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16. Abstract

An analytical study of H15 bridges in Texas has resulted in an improved procedure for issuing overweight truck permits. Single and multiple axle limits, specifically to protest pavements rather than bridges, are assumed. The weight restrictions limit any (and all) axle groups as a function of the gage and tire contact widths. The restrictions were derived to assure that the maximum stress in any bridge member never exceeds the operating level. Only one vehicle is allowed on the bridge at a time but it is assumed to be traveling at a speed such that the full impact allowance is generated.

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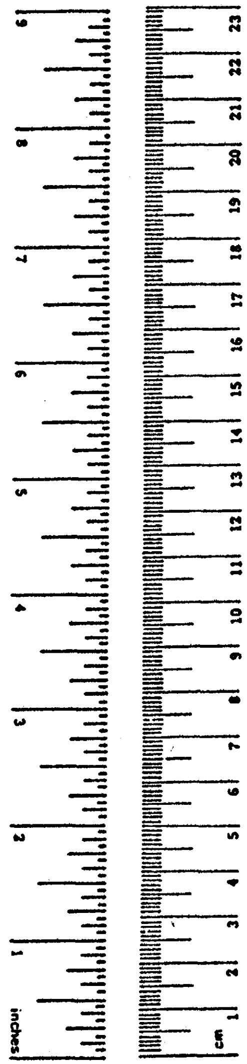
METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	2.54	centimetres	cm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
	AREA			
in^2	square inches	645.2	centimetres squared	cm^2
ft^2	square feet	0.0929	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
mi^2	square miles	2.59	kilometres squared	km^2
ac	acres	0.395	hectares	ha
	MASS (weight)			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
	VOLUME			
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.0328	metres cubed	m^3
yd^3	cubic yards	0.0765	metres cubed	m^3

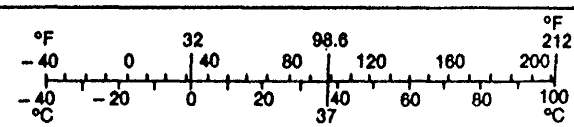
NOTE: Volumes greater than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
	AREA			
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
km^2	kilometres squared	0.39	square miles	mi^2
ha	hectores (10 000 m^2)	2.53	acres	ac
	MASS (weight)			
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T
	VOLUME			
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
	TEMPERATURE (exact)			
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



These factors conform to the requirement of FHWA Order 5190.1A.

* SI is the symbol for the International System of Measurements

Overload Permit Procedures

Interim Report

December, 1992

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Study No. 2/10-5-91-1266
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SUMMARY

The research reported herein developed criteria for issuing permits for overweight vehicles passing over H15 simply supported bridges in the state of Texas. Two sets of criteria were developed.

The first set is determined as a function of only the dimensions of the vehicle as the current Texas permit rules are based upon. Texas Department of Transportation permits rules extend only up to 80 ft. wheelbases. The first set of criteria are determined for wheelbases up to 120 ft. This set of criteria when compared to that currently used by the Texas Department of Transportation is found to be somewhat more restrictive.

A second set of criteria are developed based on the dimensions of the vehicle and the span length of any bridges that may be on the route of the permitted vehicle. By knowing the bridge span length higher vehicle weights are allowed. The formula developed for these criteria can be easily incorporated into a computerized permitting system. Also, higher permit weights can be authorized without additional analysis by an engineer, saving time and money.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and their accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Texas Department of Transportation. This report does not constitute a standard, specification, or regulation, It is not intended for construction, bidding, or permit purposes.

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1. INTRODUCTION

The issuing of overweight permits has been an issue of major importance to highway departments. On the highways of Texas there are many lightweight H15 type bridges. It is these bridges which are most susceptible to damage from overweight vehicles. Significant advances have been made in highway and bridge design in the past twenty to thirty years. These advances have made it possible to allow heavier loads on the nation's Interstate Highway System. However, despite these advances there are many older H15 type bridges that are still in service, which were designed twenty, thirty, or more years ago. It is these bridges that must be taken into consideration when formulating overweight permit rules.

The Texas Department of Transportation (TxDOT) currently issues over 32,000 oversize and/or overweight permits each year. The restrictions for issuing overweight permits were adopted by the Texas legislature into the *Texas Administrative Code* on May 29, 1991. The basis for these restrictions is a statistical analysis of permits previously issued. These restrictions probably protect most bridges in the state from significant damage or failure. However, an independent engineering based analysis has never been done to confirm the current restrictions. Also the current restrictions do not make provisions for allowing heavier permit loads for different bridge span lengths.

The primary objective of this research is to develop criteria to determine if a vehicle can safely pass over a simply supported H15 bridge. H15 type bridges are examined because they tend to be at more risk than any other type of bridge. Two different approaches are used when developing the rules for issuing permits. The first involves developing a formula to determine an allowable maximum gross vehicle weight based on the wheelbase of the truck. The formula allows the permitted vehicle to safely pass over a bridge of any span length if a specific route is unknown. The gross vehicle weights resulting from this formula are then compared to the current weight restrictions that the state of Texas currently uses. A second, more liberal formula is also developed. This formula assumed that the route of the permitted vehicle and therefore, the length of all bridge spans is known. By knowing the span length the allowable gross vehicle weight can be increased.

In determining the weight restrictions, a formula for a reduction factor based upon the gage width of the truck is also developed. This reduction factor formula for gage is also compared to the current TxDOT formula.

2. CURRENT TXDOT PERMIT RULES

2.1 TxDOT Permitting Procedures

Existing TxDOT permit rules for overweight vehicles are based on the wheelbase length and width. Wheelbase length is the distance from the center to the first axle to the center of the last axle in any axle group. The wheelbase width is referred to as "gage". Typical truck and trailer rigs have a standard gage of 6 ft. Gage is defined as the transverse spacing distance between tires on an axle, expressed in feet and measured to the nearest inch. Gage is measured from center of tire to center of tire on an axle equipped with only two tires, or measured from the center of the dual wheels on one end of the axle to the center of the dual wheels on the opposite end of the axle. The gage distance for different tire and axle configurations is shown in Fig. 1.

The *Texas Administrative Code* imposes restrictions on axle groups of overweight vehicles as shown in Table 1. In addition, a restriction for any axle of 850 lb/in. of tire width or 25,000 lbs, whichever is less, is also imposed. These restrictions are primarily for the purpose of protecting the pavement. However, for a vehicle that either of exceeds these restrictions a permit may still be issued by determining an equivalent distributed load.

Number of Axles in Group	Maximum Permissible Axle Group Weight
1	25,000 lbs.
2	45,000 lbs.
3	45,000 lbs.
4	70,000 lbs.
5	81,400 lbs.

Table 1: Axle group weigth restrictions.

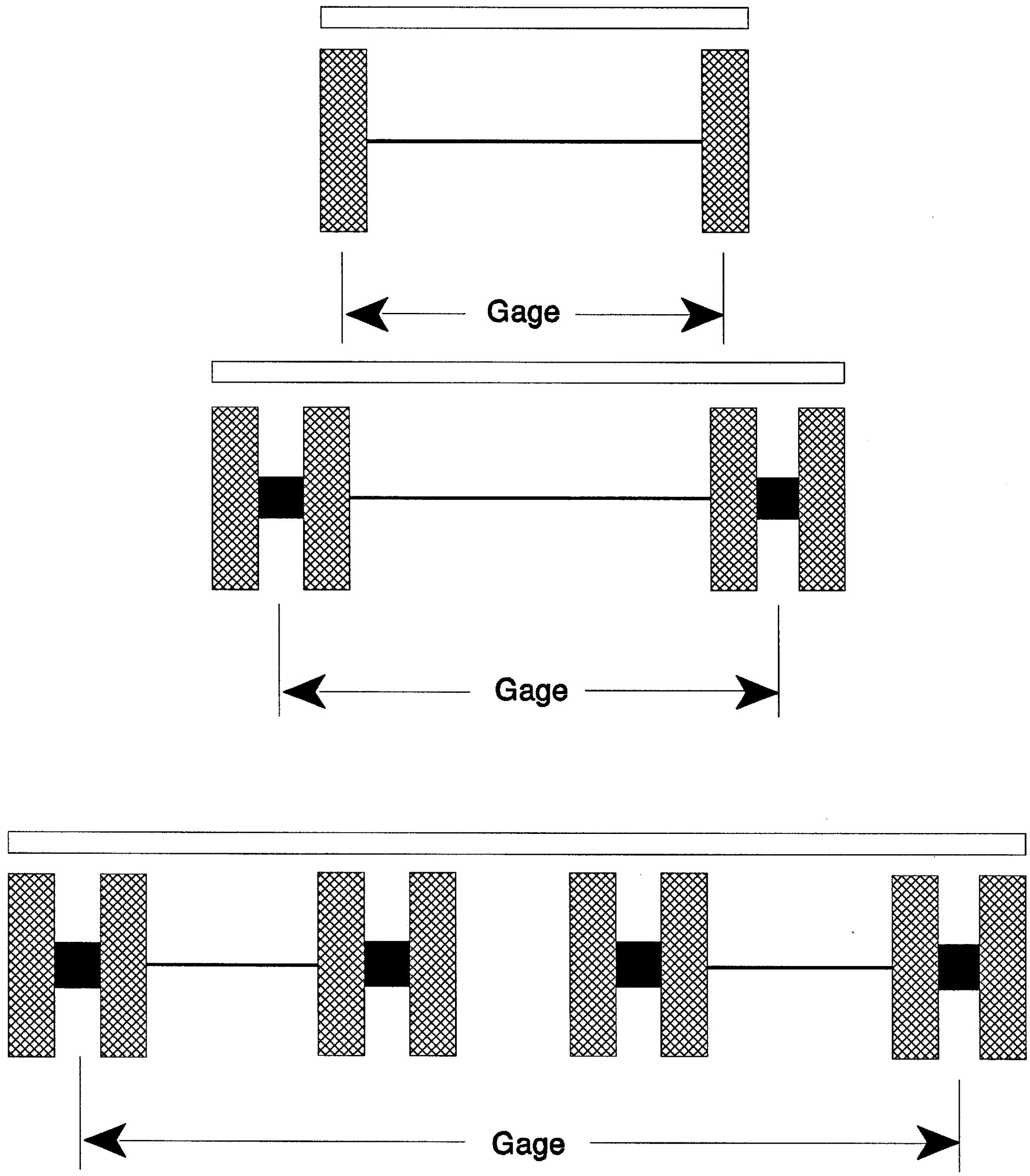


Figure 1: Gage distance for various axle confirgurations.

The *Texas Administrative Code* specifies the maximum permittable axle group distributed load for overweight vehicles is determined using the formula:

$$W_{un} = \frac{T}{WB + 4} \tag{1}$$

where

W_{un} = the unmodified equivalent distributed load per linear foot,

T = the summation of axle loads of group of two or more axles; any combination of axle loads may be considered as a group up to the total number of axles for the vehicle,

WB = wheelbase length (feet).

A vehicle with axle groups composed of eight or more tires per axle, or with axle groups having a gage greater the 6.0 ft. on an axle, may have additional reduction factors applied to each axle. This is done before summing or totaling the axle loads for the vehicle. The revised equivalent axle load is calculated by rewriting Eqn. 1 in the following form:

$$W_{rev} = \frac{\sum_{i=1}^{n^{*}} (R_i * S_i * T_i)}{WB + 4} \tag{2}$$

where

W_{rev} = revised equivalent distributed load per linear foot,

S_i = reduction factor accounting for each axle which may have more than four tires on the axle line,

= 1.0 for axles with four tires or fewer,
= 0.96 for axles with eight or more tires,

R_i = reduction factor accounting for wider gage axle groups and is calculated by the following formula:

where

$$R_i = \frac{6 + G}{2G} \tag{3}$$

G = the gage (feet),

n = number of axles.

The equivalent distributed load per linear foot is then compared to the corresponding maximum permit weight specified by the *Texas Administrative Code* shown in column 2 of Table 2. A vehicle that exceeds the values in column 2 of Table 2 is denied a permit. The vehicle may then be analyzed by the bridge division of TxDOT. An engineer in the bridge division must do an analysis of each bridge on the route to be traveled to determine if a permit can be issued.

Column 3 of Table 2 was calculated by multiplying the distributed load of column 2 by the wheelbase plus four feet. Therefore, column 3 is the summation of the axle loads for a the corresponding wheelbase (i.e. T from Eqns 1 and 2). Typically a weight in pounds or kips is easier to conceptualize than a distributed load in pounds per foot or kips per foot. Therefore, some subsequent calculations will be compared to column 3 when necessary.

2.2 Example Permit Calculation

To better understand the current Texas permit rules, the calculations involved in issuing a permit for an overweight vehicle will be done. An example configuration of an overweight truck is shown in Fig. 2.

Wheelbase (feet)	Distributed Load(lb/ft)	Weight (kips)	Wheelbase (feet)	Distributed Load(lb/ft)	Weight (kips)
			42	3354	154.284
4	7250	58.000	43	3333	156.651
5	6345	57.105	44	3313	159.024
6	5947	59.470	45	3293	161.357
7	5698	62.678	46	3274	163.700
8	5500	66.000	47	3255	166.005
9	5326	69.238	48	3236	168.272
10	5169	72.366	49	3218	170.554
11	5027	75.405	50	3200	172.800
12	4898	78.368	51	3182	175.010
13	4781	81.277	52	3164	177.184
14	4675	84.150	53	3146	179.322
15	4579	87.001	54	3128	181.424
16	4492	89.840	55	3111	183.549
17	4413	92.673	56	3094	185.640
18	4340	95.480	57	3077	187.697
19	4272	98.256	58	3061	189.782
20	4208	100.992	59	3045	191.835
21	4146	103.650	60	3030	193.920
22	4087	106.262	61	3015	195.975
23	4030	108.810	62	3000	198.000
24	3974	111.272	63	2985	199.995
25	3920	113.680	64	2971	202.028
26	3867	116.010	65	2957	204.033
27	3815	118.265	66	2943	206.010
28	3764	120.448	67	2929	207.959
29	3714	122.562	68	2915	209.880
30	3676	124.984	69	2901	211.773
31	3646	127.610	70	2887	213.638
32	3616	130.176	71	2874	215.550
33	3586	132.682	72	2861	217.436
34	3557	135.166	73	2848	219.296
35	3529	137.631	74	2835	221.130
36	3501	140.040	75	2822	222.938
37	3474	142.434	76	2809	224.720
38	3448	144.816	77	2796	226.476
39	3423	147.189	78	2783	228.206
40	3399	149.556	79	2771	229.993
41	3376	151.920	80	2759	231.756

Table 2: TxDOT Maximum Permit Weight Table.

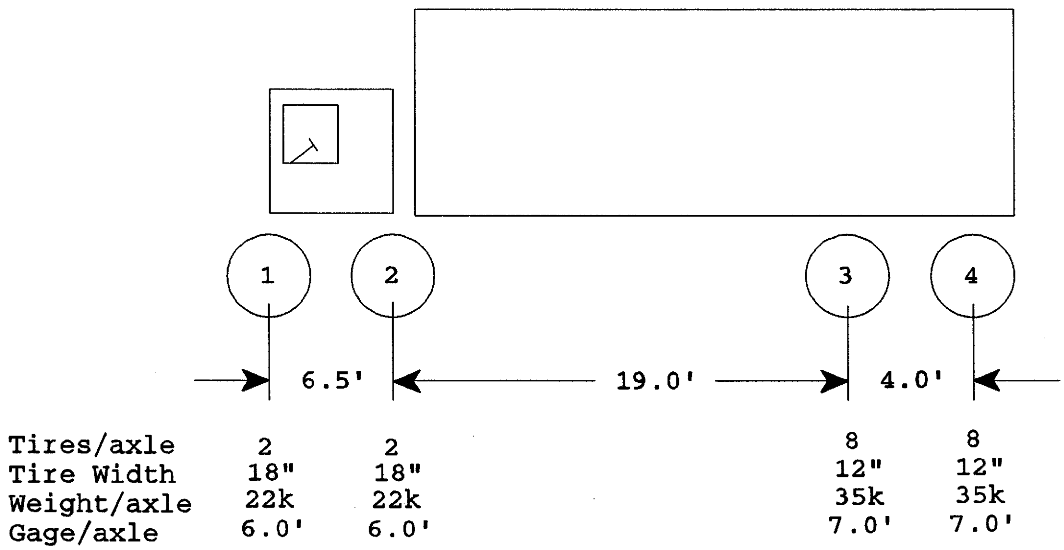


Figure 2: Overweight vehicle configuration example.

The two front axles (1 and 2) fall within the single axle group restriction of 25,000 lbs. Also, each of the front axles has a total tire width of 36 inches. Dividing 22,000 lbs. by 36 inches results in a load of 611 lb/in. of tire width for each front axle. Hence, the two front axles also meet the individual axle restriction of 850 lb/in. of tire width.

However, the two rear axles (3 and 4) of 35,000 lbs. each clearly violate the two axle group restriction of 45,000 lbs. By using Eqns. 2 and 3, an equivalent distributed load per foot can be calculated that may fall within the restrictions of Table 2. Due to the configuration of the two rear axles the reduction factor S, for number of tires and R, for gage may be used. Since each axle has eight tires, S = 0.96. Substituting a gage of seven feet into Eqn. 3 results in a gage reduction factor of R = 0.929. The wheelbase for the rear axle group as seen in Fig. 2 is WB = 4 ft. The summation of the axle loads for the rear axle group is T = 70 kips.

Substitution of R, S, T and WB into Eqn. 2 results in an equivalent distributed load of W = 7.804 k/ft. According to Table 1 the maximum allowable distributed for an axle group with a four foot wheelbase is 7.250 k/ft.

A similar calculation would be done to determine the equivalent distributed load for axle group 1, 2, 3, and 4. The values for R, S, and T for axles 3 and 4 would remain the same. For axles 1 and 2, $R_{1,2}$ = 1.0, $S_{1,2}$ = 1.0, and $T_{1,2}$ = 22 kips. Since axles 1 through 4 are under consideration the wheelbase is WB = 29.5 ft. Substituting these values into Eqn. 2 results in an equivalent distributed load of 3.177 k/ft.

Similar analysis for axle groups 1,2 and 3; 2 and 3; and 2, 3, and 4 would result in equivalent distributed loads as summarized in Table 3.

Axles	Wheelbase Length (feet)	Equivalent Distributed Load (k/ft.)	TxDOT Restriction (from Table 2)	Issue Permit?
1,2,3	25.5	2.55	3.894	Yes
1,2,3,4	29.5	3.177	3.695	Yes
2,3	19	2.314	4.272	Yes
2,3,4	23	3.127	4.03	Yes
3,4	4	7.804	7.25	No

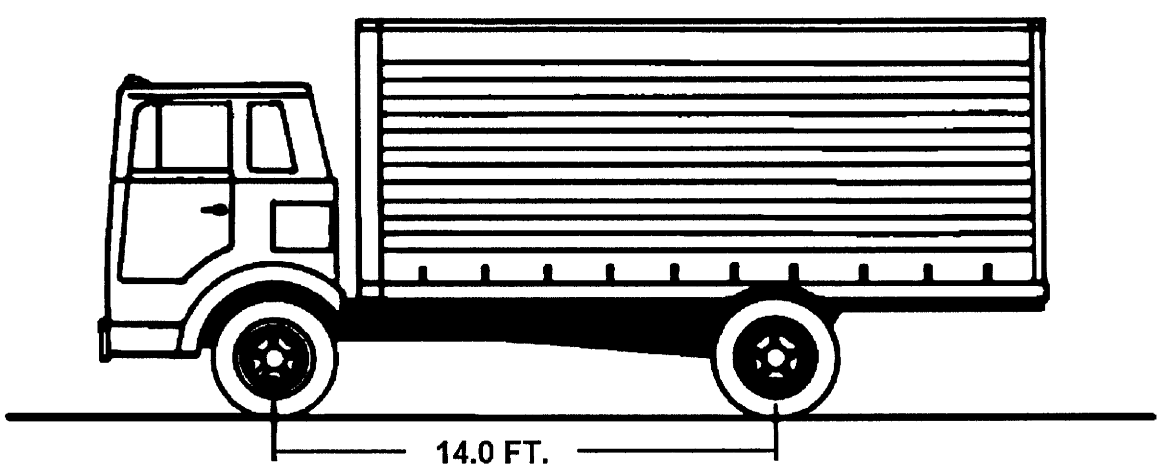
Table 3: Summary of distributed loads for example vehicle.

Since the example vehicle violates the distributed load restrictions for axles 3 and 4 a permit would be denied. The bridge division of TxDOT would then have to perform a structural analysis of the bridges along the vehicle route to determine if a permit could still be issued.

3. PROCEDURE OVERVIEW

The problem of determining an overweight permit formula was approached by first determining estimates of the dead-load for different span lengths of lightweight, H15, simple span bridges. This will allow dead loads for the entire bridge width to be related to bridges in Texas. Later in the analysis the factor of safety that increased the actual dead-load for design purposes will be changed such that the design dead-load will be decreased. The decrease in the design dead-load will then be added to the allowable live-load. This is done so that allowable live loads for overweight permits may be maximized.

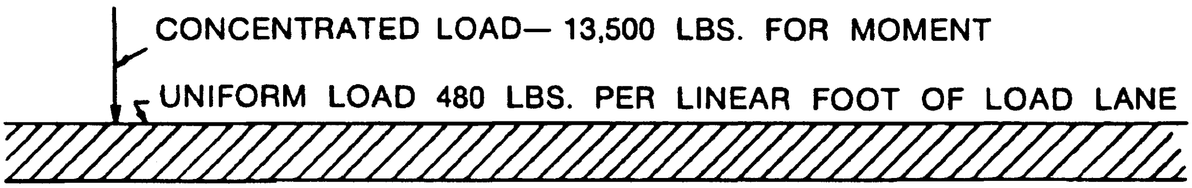
An H15 bridge is designed using an H15 truck as the live load. The AASHTO *Standard Specifications for Highway Bridges* specifies two different types of loading for an H15 truck shown in Figs. 3 and 4. The first AASHTO loading is the standard truck loading is shown in Fig. 3.



H 15-44 6,000 LBS. 24,000 LBS.

Figure 3: AASHTO H20 truck loading.

In addition, AASHTO also requires a standard lane loading shown in Fig. 4. The distributed load is placed along the length of the bridge. The concentrated load is placed on the bridge in such a position so as to cause a maximum moment. For a simple span this load is positioned at the midspan of the bridge.



H15-44 LOADING

Figure 4: AASHTO H15 lane loading.

According to the AASHTO *Standard Specifications for Highway Bridges,* the controlling live load for a given span length is the truck or lane loading that causes a maximum moment.

Seelye (1957) provides design specifications for typical bridge spans. Noel (1985) presents data that was used in relating the information from Seelye to Texas bridges. The data from Seelye (1957) was used to determine dead-load to live-load-plus-impact moment ratios for concrete slab and steel stringer bridges of various span lengths. An overload of two H15 trucks was placed on the bridge so as to cause a maximum moment. Using this overloading condition in combination with the moment ratios allowed the determination of an allowable live load moment based on operational stress as defined by AASHTO.

When designing bridges with the Allowable Stress Design (ASD) method as specified in the AASHTO *Manual for Maintenance Inspection of Bridges* an "inventory" stress of $0.55F_y$ is used. "Operational" stress, which was used in the following analysis, is defined as $0.75F_y$. Operating stresses were used because according to the AASHTO *Manual for Maintenance Inspection of Bridges*:

"The Operating Rating will result in the absolute maximum permissible load level to which the structure may be subjected. Special permits for heavier than normal vehicles may be issued only if such loads are distributed so as not to exceed the structural capacity determined by the Operating Rating."

Once an allowable moment based on operational stress was known, distributed loads were determined for various wheelbase lengths. This was done by placing a distributed load of a particular length (i.e. wheelbase length), on the bridge span as shown in Fig. 5.

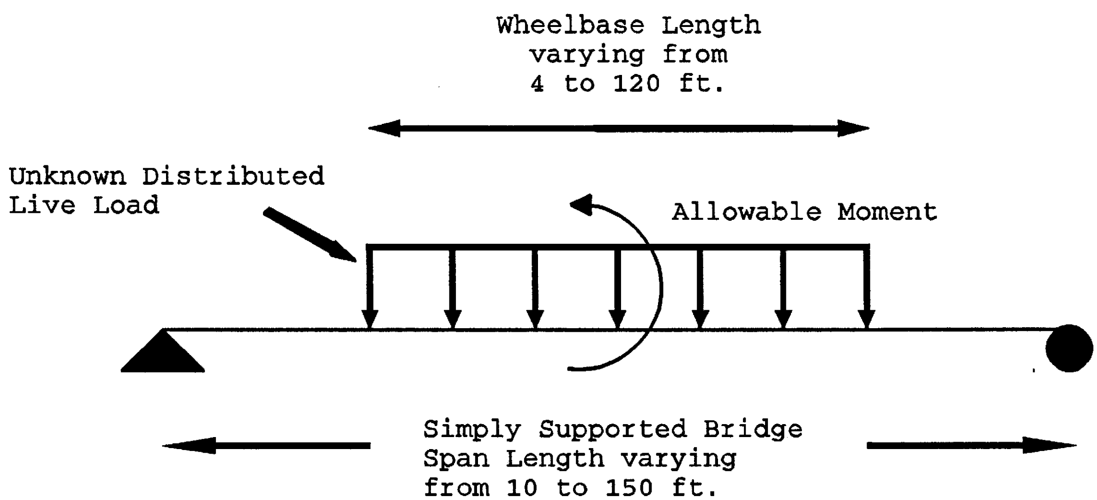


Figure 5: Unknown distributed load illustration.

For each bridge span length there was an allowable moment. The distributed load, of unknown magnitude, was positioned about the center of the bridge span. The magnitude of the distributed load necessary to cause the allowable moment on a simply supported span was then calculated. This was repeated for wheelbases from 4 to 120 ft. and bridge span lengths from 10 to 150 ft.

4. DETERMINATION OF MOMENT RATIOS

4.1 Bridge Design Specifications

The two types of bridges studied were reinforced concrete slab and steel I-beam bridges. As stated in the introduction, older bridge designs are more prone to damage then newer bridges utilizing the latest design techniques Therefore, design specifications and data for older bridge designs was needed. Seelye (1957) was an excellent source of data for typical older bridge designs. Seelye (1957) provided design specifications for span lengths up to 80 ft. for these two types of bridges. The two typical bridge cross sections from Seelye (1957) are shown in Figs. 6 and 7.

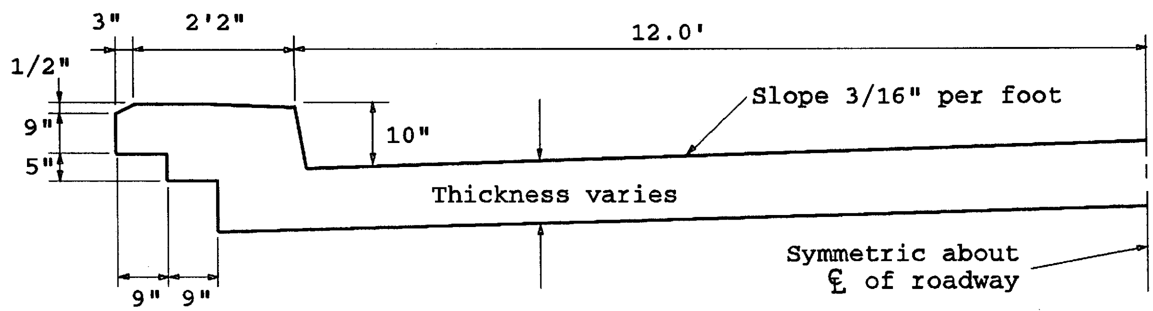


Figure 6: Typical H15 simple span reinforced concrete bridge.

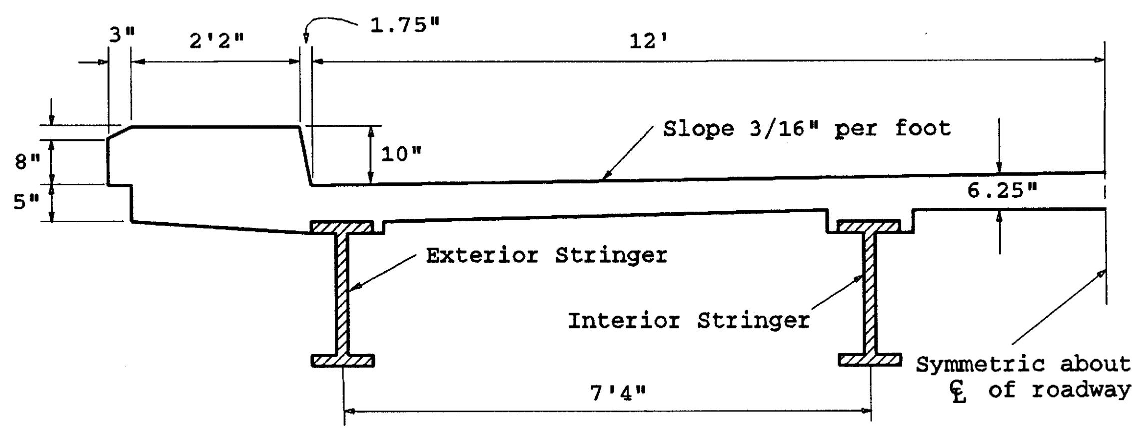


Figure 7: Typical H15 simple span steel I-beam bridge.

Table 4 shows slab thicknesses for various H15 reinforced concrete bridges. Table 5 shows the types of steel I-beams used for different span lengths. The density of concrete was assumed to be 0.15 k/ft^3. As seen in Figs. 6 and 7 the width of the clear roadway was 24 ft. This is about the minimum width that may commonly be found in bridge design.

Span Length (feet)	Thickness (in.)
20	10.5
25	12.5
30	14.5
35	17.5

Table 4: Reinforced concrete slab thicknesses.

Span Length (feet)	Exterior I-beams	Interior I-beams
20	18 WF 50	18 WF 60
25	21 WF 62	21 WF 68
30	24 WF 76	24 WF 76
35	24 WF 84	24 WF 94
40	27 WF 94	27 WF 102
45	30 WF 108	30 WF 116
50	33 WF 130	33 WF 130
60	36 WF 150	36 WF 160
70	33 WF 220	33 WF 220
80	36 WF 260	36 WF 260

Table 5: Steel I-beams for various span lengths.

4.2 Calculation of Moment Ratios

Dead load moments for a single line of wheels were determined based on AASHTO *Standard Specifications for Highway Bridges*. Distribution factors were used to determine the width of the slab over which the load of a single line of wheels was distributed as shown in Fig. 8.

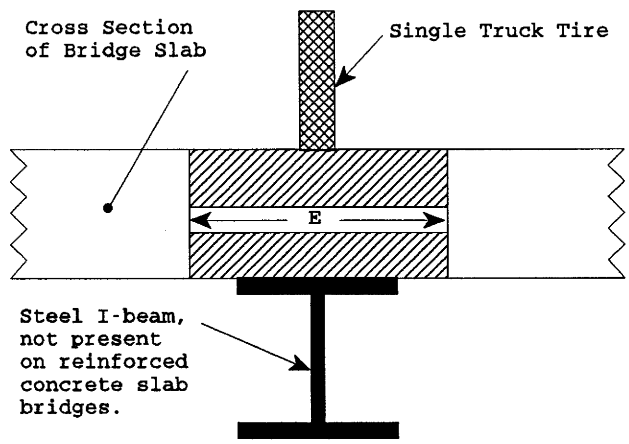


Figure 8: Illustration of Distribution Factor E.

The distribution factor for a reinforced concrete slab is given in the AASHTO specifications as:

$$E = 4 + 0.06L \leq 7.0 \tag{4}$$

where

L = the bridge span length (feet).

In the case of steel I-beam bridges the distribution factor is given as:

$$E = \frac{S}{5.5} \tag{5}$$

where

S = the center-to-center distance between the steel I-beams (feet).

Using these distribution factors the width of the slab needed for determining the dead load was calculated. Once the slab width was known, various dead loads were found for different span lengths. Each dead load was uniformly distributed over the bridge length. This distributed dead load was then used to determine the maximum dead load moment, which occurred at the center of the bridge span.

For example, the distribution factor for a 30 ft. concrete slab bridge would be 5.8 ft. Multiplying this by a slab thickness of 14.5 inches from Table 4 results in a cross sectional area of 7.0 ft^2. With a concrete density of $0.15 k/ft^3$ the distributed dead load is 1.05k/ft. as seen in Table 6. The calculations for a steel stringer bridge would be similar except the distribution factor would be determined using Eqn. 5 to determine the cross sectional area of concrete. Also the weight of a steel I-beam would be added to the dead load due to the concrete. The results of these calculations are summarized in Table 6. For certain span lengths of concrete slab and steel I-beam bridges no values appear in Table 6. Typically for shorter span lengths concrete slab bridges are easier and cheaper to build than steel I-beam bridges. Steel I-beam bridges have the same advantages over concrete slab bridges for longer span lengths.

Span Length (feet)	Distributed Dead Load (k/ft.)	
	Concrete Slab Bridges	Steel I-beam Bridges
10	0.486	
15	0.573	
20	0.683	0.466
25	0.859	0.497
30	1.05	0.529
35	1.33	0.571
40	1.66	0.602
45		0.640
50		0.677
55		0.692
60		0.707
70		0.767
80		0.807

TABLE 6. Summary of Distributed Dead Loads.

The equation for determining the maximum moment due to a distributed load and a simply supported span is shown in Eqn. 6:

$$M = \frac{wL^2}{8} \tag{6}$$

where

M	=	the moment (k-feet),
w	=	total distributed dead load (kips/ft.),
L	=	bridge span length (feet).

Substituting w = 1.05 k/ft. and L = 30 ft. results in a dead load moment of 118 k·ft. The live-load-plus-impact for a single line of wheels was determined according to the AASHTO

Standard Specifications for Highway Bridges. The basic live load was found by referring to Appendix A of the AASHTO specifications for H15 loading shown in Table 6. The moment in Table 6 was divided by 2, since only a single line of wheels was being used.

Bridge Span Length	Maximum Moment (k.ft.	Bridge Span Length	Maximum Moment (k.ft.	Bridge Span Length	Maximum Moment (k.ft.
1	6.0	27	162.7	64	461.8
2	12.0	28	170.1	66	484.1
3	18.0	29	177.5	68	506.9
4	24.0	30	185.0	70	530.3
5	30.0	31	192.4	75	590.6
6	36.0	32	199.8	80	654.0
7	42.0	33	207.3	85	720.4
8	48.0	34	214.7	90	789.8
9	54.0	35	222.2	95	862.1
10	60.0	36	229.6	100	937.5
11	66.0	37	237.1	110	1097.3
12	72.0	38	244.5	120	1269.0
13	78.0	39	252.0	130	1452.8
14	84.0	40	259.5	140	1648.5
15	90.0	41	266.9	150	1856.3
16	96.0	42	274.4	160	2076.0
17	102.0	44	289.3	170	2307.8
18	108.0	46	304.3	180	2551.5
19	114.0	48	319.2	190	2807.3
20	120.0	50	334.2	200	3075.0
21	126.0	52	349.1	220	3646.5
22	132.0	54	364.1	240	4266.0
23	138.0	56	379.1	260	4933.5
24	144.0	58	397.6	280	5649.0
25	150.0	60	418.5	300	6412.5
26	156.0	62	439.9		

Table 7: Table of Moments for H15 Truck Loadings on Simple Span Bridges from Appendix A of AASHTO *Standard Specifications for Highway Bridges*.

The load due to impact was found by multiplying the live load by an impact factor, I which is defined as:

$$I = \frac{50}{L + 125} \leq 0.3 \qquad (7)$$

where

L = bridge span length (feet).

For a 30 ft. bridge span length the live load moment from Table 7 is 185 k-ft. Dividing this value by 2 results in a live load of 92.5 k-ft. for a single line of wheels. For a 30 ft. span the impact factor from Eqn. 7 is 0.3. Multiplying the live load of 92.5 k-ft. by the impact factor results in a moment due to impact of 27.8 k-ft. Adding the live and impact load moments results in a live load plus impact of 120 k-ft. for a single line of wheels. The dead-load to live-load-plus-impact moment ratios for a single line of wheels were then computed. In the case of a 30 ft. concrete slab bridge this would be 0.983.

Dead-load to live-load-plus-impact moment ratios for the entire bridge width were also determined. This was done since there was assumed to be an overload of two H15 trucks on the bridge. Since the two trucks occupied both lanes of the bridge the entire bridge width was used in determining the dead load. The trucks were placed on the bridge to cause a maximum moment. Since these moments were for the entire bridge width, the dead and live loads were not multiplied by the distribution factors previously used to determine the moments for a single line of wheels. The live load was found by multiplying the values in Table 6 by a factor of 2 since two H15 trucks were on the bridge. The impact factor I, was used as before. For a 30 ft. span the live load for two H15 trucks would be 370 k-ft. Using the impact factor of 0.3, previously determined the impact loading would be 111 k-ft. The resulting live load plus impact for an overload of H15 trucks is then 481 k-ft.

The dead load for the entire width of a 30 ft. concrete slab bridge can be determined using the dimensions in Fig. 6 and Table 3. Using these dimensions the cross sectional area is approximately 34.2 ft^2. With a concrete density of 0.15 k/ft^3 the distributed dead load is 5.13 k/ft. Substituting w = 5.13 k/ft. and L = 30 ft. into Eqn.6 results in a dead load moment of 577 k-ft. Dividing the dead load moment of 577 k-ft. by the live load moment of 481 k-ft. results in a moment ratio of 1.20.

Dead-load to live-load-plus-impact moment ratios for the entire bridge width with an overload of two H15 trucks were related to Texas bridges in the following manner. The values from Noel (1985) for Texas bridges, shown in Table 8, were divided by the values determined from Seelye (1957) for a single line of wheels and then multiplied by the values from Seelye (1957) for the entire bridge width. For example, using the values determined above for a 30 ft. concrete slab bridge this would be 1.085/0.983*1.20 which is equal to 1.32. This point is plotted in Fig. 9 for a 30 ft. reinforced concrete slab bridge.

Bridge Span Length (feet)	Reinforced Concrete Slab Bridges	Steel I-Beam Bridges
10	0.147	-
15	0.294	-
20	0.500	-
25	0.771	0.482
30	1.085	0.464
35	1.455	0.560
40	2.087	0.651
45	-	0.741
50	-	0.827
55	-	0.921
60	-	1.006
65	-	1.068
70	-	1.154
80	-	1.324

Table 8: Dead-Load to Live-Load-Plus-Impact Moment Ratios for Texas Bridges from Noel (1985).

The moment ratios determined for reinforced concrete slab and steel I-beam bridges were calculated for spans up to 100 ft. Design parameters were available for spans extending to 80 ft. The moment ratios were then extrapolated up to 100 ft. This extrapolation was done using a scientific graphing computer program, SigmaPlot, which utilizes the Marquardt-Levenberg algorithm as a curve fitter. Since it was desired to develop weight restrictions for wheelbases up to 120 ft. additional dead-load to live-load-plus-impact moment ratio data was needed. Sears (1984) of the Federal Highway Administration provided data on moment ratios extending up to 150 ft.

The results of the calculations for reinforced concrete slabs and steel I-beam bridges along with the FHWA data are plotted in Fig. 9 versus span length. In the subsequent analysis these moment ratios will be used to decrease the design dead load. The decrease in the design dead load will then be added to the allowable live load. A bridge with a high moment ratio would transfer more weight to the live load than a bridge with a lower moment ratio. Therefore, to protect light bridge spans the minimum ratios from Fig. 9 were used.

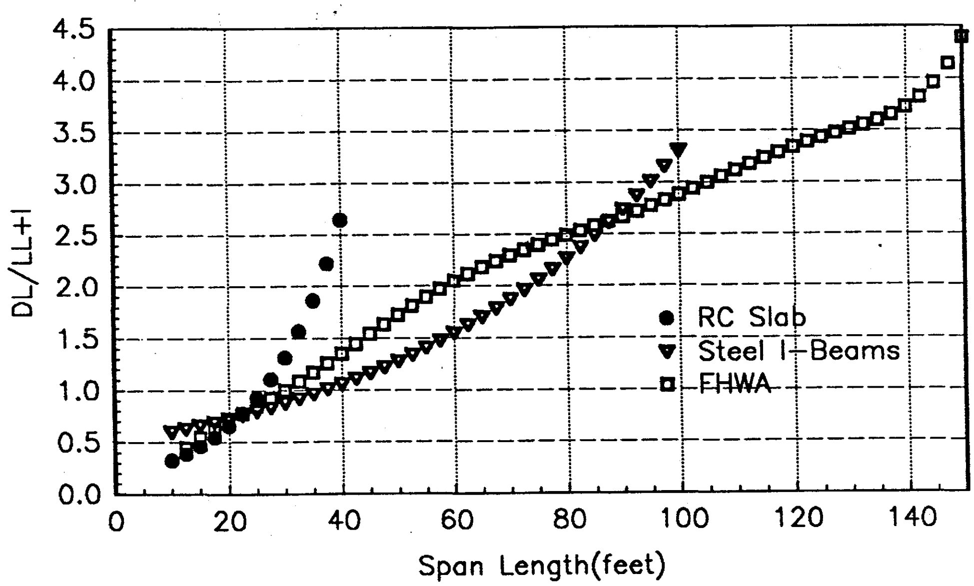


Figure 9: Dead-load to Live-load-plus-impact Moment ratios for H15 Bridge Spans.

5 DETERMINATION OF GROUP WEIGHT BASED ON A GAGE OF 16 FT.

5.1 Development of Allowable Moments

For various span lengths from 10 to 150 ft. it was desired to determine an allowable live load "operational moment." This moment was based on the moment ratios previously determined using operational limits rather than inventory limits. As stated previously, according to the AASHTO *Manual for Maintenance Inspection of Bridges*, a factor of 0.55 is used to obtain inventory level allowable stresses. A factor of 0.75 is used to obtain operating stress values. While using operational moments will not cause failure, the loads allowed will do more long term damage to the bridge than inventory values. Therefore, the lifetime of a bridge will be shortened if repeated loadings based on operational values pass over it.

It was assumed that most bridges in use were designed using inventory values, and a factor of 0.55 was used in the design. To determine the live load "allowable moment", based on operational values the total dead load and live load moment was divided by 0.55 and then multiplied by 0.75. In determining allowable moments, it was assumed that there was an overload of two H15 trucks at the center of the bridge. The vehicle speed was assumed to be of a magnitude such that full impact according to the AASHTO specifications was generated. The live load allowable moment, was determined by developing the following formula:

$$\begin{matrix}\text{Allowable}\\ \text{Moment}\end{matrix} = \left[\frac{0.75}{0.55}\right]\frac{[(DLIMR \cdot LI) + LI] - [DLIMR \cdot LI]}{1 + I} \tag{8}$$

where

DLIMR	=	minimum dead-load to live-load-plus-impact moment ratio from Fig. 9,
LI	=	live-load-plus-impact moment for two H15 trucks,
I	=	AASHTO impact factor given in Eqn. 7.

For example, the minimum dead-load to live-load-plus-impact moment ratio for a 120 ft. bridge span is about 3.4. The live load plus impact for two H15 trucks on a 120 ft. bridge span is 3060 k-ft. Substituting into Eqn. 8 results in an allowable moment of 6630 k-ft. This was done for spans from 10 to 150 ft. for the minimum moment ratios from Fig. 9.

5.2 Calculation of Distributed Loads and Group Weight

Having determined the allowable moment, various distributed loads for different span lengths and wheel bases were calculated. Since the allowable moments were determined using two H15 trucks, the gage or width of the distributed load was taken to be 16 ft. This is the minimum width that two H15 trucks can occupy according to AASHTO, where each truck is 6 ft. wide with 4 ft. separating them. Wheelbases from 4 to 120 ft. were considered. As discussed in the Procedure Overview, for each wheelbase a distributed load (kip/ft.) was determined that induced a moment equal to the allowable moment. The distributed loads were placed on the bridge at the center of the span as shown in Fig 5. The unknown distributed load was determined by deriving the following equation:

$$w = \frac{8M}{WB(2L - WB)} \tag{9}$$

where

w	=	unknown distributed load (k/ft.),
M	=	allowable moment (k-ft.),
L	=	bridge span length (feet),
WB	=	wheelbase (feet).

This was done for span lengths from 10 to 150 ft. For example, the allowable moment previously determined for a bridge span of L = 120 ft. is M = 6630 k-ft. Substituting these values and a wheelbase of WB = 70 ft. into Eqn. 9 results in a distributed load of 5.6 k/ft. This calculation was repeated for each wheelbase length for span lengths and allowable moments from 10 to 150 ft. For each wheelbase a minimum distributed load occurred, as illustrated in Fig. 10.

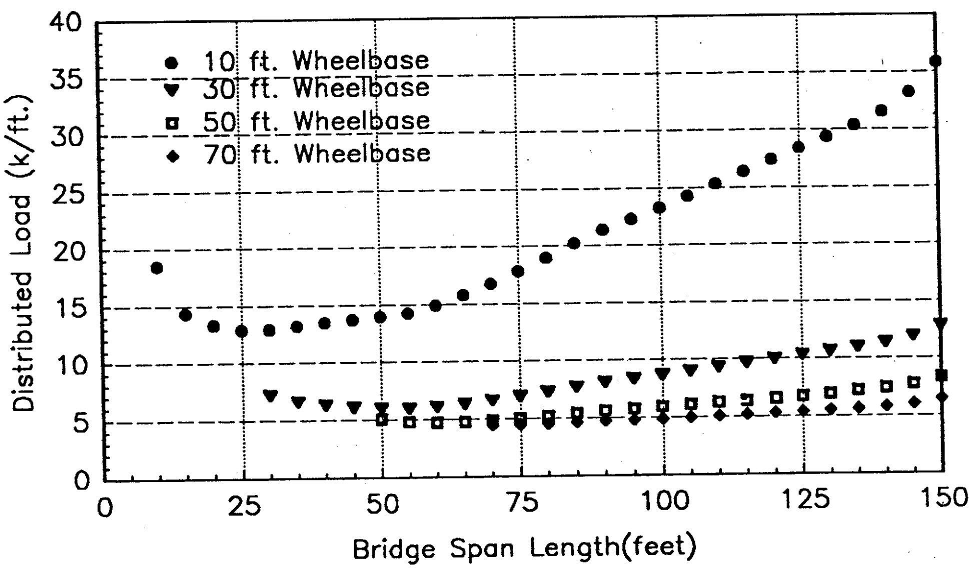


Figure 10: Illustration of Minimum Distributed Load Determination.

These minimum distributed loads along with the corresponding group weight are summarized below in Table 8. For example, in looking at Fig. 10 the minimum distributed load for the 10 ft. wheelbase is about 12.9 k/ft. This number corresponds to 12.8880 k/ft. for a 10 ft. wheelbase shown in Table 9.

Wheelbase (feet)	Minimum Distributed Load (kips/ft)	Group Weight (kips)	Wheelbase (feet)	Minimum Distributed Load (kips/ft)	Group Weight (kips)
			62	4.4911	278.45
4	2	110.26	64	4.4767	286.51
6	19.5272	117.16	66	4.4584	294.26
8	15.3428	122.74	68	4.4475	302.43
10	12.8880	128.88	70	4.4439	311.07
12	11.2282	134.74	72	4.4392	319.62
14	10.0426	140.60	74	4.4329	328.03
16	9.1611	146.58	76	4.4550	338.58
18	8.4512	152.12	78	4.4466	346.84
20	7.8395	156.79	80	4.4438	355.51
22	7.3230	161.11	82	4.4830	367.61
24	6.8894	165.35	84	4.4781	376.16
26	6.5192	169.50	86	4.4859	385.79
28	6.2012	173.63	88	4.4699	393.35
30	5.9325	177.97	90	4.4580	401.22
32	5.7043	182.54	92	4.4470	409.12
34	5.5100	187.34	94	4.4345	416.84
36	5.3446	192.40	96	4.4256	424.86
38	5.2039	197.75	98	4.4203	433.19
40	5.0850	203.40	100	4.4125	441.25
42	4.9853	209.38	102	4.4061	449.42
44	4.8969	215.46	104	4.4029	457.90
46	4.8106	221.29	106	4.3993	466.32
48	4.7382	227.43	108	4.3949	474.65
50	4.6786	233.93	110	4.3935	483.28
52	4.6310	240.81	112	4.3894	491.61
54	4.5946	248.11	114	4.3867	500.09
56	4.5690	255.86	116	4.3816	508.27
58	4.5341	262.98	118	4.3757	516.33
60	4.5082	270.49	120	4.3689	524.27

Table 9: Summary of minimum distributed loads.

The minimum values from Fig. 10 were used so that a truck with any wheelbase length up to 120 ft. could safely pass over any bridge span up to 150 ft. The group weights were found by multiplying the distributed load by the wheelbase length. The resulting group weights are shown in Table 8 and are graphed versus wheelbase in Fig. 11. Group weights were plotted rather than distributed loads since a weight of 146 kips is easier to conceive of than a distributed load of 9.2 kips/ft. especially with regard to trucks.

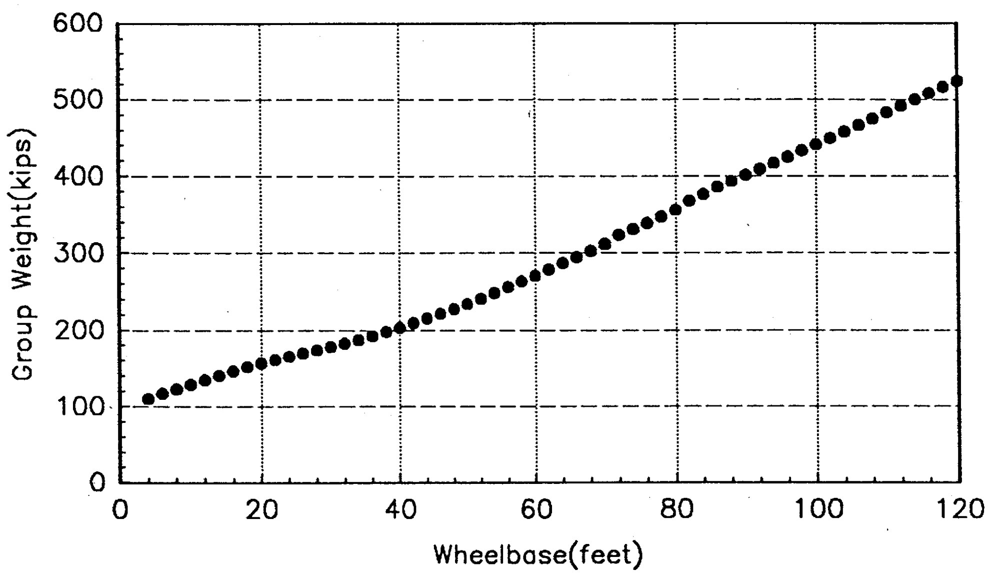


Figure 11: Calculated Group Weight versus Wheelbase for a Gage of 16 ft.

However, these values were determined with allowable moments that were based on an overload of two H15 trucks. Two H15 trucks side by side on a bridge effectively have a gage of 16 ft. The values that TxDOT uses for permitting loads shown in Table 2 are based on a truck with a 6 ft. gage.

6. DETERMINATION OF GROUP WEIGHT AND GAGE REDUCTION FORMULA BASED ON A 6 FT. GAGE

It was necessary to adjust the calculated values to be based on a truck with a 6 ft. gage so they could be compared with those values that TxDOT currently uses. It was also of interest to simultaneously to determine a formula for the reduction factor as a function of gage. This reduction factor formula for gage could then be compared to that currently used by TxDOT, shown in Eqn. 3. As seen in Fig. 12, by decreasing the gage, the intensity of the distributed load increases, but the resulting weight decreases. Since the distributed load is more highly concentrated the resultant weight must decrease so that the stress remains constant.

Since a 16 ft. gage load is distributed over a greater area than a 6 ft. gage, the overall load is greater for a 16 ft. gage than a 6 ft. gage. The formula for the reduction factor was found by using the group weights for wheelbases with a 16 ft. gage previously determined as a starting point. A finite element analysis program was then used to determine an equivalent load for gages less than 16 ft. and greater than 6 ft. The *SAFE (Slab Analysis by the Finite Element Method)* computer program was utilized in the analysis. *SAFE* is specifically designed to analyze slab type structures utilizing two types of elements. *SAFE* utilizes plate elements for modeling the slab and beam elements for modeling the steel I-beams, as shown in Fig. 13.

6.1 Gage Reduction Factor Formula

The program uses four node plate elements and two node beam elements with three degrees of freedom at each node. The beam elements may exist between any two nodes. The beam element in Fig. 13 with global node numbers 1 and 2 would exist along the length of the plate element with the same global node numbers.

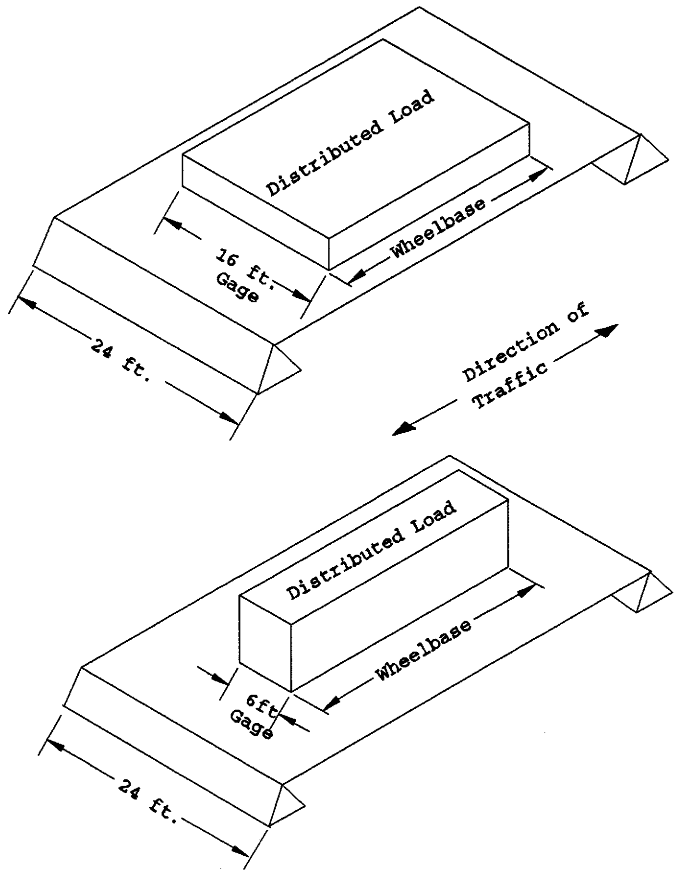


Figure 12: Change in distributed load when gage decreases with constant wheelbase.

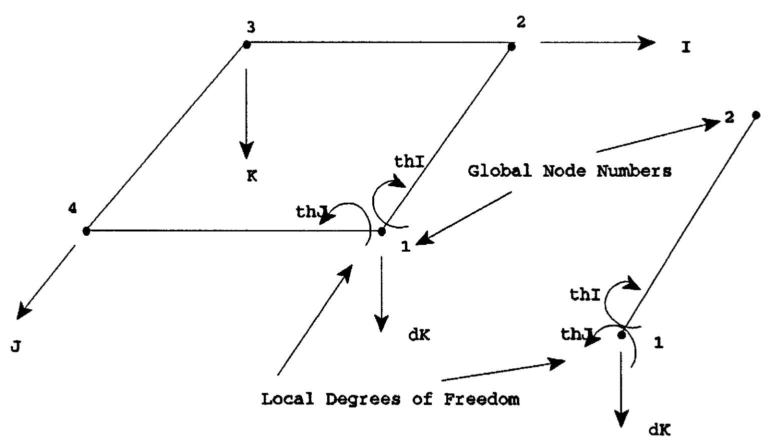


Figure 13: Slab and beam elements used in *SAFE*.

As discussed earlier, the values for the group weights were determined for wheelbases with a 16 ft. gage. The group weight for a particular wheelbase was divided by the gage of 16 ft. and by the wheelbase resulting in a distributed load in pounds per square foot. This distributed load was then put on the finite element model of the bridge. In Fig. 14 a typical mesh that was used is shown with the placement of the distributed load shown.

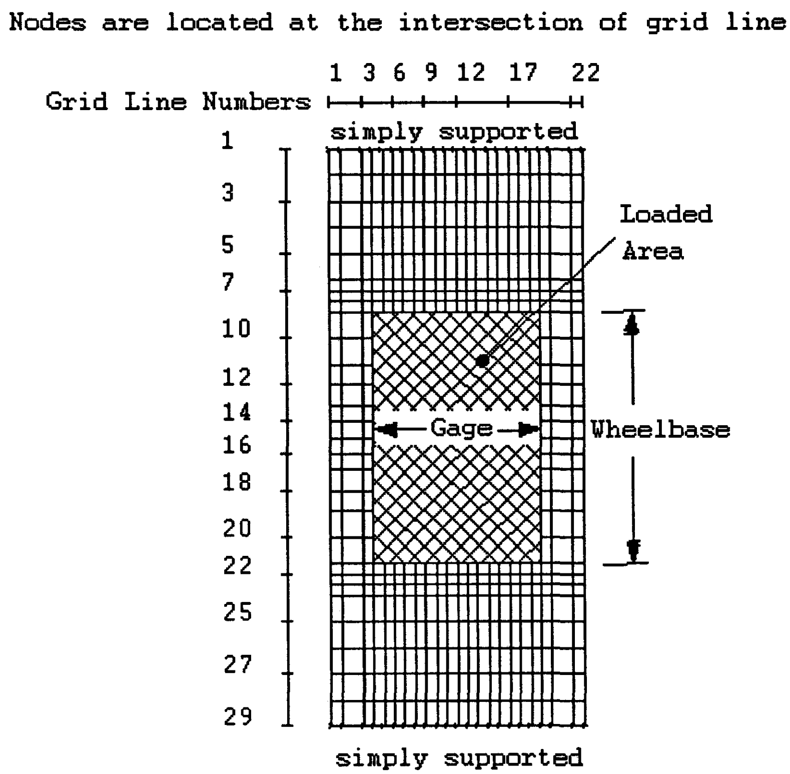


Figure 14: Typical finite element mash for a concrete slab bridge used with *SAFE*.

The wheelbase being examined determined the span length that the load was placed on. This span length used was that span which was associated with the minimum distributed load for a given wheelbase. Anywhere from 308 to 616 elements were used for spans ranging from 15 ft. to 55 ft. respectively. The large number of elements used is mainly due to two reasons. First, because of the variety of loading placed on each span, large numbers of nodal points were needed, which resulted in a large number of elements. In most cases elements that were one foot on a side were used. This was done since the width and length of the distributed loads were varied in one or two foot increments. Secondly, as a result of the large number of elements used, more accurate and reliable data resulted. The program output indicated the resulting moment about the transverse axis on each individual finite element for the entire bridge. Different distributed loads with gages between 6 ft. and 14 ft. in increments of 2 ft. were then used in the program. For each gage it was necessary to make several computer runs to obtain a distributed loading that caused a maximum moment equal to the moment caused by the truck with a 16 ft. gage. Results from the SAFE program then provided the means to develop a formula for a reduction factor for trucks with gages between 6 and 16 ft.

For example, for a wheelbase of 8 ft. and a gage of 16 ft. a load of magnitude 1.0 was placed on the bridge. A load magnitude of 1.0 was used since it was only of interest to determine how much the load changed from one gage to another. By using with 1.0, this simplified the calculation for determining the reduction factor. The magnitude of the resulting moment was 0.0513. To generate the same moment of magnitude 0.0513 for the same wheelbase but with a gage of 14 ft. the magnitude of the load was 0.925. Dividing 0.925 by 1.000 results in a reduction factor of 0.925. This value was then plotted in Fig. 15 for a 8 ft. wheelbase and a 14 ft. gage.

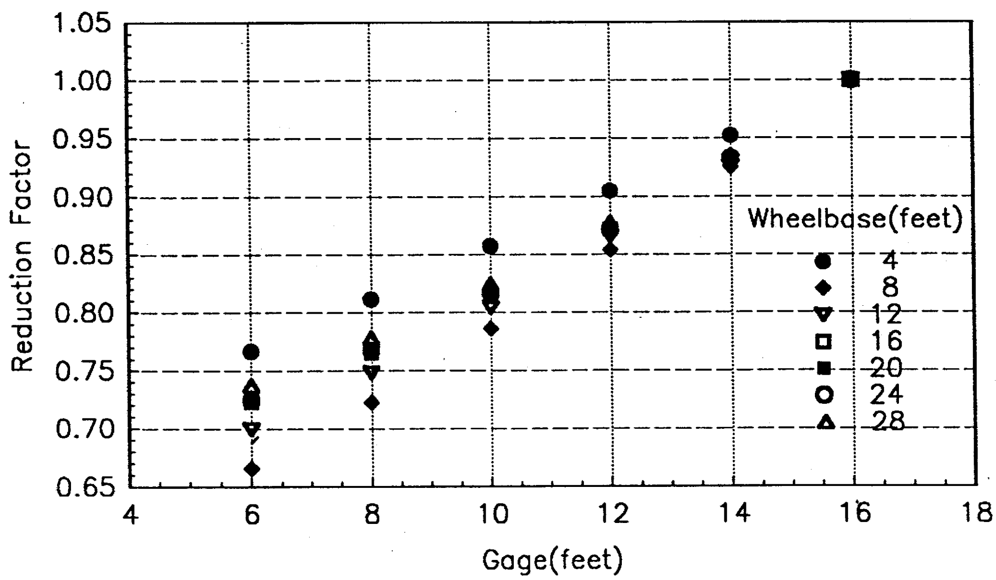


Figure 15: Reduction Factor versus Gage for Wheelbases Ranging from 4 to 28 ft. Normalized for a 16 ft. Gage.

As can be seen in Fig. 15, the formula for gage varied depending on the wheelbase. Only wheelbases from 4 to 28 ft. in increments of 4 ft. were examined. It was apparent that the reduction factor converged for the larger wheelbases. The data for a truck with an 8 ft. wheelbase was used as the basis for a formula for the reduction factor. By using the data for an 8 ft. wheelbase, the resulting loads for a 6 ft. gage were less than what could have been obtained using the data based on another wheelbase. By reducing the loads as much as possible critical bridge span lengths could be protected from excessive loads. This is the case for the 8 ft. wheelbase data as seen in Fig. 15.

The reduction factors in Fig. 15 were based on a truck with a 16 ft. gage having a reduction factor equal to one. The TxDOT formula in Eqn. 3 is such that the reduction factor for a 6 ft. gage is equal to one. The data for a 8 ft. wheelbase had to be normalized so that a truck with a 6 ft. gage would have a reduction factor equal to one. For example, for a wheelbase of 8 ft. and a gage of 14 ft. a load of magnitude 0.925 resulted in a moment of magnitude 0.0513. To generate the same moment of magnitude 0.0513 for the same wheelbase but with a gage of 6 ft. the magnitude of the load was 0.666. Dividing 0.666 by 0.925 results in a reduction factor of 0.720. This value was then plotted in Fig. 16 for a 8 ft. wheelbase and a 14 ft. gage. A graph of the reduction factor formula normalized for a 6 ft. gage is shown in Fig. 16.

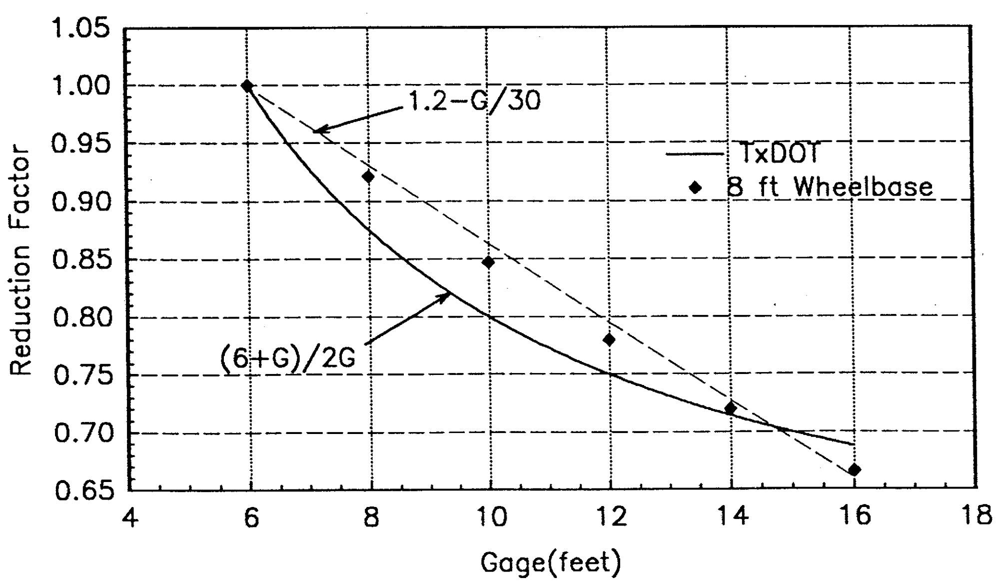


Figure 16: Reduction factor versus gage for 8 ft. Wheelbase normalized for a 6 ft. Gage.

It should be noted that the formula for gage based on the SAFE analysis indicates that the reduction factor formula should be more linear than currently used by the TxDOT as shown in Eqn. 3. The normalized formula for a 6 ft. gage fit to the data for an 8 ft. wheelbase is given as:

$$\text{ReductionFactor} = 1.2 - \frac{G}{30} \qquad (10)$$

where

G = gage (feet).

6.2 Group Weight Based on a 6 ft. Gage

The group weight of a wheelbase with a 6 ft. gage was found by setting G equal to 6 ft. in Eqn. 10 and multiplying the result by the group weight for a given wheelbase with a 16 ft. gage. This was done for wheelbases ranging from 4 to 120 ft. in one foot increments. The results of this are shown in Fig. 17 and tabulated in Table 10. To be consistent with column 3 of the TxDOT vales in Table 2, the group weight was divided by the wheelbase plus four feet to determine the distributed load.

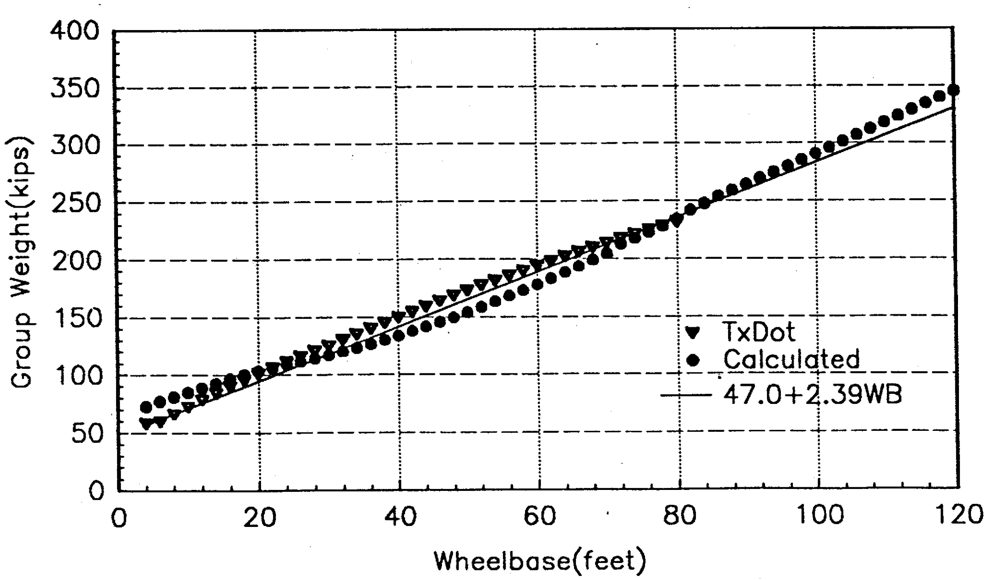


Figure 17: Calculated group weight versus wheelbase compared to TxDOT values for a 6 ft. Gage.

Wheelbase (feet)	Distributed Load (kips/ft)	Group Weight (kips)	Wheelbase (feet)	Distributed Load (kips/ft)	Group Weight (kips)
			62	2.7771	183.29
4	9.0721	72.58	63	2.7764	186.02
5	8.3528	75.17	64	2.7734	188.59
6	7.7122	77.12	65	2.7697	191.11
7	7.1741	78.92	66	2.7670	193.69
8	6.7328	80.79	67	2.7654	196.34
9	6.3665	82.76	68	2.7649	199.07
10	6.0596	84.83	69	2.7654	201.87
11	5.7920	86.88	70	2.7670	204.76
12	5.5431	88.69	71	2.7697	207.73
13	5.3281	90.58	72	2.7683	210.39
14	5.1415	92.55	73	2.7678	213.12
15	4.9791	94.60	74	2.7683	215.92
16	4.8242	96.48	75	2.7697	218.80
17	4.6811	98.30	76	2.7858	222.87
18	4.5515	100.13	77	2.7846	225.55
19	4.4240	101.75	78	2.7842	228.30
20	4.3002	103.21	79	2.7846	231.12
21	4.1880	104.70	80	2.7858	234.01
22	4.0787	106.05	81	2.8148	239.26
23	3.9787	107.42	82	2.8137	241.98
24	3.8871	108.84	83	2.8133	244.76
25	3.8020	110.26	84	2.8137	247.60
26	3.7190	111.57	85	2.8148	250.52
27	3.6424	112.92	86	2.8216	253.94
28	3.5716	114.29	87	2.8177	256.41
29	3.5062	115.70	88	2.8144	258.92
30	3.4456	117.15	89	2.8117	261.48
31	3.3895	118.63	90	2.8096	264.10
32	3.3376	120.15	91	2.8081	266.77
33	3.2896	121.71	92	2.8052	269.30

34	3.2452	123.32	93	2.8023	271.82
35	3.2041	124.96	94	2.7998	274.38
36	3.1662	126.65	95	2.7979	277.00
37	3.1313	128.38	96	2.7966	279.66
38	3.0992	130.17	97	2.7958	282.37
39	3.0698	132.00	98	2.7955	285.14
40	3.0429	133.89	99	2.7945	287.83
41	3.0184	135.83	100	2.7928	290.45
42	2.9962	137.82	101	2.7916	293.11
43	2.9762	139.88	102	2.7908	295.83
44	2.9547	141.83	103	2.7906	298.59
45	2.9330	143.72	104	2.7908	301.41
46	2.9132	145.66	105	2.7916	304.28
47	2.8952	147.66	106	2.7905	306.95
48	2.8790	149.71	107	2.7898	309.67
49	2.8644	151.81	108	2.7896	312.43
50	2.8515	153.98	109	2.7898	315.25
51	2.8403	156.21	110	2.7905	318.11
52	2.8306	158.51	111	2.7903	320.88
53	2.8224	160.88	112	2.7896	323.60
54	2.8158	163.32	113	2.7894	326.36
55	2.8106	165.83	114	2.7896	329.18
56	2.8070	168.42	115	2.7890	331.89
57	2.7989	170.73	116	2.7880	334.56
58	2.7920	173.10	117	2.7875	337.28
59	2.7864	175.54	118	2.7858	339.87
60	2.7820	178.05	119	2.7843	342.46
61	2.7789	180.63	120	2.7830	345.10

Table 10: Calculated maximum permit weight table.

Also shown in Fig. 17 are the group weights that are used by the TxDOT, which extend up to an 80 ft. wheelbase. The actual values that the TxDOT uses are shown in Table 1. Using *SigmaPlot*, a simple linear regression of the calculated values in Fig. 17 results in the following formula for the gross weight of a truck axle group as a function of wheelbase with a 6 ft. gage:

$$GW = 47.0 + 2.39 * WB \tag{11}$$

where

GW = group weight (kips),

WB = wheelbase (feet).

As was done in Table 10, to be consistent with current TxDOT methods the group weight in Eqn. 11 must be divided by the wheelbase plus four feet, resulting in an allowable distributed load which is then compared to the distributed load determined using Eqn. 2.

Current axle weight restrictions for wheelbases less than 22 ft. should be maintained. With this restriction the possibility of significant damage to the pavement is eliminated. For the most part the distributed load values in Table 10 are more restrictive than the values in Table 2 that the TxDOT currently uses. This would seem to indicate that an excessive amount of damage occurs to bridges each time a permitted load passes under the current weight restrictions.

7. DETERMINATION OF A WEIGHT FORMULA AS A FUNCTION OF WHEELBASE AND BRIDGE SPAN LENGTH

The previous weight restrictions were only a function of the wheelbase of a truck. This was done to ensure that the vehicle could safely pass over a bridge of any span length. Because of this, those weight restrictions, in some cases, severely limited the permitted weights. When the route of the vehicle to be permitted is known or to accommodate a heavier load by specifying a particular route, a greater weight may be allowed. This is due to the fact that if a particular route is known or is to be specified, in addition to the wheelbase, the span length of bridges encountered will be known.

It was of interest to develop a formula that was a function of wheelbase and bridge span length. In the future, if TxDOT were to automate permitting trucks with computers, such a formula could be used to assist in specifying a route so the permit weight could be maximized. With this additional information heavier weights could be specified.

7.1 Determination of Weight Formula

By using the Marquardt-Levenberg algorithm in *SigmaPlot* a formula for the allowable distributed load as a function of wheelbase and bridge span length was determined. The formula is based upon a truck with a gage of 6 ft. and axles with less than 8 tires per axle. By using Eqn. 2 an equivalent distributed load is determined for vehicles with different gage widths and with axles having 8 or more tires. The equivalent distributed load is then compared to the allowable distributed load to determine if a permit may be issued.

First, for twenty-three different span lengths ranging from 10 to 120 ft. the form of the equation for determining the allowable distributed load as a function of wheelbase was determined by trial and error. The form of the equation for the allowable distributed load with the least amount of error for all twenty-three different span lengths was of the form:

$$w = \frac{1.0}{(A + B * WBL)\,WBL} \tag{12}$$

where

$\underline{w}_{all}$	=	allowable distributed load (kips/ft.),
A,B	=	variables as functions of span length, L,
WBL	=	WB, wheelbase (feet) when WB ≤ L,
	=	L, span length (feet) when WB > L.

As seen in Figs. 18 and 19, the values for A and B are different for each span length. Hence, it was necessary to determine equations for A and B as functions of span length.

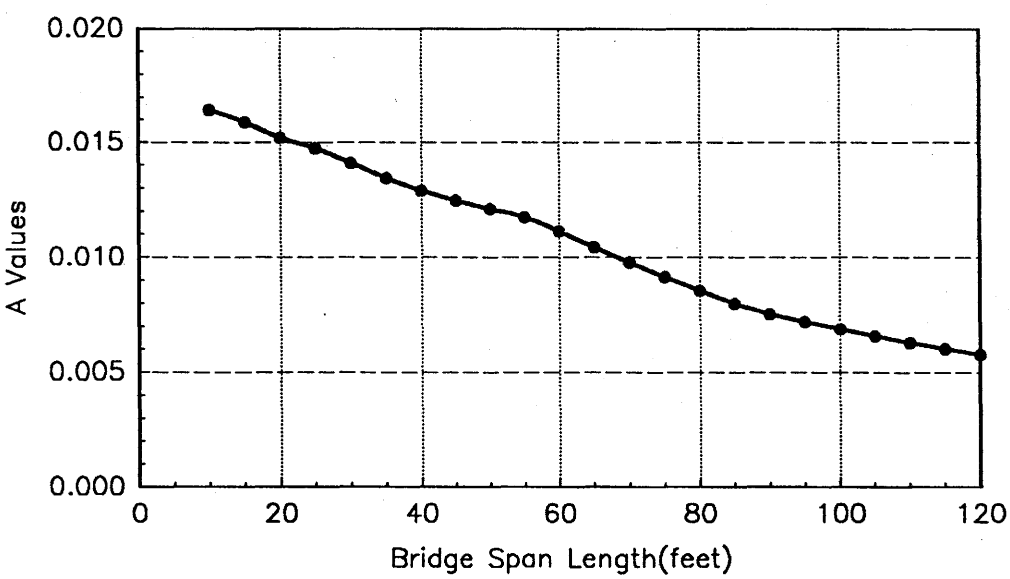


Figure 18: Plot of A values versus bridge span length.

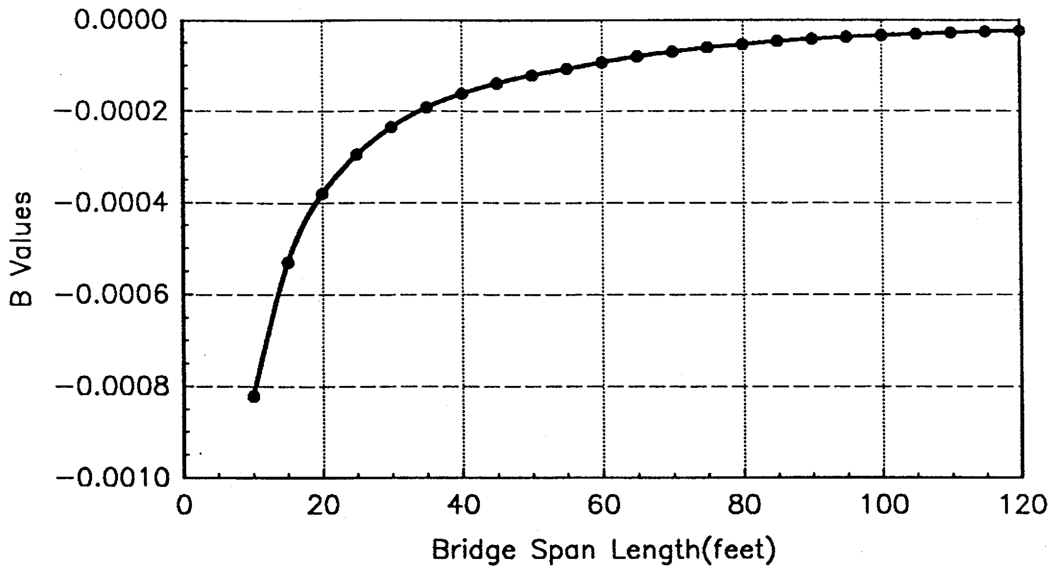


Figure 19: Plot of B values versus bridge span length.

Again, using the Marquardt-Levenberg algorithm in SigmaPlot the following equations were determined for A and B:

$$A = 0.01884 * e^{(-0.009715 * L)} \tag{13}$$

$$B = 4.663 \times 10^{-5} - \frac{0.008302}{L} - \frac{0.004034}{L^2} \tag{14}$$

where

L = bridge span length (feet).

Eqns. 13 and 14 were then substituted into Eqn. 12 resulting in an equation for the allowable distributed load as function of the wheelbase and bridge span length shown in

Eqn. 15:

$$w_{all} = \frac{1.0}{\left[0.01884 * e^{(-0.09715 * L)} + \left(4.663 \times 10^{-5} - \frac{0.008302}{L} - \frac{0.004034}{L^2}\right) * WBL\right] * WBL}$$

(15)

where

w_{all}	=	allowable distributed load (kips/ft.),
L	=	bridge span length (feet),
WBL	=	WB, wheelbase (feet) when WB ≤ L,
	=	L, span length (feet) when WB > L.

Group weight as a function of wheelbase and bridge span length is graphed in Fig. 20. The group weight in Fig. 20 was determined by multiplying the distributed load determined with Eqn. 15 by the wheelbase length as shown in Eqn. 16.

$$GW = w_{all} * WB \tag{16}$$

where

GW	=	gross weight (kips),
w_{all}	=	allowable distributed load from Eqn. 15 (k/ft.),
WB	=	wheelbase (feet).

As can be seen in Fig. 20 when the bridge span length is considered, significantly higher weights may be allowed for various span lengths.

When compared to the actual calculated values, Eqn. 15 provided very accurate values of the allowable distributed load for all spans from 10 to 120 ft. and wheelbases from 4 to 120 ft. Use of Eqn. 15 is much easier than using a table of values, since a table would be large and interpolation would be difficult to do for unusual wheelbase and/or bridge span lengths.

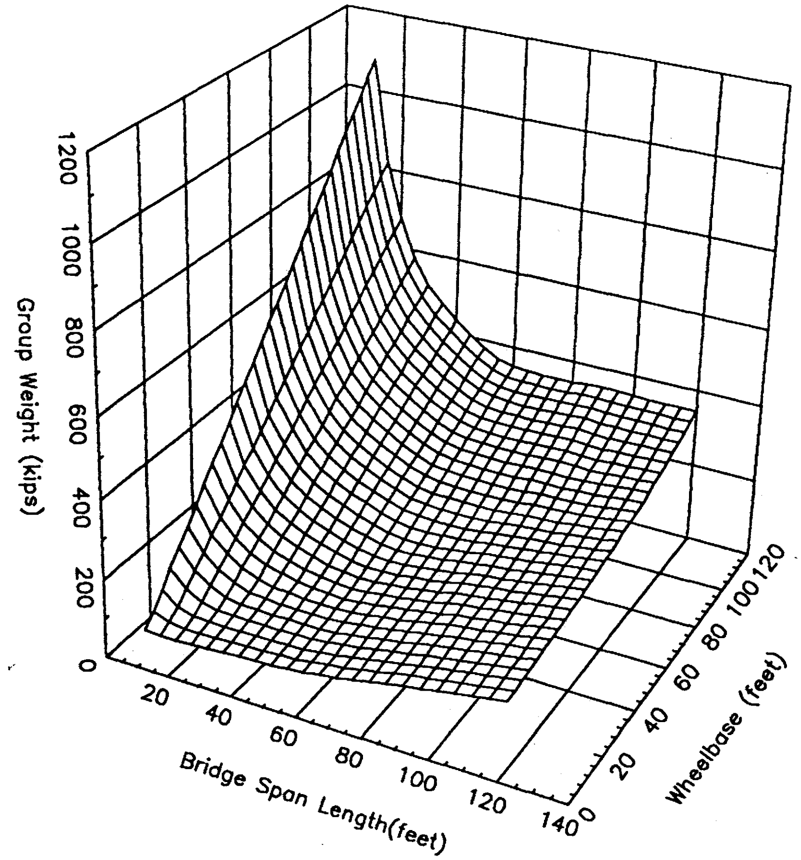


Figure 20: Calculated group weight versus wheelbase and bridge span length for a 6 ft. Gage.

7.2 Example Calculation Utilizing Equation. 15

To better demonstrate how Eqn. 15 should be utilized the truck in Fig. 2 will be reexamined. That truck was denied a permit because axles 3 and 4 exceeded TxDOT restrictions. This example will demonstrate how a permit may be issued utilizing Eqn. 15. Axles 3 and 4 had an equivalent distributed load of 7.804 k/ft. which exceeded the TxDOT restriction of 7.250 k/ft. shown in Table 2. Only axles 3 and 4 need to be examined since that was the only axle group that exceeded TxDOT restrictions. If the other axle groups had not been within TxDOT limits it would have been necessary to determine a higher allowable distributed load for those axle groups too.

It will be assumed that the vehicle in Fig. 2 will pass over a single bridge with a span of 45 ft. Since the bridge span length is known a higher permit weight may be determined using Eqn. 15. The maximum allowable distributed load for the bridge in question will be determined. The bridge span length, L = 45 ft. and wheelbase for axles 3 and 4, WB = 4.0 ft. are substituted into Eqn. 15. This results in a maximum allowable distributed load of, 21.54 k/ft. Since the equivalent distributed load of the axles 3 and 4 is only 7.804 k/ft. the permit would be granted.

Although it is not necessary, the allowable distributed load for any other group of axles could be determined too. For example, axle group 1, 2, 3, and 4 has a wheelbase of 29.5 ft. Substituting this value and a span length of 45 ft. into Eqn. 15 results in an allowable distributed load of 4.215 k/ft. As shown in Table 2 the equivalent distributed load for this axle group is 3.177 k/ft. Since the equivalent distributed load is less than the allowable distributed load the permit would be issued.

8. CONCLUSIONS

8.1 Summary of Results

In summary, new values for the group weight for wheelbases greater than 22 ft. shown in Table 10 and Eqn 11 are proposed. In addition, a new formula for the reduction factor for gages other than 6 ft. shown in Eqn. 10 is proposed. Finally, a formula for determining the allowable distributed load when the wheelbase and bridge span length are known as shown in Eqn. 15 is proposed. These formulas are summarized below.

Group Weight as a function of wheelbase only and based on a 6 ft. gage:

$$GW = 47.0 + 2.39 * WB \qquad (11)$$

where

GW	=	group weight (kips),
WB	=	wheelbase (feet).

As was done in Table 10 the group weight must be divided by the wheelbase plus four feet to determine the corresponding distributed load. If the issuance of permits were to be computerized by TxDOT Eqn. 11 could be utilized. Under current TxDOT permit procedures however, Table 9 should be used.

Reduction Factor:

$$\text{ReductionFactor} = 1.2 - \frac{G}{30} \qquad (10)$$

where

G	=	gage (feet).

The reduction factor formula is used to determine the value of R_i in Eqn. 2. The reduction factor is used to normalize the load for axles whose gage width is not equal to the standard gage of 6 ft.

When the vehicle wheelbase and bridge span length are known an allowable distributed load may be determined using Eqn. 15. When the bridge span length is known higher vehicle weights may be allowed than what TxDOT rules currently permit which are based only on the wheelbase length.

$$w_{all} = \frac{1.0}{\left[0.01884 * e^{(-0.09715*L)} + \left(4.663 \times 10^{-5} - \frac{0.008302}{L} - \frac{0.004034}{L^2}\right) * WBL\right] * WBL} \tag{15}$$

where

w_{all}	=	allowable distributed load (kips/ft.),
L	=	bridge span length (feet),
WBL	=	WB, wheelbase (feet) when WB $\leq$ L,
	=	L, span length (feet) when WB > L.

The proposed criteria for the weights of axle groups are more restrictive than the current TxDOT weight criteria shown in Table 2. The proposed changes to Table 2, although more restrictive, are not necessarily a matter for concern with regard to bridge failure. As stated earlier, by allowing heavier loads on a bridge, the service life of the bridge will be shortened.

Of possible greater concern is the formula for the gage reduction factor. The proposed formula for gage is less restrictive than TxDOT's current formula, particularly for gages from 8 to 12 ft. as shown in Fig. 16. The proposed reduction factors in this range are greater than the current TxDOT values as shown in Fig. 16. The result is that when axle weights in this range are normalized to a 6 ft. gage the proposed formula does not reduce the weight as much as the TxDOT formula.

8.2 Future Study

The proposed weight restrictions and formulas were developed using simply supported bridges. Future study should focus primarily in two areas. First, continuous bridge spans should be examined. The current and proposed restrictions may significantly overstresses these bridges. Secondly, automating the issuing of permits should be studied and implemented. By using a computer system overweight and oversize permits could be more efficiently issued. Such a system with a database of the state highways and bridges would allow the issuance of heavier permits without time consuming and costly analysis. Currently when a permit is issued the span lengths and configurations of bridges along the truck's route cannot be determined. If the bridge configurations along the route were known heavier permits could be issued without additional analysis.

9. REFERENCES

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