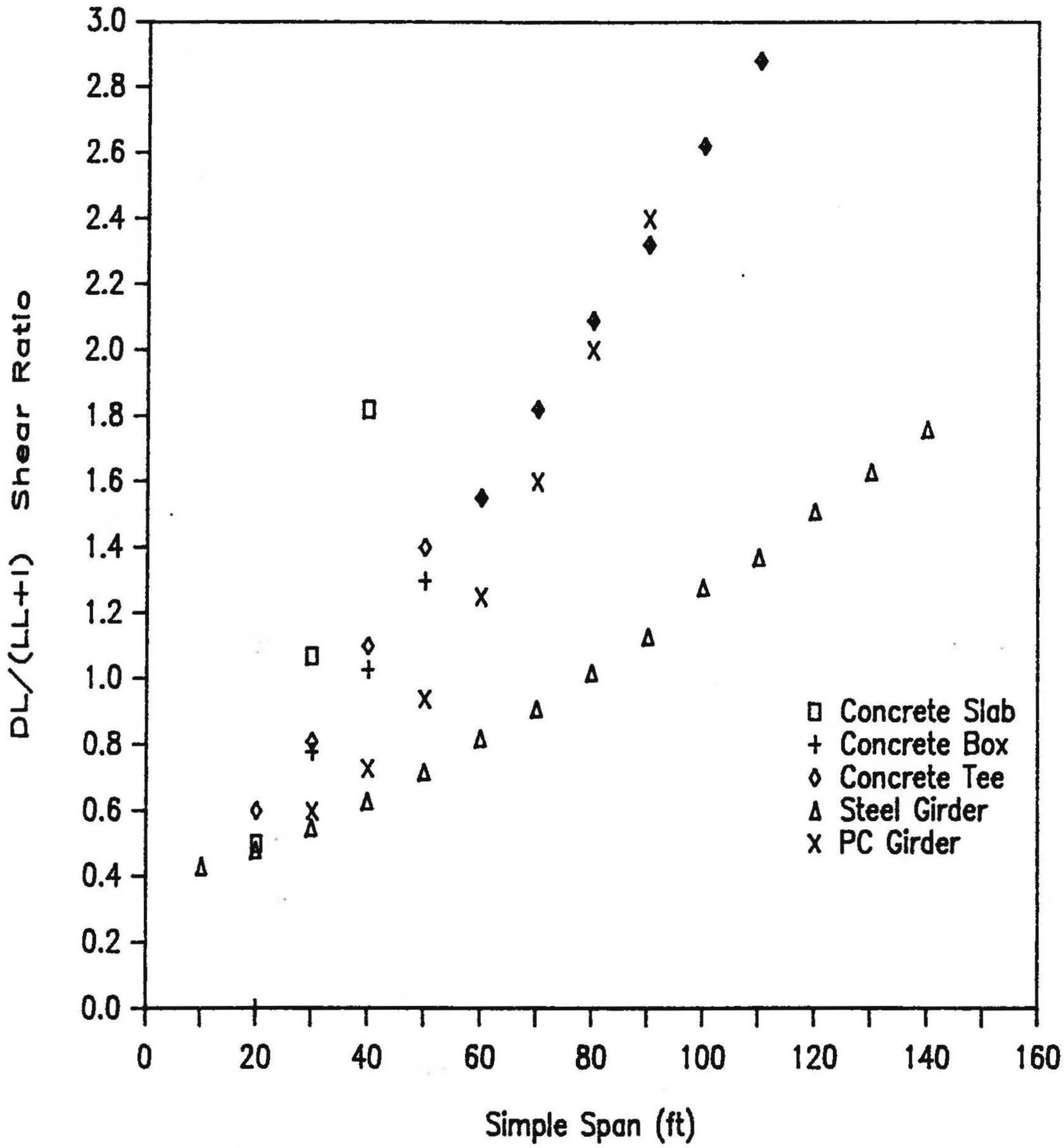


Figure 11. Dead Load Shear Ratios DL/(LL+I) for Simple Span, Noncomposite H15 Bridges

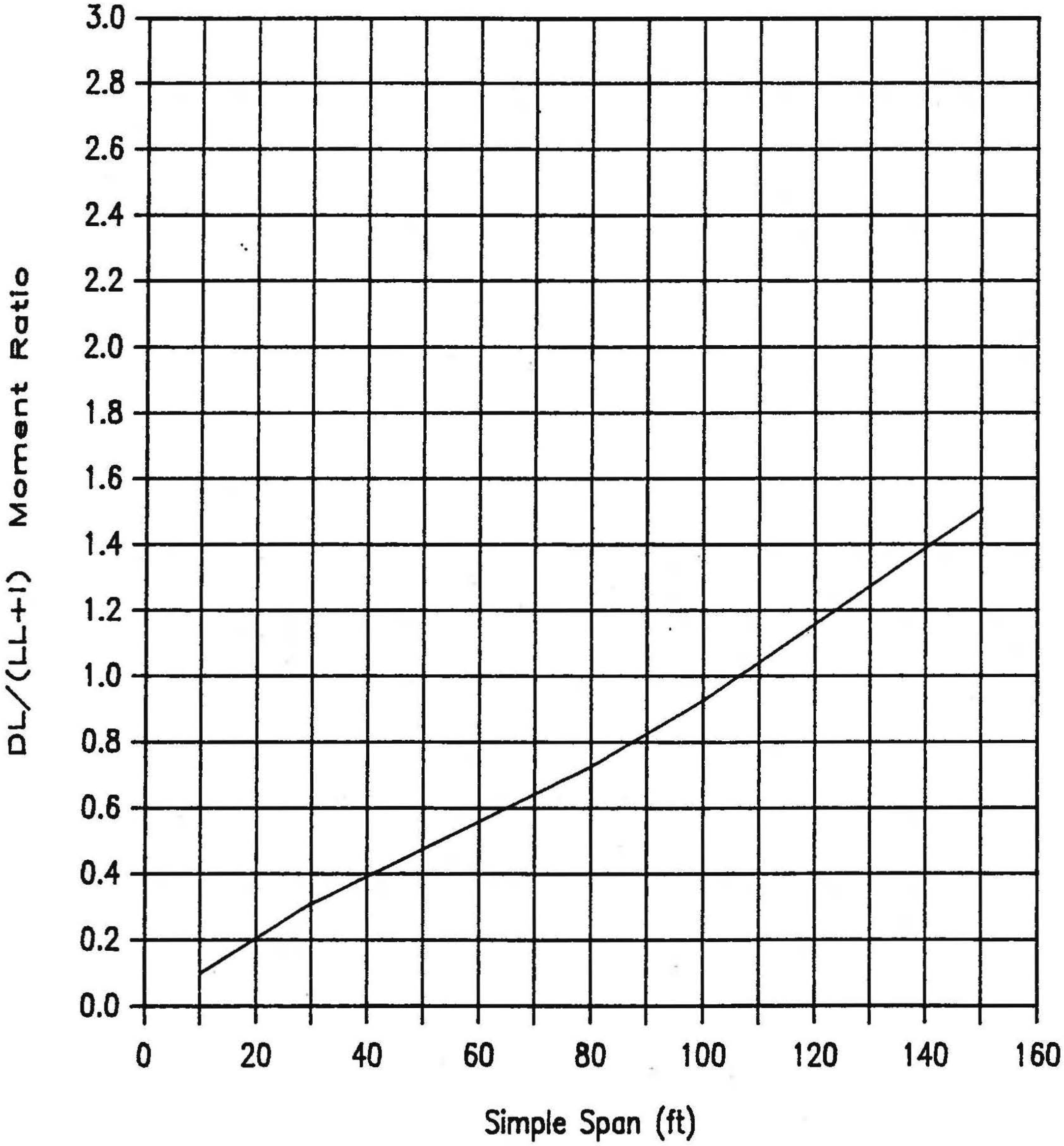


Figure 12. Idealized Moment Ratios DL/(LL+I) for HS20 Bridges

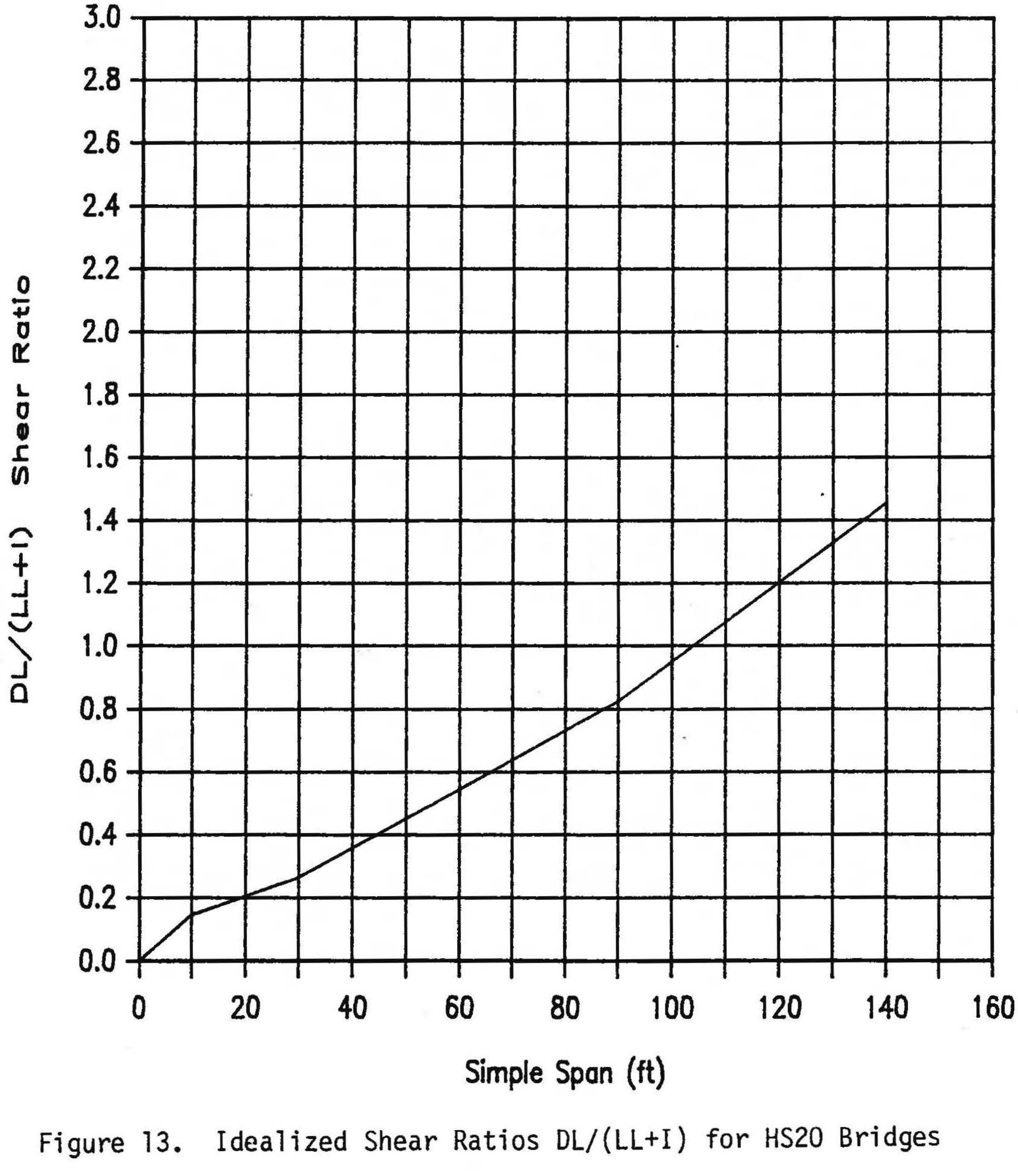


Figure 13. Idealized Shear Ratios DL/(LL+I) for HS20 Bridges

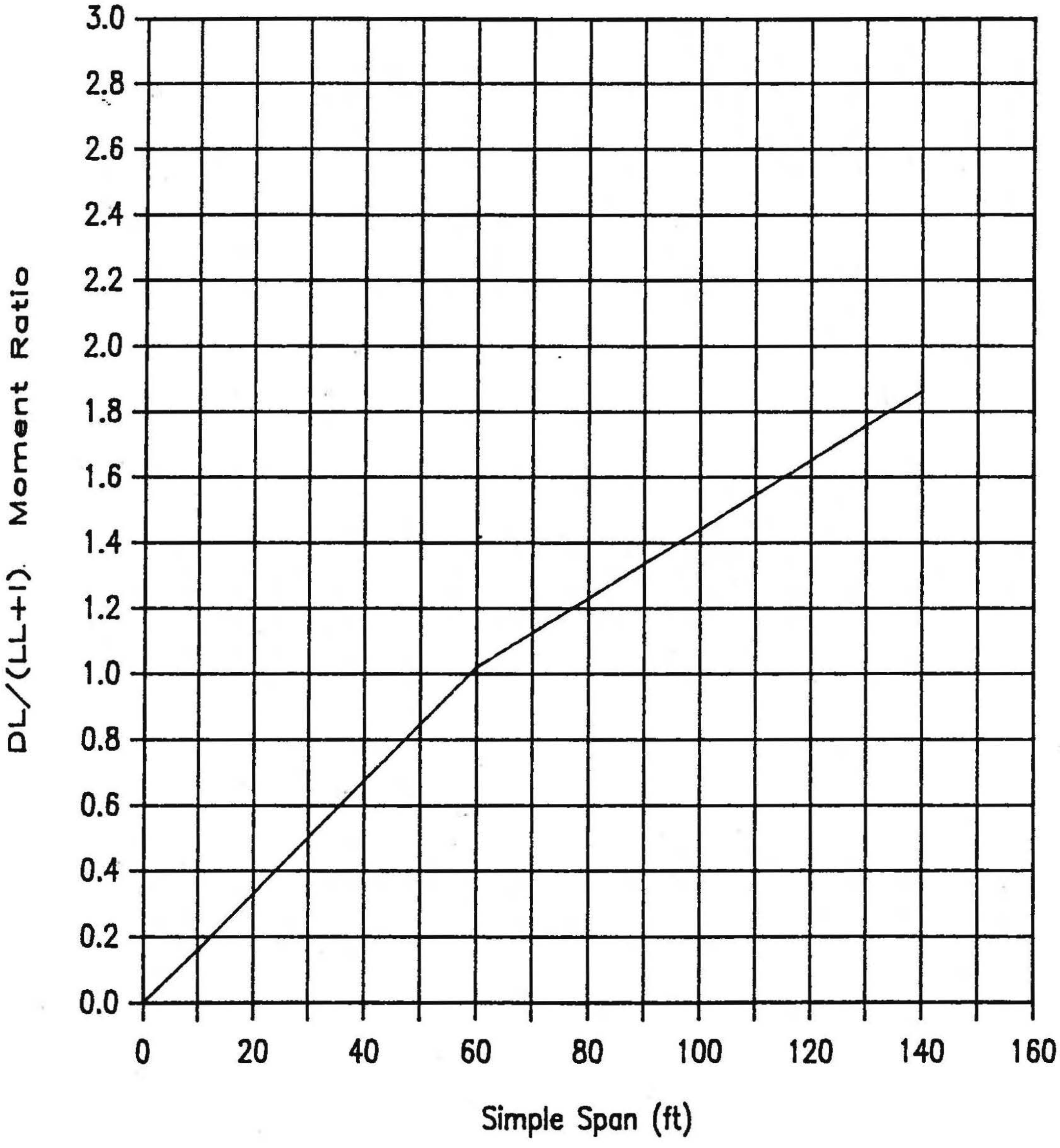


Figure 14. Idealized Moment Ratios DL/(LL+I) for H15 Bridges

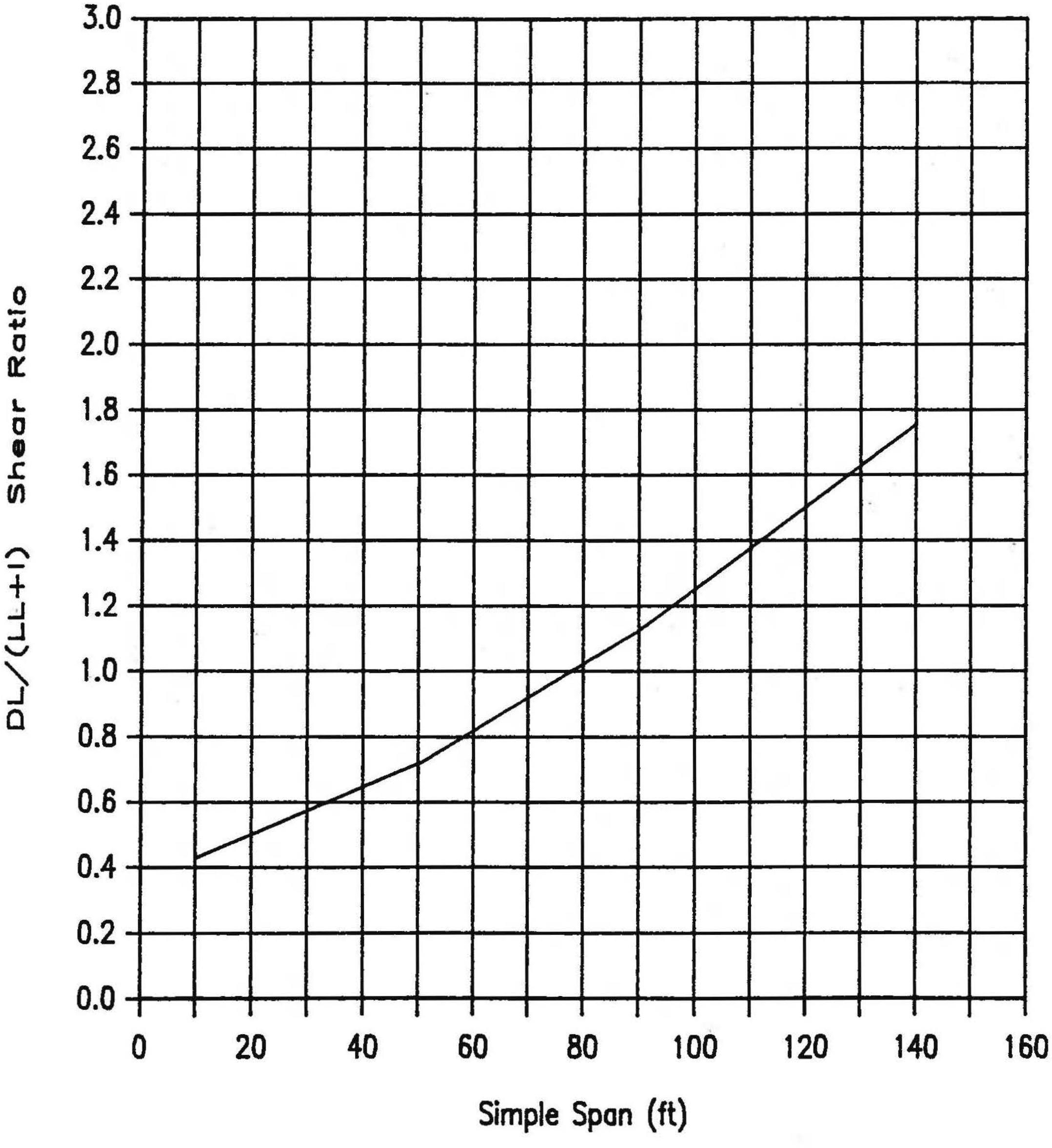


Figure 15. Idealized Shear Ratios DL/(LL+I) for H15 Bridges

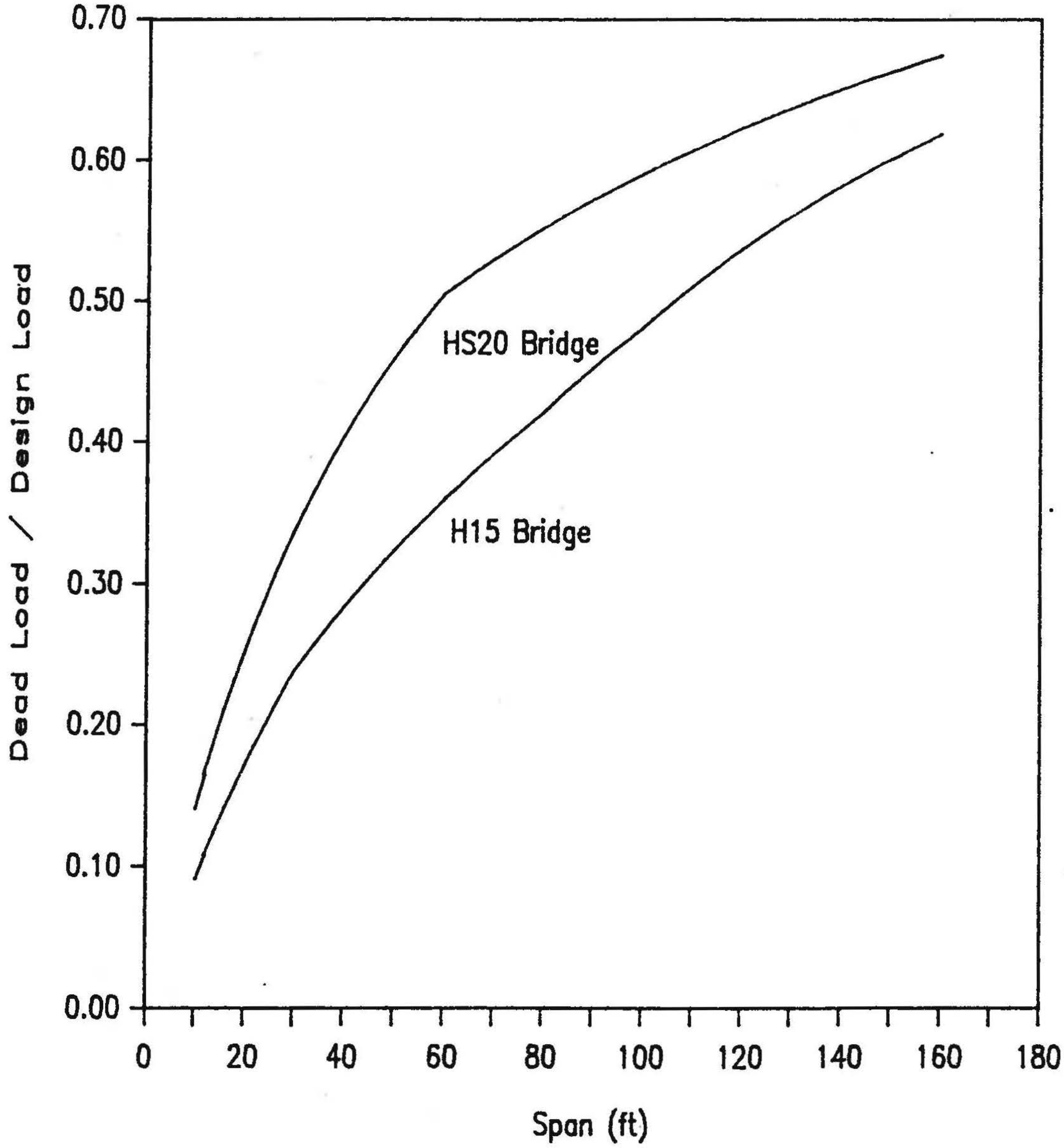


Figure 16. Dead Load Moment Ratios DL/(DL+LL+I) for H15 and HS20 Bridges

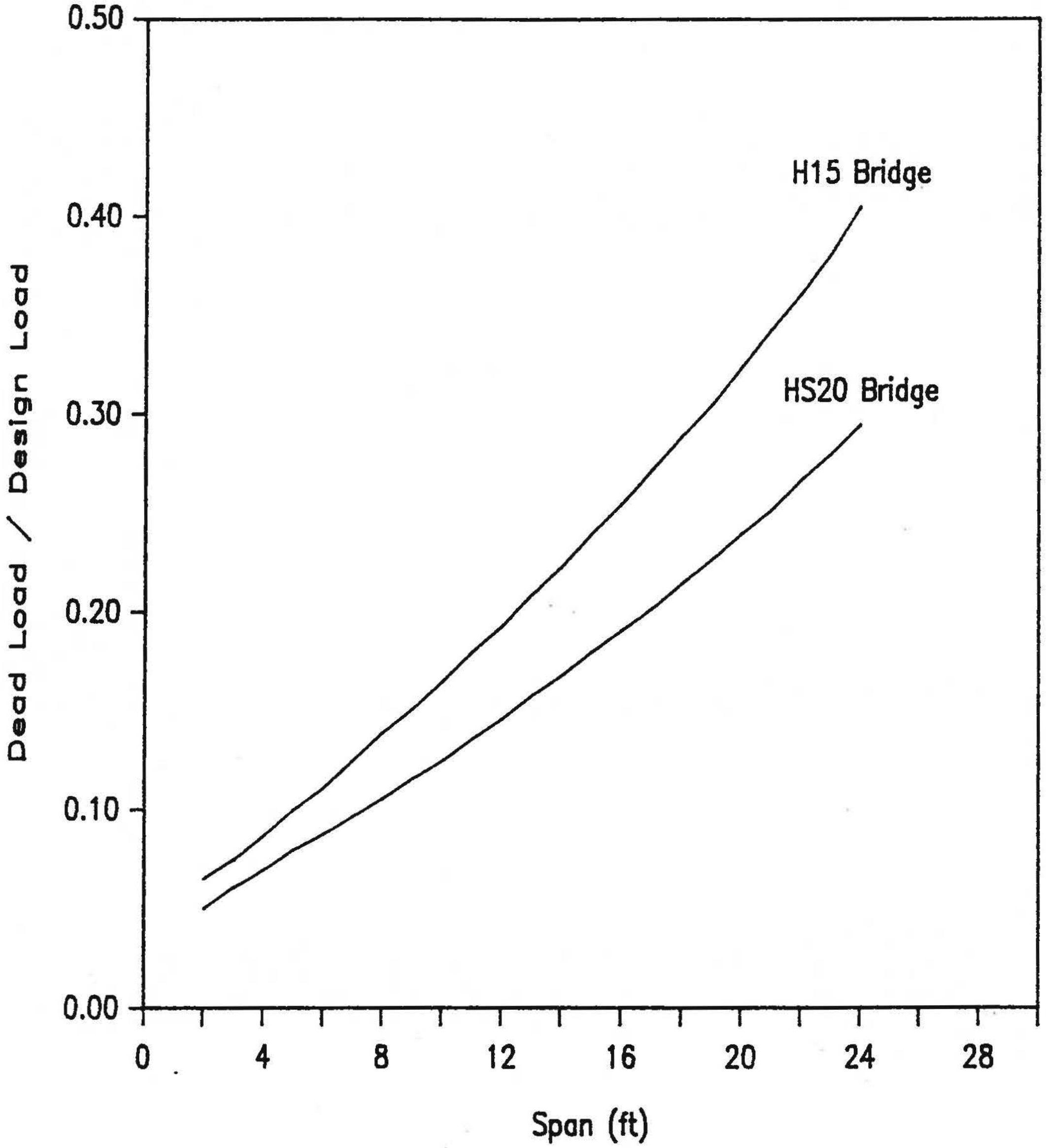


Figure 17. Dead Load Shear Ratios DL/(DL+LL+I) for H15 and HS20 Bridges

Table 8. Dead Load Ratios DL/(LL+I) -- HS20 Bridges

Composite Welded Plate Girders
"Highway Structures Design Handbook
Vol. II. Application Examples"
United States Steel Corp. (From Reference 18)

SPAN	MOMENT		SHEAR	
(ft)	B1	B2	B1	B2
80	0.658	0.121	0.603	0.111
100	0.555	0.102	0.574	0.106
100	0.458	0.100	0.574	0.106
128	0.351	0.186		
273	0.839	0.257	1.403	0.318

1 ft = 0.3048 m

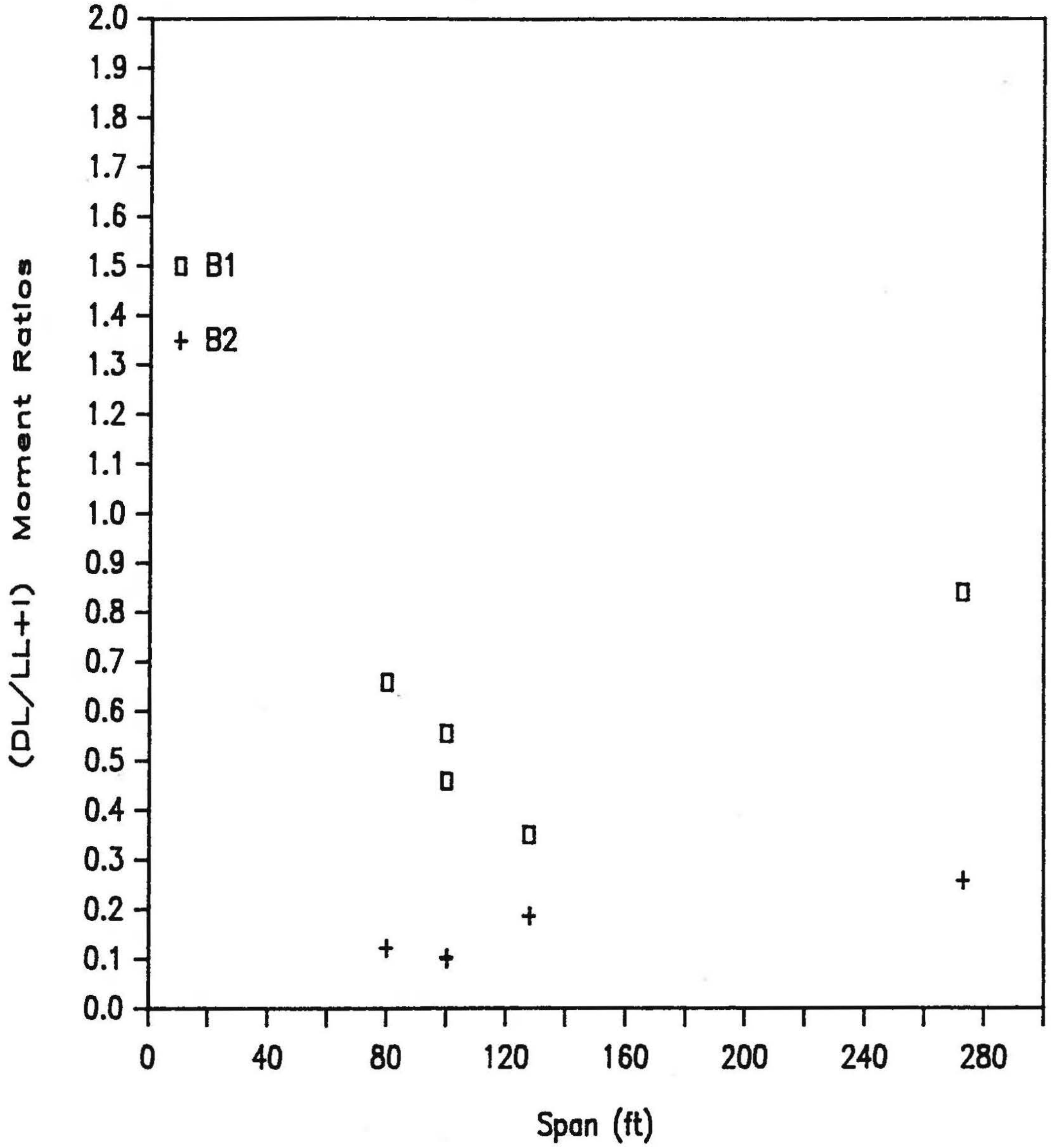


Figure 18. Dead Load Moment Ratios DL/(LL+I) for Continuous, Composite Welded Plate Girders -- HS20

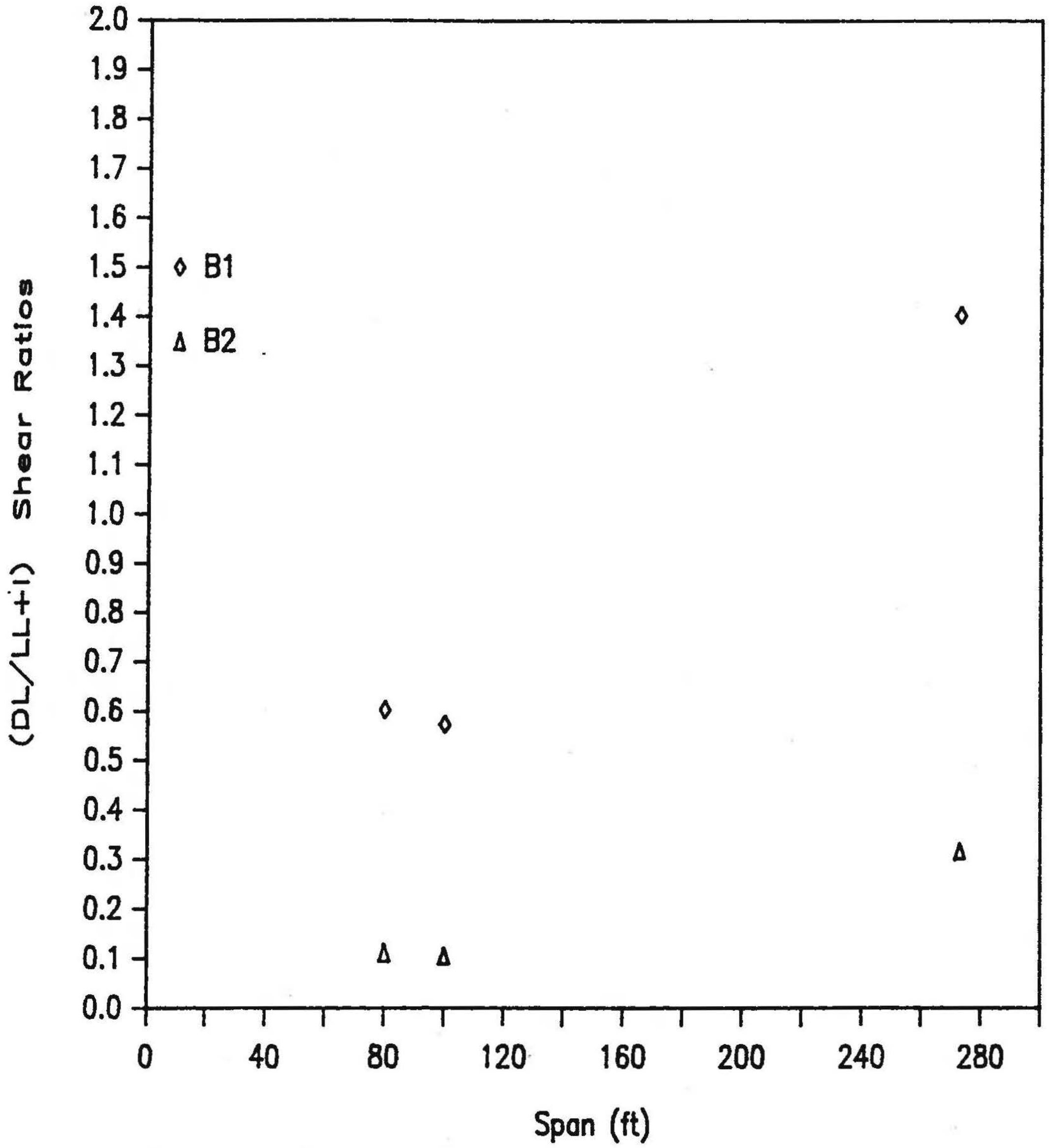


Figure 19. Dead Load Shear Ratios DL/(LL+I) for Continuous, Composite Welded Plate Girders -- HS20

CALCULATION OF SERVICE LOADS TO CAUSE SPECIFIED STRESS RATIOS

Bridges Designed by the Working Stress Design (WSD) Procedure

As discussed above, data for dead load moment and shear ratios was collected for H15 and HS20 designed bridges. These moment and shear dead load stress ratios are tabulated and plotted in the preceding section in the form of

$$B = DL/(LL+I) \tag{3}$$

Also shown are piecewise linear models representing the lower bounds, or the most critical values, of the stress ratios versus span length. These piecewise linear simplifications can be used in calculations of overstress ratios for various axle configurations.

Ratios of DL/(DL+LL+I) or LL/(DL+LL+I) can be expressed in terms of the given B ratios using the following identities:

$$DL/TL = B/(1+B) \tag{4}$$

$$(LL+I)/TL = 1/(1+B) \tag{5}$$

where TL = DL+LL+I is the total design load. Plots of DL/TL are shown in figures 16 and 17.

For simple span, noncomposite bridges designed by the working stress design (WSD) method, these B ratios can be used to calculate an allowable live load (LL+I) shear $\bar{V}_L$ or moment $\bar{M}_L$ for a specified allowable overstress ratio as follows:

$$\bar{M}_L = M_L\,[\Omega + (\Omega - 1)B_M] \tag{6}$$

$$\bar{V}_L = V_L\,[\Omega + (\Omega - 1)B_V] \tag{7}$$

where M_L is the design (LL+I) moment,

V_L is the design (LL+I) shear,

B_M is the DL/(LL+I) moment ratio,

B_V is the DL/(LL+I) shear ratio, and

Ω is the specified overstress ratio.

Bridges Designed by the Load Factor Design (LFD) Procedure

Since 1973 the load factor design (LFD) procedure has been used to design increasing numbers of highway bridges. The LFD bridges are still certainly in the minority, and the LFD method may never completely replace the working stress design (WSD) procedure for design of highway bridges, but the economics of the LFD procedure are most likely to be beneficial to long span structures where the dead load ratios are the greatest. The use of the LFD method will result in lower dead load ratios however, and because the proposed truck weight formula is based on a lower bound of the dead load ratios of data collected mostly from WSD bridges, the effects of the proposed truck weight formula on LFD bridges must be evaluated.

First, a summary of the LFD method, as applied to highway bridges, is appropriate. The LFD method is outlined in the AASHTO Standard Specifications(9), article 1.2.22, and sections 1.5 and 1.7. It can be briefly summarized as follows for main structural members made of steel or reinforced concrete:

Steel Structures

Strength Considerations:

Maximum Design Load: Internal resultants due to

$1.3(DL+1.67(LL+I)) < \text{Strength}$

For bridges designed for trucks lighter than H20, the resultants due to the following factored loads (without concurrent loading of adjacent lanes) are also checked against the factored strength:

$1.3(DL+2.20(LL+I))$

Serviceability Considerations:

Service Load: Stress due to $DL+(LL+I) <$ Allowable stress for repeated loadings

and Stress due to $(LL+I) <$ Allowable stress range

Overload: Stress due to DL+1.67(LL+I) < Stress causing permanent deformations

Concrete Structures

Strength Considerations:

Maximum Design Load: Internal resultants due to
1.3(DL+1.67(LL+I) < Factored strength (strength factors vary from 0.7 to 0.9)

For bridges designed for trucks lighter than H20, the resultants due to the following factored loads (without concurrent loading of adjacent lanes) are also checked against the factored strength:

1.3(DL+2.20(LL+I))

Serviceability Considerations:

Service Load: Stresses in concrete and reinforcement due to
DL+(LL+I) < Allowable stresses for repeated loads

Deflections due to
DL+(LL+I) < Allowable deflections

The proposed truck weight limiting formula has been developed based on an analysis of the critical weights of various vehicles on typical WSD bridges, therefore the effects of increased truck weights on LFD bridges must be considered separately.

The effect of a proposed formula which might allow higher truck weights is to cause higher service loading for an LFD structure. Since the procedure for LFD for service loading is identical to the WSD procedure, trucks which cause a 5% overstress in HS20 WSD bridges will not cause more than a 5% increase in the design service stress in an HS20 LFD bridge. The factor of safety against ultimate capacity for the two bridges under the same truck will in general be different, however. The occasional overload provision of the LFD procedure will not be violated as long as the ratio of the overload (LL+I) moment (caused by a legal truck under a proposed new formula) to the design (LL+I) moment (caused by the design truck) does not exceed 5/3.

Composite Steel-Concrete Bridges

The effects of a proposed formula which might allow higher truck weights on composite bridges is to increase the service load stresses on the composite section. The expressions for the maximum allowable overload moment, i.e., that moment which causes a specified overstress ratio, are as follows:

$$\bar{M}_L = M_L\,[\Omega + (\Omega - 1)B_2 + (\Omega - 1)(S_{b2}/S_{b1})B_1] \qquad (8)$$

for stresses at the bottom of the composite section, and

$$\bar{M}_L = M_L\,[\Omega + (\Omega - 1)B_2] \qquad (9)$$

for stresses at the top of the composite section. In these two expressions,

$\bar{M}_L$ is the allowable overload (LL+I) moment,

M_L is the design (LL+I) moment,

Ω is the specified overstress ratio (i.e., 1.05 for HS20 bridges),

B_1 is the ratio of DL/(LL+I) for the DL_1, which acts only on the noncomposite section, due to the girders and deck,

B_2 is the ratio of the DL/(LL+I) for the superimposed DL_2, which acts on the composite section, due to the curbs, wearing surface, etc.,

S_{b1} is the noncomposite bottom fiber section modulus, and

S_{b2} is the composite bottom fiber section modulus.

RATIONALE FOR THE FORMULA

Consider a given load distributed equally among a specified number of wheels, N, equally spaced along a simply supported beam assuming that the beam span is greater than the outside dimension of the wheels. As the number of wheels increases, so does the maximum moment. Taken to the limit, this means that the maximum moment occurs at the center of the beam as the number of wheels approaches infinity, i.e., a uniform load.

With this rationale in mind, uniform loads with overall lengths varying in 1 ft (0.3048 m) intervals were placed on simple spans having the dead load to design load ratios shown in figure 16 for moment and figure 17 for shear. The magnitude of the uniform load required to cause a moment (or shear) equal to or greater than 1.05 times that used for the design of an HS20 bridge and 1.3 times that used for the design of an H15 bridge was calculated. This calculation was made for each span and resulted in curves such as those shown in figure 20, one for the HS20 and one for the H15. Figure 20 illustrates the calculation for a 24 ft (7.315 m) uniform load; but the same calculations were made, in 1 ft (0.3048 m) intervals, for all load lengths from 8 to 120 ft (2.438 to 36.58 m). These calculations and curves result in a unique critical span for each condition. This critical span defines the maximum uniform load of the given length that can be allowed. Any greater load would cause the stress ratios of 1.05 or 1.3 to be exceeded in the respective critical spans.

These maximum uniform loads were then plotted as a function of their lengths. See figure 21. It is interesting that H15 bridges with the 1.3 factor control the maximum uniform loads up to near 70 ft (21.34 m), but that HS20 bridges with the smaller 1.05 factor control the longer load lengths. This occurs because long bridge spans control the total load on long wheelbases (uniform loads). As the dead load to total load ratios become large, the 1.3 factor applied to the H15 live load moment becomes larger than the 1.05 increase in the HS20 live load moment. Although this result has several implications, the most important is that consideration of HS20 bridges would be the primary criterion for the gross weight of the very

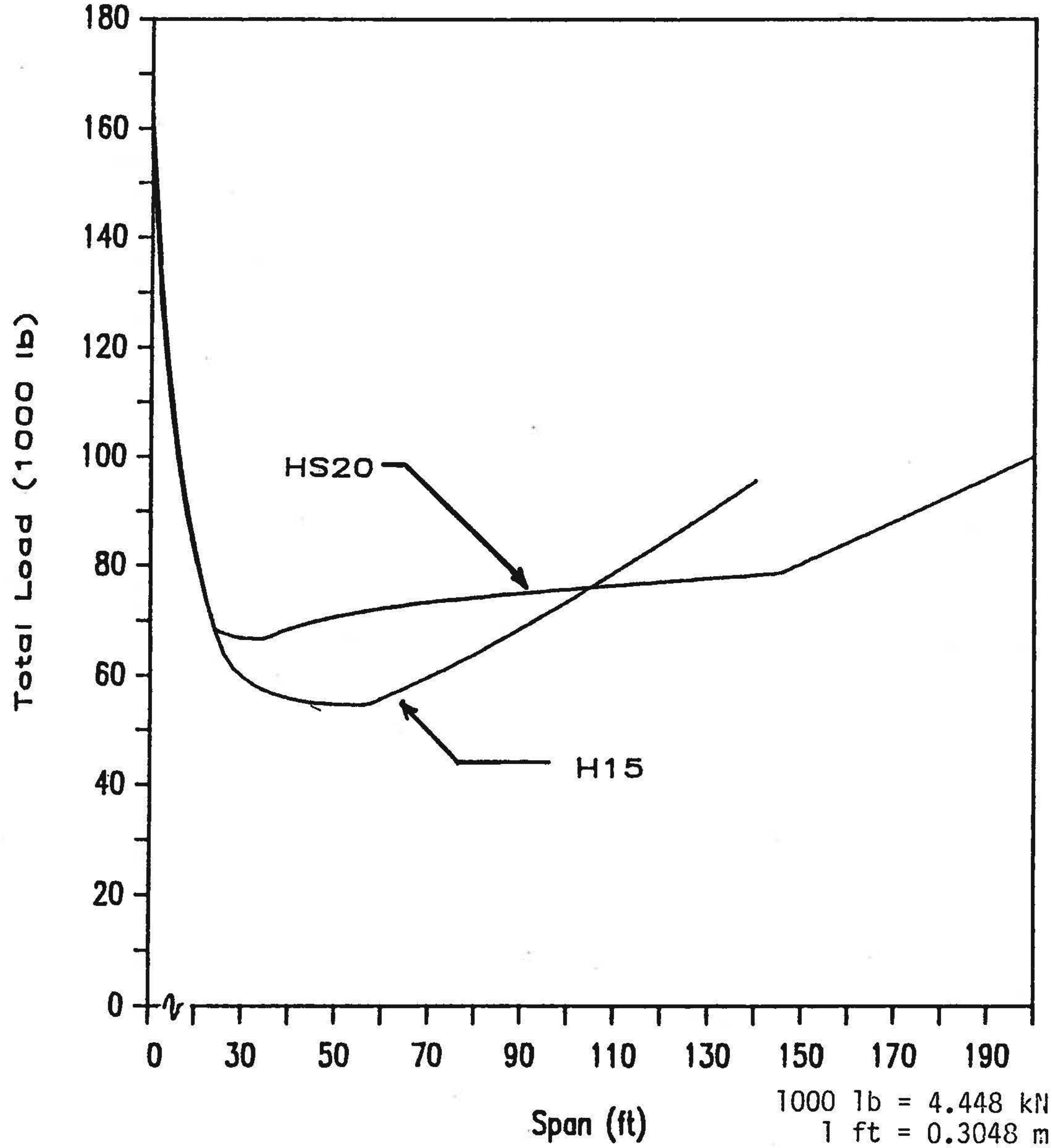
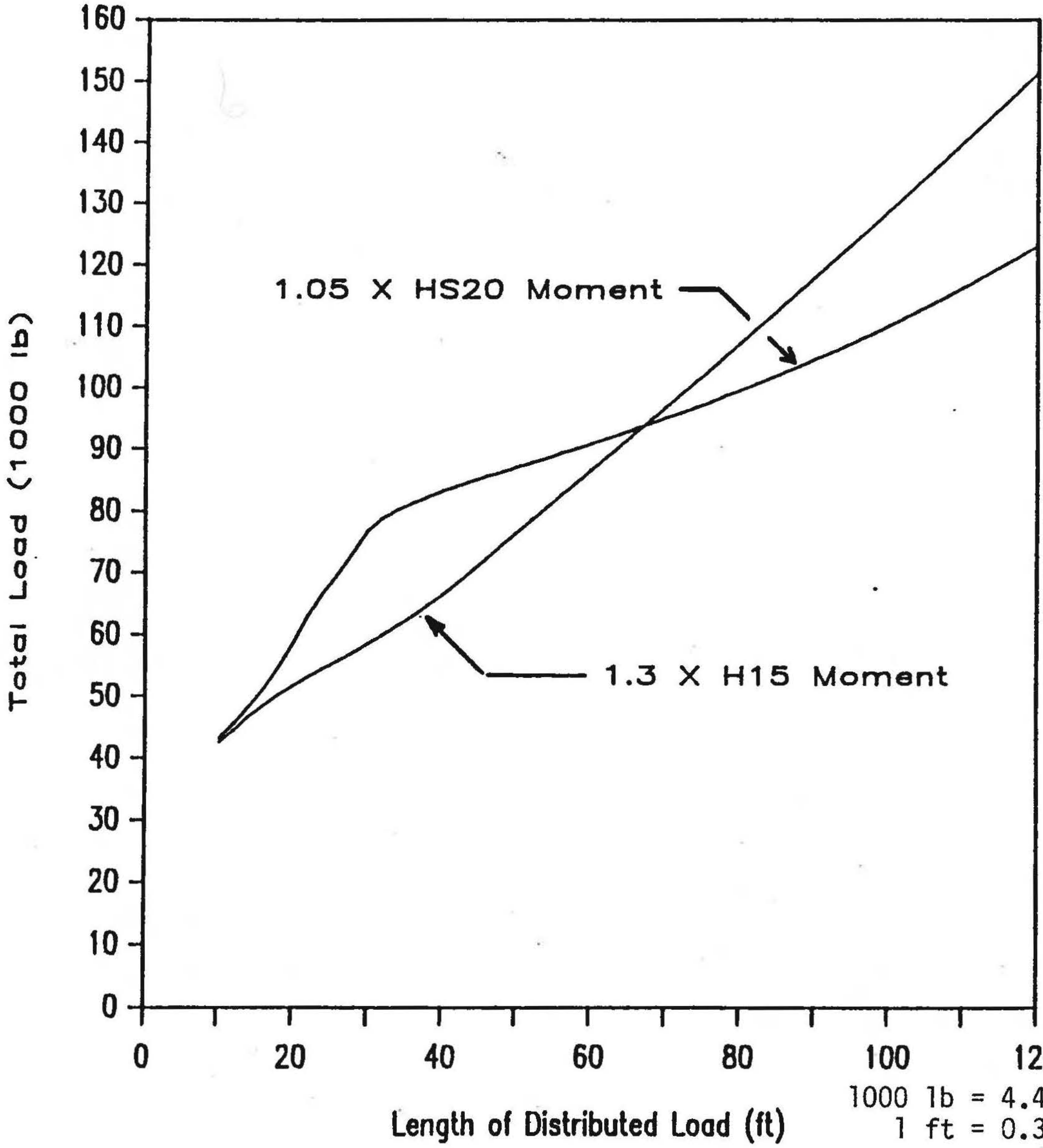


Figure 20. The Maximum Uniform Load, 24 ft (7.32 m) Long, Required to Cause the Specified Ratios of Design Moments on Simple Bridge Spans. Note that a 33 ft (10.06 m) Span is Critical for an HS20 Bridge, while a 57 ft (17.37 m) Span is Critical for an H15 Bridge



1000 lb = 4.448 kN
1 ft = 0.3048 m

Figure 21. The Maximum Uniform Loads, as a Function of Load Length, Defined by the Stress Ratio. Note that H15 Bridges are Critical for Vehicle Lengths up to 67 ft (20.42 m), but HS20 Bridges are Critical for the Longer Lengths

long vehicles should they ever be allowed. These maximum uniform loads provided the bases for the truck formula being suggested.

PROPOSED BRIDGE FORMULA

The maximum uniform load curves were used as a guide to draw the two straight lines of figure 22. The equation of each straight segment and the wheelbases over which they are valid are

$$W = (34 + L)1000 \text{ lb} \qquad 8 \text{ ft} \le L \le 56 \text{ ft } (2.438 \text{ m} \le L \le 17.07 \text{ m}) \qquad (10)$$

$$W = (62 + L/2)1000 \text{ lb} \qquad 56 \text{ ft} \le L \quad (17.07 \text{ m} \le L)$$

where W = total weight in lb over a wheelbase L in ft.

A table, based on the two straight line formula, has been completed. See table 9. The maximum weights for any group of axles up to a wheelbase of 120 ft (36.58 m) are shown in thousands of pounds. As implied by the term "bridge formula", these allowable weights are equally applicable to all the wheels under a truck or to any subgroup of axles using the outside dimension of the subgroup as the length.

This formula does not guarantee that the prescribed bridge stresses will never be exceeded. Some of the examples shown below clearly illustrate truck configurations that will be legal yet cause more than 1.3 times the H15 bridge design stresses. Other examples show that some legal trucks can cause stresses slightly greater than 1.05 times the HS20 design stresses. However, it is believed these exceptions represent rare vehicular configurations and that the suggested formula is an improvement from the standpoint of utilizing, but not abusing, the nation's bridges.

The enforcement of the proposed formula should be easier than Table B primarily because the number of axles is not a factor. Associated with the out-to-out length of any group of wheels is a specific gross weight. It is, of course, implied that the single and tandem axle maximums may not be exceeded. The 5 percent average allowed for the design stress of the HS20 bridges further implies an allowance for weighing errors and inadvertent load shifting, thus making the formula loads absolute maximums.

ASSUMPTIONS

The calculations leading to the two straight line formula all assume the following conditions in consonance with the AASHTO Bridge Specification.[9]

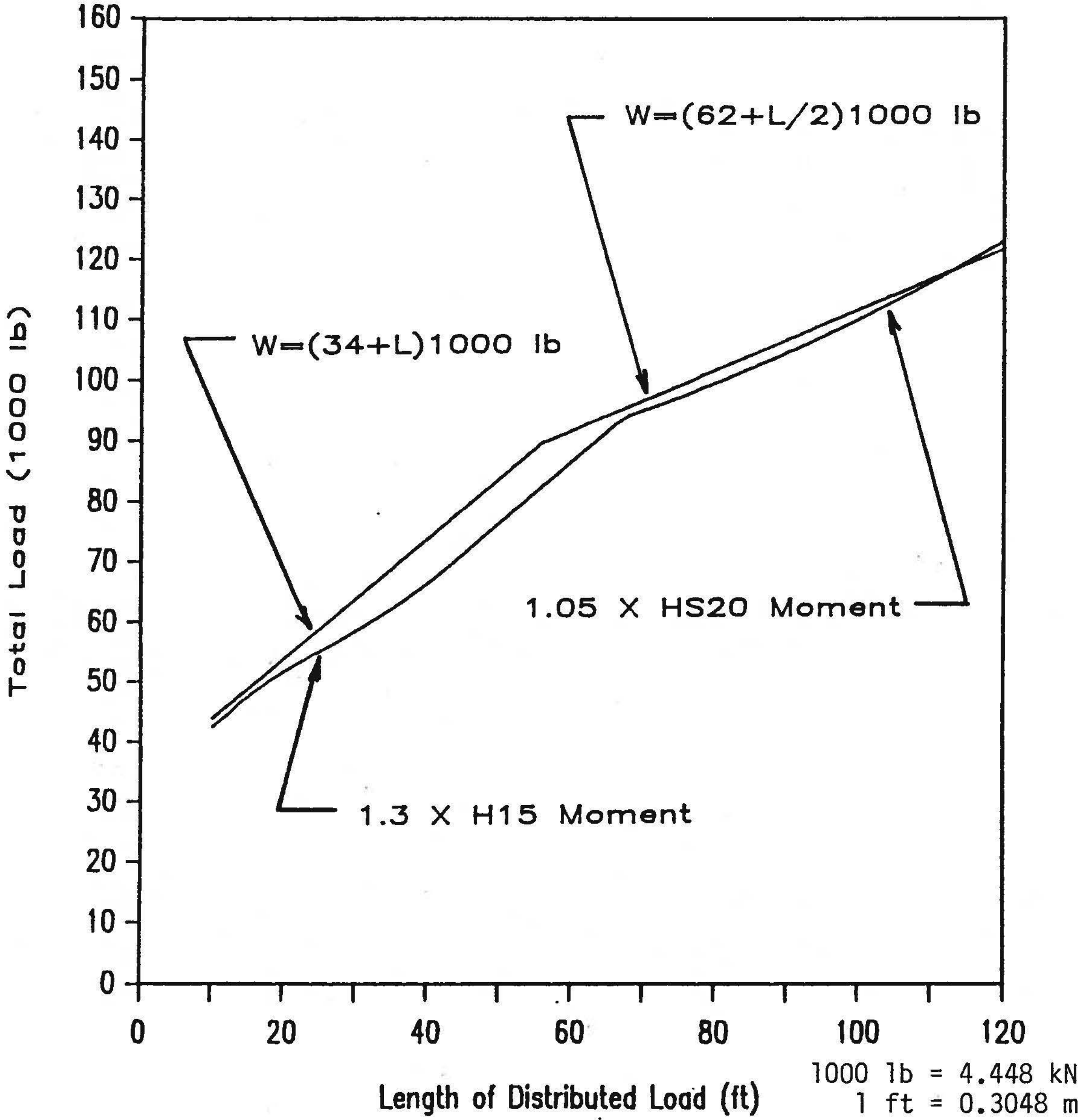


Figure 22. The Proposed Bridge Formula Compared to the Derived Maximum Uniform Loads that Cause the Specified Overstresses (1.05 for HS20, 1.3 for H15 Bridges)

Table 9. Permissible Loads Based on Proposed Formula

Wheelbase (ft)	Number of Axles 2	3	4	5	6	7	8	9
4	34.0							
5	34.0							
6	34.0							
7	34.0							
8	34.0	42.0						
9	39.0	43.0						
10	40.0	44.0						
11		45.0						
12		46.0	46.0					
13		47.0	47.0					
14		48.0	48.0					
15		49.0	49.0					
16		50.0	50.0	50.0				
17		51.0	51.0	51.0				
18		52.0	52.0	52.0				
19		53.0	53.0	53.0				
20		54.0	54.0	54.0	54.0			
21		55.0	55.0	55.0	55.0			
22		56.0	56.0	56.0	56.0			
23		57.0	57.0	57.0	57.0			
24		58.0	58.0	58.0	58.0	58.0		
25		59.0	59.0	59.0	59.0	59.0		
26		60.0	60.0	60.0	60.0	60.0		
27		60.0	61.0	61.0	61.0	61.0		
28		60.0	62.0	62.0	62.0	62.0	62.0	
29		60.0	63.0	63.0	63.0	63.0	63.0	
30		60.0	64.0	64.0	64.0	64.0	64.0	
31		60.0	65.0	65.0	65.0	65.0	65.0	
32		60.0	66.0	66.0	66.0	66.0	66.0	66.0
33			67.0	67.0	67.0	67.0	67.0	67.0
34			68.0	68.0	68.0	68.0	68.0	68.0
35			69.0	69.0	69.0	69.0	69.0	69.0
36			70.0	70.0	70.0	70.0	70.0	70.0
37			71.0	71.0	71.0	71.0	71.0	71.0
38			72.0	72.0	72.0	72.0	72.0	72.0
39			73.0	73.0	73.0	73.0	73.0	73.0
40			74.0	74.0	74.0	74.0	74.0	74.0
41			75.0	75.0	75.0	75.0	75.0	75.0
42			76.0	76.0	76.0	76.0	76.0	76.0
43			77.0	77.0	77.0	77.0	77.0	77.0
44			78.0	78.0	78.0	78.0	78.0	78.0
45			79.0	79.0	79.0	79.0	79.0	79.0

$W = 34 + L$ $\quad 8 \leq L \leq 56$ ft

$W = 62 + L/2$ $\quad L \geq 56$ ft

except for $N = 2$

1 ft = 0.3048 m

1000 lb = 4.448 kN

Table 9. Permissible Loads Based on Proposed Formula (continued)

	Number of Axles							
Wheelbase (ft)	2	3	4	5	6	7	8	9
46			80.0	80.0	80.0	80.0	80.0	80.0
47			80.0	81.0	81.0	81.0	81.0	81.0
48			80.0	82.0	82.0	82.0	82.0	82.0
49			80.0	83.0	83.0	83.0	83.0	83.0
50			80.0	84.0	84.0	84.0	84.0	84.0
52			80.0	86.0	86.0	86.0	86.0	86.0
54			80.0	88.0	88.0	88.0	88.0	88.0
56			80.0	90.0	90.0	90.0	90.0	90.0
58				91.0	91.0	91.0	91.0	91.0
60				92.0	92.0	92.0	92.0	92.0
62				93.0	93.0	93.0	93.0	93.0
64					94.0	94.0	94.0	94.0
66					95.0	95.0	95.0	95.0
68					96.0	96.0	96.0	96.0
70					97.0	97.0	97.0	97.0
72					98.0	98.0	98.0	98.0
74					99.0	99.0	99.0	99.0
76					100.0	100.0	100.0	100.0
78					101.0	101.0	101.0	101.0
80					102.0	102.0	102.0	102.0
82					103.0	103.0	103.0	103.0
84					104.0	104.0	104.0	104.0
86					105.0	105.0	105.0	105.0
88					106.0	106.0	106.0	106.0
90					107.0	107.0	107.0	107.0
92					108.0	108.0	108.0	108.0
94					109.0	109.0	109.0	109.0
96					110.0	110.0	110.0	110.0
98					111.0	111.0	111.0	111.0
100					112.0	112.0	112.0	112.0
102					113.0	113.0	113.0	113.0
104					114.0	114.0	114.0	114.0
106					115.0	115.0	115.0	115.0
108					116.0	116.0	116.0	116.0
110					117.0	117.0	117.0	117.0
112					118.0	118.0	118.0	118.0
114					119.0	119.0	119.0	119.0
116					120.0	120.0	120.0	120.0
118					120.0	121.0	121.0	121.0
120					120.0	122.0	122.0	122.0

$W = 34 + L \quad 8 \leq L \leq 56$ ft

$W = 62 + L/2 \quad L \geq 56$ ft

except for $N = 2$

1 ft = 0.3048 m

1000 lb = 4.448 kN

- The impact factor

$$I = \frac{50}{L + 125} \qquad (11)$$

$$I = 0.3 \text{ for } L \geq 41.67 \text{ ft} \quad (12.71 \text{ m})$$

where I is the fractional increase of the live load due to impact and L is the span length in feet.
- In width, only one truck per lane is allowed. For longer spans each lane is considered to have a truck train with the same characteristics that led to the AASHTO design lane loadings.
- The side by side spacing of adjacent vehicles is considered to be 4 ft (1.219 m). Further, in the case of a curb, the spacing to the center of the wheel(s) is 2 ft (0.6096 m).
- The distribution of the wheel loads to longitudinal stringers, whether steel or concrete, was assumed to be that recommended by the Design Specification for both moment and shear. Many consider these distributions to be overly conservative,but they represent current design practice.

DISCUSSION OF THE PROPOSED FORMULA

To examine the possible implications of the new formula a table, identical in format to table 9, was constructed showing the differences from the current truck weight formula. This tabulation, table 10, simply shows the changes, in thousands of pounds, from the current STAA bridge weight law to that being suggested as the replacement. Numbers in parentheses mean that the new formula allows less weight; all others, more weight.

For axle groupings and wheelbases of three or less axles there is no reduction in the maximum weight, but for short coupled groupings of four or more axles there are significant reductions. These reductions are not without good reason. Virtually without exception the weights allowed by Table B on the short-coupled groupings of four or more axles exceed the tolerable stresses in H15 bridges. This is exactly the phenomenon recognized in the long forgotten footnote to the original Table B.[5]

OBSERVED AXLE GROUP WEIGHTS

Tapes of data collected in 1983 reflecting the results of loadometer surveys performed by several States in cooperation with the Federal Highway Administration were obtained for analysis and comparison with the proposed formula. These data include information defining the type of truck and the weights and spacings of the individual axles. This made it possible to easily tabulate the weights of all axle groupings, i.e., all single axles and groups of two axles, three axles, four axles, and five axles. The details of this procedure are described in Appendix B. But for clarity it is reiterated that these groupings are observations from all vehicles, not just those with the specified number of axles. For example, a vehicle with three total axles yields two data points in the two-axle group, the first and second axle and the second and third axle.

These tabulations were summarized in increments of length of 8 ft (2.438 m) and in increments of weight of 10,000 lb (4.448 kN). Tables 11 to 14 show the results. Superposed over the tables are lines showing both the current Formula B, for the specified number of included axles, and the proposed formula, which is, of course, independent of the number of included axles.

Table 10. Changes in Allowable Weights, in Thousands of Pounds, from Formula B (Current Law) to the Proposed Formula.

1 ft = 0.3048 m
1000 lb = 4.448 kN

Wheelbase	Number of Axles 3	4	5	6	7	8
4						
6						
8	(0.00)*					
10	0.50					
12	1.00	(4.00)				
14	1.50	(3.33)				
16	2.00	(2.67)	(8.00)			
18	2.50	(2.00)	(7.25)			
20	3.00	(1.33)	(6.50)	(12.00)		
22	3.50	(0.67)	(5.75)	(11.20)		
24	4.00	(0.00)	(5.00)	(10.40)	(16.00)	
26	4.50	0.67	(4.25)	(9.60)	(15.17)	
28	3.00	1.33	(3.50)	(8.80)	(14.33)	(18.00)
30	1.50	2.00	(2.75)	(8.00)	(13.50)	(16.00)
32	0.00	2.67	(2.00)	(7.20)	(12.67)	(14.00)
34		3.33	(1.25)	(6.40)	(12.00)	(12.00)
36		2.00	(0.50)	(5.60)	(10.00)	(10.00)
38		4.00	0.25	(4.80)	(8.00)	(8.00)
40		5.33	1.00	(4.00)	(6.00)	(6.00)
42		6.00	1.75	(3.20)	(4.00)	(4.00)
44		6.67	2.50	(2.00)	(2.00)	(2.00)
46		7.33	3.25	(0.00)	(0.00)	(0.00)
48		6.00	4.00	2.00	2.00	2.00
50		4.67	4.75	4.00	4.00	4.00
52		3.33	6.00	6.00	6.00	6.00
54		2.00	8.00	8.00	8.00	8.00
56		0.67	10.00	10.00	10.00	10.00
58			11.00	11.00	11.00	11.00
60			12.00	12.00	12.00	12.00

*Parenthesis means that the proposed formula will allow less weight.

Table 10. Changes in Allowable Weights, in Thousands of Pounds, from Formula B (Current Law) to the Proposed Formula. (continued)

	Number of Axles					
Wheelbase	3	4	5	6	7	8
62			13.00	13.00	13.00	13.00
64			14.00	14.00	14.00	14.00
66			15.00	15.00	15.00	15.00
68			16.00	16.00	16.00	16.00
70			17.00	17.00	17.00	17.00
72				18.00	18.00	18.00
74				19.00	19.00	19.00
76				20.00	20.00	20.00
78				21.00	21.00	21.00
80				22.00	22.00	22.00
82				23.00	23.00	23.00
84				24.00	24.00	24.00
86				25.00	25.00	25.00
88				26.00	26.00	26.00
90				27.00	27.00	27.00
92				28.00	28.00	28.00
94				29.00	29.00	29.00
96				30.00	30.00	30.00
98				31.00	31.00	31.00
100				32.00	32.00	32.00
102				33.00	33.00	33.00
104				34.00	34.00	34.00
106				35.00	35.00	35.00
108				36.00	36.00	36.00
110				37.00	37.00	37.00
112					38.00	38.00
114					39.00	39.00
116					40.00	40.00
118					41.00	41.00
120					42.00	42.00

1 ft = 0.3048 m
1000 lb = 4.448 kN

TABLE 11. Existing and Proposed Bridge Weight Formulas Superposed Over Tables Reflecting the Weights of Single Axles and Two-Axle Groups Observed for All Vehicle Types in Loadmeter Surveys Conducted in 1982. Note that the Proposed and Current Formulas are identical for these Groupings

a) Single Axles

449,660 Observations

Gross Weight of Wheel Group (1000 lb)	
TOTAL	449660
TOTAL	100
100–110	
90–100	4 0.00
80–90	4 0.00
70–80	5 0.00
60–70	4 0.00
50–60	8 0.00
40–50	15 0.00
30–40	73 0.01
20–30	6537 1.31
10–20	227703 45.57
0–10	265307 53.10

b) Two Axle Group

381,639 Observations

Gross Weight of Wheel Group (1000 lb) \ Outside Spacing of Wheel Group (ft)	0–8	8–16	16–24	24–32	32–40	40–48	48–56	56–64	64–72	72–80	80–88	88–96
TOTAL	168956	106521	26466	66054	11273	325	25	3	7	4	2	3
TOTAL	44.27	28.19	6.93	17.85	2.94	0.07	0.00	0.00	0.00	0.00	0.00	0.00
100–110	4 0.00	0 0.00	1 0.00	1 0.00	1 0.00	0 0.00	0 0.00	0 0.00				
90–100	3 0.00	2 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00				
80–90	4 0.00	4 0.00	0 0.00	1 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	
70–80	7 0.00	4 0.00	0 0.00	3 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	
60–70	12 0.00	3 0.00	1 0.00	1 0.00	3 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	
50–60	119 0.03	8 0.00	5 0.00	21 0.00	1 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	
40–50	1950 0.52	118 0.02	125 0.02	464 0.14	107 0.02	1 0.00	1 0.00	0 0.00	0 0.00	0 0.00	0 0.00	
30–40	48790 12.78	3510 0.92	3595 0.94	18625 4.89	2689 0.71	76 0.01	10 0.00	2 0.00	2 0.00	1 0.00	0 0.00	
20–30	44836 11.76	48820 13.09	11122 2.91	19274 5.05	4080 1.06	137 0.04	12 0.00	0 0.00	2 0.00	1 0.00	1 0.00	1 0.00
10–20	52871 13.85	42444 11.12	10389 2.73	22910 6.00	3782 0.99	100 0.02	2 0.00	1 0.00	3 0.00	2 0.00	1 0.00	2 0.00
0–10	20359 5.33	11608 3.04	1228 0.33	6755 1.77	610 0.16	11 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00

2-Axle Group

Legend

— Current Formula

-- Proposed Formula

observations

frequency

per cent

Outside Spacing of Wheel Group (ft)

1000 lb = 4.448 kN
1 ft = 0.3048 m

TABLE 12. Existing and Proposed Bridge Weight Formulas Superposed Over Tables Reflecting the Weights of Three-Axle Groups Observed for All Vehicle Types in Loadmeter Surveys Conducted in 1982. (263,326 Observations)

Gross Weight of Wheel Group (1000 lb) by Outside Spacing of Wheel Group (ft)

Gross Weight (1000 lb)	0–8	8–16	16–24	24–32	32–40	40–48	48–56	56–64	64–72	72–80	80–88	88–96
TOTAL	31	35737	51456	57708	111531	6661	426	42	12	7	8	7
TOTAL	0.00	13.55	20.18	21.88	42.24	2.79	0.15	0.00	0.00	0.00	0.00	0.00
100–110	0 0.00	2 0.00	1 0.00	3 0.00	2 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00
90–100	0 0.00	0 0.00	0 0.00	2 0.00	5 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00
80–90	0 0.00	2 0.00	4 0.00	7 0.00	3 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00
70–80	0 0.00	5 0.00	23 0.01	31 0.01	76 0.02	5 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00
60–70	0 0.00	33 0.01	111 0.04	387 0.15	802 0.29	39 0.01	2 0.00	0 0.00	1 0.00	0 0.00	0 0.00	0 0.00
50–60	3 0.00	421 0.16	1132 0.[illegible]	51[illegible] 1.96	11583 4.39	394 0.42	16 0.00	9 0.00	2 0.00	2 0.00	1 0.00	0 0.00
40–50	10 0.00	8961 3.40	17[illegible]3 6.75	16247 6.15	33934 12.86	1024 0.39	71 0.02	9 0.00	5 0.00	1 0.00	1 0.00	1 0.00
30–40	2 0.00	1[illegible]17 4.32	11492 4.35	9869 3.74	19419 7.37	2086 0.80	139 0.05	10 0.00	2 0.00	0 0.00	3 0.00	0 0.00
20–30	13 0.00	11845 4.50	15261 5.79	10784 4.09	21019 7.96	2429 0.92	162 0.07	9 0.00	1 0.00	3 0.00	1 0.00	5 0.00
10–20	1 0.00	3041 1.16	5529 2.71	15003 5.69	24256 9.19	670 0.25	33 0.01	2 0.00	1 0.00	1 0.00	2 0.00	1 0.00
0–10	2 0.00	10 0.00	50 0.02	228 0.09	432 0.16	14 0.00	3 0.00	3 0.00	0 0.00	0 0.00	0 0.00	0 0.00

3-Axle Group

Legend

— Current Formula

-- Proposed Formula

observations: 1

frequency per cent: 0.00

Outside Spacing of Wheel Group (ft)

1000 lb = 4.448 kN
1 ft = 0.3048 m

TABLE 13. Existing and Proposed Bridge Weight Formulas Superposed Over Tables Reflecting the Weights of Four-Axle Groups Observed for All Vehicle Types in Loadmeter Surveys Conducted in 1982. (168,334 Observations)

Gross Weight of Wheel Group (1000 lb)	0–8	8–16	16–24	24–32	32–40	40–48	48–56	56–64	64–72	72–80	80–88	88–96	96–104	104–112	112–120
TOTAL TOTAL		51 0.00	816 0.45	5013 2.96	67334 39.96	74118 43.94	19710 11.69	1064 0.79	191 0.09	13 0.00	6 0.00	7 0.00	11 0.00		
100–110		0 0.00	0 0.00	0 0.00	14 0.00	4 0.00	2 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	4-Axle Group	
90–100		0 0.00	1 0.00	3 0.00	59 0.04	17 0.01	1 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00		
80–90		0 0.00	8 0.00	43 0.02	413 0.25	135 0.08	28 0.01	4 0.00	1 0.00	1 0.00	0 0.00	0 0.00	0 0.00		
70–80		1 0.00	48 0.03	189 0.11	2422 1.44	1082 0.[illegible]	316 0.19	45 0.03	8 0.00	0 0.00	1 0.00	0 0.00	1 0.00		
60–70		23 0.01	157 0.09	1348 0.[illegible]	1[illegible]162 10.21	9505 5.63	3077 1.83	236 0.15	20 0.01	6 0.00	3 0.00	0 0.00	0 0.00		
50–60		17 0.01	13[illegible] 0.08	860 0.51	10578 6.27	19428 11.52	6031 3.57	305 0.18	41 0.02	4 0.00	1 0.00	3 0.00	2 0.00		
40–50		3 0.00	48 0.02	416 0.24	8074 4.79	12151 7.21	3282 1.95	327 0.20	65 0.03	1 0.00	0 0.00	2 0.00	0 0.00		
30–40		4 0.00	61 0.03	509 0.31	8300 4.92	12840 7.61	3205 1.9	265 0.15	29 0.01	0 0.00	1 0.00	1 0.00	5 0.00		
20–30		2 0.00	203 0.11	1233 0.73	14990 8.89	17773 10.54	3642 2.16	142 0.08	23 0.02	1 0.00	3 0.00	1 0.00	3 0.00		
10–20		1 0.00	152 0.09	410 0.24	5269 3.12	1176 0.70	124 0.07	21 0.00	4 0.00	0 0.00	0 0.00	0 0.00	0 0.00		
0–10		0 0.00	1 0.00	2 0.00	41 0.03	5 0.00	1 0.00	1 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00		

Outside Spacing of Wheel Group (ft)

Legend

— Current Formula

-- Proposed Formula

observations: 1 = frequency, 0.00 = per cent

1000 lb = 4.448 kN
1 ft = 0.3048 m

TABLE 14. Existing and Proposed Bridge Weight Formulas Superposed Over Tables Reflecting the Weights of Five-Axle Groups Observed for All Vehicle Types in Loadmeter Surveys Conducted in 1982. (79,109 Observations)

Gross Weight of Wheel Group (1000 lb)	0–8	8–16	16–24	24–32	32–40	40–48	48–56	56–64	64–72	72–80	80–88	88–96	96–104	104–112	112–120
TOTAL TOTAL			35 0.03	163 0.16	1539 1.99	21816 26.91	48491 62.37	6114 7.52	853 1.05	73 0.07	11 0.00	6 0.00	8 0.00		
100–110			0 0.00	0 0.00	2 0.00	21 0.02	51 0.05	11 0.01	0 0.00	1 0.00	0 0.00	0 0.00	0 0.00		
90–100			0 0.00	1 0.00	17 0.01	153 0.18	362 0.65	63 0.0[illegible]	[illegible] 0.01	3 0.00	0 0.00	1 0.00	1 0.00		
80–90			0 0.00	7 0.00	71 0.08	810 1.00	[illegible] 2.54	452 0.56	82 0.10	7 0.00	3 0.00	0 0.00	0 0.00		
70–80			20 0.02	31 0.04	307 0.3[illegible]	[illegible] 5.81	14342 17.7	1395 1.70	129 0.16	9 0.01	2 0.00	3 0.00	3 0.00		
60–70			10 0.01	[illegible] [illegible]	228 0.28	3097 3.82	7893 9.74	1221 1.51	149 0.18	16 0.02	2 0.00	0 0.00	1 0.00		
50–60			0 0.00	13 0.01	112 0.14	2526 3.12	5579 6.88	1038 1.28	193 0.25	10 0.01	2 0.00	0 0.00	1 0.00		
40–50			4 0.00	15 0.01	122 0.15	2751 3.40	3583 6.76	746 0.91	144 0.18	13 0.02	1 0.00	2 0.00	2 0.00		
30–40			1 0.00	30 0.04	322 0.40	4436 5.48	8927 11.02	944 1.16	97 0.12	8 0.01	0 0.00	0 0.00	0 0.00		
20–30			0 0.00	37 0.04	356 0.44	3247 4.02	5616 6.93	245 0.31	39 0.04	5 0.00	1 0.00	0 0.00	0 0.00		
10–20			0 0.00	0 0.00	2 0.00	42 0.05	82 0.10	9 0.00	10 0.01	1 0.00	0 0.00	0 0.00	0 0.00		
0–10			0 0.00	0 0.00	0 0.00	0 0.00	1 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00		

Outside Spacing of Wheel Group (ft)

5-Axle Group

Legend

— Current Formula

-- Proposed Formula

observations

1 ← frequency

0.00 ← per cent

1000 lb = 4.448 kN
1 ft = 0.3048 m

Formula B is shown by the solid line and the proposed formula by the broken one. For single axles and for two-axle groups, the two formulas are identical.

Table 12, for groups of three axles, shows the proposed formula to be more liberal for total axle spacings from 8 to 32 ft (2.438 to 9.754 m) which includes a larger percentage of the three-axle groups. Above 32 ft (9.754 m) the allowed weight by both formulas is 60,000 lb (266.9 kN) and is controlled by the 20,000 lb (88.96 kN) single-axle maximum. The maximum liberalization occurs at a length of 26 ft (7.925 m) where the proposed formula would allow an extra 4,500 lb (20.02 kN) of weight.

Table 13 is for groups of four axles. The proposed formula is more restrictive for lengths shorter than 24 ft (7.315 m) than is the current formula. However, for lengths between 24 and 57 ft (7.315 and 17.37 m), the proposed formula allows higher weights, ranging up to a maximum of 7,666 lb (34.10 kN). The table shows this range of lengths contains more than 98 percent of the four-axle groupings.

Table 14 is for groups of five axles. The current formula is drawn in accordance with the current law which restricts gross vehicle weights to 80,000 lb (355.8 kN) while the proposed formula is shown extended on up to 100,000 lb (444.8 kN) reflecting a limit based on 20,000 lb (88.96 kN) per axle. The transition to a shallower slope at 56 ft (17.07 m), a feature of the proposed formula, is also shown. For lengths less than 37.3 ft (11.38 m) the proposed formula is more restrictive. But this represents only about 2 percent of the observations. If the maximum allowable gross weight of 80,000 lb (355.8 kN) is maintained with the proposed formula, then the proposed formula liberalizes the overall loads only on lengths between 37.3 and 51.2 ft (11.38 and 15.61 m) with the maximum increase of 3,250 lb (14.46 kN) at 46 ft (14.02 m). While the increases are small, this range of lengths includes about 50 percent of the observed five-axle groups.

For groups of six axles and more the proposed formula is more restrictive than the current formula, at least up to 80,000 lb (355.8 kN).

BOGIES

The capability of legal three-, four-, and five-axle bogies to exceed the tolerable stress levels in H15 bridges is dramatically illustrated in graphs of figure 23. A bogie is considered to be adjacent axles, equally spaced, with a suspension designed to equalize the distribution of the load. For the purpose of these calculations they were considered as equally loaded axles, equally spaced.

Remembering that for three-axle groupings the new formula does not mandate any load reduction, one looks at the stress ratios generated by 3-axle bogies in figure 23a. For the shortest axle spacing of 8 ft (2.438 m) overall, one sees that the weight of 42,000 lb (186.8 kN) allowed by both the old and new formulas generates a stress ratio of just over 1.27 in an H15 bridge. Following Table B, the ratio decreases with longer wheelbases, while the proposed formula, allowing for more weight, keeps the ratio near 1.27.

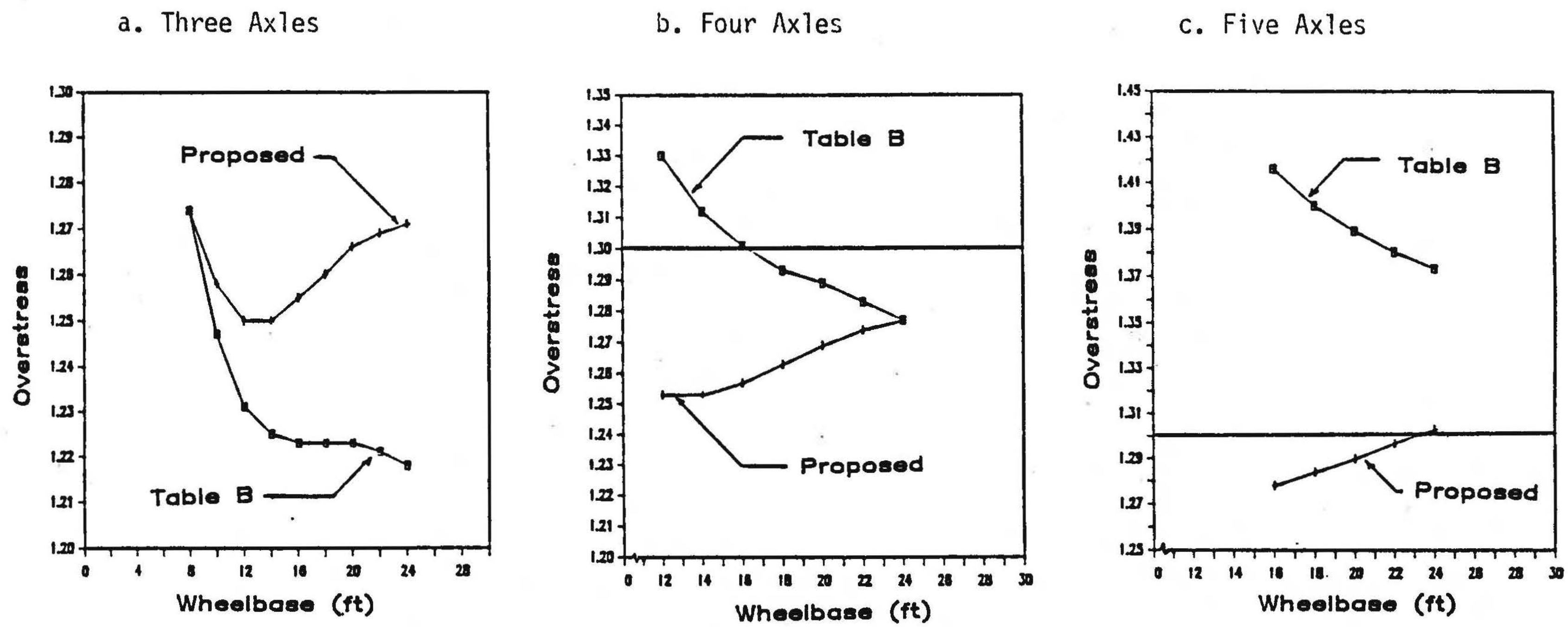
Figure 23b, for a 4-axle bogie, shows clearly, however, that the weight allowed on the shortest spacing, 50,000 lb (222.4 kN) with a 12 ft (3.658 m) outside dimension, causes a stress ratio of 1.33. Correspondingly, with the proposed formula limits the ratio is less than 1.26. Finally, for the 5-axle bogie, figure 23c shows that Table B allows stress ratios as great as 1.41, while new maximums would, in general, be below 1.30.

SHORT MULTIAXLE VEHICLES

Consider some conventional multiaxled vehicles with very short wheelbases. Such trucks include those with four or more axles with outside lengths such that both table B and the recommended formula allow the same gross weights. A review of table 10 shows that these crossover points are:

Number of Axles	Length	
4	24 ft	(7.315 m)
5	37 1/3ft	(11.38 m)
6 and more	46 ft	(14.02 m)

For shorter lengths, the recommended formula specifies a smaller gross weight; for longer lengths, it allows larger gross weights.



1000 lb = 4.448 kN
1 ft = 0.3048 m

Figure 23. Stress Ratios Generated in H15 Bridges by Multiaxled Bogies Having Maximum Weights Allowed by Table B Compared with those Having Maximum Weights Allowed by the Proposed Formula

Figures 24 and 25 show calculated H15 bridge stress ratios resulting from 3S2 and 3S3 semitrailer trucks with very short wheelbases. The figures are intended to compare the resulting stress ratios between legal trucks under Table B with those under the proposed formula. Since these wheelbases are near the crossover lengths between the two formulas, the differences are small. However, when there are differences they show the new formula to be superior both in suppressing stress ratios greater than 1.3 and increasing stress ratios below 1.3.

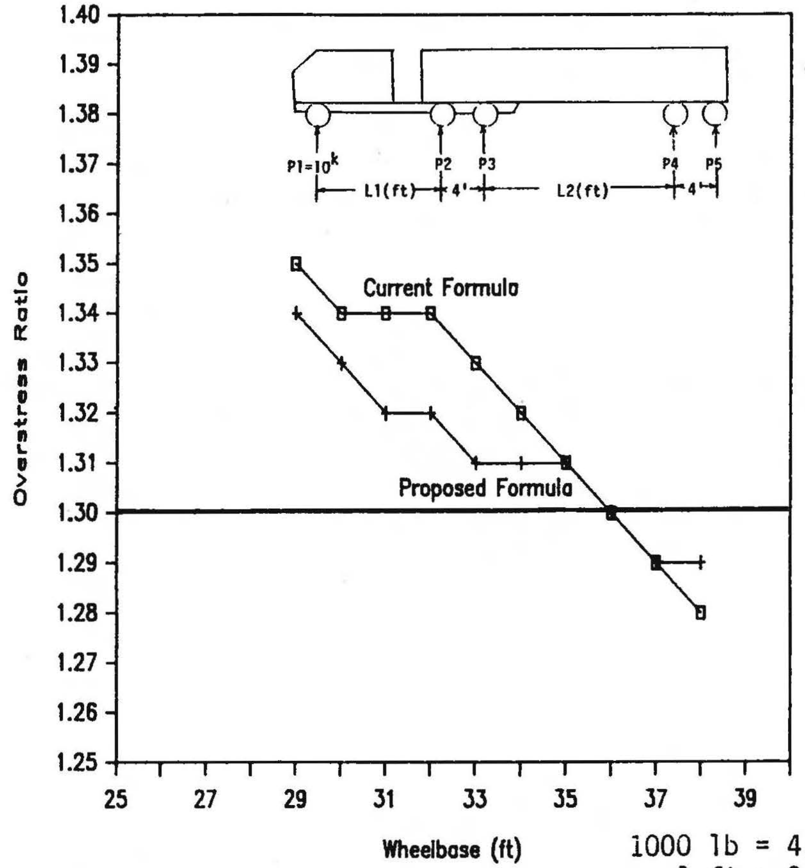
Figure 26 shows the overstress ratios on H15 bridges due to 3S2 trucks having more practical wheelbases of 44 to 56 ft (13.41 to 17.07 m). The outside spacing of the two tandems was 36 ft (10.97 m), and each tandem was loaded to the maximum of 34,000 lb (151.2 kN). The current law contains an 80,000 lb (355.8 kN) gross weight limit for wheelbases greater than 51 ft (15.54 m). However, for all of these configurations, the new formula allows a higher gross weight than Table B.

CONVENTIONAL LENGTH VEHICLES

The gross vehicle weights on several conventional vehicle geometries required to generate the specified overstresses on critical spans were calculated. The conventional vehicle geometries are representative of those observed in the 1982 loadometer survey.

The critical spans for each of these representative vehicle geometries and proportional wheel loadings were determined. Then the gross weights to generate stress ratios of 1.05 in HS20 bridges and 1.30 in H15 bridges for the critical bridges were calculated. These gross vehicle weights are those plotted in figures 27 and 28, and the corresponding proportional wheel loadings are summarized in table 15.

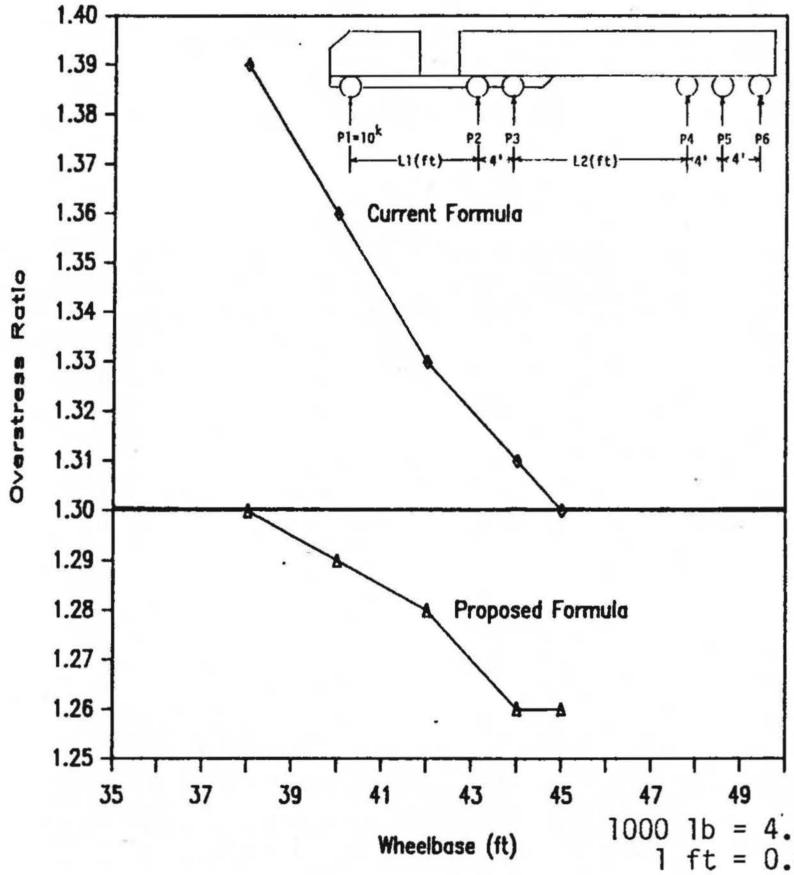
When the practical vehicles have gross weights above the lines representing the proposed formula, it means the formula is too restrictive. When the weights plot below the formula, it is not restrictive enough. Ideally, the formula should provide an envelope along the lower limit of all the points. It should be noted that the geometries and load distributions



1000 lb = 4.448 kN
1 ft = 0.3048 m

Axle Spacing		Overstress Ratios by Selected Trucks on H15 Bridges											
		Formula B						Proposed Formula					
		Axle Loading				Totals		Axle Loading				Totals	
L1	L2	P2	P3	P4	P5	P	L	P2	P3	P4	P5	P	L
12	18	12	17.5	17.5	12	69.0	38	12	18	18	12	70.0	38
12	17	12	17.3	17.3	12	68.6	37	12	17.5	17.5	12	69.0	37
12	16	12	17	17	12	48.0	34	12	17	17	12	68.0	36
12	15	11	17.8	17.8	11	47.4	35	11	17.5	17.5	11	67.0	35
12	14	11	17.3	17.3	11	66.6	34	11	17	17	11	66.0	34
12	13	11	17	17	11	66.0	33	11	16.5	16.5	11	65.0	33
12	12	10.5	17.3	17.3	10.5	65.4	32	10	17	17	10	64.0	32
12	11	10	17	17	10	64.0	31	10	16.5	16.5	10	63.0	31
11	11	10	17	17	10	64.0	30	10	16.5	16.5	10	63.0	30
11	10	10	17	17	10	64.0	29	10	16.5	16.5	10	63.0	29

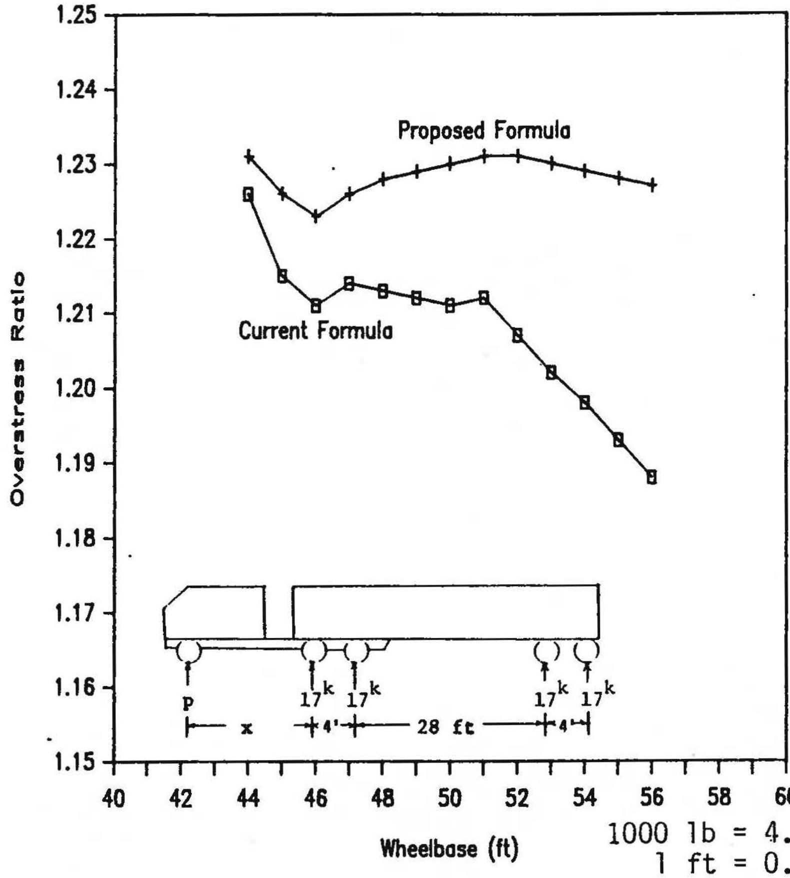
Figure 24. Graph Contrasting the Stress Ratios Generated in H15 Bridges by the Legal Maximum Loads on 3S2 Vehicles According to Formula B with Those by the Proposed Formula



1000 lb = 4.448 kN
1 ft = 0.3048 m

Axle Spacing		Overstress Ratios by Selected Trucks on H15 Bridges														
		Formula B							Proposed Formula							
		Axle Loading					Totals		Axle Loading					Totals	
L1	L2	P2	P3	P4	P5	P6	P	L	P2	P3	P4	P5	P6	P	L
14	19	11.5	16.5	18	14	8	77.5	45	12	15	17	13	8	75.0	45
14	18	11.5	16	17.5	14	8	77.0	44	12	15	16	13	8	74.0	44
14	16	11	16	17	14	8	76.0	42	11	15	16	12	8	72.0	42
14	14	10	15	17	14	8	75	40	10	15	15	12	8	70.0	40
14	12	10	14	17	14	8	73	38	10	14	14	12	8	68.0	38

Figure 25. Graph Contrasting the Stress Ratios Generated in H15 Bridges By the Legal Maximum Loads on 3S3 Vehicles According to Formula B with Those by the Proposed Formula



1000 lb = 4.448 kN
1 ft = 0.3048 m

Wheelbase (ft)	Formula B (kips)		Proposed Formula (kips)	
	P	Total P	P	Total P
44	7.5	75.5	8.0	76.0
45	8.0	76.0	9.0	77.0
46	8.5	76.5	10.0	78.0
47	9.5	77.5	11.0	79.0
48	10.0	78.0	12.0	80.0
49	10.5	78.5	13.0	81.0
50	11.0	79.0	14.0	82.0
51	12.0	80.0	15.0	83.0
52	12.0	80.0	16.0	84.0
53	12.0	80.0	17.0	85.0
54	12.0	80.0	18.0	86.0
55	12.0	80.0	19.0	87.0
56	12.0	80.0	20.0	88.0

Figure 26. Graph Showing Stress Ratios of Selected 3S2 Vehicles on H15 Bridges

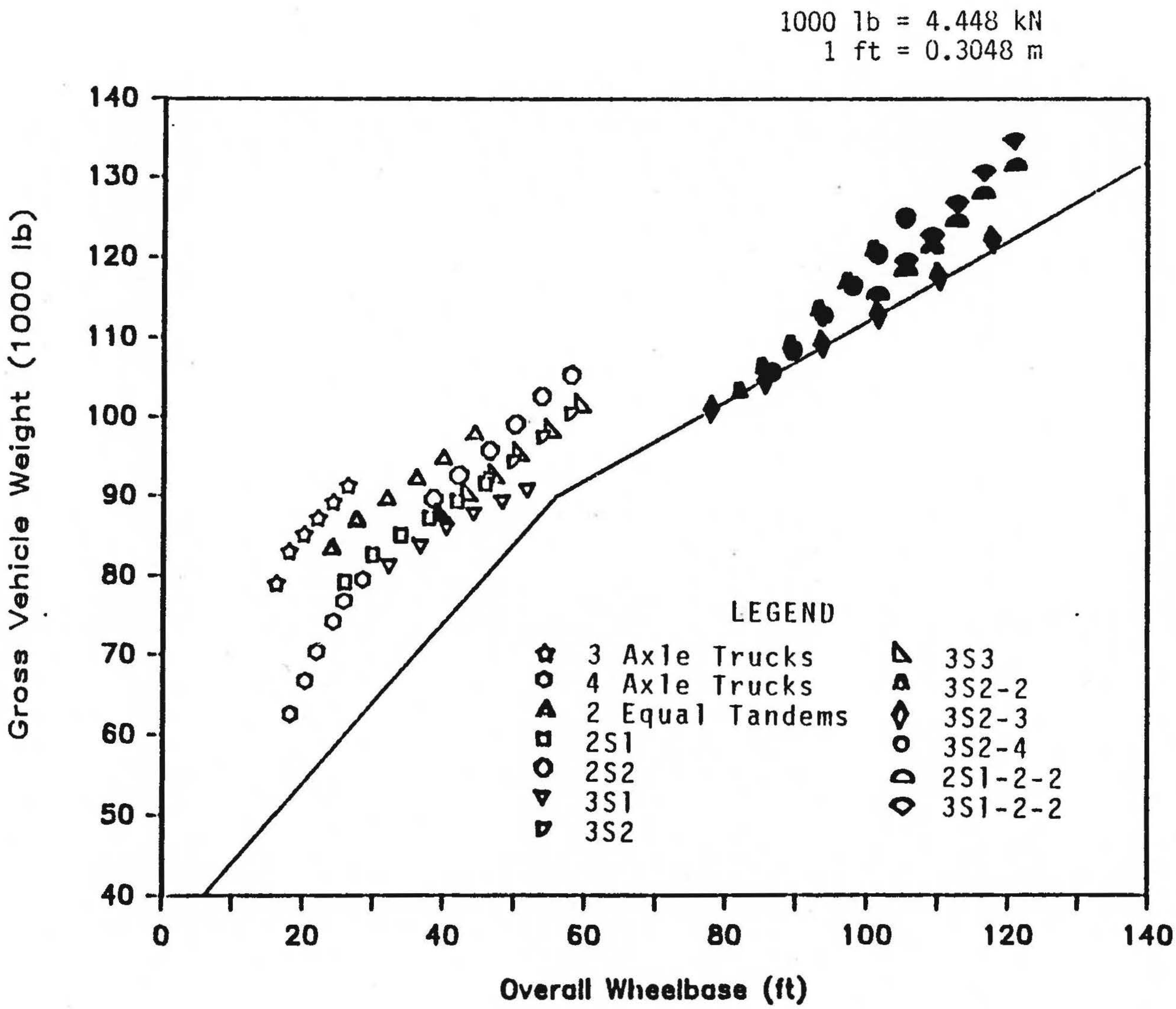


Figure 27. Critical Weights for Selected Practical Vehicles Causing Specified Overstresses of 105% on HS20 Bridges Compared to Proposed Formula

1000 lb = 4.448 KN
1 ft = 0.3048 m

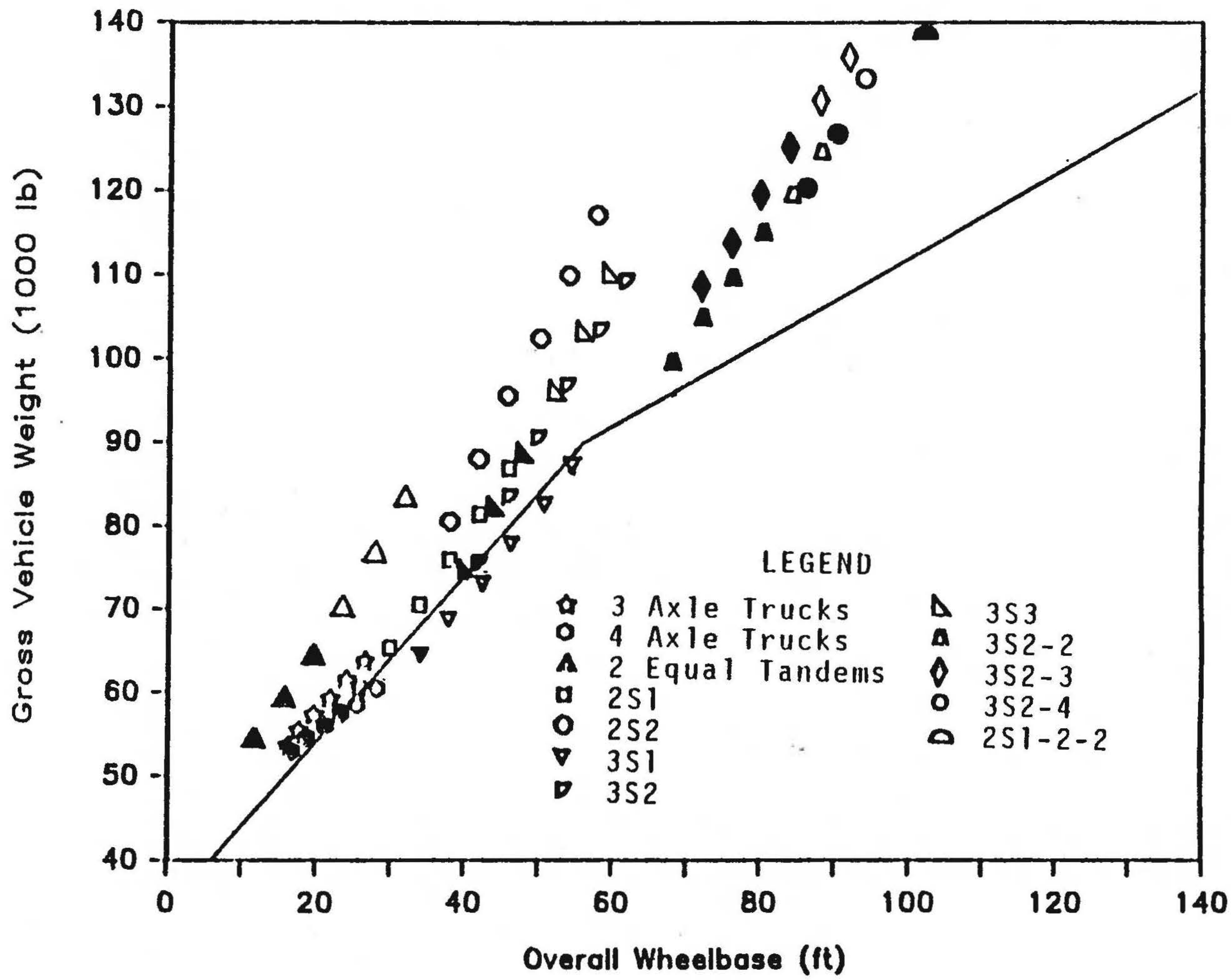


Figure 28. Critical Weights for Selected Practical Vehicles Causing Specified Overstresses of 130% on H15 Bridges Compared to Proposed Formula

Table 15. Maximum Legal Weights of Selected Practical Vehicles Under Existing and Proposed Formulas when H15 Bridges Govern

Vehicle	No. Axles	Axle Spacings in feet	Axle Weight Fractions[a] (% GVW)
3-axle truck	3	12[b] 4	37 31 31
4-axle truck	4	12[b] 4 4	25 25 25 25
2 tandems	4	4 28[b] 4	25 25 25 25
2S1	3	14 12[b]	26 37 37
2S2	4	14 20[b] 4	22 29 25 25
3S1	4	14 4 16[b]	18 26 26 30
3S2	5	14 4 20[b] 4	15 21 21 21 21
3S3	6	12 4 16[b] 4 4	14 19 19 16 16 16
3S2-2	7	12 4 20[b] 4 8 20[b]	11 15 15 15 15 11 18
3S2-3	8	12 4 20[b] 4 8 20[b] 4	10 13 13 13 13 10 13 13
3S2-4	9	18 4 20[b] 4 8 4 24 4	12 13 13 10 10 10 10 13 9
2S2-2-2	7	18 20[b] 8 24 8 24	12 16 16 16 12 12 16
3S1-2-2	8	14 4 24[b] 8 24 8 24	10 11 11 13 13 13 13 13

[a]Roundoff errors may cause total of axle weight fractions to differ from 100%.
[b]Variable spacing between axles.

NOTE: 1 ft = 0.305 m
1 kip = 1000 lb
1 lb = 4.45 N

shown in the figures often lead to vehicles that are illegal from other standpoints, i.e., either the single- or tandem-axle load limits are violated. Vehicles which are prohibited are shown by the open symbols while those that are not are shown by the solid symbols. Of course, all vehicles with gross weights greater than 80,000 lb (355.8 kN) are prohibited by the current law.

A second item to note is that the necessity to protect the H15 bridges is the criteria used to define the steeper sloped portion of the proposed formula. Similarly, the shallower part, that part applicable to the longer wheelbases and, in turn, heavier loads, is governed by the HS20 bridges.

Single unit vehicles with four axles and some 3S1 vehicles fall below the formula. So some variations of these geometries may comply with the formula and at the same time cause overstresses larger than those prescribed. But these geometries constitute a small proportion of the observed traffic and the margins above the prescribed overstresses are small. One certain conclusion is that the proposed formula is an improvement over the current formula, at least from the standpoint of overstresses in simple span bridges.

BRIDGE FATIGUE

The fatigue behavior of highway bridges is influenced primarily by stress range. The stress range is equal to the LL + I stresses, therefore any changes in truck weights will result in increased fatigue loading on highway bridges and a corresponding increase in maintenance costs if the increased fatigue loading causes stresses that are above the fatigue endurance limits. To evaluate the significance of the proposed formula on the fatigue lives of highway bridges, it is necessary to make several simplifying assumptions. It is assumed that existing bridges are loaded in flexure to design allowable stresses by design vehicles., i.e.,

$$F = 0.55\ F_y - 0.46\ F_u - F_{sr} \qquad (12)$$

where F_{sr} is the allowable stress for repeated loadings, a function of the design lifetime in loading cycles and the weld detail category. It is assumed that flexure governs, and shear is not checked. If existing single, tandem, and triple axle bogie limits are not changed, shear stresses are not expected to increase as significantly as flexure stresses. Further, only simple spans were evaluated.

For each span checked, the maximum moment caused by the maximum legal weight vehicles and the maximum moment due to the design vehicle (or lane loading) were calculated. With the assumption that the stress range due to the design loading equals the allowable stress range, the stress range due to the maximum weight vehicles is calculated by multiplying the appropriate moment ratio.

The calculated stress ranges are compared to the allowable fatigue stress ranges in figures 29 and 30 for two representative checks. In this manner, it was determined that the ratio of the calculated stress range to the allowable stress range does not exceed 1.05 except for a small range of span for any specific vehicle configuration. Similar calculations were made for all the practical vehicles described in table 15, and the maximum calculated stress range along with the critical span is tabulated in tables 16 and 17. Table 16 summarizes the results for A36 steel structures, and table 17 summarizes the results for A514 steel structures. These two steels have strengths bracketing the range of commonly used steels for bridge structures, and the calculated stress ranges for other steels are bounded above by the calculated stress ranges for A514 steel and below by the calculated stress ranges for A36 steels. For most spans and detail categories, the increased stress range is still well below the allowable stress range. Span-detail combinations which are most affected by the proposed formula are the more severe details E, F, E', in maximum moment regions of longer (120-160 ft) spans.

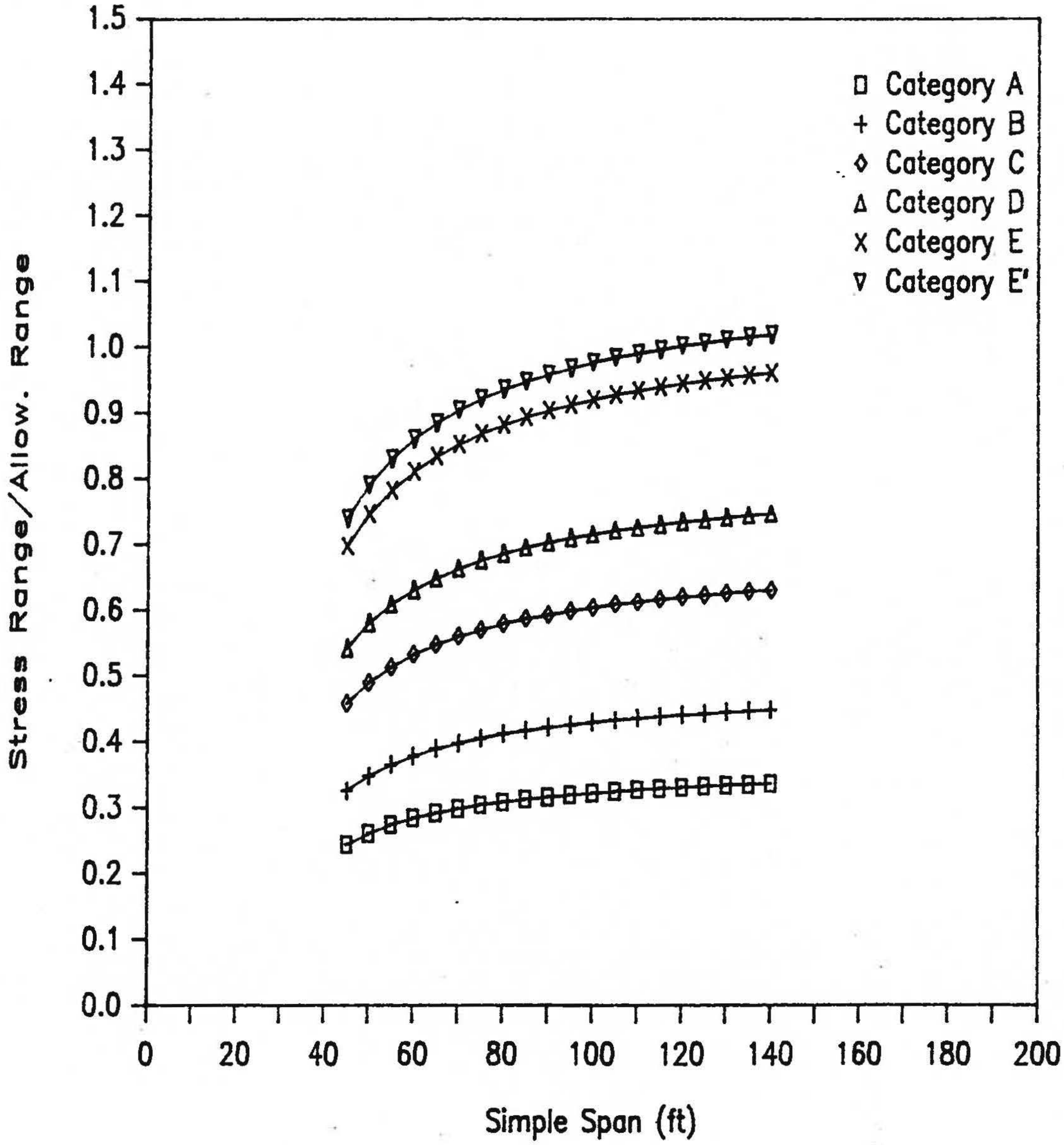


Figure 29. Live Load Plus Impact Stress Range Caused by Maximum Legal Weight 3S2 Vehicle According to Proposed Formula for A36 Steel Stringers Designed for 100,000 Cycles of HS20 Load.

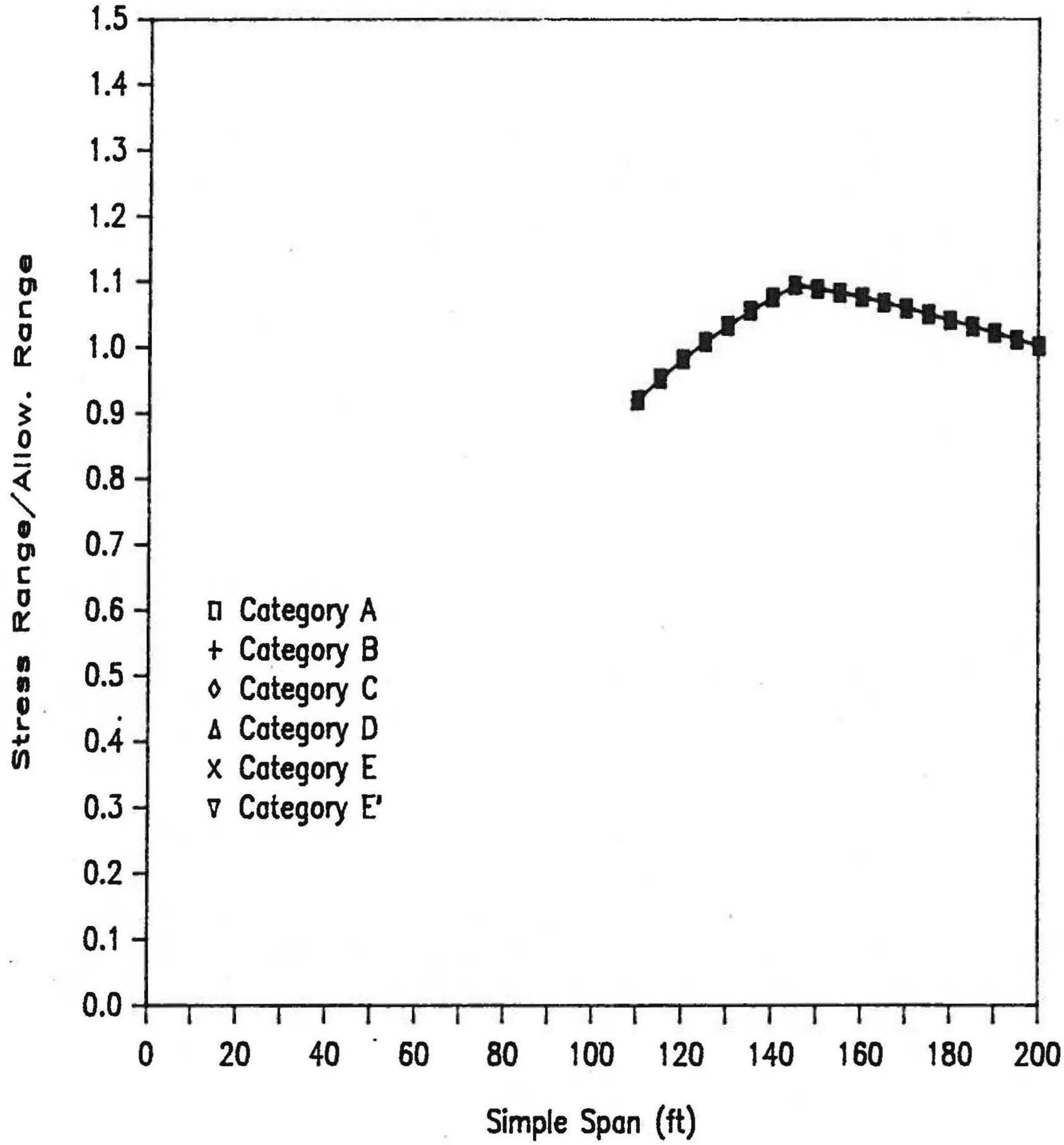


Figure 30. Live Load Plus Impact Stress Range Caused by Maximum Legal Weight 2S2-2-2 Vehicle According to Proposed Formula for A514 Steel Stringers Designed for More than 2,000,000 Cycles of HS20 Loading.

TABLE 16. MAXIMUM CALCULATED STRESS RANGES[a] FOR VARIOUS SELECTED VEHICLES ON HS20 DESIGNED SIMPLE SPANS USING A36 STEEL

A36 Steel-Redundant Load Path Structures									
Vehicle Type	Weight in Pounds	Design Life in Cycles of Truck Loading							
		100,000		500,000		2,000,000		>2,000,000	
		Detail A	Category F	Detail A	Category E'	Detail A	Category E'	Detail A	Category E'
Tandem	34,000	17.2 (25)[b]	17.2 (25)	17.2 (25)	8.2 (25)	17.2 (25)	5.0 (25)	17.2 (25)	2.3 (25)
Triple	42,000	19.7 (25)	19.7 (25)	19.7 (25)	9.4 (25)	19.7 (25)	5.8 (25)	19.7 (25)	2.6 (25)
3-Axle Truck	54,000	18.0 (25)	18.0 (25)	18.0 (25)	8.6 (25)	18.0 (25)	5.3 (25)	18.0 (25)	2.4 (25)
4-Axle Truck	56,000	17.7 (30)	17.7 (30)	17.7 (30)	8.4 (30)	17.7 (30)	5.2 (30)	17.7 (35)	2.3 (35)
2 Tandems	68,000	16.5 (145)	16.5 (145)	16.5 (145)	7.8 (145)	16.5 (145)	4.8 (145)	16.5 (145)	2.2 (145)
2S1	54,000	14.0 (135)	14.0 (135)	14.0 (135)	6.7 (135)	4.0 (135)	4.1 (135)	14.0 (135)	1.8 (135)
2S2	69,000	17.2 (140)	17.2 (140)	17.2 (140)	8.1 (140)	17.2 (140)	5.0 (140)	17.2 (140)	2.3 (140)
3S1	66,000	17.5 (135)	17.5 (135)	17.5 (135)	8.3 (135)	17.5 (135)	5.1 (135)	17.5 (135)	2.3 (135)
3S2	80,000	20.2 (140)	20.2 (140)	20.2 (140)	9.6 (140)	20.2 (140)	5.9 (140)	20.2 (140)	2.7 (140)
3S3	82,000	20.9 (140)	20.9 (140)	20.9 (140)	9.9 (140)	20.9 (140)	6.1 (140)	20.9 (140)	2.7 (140)
3S2-2	96,000	22.1 (145)	18.5 (145)	22.1 (145)	10.5 (145)	22.1 (145)	6.5 (145)	22.1 (145)	2.9 (145)
3S2-3	101,000	22.1 (145)	22.1 (145)	22.1 (145)	10.5 (145)	22.1 (145)	6.5 (145)	22.1 (145)	2.9 (145)
3S2-4	105,000	22.0 (145)	22.0 (145)	22.0 (145)	10.5 (145)	22.0 (145)	6.5 (145)	22.0 (145)	2.9 (145)
2S1-2-2	115,000	21.5 (145)	15.0 (145)	21.5 (145)	10.2 (145)	21.5 (145)	6.3 (145)	21.5 (145)	2.8 (145)
3S1-2-2	115,000	21.2 (145)	14.5 (145)	21.3 (145)	10.1 (145)	21.3 (145)	6.2 (145)	21.3 (145)	2.8 (145)

[a]In thousands of pounds per square inch.
[b]Numbers in parentheses are critical span lengths in feet.

NOTE: 1 ksi = 1,000 psi = 6.89 MPa
1 ft = 0.305 m

TABLE 17. MAXIMUM CALCULATED STRESS RANGES[a] FOR VARIOUS SELECTED VEHICLES ON HS20 DESIGNED SIMPLE SPANS USING A514 STEEL

A514 Steel-Redundant Load Path Structures									
Vehicle Type	Weight in Pounds	Design Life in Cycles of Truck Loading							
		100,000		500,000		2,000,000		>2,000,000	
		Detail A	Category F	Detail A	Category E'	Detail A	Category E'	Detail A	Category E'
Tandem	34,000	43.9 (25)	13.0 (25)	31.2 (25)	8.2 (25)	20.8 (25)	5.0 (25)	20.8 (25)	4.3 (25)
Triple	42,000	50.4 (25)	14.9 (25)	35.9 (25)	9.4 (25)	23.9 (25)	5.8 (25)	23.9 (25)	5.0 (25)
3-Axle Truck	54,000	46.1 (25)	13.7 (25)	32.8 (25)	8.6 (25)	21.9 (25)	5.3 (25)	21.4 (25)	4.6 (25)
4-Axle Truck	56,000	45.2 (30)	13.4 (30)	32.2 (30)	8.4 (30)	21.4 (30)	5.2 (30)	20.9 (30)	4.5 (30)
2 Tandems	68,000	42.1 (145)	12.5 (145)	30.0 (145)	7.8 (145)	20.0 (145)	4.8 (145)	20.0 (145)	4.2 (145)
2S1	54,000	35.9 (135)	10.6 (135)	25.5 (135)	6.7 (135)	17.0 (135)	4.1 (135)	17.0 (135)	3.5 (135)
2S2	69,000	43.8 (140)	13.0 (140)	31.2 (140)	8.1 (140)	20.8 (140)	5.0 (140)	20.8 (140)	4.3 (140)
3S1	66,000	44.6 (135)	13.2 (135)	31.7 (135)	8.3 (135)	21.2 (135)	5.1 (135)	21.2 (135)	4.4 (135)
3S2	80,000	51.6 (140)	15.3 (140)	36.7 (140)	9.6 (140)	24.5 (140)	5.9 (140)	24.5 (140)	5.1 (140)
3S3	82,000	53.3 (140)	15.8 (140)	37.9 (140)	9.9 (140)	25.3 (140)	6.1 (140)	25.3 (140)	5.3 (140)
3S2-2	96,000	56.2 (140)	16.8 (145)	40.2 (145)	10.5 (145)	26.8 (145)	6.5 (145)	26.8 (145)	5.6 (145)
3S2-3	101,000	56.6 (145)	16.8 (145)	40.3 (145)	10.5 (145)	26.8 (145)	6.5 (145)	26.8 (145)	5.6 (145)
3S2-4	105,000	56.3 (145)	16.7 (145)	40.1 (145)	10.5 (145)	26.7 (145)	6.5 (145)	26.7 (145)	5.6 (145)
2S1-2-2	115,000	55.0 (145)	16.3 (145)	39.1 (145)	10.2 (145)	26.1 (145)	6.3 (145)	26.1 (145)	5.4 (145)
3S1-2-2	115,000	54.3 (145)	16.1 (145)	38.7 (145)	10.1 (145)	25.8 (145)	6.2 (145)	25.7 (145)	5.4 (145)

[a]In thousands of pounds per square inch.
[b]Numbers in parentheses are critical span lengths in feet.

NOTE: 1 ksi = 1,000 psi = 6.89 MPa
1 ft = 0.305 m

PAVEMENT CONSIDERATIONS

Recognizing that the passage of heavy vehicles causes fatigue damage to pavements as well as to bridges and that the country's investment in pavement is several times larger than that of bridges, no change should be made in the bridge formula without considering the consequences of the change to the pavements. The analytical assessment of the impact of such a change on pavement life is not so straightforward as it is for bridges. It is generally accepted that heavy axles, and very short groupings of axles, are more damaging to pavements while gross vehicle weights, or the longer groupings of axles, are more damaging to bridges.

One measure of the fatigue damage heavy vehicles exert on pavements is termed the "equivalent axle load". The equivalent axle load compares the fatigue damage done by a single axle, or grouping of axles, with the damage done by an 18,000 lb (80.06 kN) axle. So an 18,000 lb (80.06 kN) single axle is arbitrarily assigned an equivalent axle load value of 1.0. A single axle, or grouping of axles, that causes twice as much damage as an 18,000 lb (80.06 kN) axle is given as equivalent axle load value of 2.0. Tables of equivalent axle loads for single and tandem axles, on different types of pavement surfaces, have been tabulated and published.(19) These tables are based primarily on the results of the AASHO Road Test completed in the late 1950's where the deterioration of various pavement surfaces under repeated heavy truck loadings was observed.

These tables make it possible to estimate the number of equivalent axle loads resulting from the passage of any given heavy truck. If a truck has two widely spaced axles weighing 18,000 lb (80.06 kN) each, for example, it could be said that the passage of that truck generated 2.0 equivalent axle loads. Another truck with three 18,000 lb (80.06 kN) axles would generate 3.0 equivalent axle loads and would be considered 50 percent more damaging to the pavement. Closely spaced axles have an interactive effect, but equivalent axle loads for tandem axles (groups of two axles jointly suspended) are also tabulated. This makes it possible to calculate the number of equivalent axle loads generated by most of the heavy truck configurations currently in use.

These calculations were made for trucks conforming to the current bridge formula and for trucks conforming to the proposed bridge formula and the results compared. These comparisons for two common truck configurations are shown in figures 31 to 34. Figures 31 and 32 are for the 3S2, a semitrailer truck with a steering axle and two tandems (commonly referred to as the eighteen wheeler). Figures 33 and 34 are for the 2S1-2, a semitrailer truck with a full trailer on two axles; so it has a steering axle with four widely spaced single axles.

For very short and very long vehicles, figures 31 and 33 show the equivalent axle loads per truck to be about the same. In fact, for the short ones, those with wheelbases less than about 36 ft (10.97 m), the proposed formula would lead to smaller equivalent axle loads per truck. If the 80,000 lb (355.8 kN) maximum gross weight per vehicle is maintained, the proposed and current formulas come together at wheelbases just over 50 ft (15.24 m) and are identical for all longer lengths. However, in the intermediate lengths, the equivalent axle loads per truck are significantly greater, in some instances by as much as 20 percent. These intermediate truck lengths, 36 to 50 ft (10.97 to 15.24 m), are very common, and the increase in equivalent axle loads would certainly have a detrimental effect on the wearout rate of our pavements.

So it appears that the average equivalent axle load per vehicle will probably increase if the proposed formula is adopted. Even so, this increase would be more acceptable if it could be shown that the payload per equivalent axle load increased as a result of the change. Figures 32 and 34 show the gross vehicle weights versus wheelbase alongside plots of the assumed payloads divided by vehicle equivalent axle loads. These payloads were calculated by subtracting an arbitrary vehicle empty weight of 25,000 lb (111.2 kN) from the gross vehicle weights. Disappointingly, the payload per equivalent axle load was found to decrease, if only slightly, for vehicles complying with the proposed formula.

The calculations and comparisons of the equivalent axle loads per truck, as shown above, are evidence that the new bridge formula, as stated and

1000 lb = 4.448 kN
1 ft = 0.3048 m

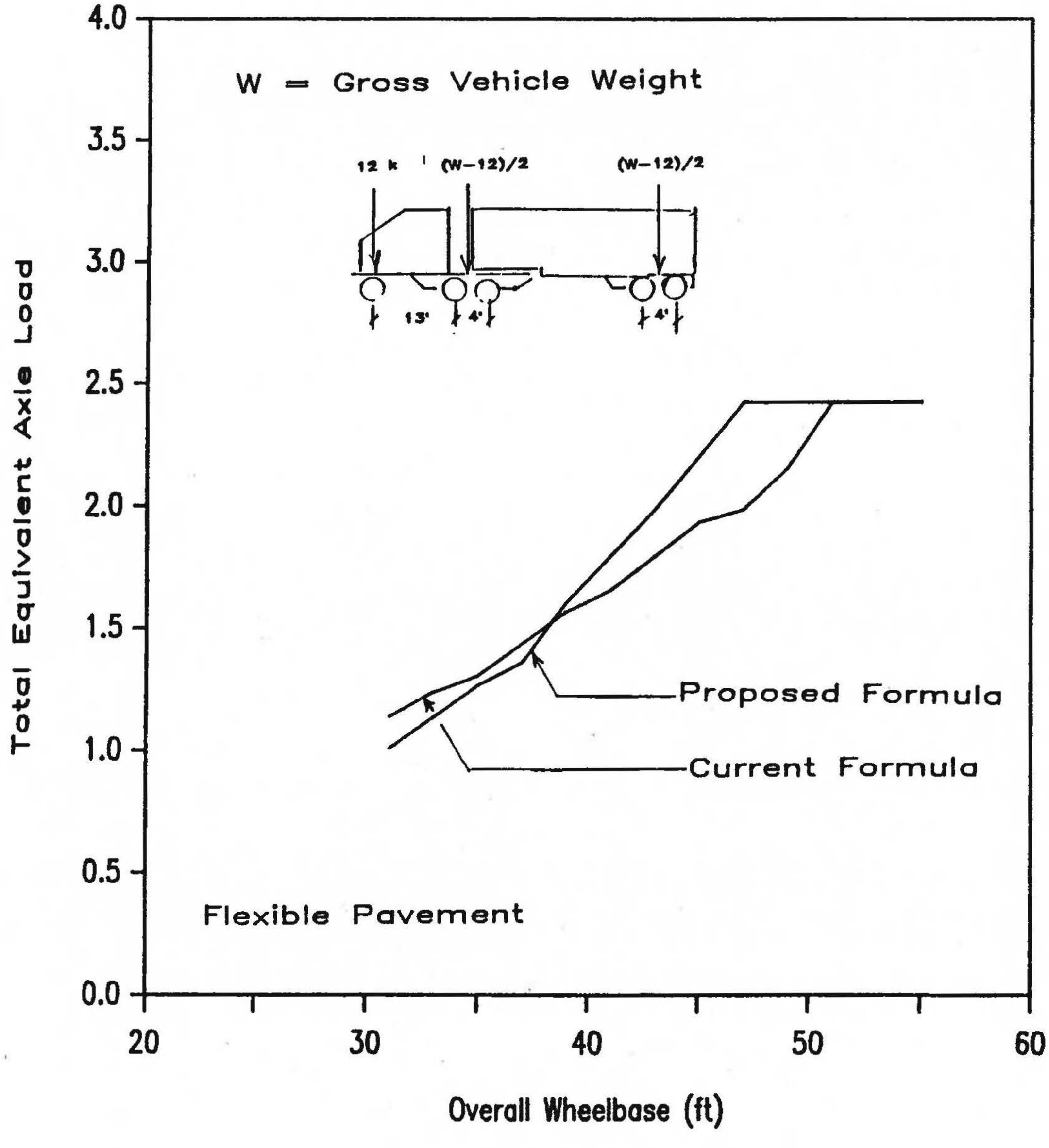


Figure 31. Equivalent Axle Loads of 3S2 Vehicles for the Proposed Formula and the Existing Formula

1000 lb = 4.448 kN
1 ft = 0.3048 m

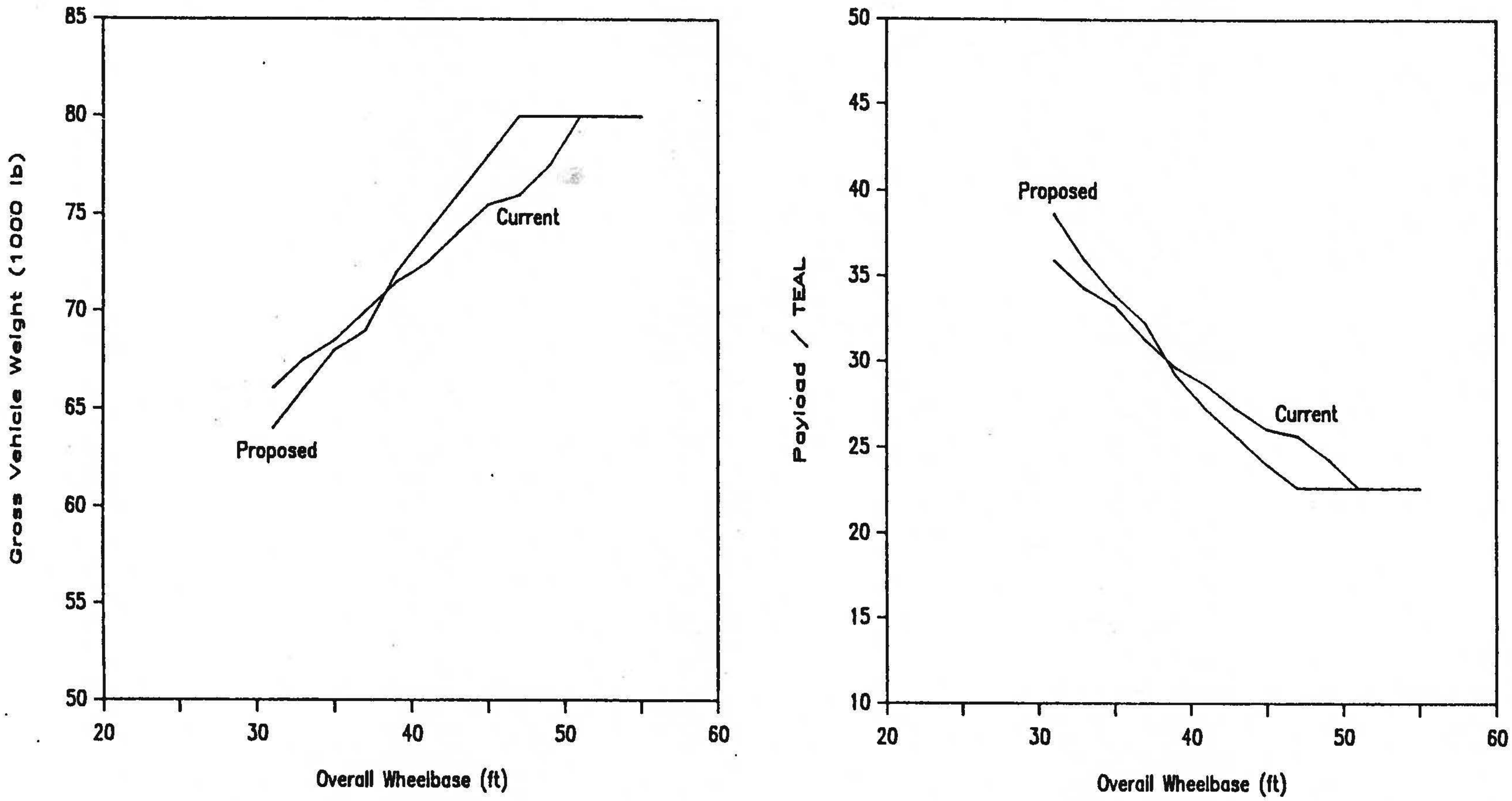


Figure 32. Curves Illustrating the Gross Vehicle Weight and Payload per Equivalent Axle Load for 3S2 Vehicles Complying with the Proposed and Current Bridge Formulas. These are the Same Vehicles as in Figure 31

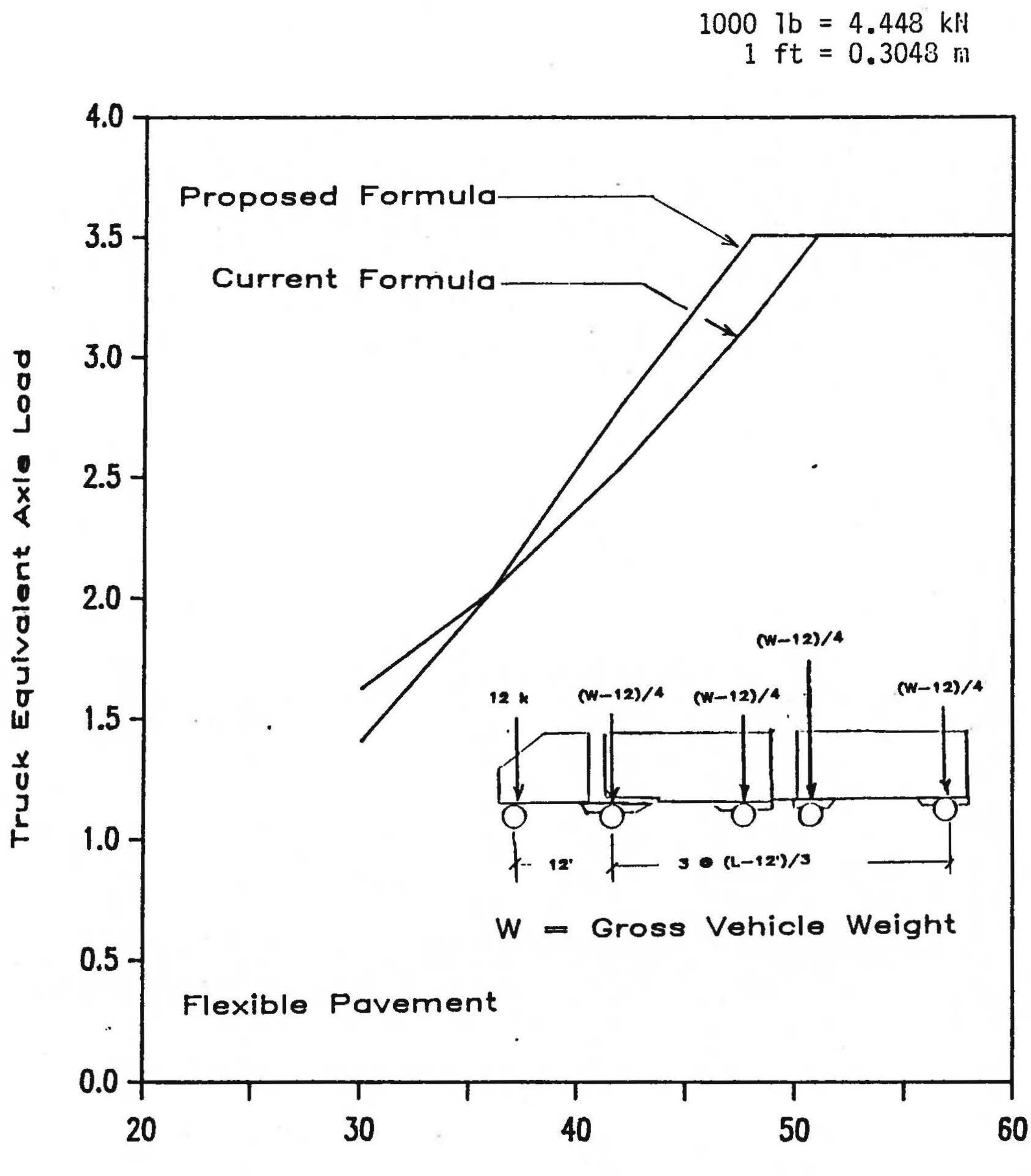


Figure 33. Equivalent Axle Loads of 2S1-2 Vehicles for the Proposed Formula and the Existing Formula

1000 lb = 4.448 kN
1 ft = 0.3048 m

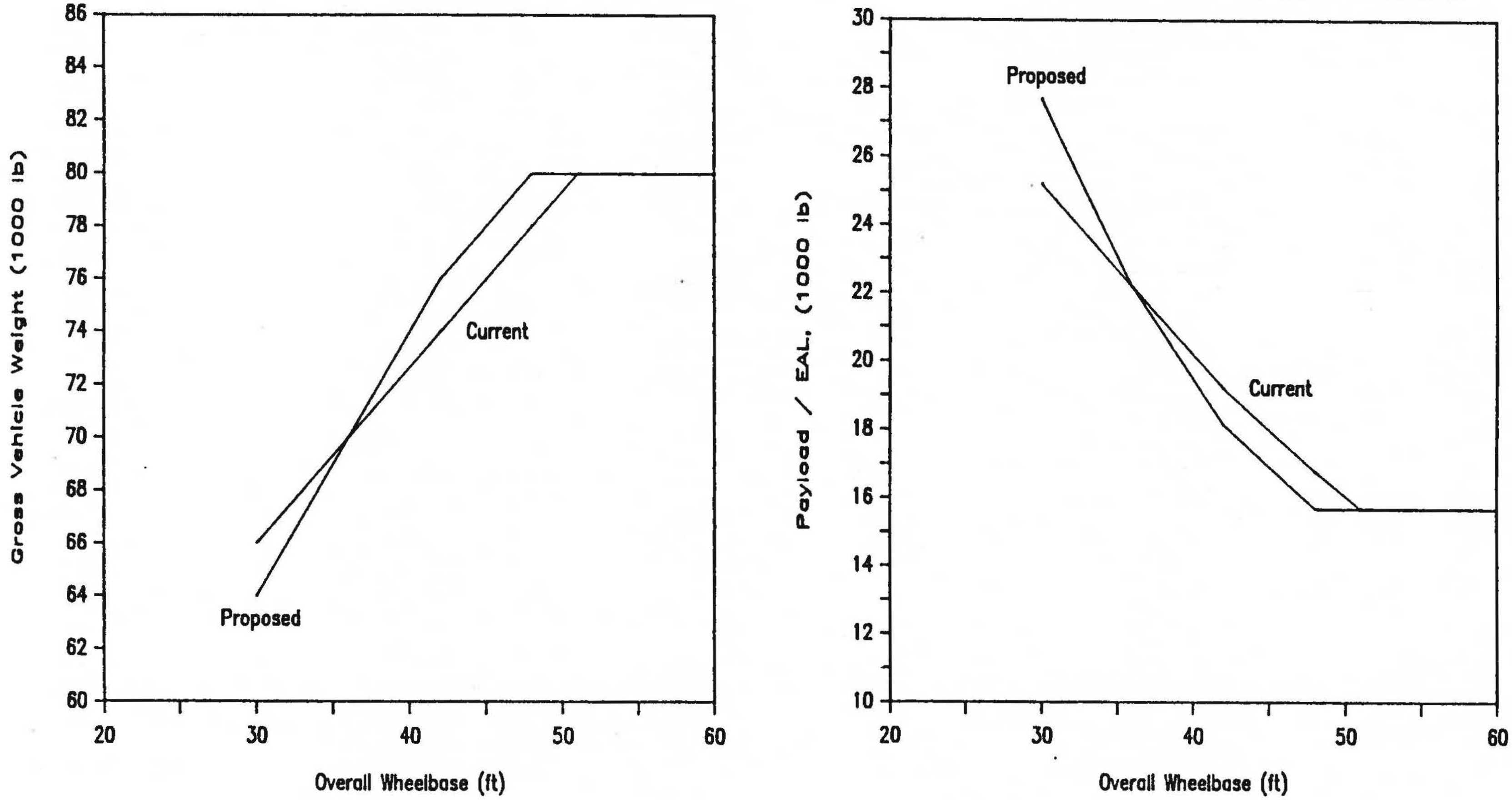


Figure 34. Curves Illustrating the Gross Weight and Payload per Equivalent Axle Load for 2S1-2 Vehicles Complying with the Proposed and Current Bridge Formulas. These are the Same Vehicles as in figure 33

without further modification, would indeed be detrimental to pavements. Currently, pavement deterioration rates are higher than ever, and a change in the bridge formula should not be allowed to magnify that problem. As a result, it is recommended that a detailed study of the influence of a bridge formula change on pavements be initiated with the goal of suggesting additional modifications that would permit the formula to be used without causing unacceptable pavement deterioration. One alternative such a study could consider would be to reduce the allowed maximum single- and tandem-axle loads to coincide with the adoption of the new formula.

CONCLUSIONS

A new bridge formula consisting simply of two straight lines relating the maximum weight allowed on any group of axles to the dimension between the extremes of the axles is being suggested. The formula is independent of the number of included axles. The formula is written

$$W = 34 + L, \qquad 8 \text{ ft} \leq L \leq 56 \text{ ft}$$
$$W = 62 + L/2, \qquad 56 \text{ ft} \leq L$$

where W is the weight in thousands of pounds and L in the outside dimension of any group of axles in feet. This formula would assure the specified overstress ratios would rarely be exceeded for all vehicle configurations even if maximum lengths or maximum gross weights were liberalized.

A further constraint on the formula is that single axles may not weigh more than 20,000 lb (88.96 kN) and tandems (2 axles) not more than 34,000 lb (151.2 kN) for spacings from 4 to 8 ft (1.219 to 2.438 m). For tandems spaced more than 8 ft (2.438 m) but less than 10 ft (3.048 m) the weight may be 30 + L, in thousands of pounds, as it is with the current law.

In bridges, the problems that might be expected with increased axle loads are fatigue damage to reinforcing steel in concrete, weld fatigue in steel elements, and increased rate of crack growth in steel.

Assuming that pavements and bridges are designed for a given traffic density, makeup, and axle loading, it must be accepted that the life of a pavement or a bridge will be reduced if the axle loads are increased. Pavements, in particular, and welded steel bridges would be affected more than reinforced or prestressed concrete bridges by those increases. Assuming a linear relationship between load and stress, an increase in a single-axle load will take a heavy toll on the life of existing pavements and bridges. The benefits received from an increase in axle load would have to be very high to make the accelerated deterioration acceptable. So an increase in the maximum single- and tandem-axle loads is not recommended.

The proposed formula is based on engineering rationale, albeit several controversial assumptions.

If the bridge formula is not enforced, irrespective of the formula being used, bridges are apt to have foreshortened service lives due to fatigue.

The indiscriminate issuing of overweight truck permits, especially those on a periodic or annual basis, are equally apt to result in foreshortened bridge service lives.

Adoption of the proposed bridge formula, without any change in the maximum single and tandem axle loads, will cause an increase in the average equivalent axle load per truck. This is often considered the primary measure of the fatigue damage a vehicle causes to pavement. So, while the proposed formula will satisfactorily protect the bridge structures, there is real concern about its effect on pavements, a consequence that should be carefully evaluated before any changes are made.

APPENDIX A

STATISTICAL ANALYSIS OF TRUCK WEIGHT DATA

INTRODUCTION

The objective of this task is to provide data that would permit a comparison of the existing bridge formula with the proposed formula. Given the form of the two laws, such data must necessarily relate the total weight supported by a single axle or a group of axles to the maximum spacing of the axles. In order to evaluate the existing formula, the number of axles in the group must also be specified.

DESCRIPTION OF THE DATA

The data used in this study has been extracted from the Annual Truck Weight Study that is conducted by the State highway agencies in cooperation with the Federal Highway Administration. The specific data used in this report was collected in 1983. The raw data was supplied by the Highway Statistics Division of the FHWA in the form of magnetic tapes. Further details specifying methods of collection, locations, weighing operations, and classification counts are described elsewhere(20,21).

The 1983 survey resulted in a sample of 127,518 vehicles. For each vehicle was provided specific information on the type of vehicle, the state in which the data was collected, individual axle weights, axle spacings, and gross weights. A typical set of 58 records is shown in figure 35. Additional information required to interpret the data is summarized in tables 18 to 20.

ANALYSIS OF THE DATA

Frequency Count by State

The data was first analyzed to provide a one-way frequency classification for the States involved in the data gathering process. The analysis was accomplished using an existing computer statistical package (22). As shown in table 21, 24 States were involved with approximately 50 percent of the data coming from Iowa, Texas, Washington, Wisconsin, and New Jersey.

DATA

```
1    70101004383070808240000352990009992204001059616019213011400019004004000002700010
2.   70101004383070808220000422990009992411001024710314400000000017000000000001700020
3.   70101004383070808332000412990009992411001047908510610108110611004031005005100030
4.   70101004383070808332000412990009992411001066509615213311916510005027004004600040
5.   70101004383070808332000212990009992331001058308810812612813310004042004006000050
6.   70101004383070808220000411990009991360001014904910000000000017000000000001700060
7.   70101004383070808332000412990009991000000021306605702902703411004024004004300070
8.   70101004383070808332000422990009992090001062711215214111111111005023004004300080
9.   70101004383070808332000212990009992242001066410213013614415214004025005004800090
10.  70101004383070808332000412990009992281001074907314615317720011004025004004400100
11.  70101004383070808322000412990009991000000021705609204102800011030004000004500110
12.  70101004383070808332000422990009992202001048707211010508711312004028004004800120
13.  70101004383070808333000212990009991331001066209809909709511409004024004004500131
14.  70101004383070808333000212991590000000000000000000000400000000000000000000000139
15.  70101004383070808332000412990009992000000025507505605603902912005029004005000140
16.  70101004383070808220000412990009991362001022806516300000000015000000000001500150
17.  70101004383070808220000412990009991262001015005709300000000015000000000001500160
18.  70101004383070808322000412990009991000000034207012307607300012030005000004700170
19.  70101004383070808332000412990009992343001038408410007107305610004032004005000180
20.  70101004383070808322000412990009992203001033207914105305900011028004000004300190
21.  70101004383070808332000412990009992365001039007908906107508610004025004004300200
22.  70101004383070808220000232990009991242001021510511000000000013000000000001300210
23.  70101004383070808332000412990009992365001057910613610711012010004031004004900220
24.  70101004383070808220000412990009991411001016407808600000000019000000000001900230
25.  70101004383070809332000212990009992242001074210416614816715712004028004004800240
26.  70101004383070809332000412990009992371401038409007307007207910005029005004900250
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28.  70101004383070809332000412990009992411001072810315513114519411005026004004600270
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32.  70101004383070809332000422990009992000000038208906909905207317005027005005400310
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47.  70101004383070809332000422990009992000000075311717316512417413004032005005400460
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50.  70101004383070809220000621990009991000000014005508500000000014000000000001400490
51.  70101004383070809332000422990009992202001065510711912815514611004026004004500500
52.  70101004383070809332000412990009992333001074411214416815816211005028004004800510
53.  70101004383070809220000412990009991372001012104907200000000014000000000001400520
54.  70101004383070809332000412990009992000000063610314113815809610004032004005000530
55.  70101004383070810220000232990009992000000014406607800000000017000000000001700540
56.  70101004383070810332000212990009992330001072820014316410311811004028004004700550
57.  70101004383070810332000422990009992200001078610417014118019112004028005004900560
58.  70101004383070810332000422990009992280001034806806606906108411004027004004600570
```

Figure 35. Typical Weight Data for 58 Vehicles, 1983

Table 18. Description of Data Field

TRUCK WEIGHT TABULATING CARO 7 [1]

COLS.	NO. OF COLS.	NUMERIC ONLY OR ALPHA-NUMERIC	OESCRIPTION OF ITEM	REF. PAGE
1	1	N	TRUCK WEIGHT TABULATION CARO CODE: 7 IN COL. NO.	
2-3	2	N	STATE CODE NUMBER: 01-66;	2-C
4-5	2	N	HIGHWAY SYSTEM: 01-12, 31, 32, 41, 42;	4-C
6-8	3	A	STATION IOENTIFICATION (ALPHA OR NUMERIC) RIGHT JUSTIFIEO;	6-C
9	1	N	OIRECTION OF TRAVEL: 1-9, 0; N=1, NE=2 - - - NW=8; BOTH N ANO S OR NE AND SW=9; BOTH E ANO W OR SE AND NW=0;	6-C
10-11	2	N	YEAR DATA WERE GATHEREO: LAST TWO DIGITS OF YEAR	
12-13	2	N	MONTH OATA WERE GATHEREO: 01=JAN.,12=OEC.	
14-15	2	N	DATE OATA WERE GATHEREO: 01-31	
16-17	2	N	HOUR OF OAY: CODE BEGINNING OF HOUR FOR WHICH COUNT IS TAKEN OO THROUGH 23, 1 P.M.=13 FOR COUNT BETWEEN 1 P.M. AND 2 P.M.	
18-23	6	N	VEHICLE TYPE CODE: SEE NEW VEHICLE CODES	32-C-44-C
24-25	2	N	BODY TYPE CODE: 11-94;	45-C-48-C
26	1	N	ENGINE: 1-4 9;	48-C
27-28	2	N	GROSS REGISTEREO WEIGHT GROUP CODE:	51-C
29-31	3	N	REGISTERED WEIGHT (THOUSANDS OF POUNOS): ZEROS IF NOT OETERMINEO;	49-C-50-C
32	1	N	BASIS OF REGISTRATION: 1-6, 9;	
33-34	2	N	MOOEL YEAR OF TRUCK OR TRACTOR: 99 IF NOT DETERMINEO;	
35	1	N	CLASS OF OPERATION: 1=PRIVATE, 2=HIRE UNDER ICC, 3=OTHER FOR HIRE, 9=DOES NOT APPLY;	52-C 53-C-69-C
36-40	5	N	COMMODITY CODE:	
41	1	N	EMPTY OR LOAOEO: 0=EMPTY, 1=LOADEO, 2=OOES NOT APPLY;	70-C
42-45	4	N	TOTAL WEIGHT OF TRUCK OR COMBINATION (HUNOREDS OF POUNDS)	
46-48	3	N	A-AXLE WEIGHT (HUNOREOS OF POUNDS)	
49-51	3	N	B-AXLE WEIGHT (HUNDREOS OF POUNOS)	
52-54	3	N	C-AXLE WEIGHT (HUNDREOS OF POUNOS) UNUSED AXLE	
55-57	3	N	O-AXLE WEIGHT (HUNDREDS OF POUNDS) WEIGHT AND	
58-60	3	N	E-AXLE WEIGHT (HUNDREDS OF POUNOS) SPACING	
61-63	3	N	(A-B) AXLE SPACING (FEET AND TENTHS) FIELDS ARE	
64-66	3	N	(B-C) AXLE SPACING (FEET ANO TENTHS) BLANK	
67-69	3	N	(C-D) AXLE SPACING (FEET AND TENTHS)	
70-72	3	N	(O-E) AXLE SPACING (FEET AND TENTHS)	
73-76	4	N	TOTAL WHEEL BASE (FEET ANO TENTHS)	
77-79	3	N	CARD SERIAL NUMBER (SAME FOR CONTINUATION CARO) THE SERIAL NUMBER SHOULD START WITH '001' FOR THE FIRST TRUCK WEIGHEO AT EACH STATION EACH SHIFT. CONTINUATION CAROS SHOULO HAVE THE SAME NUMBER AS THE CAROS THEY SUPPLEMENT.	71-C
80	1	N	(O-THIS IS THE ONLY CARO, 1=ANOTHER TO FOLLOW)	

[1]- USEO FOR VEHICLES HAVING NO MORE THAN 5 AXLES OR FOR THE FIRST 5 AXLES OF LARGER COMBINATIONS.

Table 19. Vehicle Type Coding Chart

	1st Character	2nd Character	3rd Character	4th Character	5th Character	6th Character
Passenger vehicles	basic vehicle type = 0	(C) vehicle type	(A) registration modifier	(B) light trailer modifier	State of registration	
Buses	basic vehicle type = 1	(D) vehicle type	(A) registration modifier	(E) axle & tire modifier	State of registration	
Single-unit trucks	basic vehicle type = 2	(F) total axles	(A) registration modifier	(B) light trailer modifier	State of registration	
Tractor + semitrailer	basic vehicle type = 3	total axles on power unit	(G) total axles on first trailer	code = 0	code = 0	(H) special modifier
Truck + full trailer	basic vehicle type = 4	total axles on power unit	(G) total axles on first trailer	code = 0	code = 0	(H) special modifier
Tractor + semitrailer + full trailer	basic vehicle type = 5	total axles on power unit	(G) total axles on first trailer	(G) total axles on second trailer	code = 0	(H) special modifier
Truck + full trailer + full trailer	basic vehicle type = 6	total axles on power unit	(G) total axles on first trailer	(G) total axles on second trailer	code = 0	(H) special modifier
Tractor - semitrailer + 2 full trailers	basic vehicle type = 7	total axles on power unit	(G) total axles on first trailer	(G) total axles on second trailer	(G) total axles on third trailer	(H) special modifier
Truck + 3 full trailers	basic vehicle type = 8	total axles on power unit	(G) total axles on first trailer	(G) total axles on second trailer	(G) total axles on third trailer	(H) special modifier

Table A
0 State registration not recorded
1 In-State, all
2 Out-of-State, all
3 In-State, nongovernment owned
4 In-State, government owned
5 Out-of-State, nongovernment owned
6 Out-of-State, government owned
7 Federal government owned

Table B
0 No trailer
1 Camp trailer
2 Mobile home
3 Cargo trailer
4 Boat trailer
6 Towed auto
7 Towed truck
8 "Slantback"
9 Any or all types trailed vehicles

Table C
1 Motorcycle
2 Motorscooter
3 Motorcycle or motorscooter
4 Standard auto
5 Compact auto
6 Small auto
7 Standard and compact auto
8 Compact and small auto

Table D
1 Bus, intercity, commercial
2 Bus, transit, commercial
3 Bus, sightseeing, commercial
4 Bus, commercial, other
5 Bus, commercial, any type
6 Bus, school and nonrevenue
7 Bus, camper
8 Bus, all nonrevenue type

Table E
0 Axle arrangement not recorded
1 Two-axle, four-tire
2 Two-axle, six-tire
3 Three-axle
4 Four-axles or more

Table F
0 Panel and pickup
1 Heavy two-axle, four-tire
2 Two-axle, six-tire
3 Three-axle
4 Four-axle
5 Five-axle
6 Six-axle
7 Seven-axle
8 Eight-axles or more

Table G
0 No Trailer
1 Single-axle trailer
2 Two-axle trailer
3 Three-axle trailer
4 Four-axle trailer
5 Five-axle trailer
6 Six-axle trailer
7 Two-axle trailer with one spread tandem
8 Three-axle trailer with one spread tandem
9 Four-axle trailer with one spread tandem

Table H
0 No special modification
1 One spread tandem on pavement in addition to any indicated by 7, 8, 9 in C3, C4, C5.
2 Two spread tandems on pavement in addition to any indicated by 7, 8, 9 in C3, C4, C5.
3 Three spread tandems on pavement in addition to any indicated by 7, 8, 9 in C3, C4, C5.
4 One trailer piggyback and no spread tandems except those indicated by 7, 8, 9 in C3, C4, C5.
5 One trailer piggyback and one spread tandem on pavement in addition to any indicated by 7, 8, 9 in C3, C4, C5.
6 One trailer piggyback and two sets of spread tandems on pavement in addition to any indicated by 7, 8, 9 in C3, C4, C5.
7 Two trailers piggyback and no spread tandems except those indicated by 7, 8, 9 in C3, C4, C5.
8 Two trailers piggyback and one spread tandem on pavement in addition to any indicated by 7, 8, 9 in C3, C4, C5.
9 Two trailers piggyback and two sets of spread tandems on pavement in addition to any indicated by 7, 8, 9 in C3, C4, C5.

Table 20. Codes for Census Divisions & States

Code	New England (01)
01	Connecticut
02	Maine
03	Massachusetts
04	New Hampshire
05	Rhode Island
06	Vermont
	Middle Atlantic (02)
07	New Jersey
08	New York
09	Pennsylvania
	South Atlantic (North) (03)
11	Delaware
12	District of Columbia
13	Maryland
14	Virginia
15	West Virginia
	South Atlantic (South) (04)
16	Florida
17	Georgia
18	North Carolina
19	South Carolina
	East North Central (05)
21	Illinois
22	Indiana
23	Michigan
24	Ohio
25	Wisconsin
	East South Central (06) (East of Mississippi River)
26	Alabama
27	Kentucky
28	Mississippi
29	Tennessee

Code	West North Central (07) (West of Mississippi River)
31	Iowa
32	Kansas
33	Minnesota
34	Missouri
35	Nebraska
36	North Dakota
37	South Dakota
	West South Central (08)
41	Arkansas
42	Louisiana
43	Oklahoma
44	Texas
	Mountain (09)
51	Arizona
52	Colorado
53	Idaho
54	Montana
55	Nevada
56	New Mexico
57	Utah
58	Wyoming
	Pacific (10)
61	California
62	Oregon
63	Washington
	(11)
64	Alaska
65	Hawaii
66	Puerto Rico

Table 21. Vehicle Frequency Count by State, 1983

STATE CODE	FREQUENCY	CUM FREQ	PERCENT	CUM PERCENT
1	3283	3283	2.575	2.575
7	9194	12477	7.210	9.785
8	2977	15454	2.335	12.119
9	5213	20667	4.088	16.207
14	4236	24903	3.322	19.529
21	2988	27891	2.343	21.872
22	3021	30912	2.369	24.241
25	9955	40867	7.807	32.048
31	17620	58487	13.818	45.866
32	1181	59668	0.926	46.792
33	4513	64181	3.539	50.331
34	3502	67683	2.746	53.077
37	3733	71416	2.927	56.005
41	5533	76949	4.339	60.344
42	1590	78539	1.247	61.591
43	6026	84565	4.726	66.316
44	15310	99875	12.006	78.322
52	1914	101789	1.501	79.823
53	2115	103904	1.659	81.482
54	1994	105898	1.564	83.046
56	7394	113292	5.798	88.844
63	12927	126219	10.137	98.981
64	430	126649	0.337	99.319
65	869	127518	0.681	100.000

Frequency Count by Vehicle Type

The data was also analyzed to provide a one-way frequency classification for the vehicles. As shown in table 22, 131 vehicle types were distinguished. However, the vehicle type designated 332000 (see table 22) accounted for 61.5 percent (78,474 vehicles) of the sample. In the ensuing analysis, vehicle types 200000, 220000, 230000, 321000, 322000, 332000, and 521200, which accounted for approximately 93 percent of the sample, were analyzed further.

Two-Way Cross Tabulation for Truck Weight Data

The objective of this part of the analysis was to generate a two-dimensional frequency diagram for the entire data set with the number of axles as a parameter. This objective was realized in the following manner. For each of the selected vehicles, a new data set was created. Each element of this new data set consisted of the spacing between a group of axles and the total load supported by the contributing axles. For example, a five axle vehicle produced 10 observations. For each vehicle type, this new data set was then partitioned according to the number of axles. Next, all the data sets representing the same number of contributing axles were combined into a single data set. This step resulted in a collection of four data sets: one set each for two axles, three axles, four axles, and five axles with 381,639, 263,326, 168,334, and 79,109 observations, respectively. In the final stage of the analysis, each of the four data sets was used to generate a two-way cross tabulation table with the axle spacing and the weight of the contributing axles as the variables.

A typical two-way frequency table developed using the outlined procedure is shown in table 23. From these results, coarser tables were constructed for all axle spacings as shown in tables 11 to 14.

Table 22. Vehicle Frequency by Vehicle Type

VEHICLE CODE	FREQUENCY	CUM FREQ	PERCENT	CUM PERCENT
200000	6223	6223	4.880	4.880
200079	97	6320	0.076	4.956
200100	24	6344	0.019	4.975
200200	2	6346	0.002	4.977
200300	33	6379	0.026	5.002
200400	10	6389	0.008	5.010
200500	25	6414	0.020	5.030
200600	1	6415	0.001	5.031
200791	1	6416	0.001	5.031
200800	1	6417	0.001	5.032
200900	62	6479	0.049	5.081
201000	481	6960	0.377	5.458
201100	1	6961	0.001	5.459
201200	2	6963	0.002	5.460
201300	7	6970	0.005	5.466
201700	1	6971	0.001	5.467
201900	2	6973	0.002	5.468
202000	26	6999	0.020	5.489
202200	1	7000	0.001	5.489
210000	276	7276	0.216	5.706
210079	3	7279	0.002	5.708
210300	28	7307	0.022	5.730
211000	69	7376	0.054	5.784
211200	1	7377	0.001	5.785
212000	1	7378	0.001	5.786
220000	16528	23906	12.961	18.747
220004	2	23908	0.002	18.749
220079	72	23980	0.056	18.805
220100	4	23984	0.003	18.808
220200	3	23987	0.002	18.811
220300	67	24054	0.053	18.863
220400	3	24057	0.002	18.866
220500	16	24073	0.013	18.878
220600	6	24079	0.005	18.883
220700	4	24083	0.003	18.886
220800	7	24090	0.005	18.891
220900	50	24140	0.039	18.931
220979	6	24146	0.005	18.935
221000	927	25073	0.727	19.662
221065	479	25552	0.376	20.038
221200	1	25553	0.001	20.039
222000	267	25820	0.209	20.248
230000	5462	31282	4.283	24.531
230079	3	31285	0.002	24.534
230300	5	31290	0.004	24.538
230500	1	31291	0.001	24.538
230600	1	31292	0.001	24.539
230700	7	31299	0.005	24.545
230800	5	31304	0.004	24.549
230900	8	31312	0.006	24.555
231000	201	31513	0.158	24.713
231065	96	31609	0.075	24.788
231300	1	31610	0.001	24.789
231500	26	31636	0.020	24.809
232000	49	31685	0.038	24.847
232800	4	31689	0.003	24.851

Table 22. Vehicle Frequency by Vehicle Type (continued)

VEHICLE CODE	FREQUENCY	CUM FREQ	PERCENT	CUM PERCENT
233200	1	31690	0.001	24.851
240000	273	31963	0.214	25.065
241000	88	32051	0.069	25.134
241300	2	32053	0.002	25.136
242000	5	32058	0.004	25.140
242300	1	32059	0.001	25.141
250000	1	32060	0.001	25.142
321000	2126	34186	1.667	26.809
322000	6547	40733	5.134	31.943
323000	87	40820	0.068	32.011
324000	1	40821	0.001	32.012
327000	26	40847	0.020	32.032
331000	355	41202	0.278	32.311
332000	78474	119676	61.540	93.850
332001	27	119703	0.021	93.871
332002	1	119704	0.001	93.872
332004	92	119796	0.072	93.944
332006	2	119798	0.002	93.946
333000	693	120491	0.543	94.489
334000	15	120506	0.012	94.501
337000	626	121132	0.491	94.992
338000	27	121159	0.021	95.013
341000	2	121161	0.002	95.015
342000	122	121283	0.096	95.110
343000	66	121349	0.052	95.162
344000	16	121365	0.013	95.175
354000	1	121366	0.001	95.176
355000	1	121367	0.001	95.176
421000	54	121421	0.042	95.219
421001	1	121422	0.001	95.219
422000	328	121750	0.257	95.477
423000	55	121805	0.043	95.520
427000	1	121806	0.001	95.521
431000	25	121831	0.020	95.540
432000	1325	123156	1.039	96.579
433000	158	123314	0.124	96.703
434000	51	123365	0.040	96.743
435000	7	123372	0.005	96.749
437000	2	123374	0.002	96.750
441000	1	123375	0.001	96.751
442000	24	123399	0.019	96.770
443000	15	123414	0.012	96.782
444000	20	123434	0.016	96.797
449000	1	123435	0.001	96.798
521100	1	123436	0.001	96.799
521200	2567	126003	2.013	98.812
521300	14	126017	0.011	98.823
521700	3	126020	0.002	98.825
522100	6	126026	0.005	98.830
522200	142	126168	0.111	98.941
522300	3	126171	0.002	98.944
531100	4	126175	0.003	98.947
531200	567	126742	0.445	99.391
531300	26	126768	0.020	99.412
531400	1	126769	0.001	99.413
532100	3	126772	0.002	99.415
532200	497	127269	0.390	99.805

Table 22. Vehicle Frequency by Vehicle Type (continued)

VEHICLE CODE	FREQUENCY	CUM FREQ	PERCENT	CUM PERCENT
532300	165	127434	0.129	99.934
532400	37	127471	0.029	99.963
533200	5	127476	0.004	99.967
533300	4	127480	0.003	99.970
537300	1	127481	0.001	99.971
542100	1	127482	0.001	99.972
542200	7	127489	0.005	99.977
542400	6	127495	0.005	99.982
543200	1	127496	0.001	99.983
543300	1	127497	0.001	99.984
622200	1	127498	0.001	99.984
632100	1	127499	0.001	99.985
632200	1	127500	0.001	99.986
633300	1	127501	0.001	99.987
721220	10	127511	0.008	99.995
722220	1	127512	0.001	99.995
731220	2	127514	0.002	99.997
732220	3	127517	0.002	99.999
742230	1	127518	0.001	100.000

Table 23. Two-Way Classification for Three-Axle Group

GROUP AXLE WEIGHT IN KIPS

AXLE SPACING IN FEET

FREQUENCY PERCENT ROW PCT COL PCT	0-5	5.1-10	10.1-15	15.1-20	20.1-25	25.1-30	30.1-35	35.1-40	40.1-45	45.1-50	50.1-55	55.1-60	60.1-65	65.1-70	70.1-75	75.1-80	80.1-85	85.1-90	90.1-95	95.1-100	TOTAL
0.1-4	0 0.00 0.00 0.00	1 0.00 3.57 0.15	0 0.00 0.00 0.00	1 0.00 3.57 0.00	5 0.00 17.86 0.02	6 0.00 21.43 0.03	0 0.00 0.00 0.00	2 0.00 7.14 0.01	4 0.00 14.26 0.01	6 0.00 21.43 0.02	3 0.00 10.71 0.02	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	28 0.0[illegible]
4.1-12	0 0.00 0.00 0.00	0 0.00 0.00 0.00	4 0.00 1.30 0.03	14 0.01 4.56 0.06	26 0.01 8.67 0.06	40 0.02 12.03 0.16	35 0.01 11.40 0.17	66 0.04 21.66 0.24	52 0.02 16.64 0.14	39 0.02 12.70 0.11	11 0.00 2.58 0.07	1 0.00 0.32 0.02	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	307 0.13
12.1-16	1 0.00 0.00 14.26	2 0.00 0.01 0.46	186 0.08 0.56 1.35	2480 1.06 7.15 6.43	6520 2.74 16.76 22.00	5112 2.17 15.66 22.63	5186 2.21 15.62 24.82	6025 2.56 17.24 23.62	7306 3.10 20.66 16.22	1504 0.64 6.22 4.16	225 0.14 0.96 2.25	56 0.03 0.17 1.66	16 0.01 0.05 1.66	4 0.00 0.01 1.49	1 0.00 0.00 1.16	1 0.00 -0.00 6.17	1 0.00 0.00 11.11	1 0.00 0.00 25.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	34407 14.7[illegible]
16.1-20	0 0.00 0.00 0.00	14 0.01 0.04 2.10	407 0.17 1.12 2.68	2996 1.27 6.36 10.16	7608 3.22 21.31 25.80	3645 1.55 10.16 17.05	3177 1.35 6.65 16.08	4657 2.11 13.82 15.56	8560 6.06 26.65 25.30	2842 1.21 7.62 7.86	616 0.22 1.44 2.48	112 0.05 0.21 2.60	29 0.01 0.06 2.88	4 0.00 0.01 1.46	3 0.00 0.01 2.49	0 0.00 0.00 0.00	1 0.00 0.00 11.11	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	35474 15.24
20.1-24	1 0.00 0.01 14.26	12 0.01 0.13 3.01	263 0.12 2.71 2.04	682 0.26 6.42 2.36	1610 0.60 12.08 6.74	856 0.36 7.82 6.00	868 0.36 7.64 4.00	1561 0.67 14.72 6.26	3017 1.26 26.01 7.95	1561 0.66 14.46 6.22	276 0.16 2.46 2.52	84 0.04 0.78 2.66	27 0.01 0.25 2.86	10 0.00 0.08 3.62	4 0.00 0.04 6.66	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	10773 4.58
24.1-28	0 0.00 0.00 0.00	30 0.01 0.41 4.82	630 0.27 6.61 4.40	1102 0.47 15.08 3.73	645 0.27 6.76 3.16	484 0.21 6.62 2.36	636 0.16 5.86 3.07	902 0.21 6.66 1.96	1007 0.42 12.77 2.65	1757 0.76 24.18 4.66	500 0.21 6.84 2.36	155 0.06 2.09 4.60	46 0.02 0.62 4.46	6 0.00 0.11 2.90	2 0.00 0.04 2.46	1 0.00 0.01 4.17	1 0.00 0.01 11.14	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	7315 7.[illegible]
28.1-32	0 0.00 0.00 0.00	166 0.07 0.42 25.62	4580 1.96 11.56 21.87	7056 2.00 17.83 22.81	3743 1.56 8.46 12.56	2840 1.23 7.42 12.76	2882 1.23 7.25 12.97	2722 1.16 6.90 10.76	3990 1.66 10.07 10.56	7160 2.05 18.13 16.96	2395 1.44 6.51 33.63	616 0.26 1.56 16.62	236 0.10 0.60 23.61	70 0.02 0.16 26.36	16 0.01 0.04 17.44	6 0.00 0.02 25.00	4 0.00 0.01 44.44	2 0.00 0.01 50.00	0 0.00 0.00 0.00	2 0.00 0.01 100.00	39609 16.47
32.1-36	7 0.00 0.01 71.43	330 0.74 0.40 50.83	7021 2.98 6.46 46.02	12266 5.26 14.63 41.96	7221 3.11 6.83 24.67	5681 2.64 7.20 27.88	5152 2.04 7.41 26.18	6742 2.86 8.13 36.81	9617 6.17 11.63 25.86	17771 7.20 20.60 47.50	7616 3.32 6.46 62.87	1564 0.66 1.46 46.44	481 0.20 0.64 47.61	126 0.06 0.17 50.36	51 0.02 0.06 55.30	12 0.01 0.02 56.17	2 0.00 0.00 33.33	1 0.00 0.00 25.00	2 0.00 0.00 75.00	0 0.00 0.00 0.00	83016 35.26
36.1-40	1630 0.66 6.02 10.86	622 0.16 2.12 13.77	136 0.06 0.66 12.35	36 0.02 0.16 13.04	6 0.00 0.03 6.66	2 0.00 0.01 12.50	0 0.00 0.00 0.00	0 0.00 0.00 0.00	1 0.00 0.00 25.00	0 0.00 0.00 0.00	1630 0.66 6.02 10.86	422 0.16 2.13 12.17	126 0.06 0.66 12.35	36 0.02 0.16 12.04	6 0.00 0.03 6.66	2 0.00 0.01 12.50	0 0.00 0.00 0.00	0 0.00 0.00 0.00	1 0.00 0.00 25.00	0 0.00 0.00 0.00	20300 8.4[illegible]
40.1-44	217 0.09 6.65 1.46	64 0.04 2.42 2.66	22 0.01 0.64 2.28	2 0.00 0.12 1.06	2 0.00 0.12 2.66	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	217 0.08 6.66 1.46	64 0.04 2.43 2.66	22 0.01 0.64 2.36	2 0.00 0.12 1.08	3 0.00 0.13 2.46	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	2453 1.0[illegible]
44.1-48	52 0.02 7.01 0.35	22 0.01 2.96 0.70	7 0.00 0.94 0.70	2 0.00 0.27 0.72	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	52 0.02 7.01 0.35	22 0.01 2.96 0.70	7 0.00 0.94 0.70	2 0.00 0.27 0.72	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	742 0.32
48.1-52	6 0.00 6.46 0.06	1 0.00 0.84 0.03	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	6 0.00 6.48 0.06	1 0.00 0.84 0.03	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	106 0.05
52.1-56	1 0.00 2.74 0.01	2 0.00 6.56 0.06	2 0.00 6.56 0.30	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	1 0.00 2.76 0.01	2 0.00 5.38 0.06	2 0.00 6.66 0.20	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	26 0.02
56.1-60	2 0.00 16.25 0.02	2 0.00 12.50 0.06	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	2 0.00 16.75 0.02	2 0.00 12.50 0.06	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	16 0.01
60.1-64	0 0.00 0.00 0.00	4 0.00 57.14 0.13	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	4 0.00 61.14 0.13	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	[illegible] 0.0[illegible]
64.1-68	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	0 0.00 0.00 0.00	[illegible] 0.0[illegible]
TOTAL	14672 6.22	2126 1.32	1004 0.42	376 0.15	86 0.04	24 0.01	6 0.00	4 0.00	4 0.00	2 0.00	14672 6.32	2126 1.32	1004 0.42	276 0.12	86 0.04	24 0.01	6 0.00	4 0.00	4 0.00	2 0.00	225423 100.00

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