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16. Abstract

This document defines standards for issuing permits for overweight vehicles crossing standard H-type and HS-type Texas highway bridges. A general formula and a bridge specific formula were developed for simple spans of both bridge types. Several reinforced concrete continuous span slab bridges were then evaluated according to the proposed criteria to ensure the validity of the proposed formulae for continuous spans as well as simple spans. The general formula limits the axle group weight according to only the "X" rating and the vehicle dimensions, while the bridge specific formulae also include the span length.

Currently, the vehicle dimensions are the only criteria used by the Texas Department of Transportation (TxDOT) to determine whether or not an overweight permit will be issued. The proposed restrictions allow only one permit vehicle on the bridge at a time and ensure that the maximum stress does not exceed the operational stress level. In addition to determining the maximum weight which may be safely carried by a given axle configuration over either a specific bridge or an unknown bridge, the proposed formulae may also yield the "X" rating for any specific truck. Being able to quickly convert any truck to an equivalent HX or HSX rating will greatly simplify and increase the accuracy of the permitting process.

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OVERWEIGHT PERMIT RULES

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IMPLEMENTATION STATEMENT

The use of the proposed formulae for bridges designed to H15, HS15, H20, and HS20 loadings will significantly expedite the issuance of permits by the Central Permit Office (CPO). These formulae better estimate the design strength of bridges typical to Texas highways by incorporating the effect of span length, span type (simple supported or continuous), and type of bridge (slab, concrete, or steel stringer). The development of H*X* and HS*X* formulae will simplify and increase the accuracy of the permitting process by quickly converting any truck to an equivalent "*X*" rating. Therefore, the routing of permit loads, especially "superheavy" vehicles, can be performed with consideration given to a specific bridge on an intended route.

DISCLAIMER

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

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LIST OF ABBREVIATIONS AND SYMBOLS

β	correction factor for concentrated loadings
A	area (in.^{2} or cm^2)
A_s	area of steel in reinforced concrete slab (in^2 or cm^2)
b	width of cross section (ft. or m)
c	distance to neutral axis of a reinforced concrete section (in. or cm)
d	depth to tensile steel (in. or cm)
D	distance between center of gravity of a given axle group and its nearest axle (ft. or m)
DF	distribution factor for steel I-beam bridges used to calculate the number of wheel loads supported by a given stringer
E	effective width which will support one line of wheels (ft. or m)
E_L	effective width which will support one lane loading (ft. or m)
f_c	allowable stress of concrete (ksi or kPa)
f_s	allowable stress of steel (ksi or kPa)
F_y	yield stress of steel (ksi or kPa)
G	truck gage - transverse distance between axles (ft. or m)
GD	greatest distance between any two axles of an axle group (ft. or m)
GW	axle group weight (k or kN)
GW_{rev}	revised allowable group weight (k or kN)
I	AASHTO impact factor
j	concrete section factor
k	concrete section factor
L	bridge span length (ft. or m)
M_{allow}	allowable live load moment which will not cause stresses greater than 0.75 F_y (k-ft. or kN-m)
M_D	moment due to dead load (k-ft. or kN-m)
M_{fc}	moment capacity of concrete (k-ft. or kN-m)
M_{fs}	moment capacity of steel (k-ft. or kN-m)

LIST OF ABBREVIATIONS AND SYMBOLS (cont.)

M_{IL}	max moment resulting from influence line analysis (k-ft. or kN-m)
M_{ol}	allowable moment capacity of continuous slab bridge (k-ft. or kN-m)
n	number of axles
OSR	overstress ratio (M_{IL} / M_{ol})
R_i	reduction factor accounting for gages wider than 1.8 m (6 ft.), TxDOT
RF	reduction factor accounting for gages wider than 1.8 m (6 ft.), TTI
S	distance between longitudinal stringers (ft. or m)
S_i	reduction factor accounting for more than four tires on a given axle
T	summation of axle loads of a given axle group (k or kN)
w	allowable distributed load (k/ft. or kN/m)
w_{rev}	revised equivalent distributed load (k/ft. or kN/m)
WB	wheelbase length of a given axle group (ft. or m)
WB_{rev}	revised wheelbase (WB/β) between axles of an axle group (ft. or m)
WBL	effective wheelbase on bridge (ft. or m)

SUMMARY

This document defines standards for issuing permits for overweight vehicles crossing standard H-type and HS-type Texas highway bridges. A general formula and a bridge specific formula were developed for simple spans of both bridge types. Several reinforced concrete continuous span slab bridges were then evaluated according to the proposed criteria to ensure the validity of the proposed formulae for continuous spans as well as simple spans. The general formula limits the axle group weight according to only the "X" rating and the vehicle dimensions, while the bridge specific formulae also include the span length.

Currently, the vehicle dimensions are the only criteria used by the Texas Department of Transportation (TxDOT) to determine whether or not an overweight permit will be issued. The proposed restrictions allow only one permit vehicle on the bridge at a time and ensure that the maximum stress does not exceed the operational stress level. In addition to determining the maximum weight which may be safely carried by a given axle configuration over either a specific bridge or an unknown bridge, the proposed formulae may also yield the "X" rating for any specific truck. Being able to quickly convert any truck to an equivalent HX or HSX rating will greatly simplify and increase the accuracy of the permitting process.

1. INTRODUCTION

1.1 Historical Overview

The maximum sizes and weights of motor vehicles allowed to operate over the nation's highways and bridges have been at the center of many debates. The economic significance of this subject impacts not only those directly involved in the transportation of goods, but also the consumers of these goods and the various public entities which design, construct, and maintain the roadway system. Therefore, the Texas Department of Transportation (TxDOT) and Texas lawmakers are constantly reassessing the permissible vehicle weights and axle configurations in an attempt to obtain an economic balance between transportation and infrastructure costs while ensuring the public's safety.

An important aspect of the maximum permissible loads concern is whether or not to issue an overweight permit. Currently, the Central Permit Office (CPO) for TxDOT issues more than 20,000 oversize and/or overweight permits each month. Some of these permit requests are "superheavy" loads which require an engineering analysis for each bridge along the proposed route. Because this analysis is specific to a particular bridge, the process is both time consuming and costly, particularly when a number of bridges must be crossed. An example of a superheavy vehicle is shown in Fig. 1-1. This vehicle weighs approximately 8 meganewtons (1.8 million pounds) and will require special provisions and/or reinforcement to permit it to cross any bridge. However, most vehicles applying for overweight permits are much closer to the bridge design load. The CPO issues overweight permits to mobile cranes like that described in Table 1-1. The gross weight of this vehicle is 885 kN (199 kips). As the number of these overweight permit requests continue to grow, the need for an easier and less costly method of analysis is becoming more imperative.

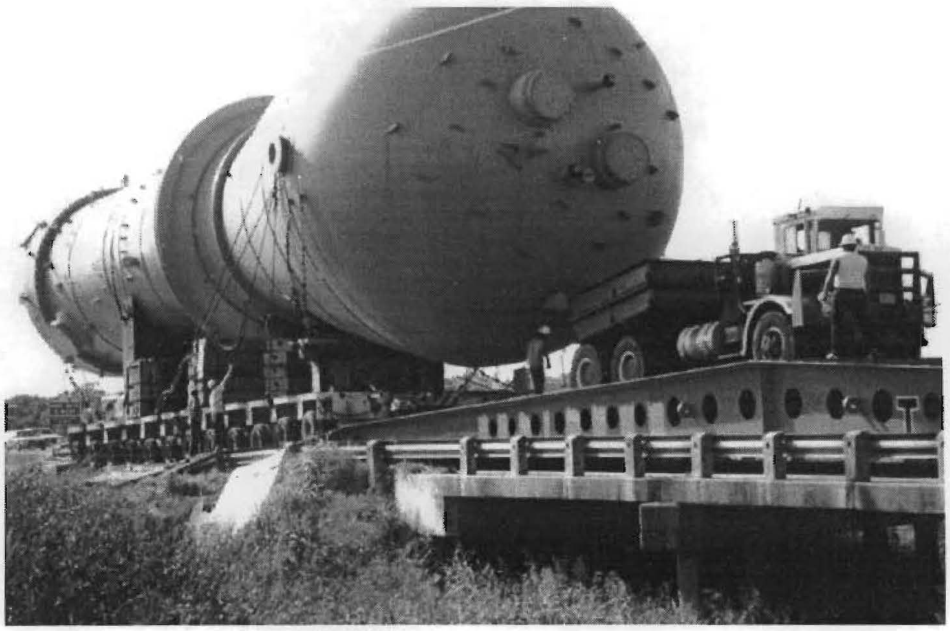

Figure 1-1: Example of 8 meganewton (1.8 million lb.) superheavy load.

Axle No.	Axle Spacing		Tires per Axle	Tire Width		Weight per Axle		Axle Gage	
	(m)	(ft.)		(mm)	(in.)	(kN)	(kips)	(m)	(ft.)
1	-	-	2	356	14.0	96.37	21.66	3.2	10.5
2	1.2	4.0	2	356	14.0	96.37	21.66	3.2	10.5
3	1.2	4.0	2	356	14.0	96.37	21.66	3.2	10.5
4	4.9	16.	4	356	14.0	123.0	27.65	3.0	9.8
5	1.2	4.0	4	356	14.0	123.0	27.65	3.0	9.8
6	1.2	4.0	4	356	14.0	123.0	27.65	3.0	9.8
7	5.2	17.	4	254	10.0	75.73	17.02	2.0	6.7
8	1.2	4.0	4	254	10.0	75.73	17.02	2.0	6.7
9	1.2	4.0	4	254	10.0	75.73	17.02	2.0	6.7

Table 1-1: Example of overweight permit vehicle (mobile crane).

On May 29, 1991, the Texas Legislature adopted into the *Texas Administrative Code* [1] the method currently used by the CPO to analyze most overweight permit requests. These rules are specifically for mobile cranes and oil well service vehicles but are applied to other overload permit requests as a screening process. These regulations limit the axle weights by two methods. The first is a limiting weight which is characterized only by the number of axles in any subgroup. This restriction does not depend on the bridge type, number of tires, axle width (gage), or axle wheelbase. A second restriction converts the total weight of any axle group to a distributed load per linear foot. This equivalent distributed load may then be modified by additional factors for wider than average axle widths, and more than four tires per axle. The resulting modified equivalent distributed load is then checked against the maximum allowed. A curve defining the allowable maximum load was derived from vehicles which had been granted permits in the past, from a thorough static analysis.

Although the current permit restrictions protect most bridges from significant damage, they leave several issues unresolved which this investigation will address. The first reason for additional research is that the current restrictions are based on limited data from overweight vehicles which had been granted permits in the past. Because the resulting forces in a bridge are dependent on the axle weights, wheelbase, gage, and span length of the bridge, a load which produces the maximum allowable stresses in one span length may develop stresses which exceed this allowable level in another span length. Since it would take an enormous amount of data to accurately represent the true "critical" axle configurations, a different approach is necessary.

Secondly, none of the current restrictions take the bridge design type into account. Most bridges in the United States are designed according to standards set by the American Association of State Highway and Transportation Officials (AASHTO) [2]. AASHTO specifies a truck loading and a lane loading for the H15, H20, HS15, and HS20 design types. The truck loading for the four design types is shown in Fig.

1-2. The second type of load condition, shown in Fig. 1-3, is the standard lane loading consisting of a distributed load and a concentrated load which is positioned to produce a maximum moment.

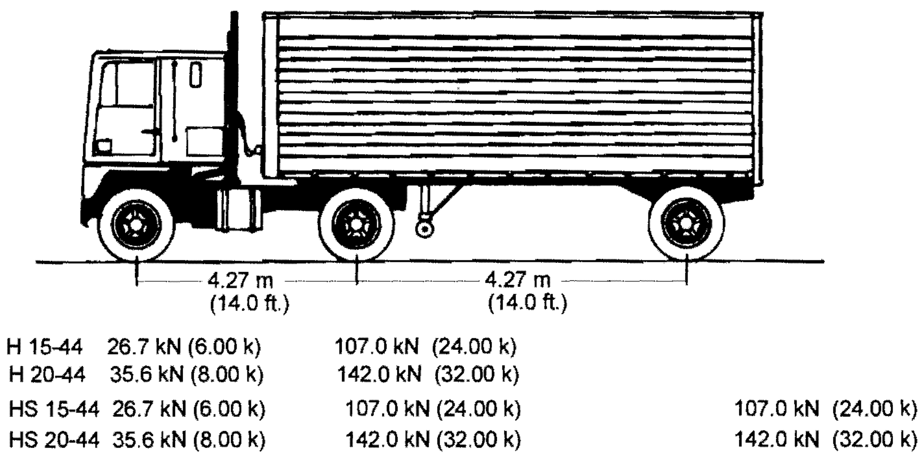

Figure 1-2: AASHTO truck loadings [2].

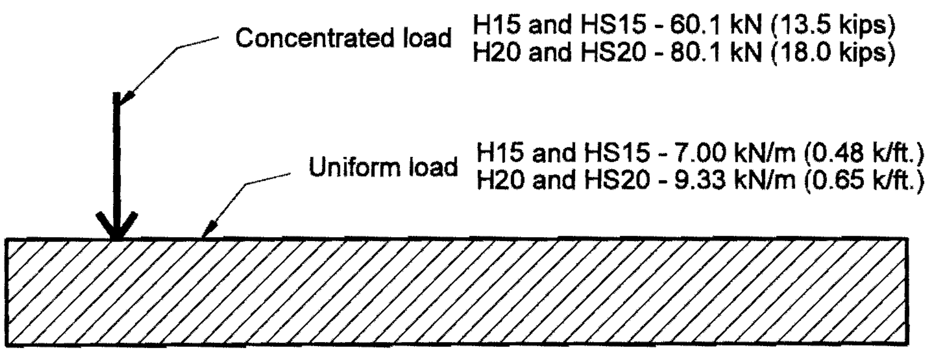

Figure 1-3: AASHTO lane loadings [2].

For a given design type, the load condition which produces the largest maximum moment governs the design of the bridge for that particular span length. The truck loadings produce the maximum moment for shorter spans, while the lane loadings control for the longer spans. Obviously, a HS20 bridge should allow greater allowable loads than a H15 bridge. However, the current permit criteria limit all vehicular weights according to H15 restrictions, even if only HS20 bridges exist along the desired route. A formula which incorporates the bridge design type will allow greater loads along HS20-only routes without a detailed engineering analysis of each bridge.

A third issue which further investigation needs to address, is to consider span length of a particular bridge in the permit process. Because a formula applicable to any span length bridge may be very conservative to the majority of bridges with other span lengths, this additional variable will raise the allowable loads for many bridges. This will be particularly useful when BRINSAP includes individual span lengths for specific bridges along a route.

The fourth goal of further research is to provide a general check for the current regulations. Although no major complications have resulted from use of the current criteria, an independent, engineering-based analysis has never been done to confirm their allowable values.

In light of above concerns, the TxDOT project 1266 was initiated to develop the general formulae and procedures for issuing the permits to overweight vehicles passing over H15, H20, and HS20 highway bridges in the state of Texas. The Texas Transportation Institute (TTI) Report 1266-4F (3) summarizes the work. The resulting formulae were developed to ensure that the maximum stress did not exceed the operational stress level. Criteria was first developed for simple span bridges. Two formulae for each bridge type, a general formula and a bridge formula, had been developed to limit the group weight on simple span bridges.

The general formula was a function of only the vehicle dimensions and was similar to the current Texas permit rules. The current TxDOT permit rules for mobile cranes and oil well service vehicles only apply for wheelbases up to 24.4 m (80 ft.). The proposed formula was calculated for wheelbases up to 36.6 m (120 ft.). The formula developed was significantly more restrictive than that currently used by TxDOT. A second formula was developed based on the vehicle dimensions and the span length of any bridge along the permitted vehicle route. With this bridge-specific formula, higher permit weights can be safely authorized without additional engineering analysis. In addition, several critical reinforced concrete continuous span slab bridges originally designed for H15 or H20 loading were checked to ensure that the formula calculated for simple span bridges did not exceed the allowable stress for these birdge types.

1.2 Objectives

The primary objective for this study is to continue the effort of TxDOT Project 1266 and define permit bridge formulae applicable to bridges designed for the AASHTO H-type and HS-type axle configurations. This will include not only the four common design types, but also other bridges which may have been designed by or reduced to another H*X* or HS*X* designation. Two types of formulae will be derived for the H*X* and HS*X* axle configurations. With the first type of formula, the bridge span length will not need to be known. This formula will have two uses. It will either calculate the maximum allowable load for a given axle configuration and bridge type or, more importantly, it will classify a given axle group as an equivalent H*X* or HS*X* truck. By reclassifying any real vehicle with a H*X* or HS*X* status, the bridges a given truck may be allowed to cross will be quickly known. This new status will also become useful when deterioration has affected the load carrying capacity of a bridge. The bridge can then be given a lower H*X* or HS*X* rating to reflect its deteriorated state and be compared directly with any axle configuration.

The second type of formula derived for the H*X* and HS*X* axle configurations will require the bridge span length to be known. While the formula for any bridge span length will be conservative for all spans except the critical one, the bridge-specific formula will not have this problem. Therefore, this formula will be the most accurate and permit the greatest allowable loads.

The high stress region at the interior supports of reinforced concrete continuous span slab bridges originally designed for H-type live load will then be evaluated according to the proposed H*X* and HS*X* formulae. This bridge type was selected because the negative moment they were designed for can be greatly exceeded by vehicles with wheel patterns that generate more negative moment than the original design truck.

Current AASHTO design procedures (2) account for vehicular dynamic loading by multiplying the live load moment by an impact factor, *I,* with maximum value of 30 percent, assuming the vehicle is traversing the bridge at full speed. TxDOT's current procedure is to use a reduced impact allowance, such as 10 percent, or no impact allowance if this will allow the overload to be issued a permit. The procedure is invoked only if the speed of the overload is appropriately restricted as a condition of permit issuance. As the result of TxDOT's acceptance of lower impact factors, formulae with an impact factor of 10 percent as well as zero percent are developed by following the same procedures in TTI Report 1266-4F [3]. It is warned, however, that the speed of the permit vehicle must be restricted if the proposed formulae associated with 10 percent and zero percent impact factor are to be used. Otherwise, the formulae with 30 percent impact factor should be used conservatively.

It should be noted that the following analysis does not take into account any reduction in service life due to accelerated deterioration rates from the permit trucks. Repeated overloading of the structures may cause permanent deformations.

Additionally, as the stress range of a particular member increases, the number of cycles needed for fatigue crack propagation and failure is reduced.

1.3 Current TxDOT Permitting Procedures for Mobile Cranes and Oil Well Service Vehicles

The TxDOT rules which currently govern the awarding of these special overweight permits are based on the wheelbase length and width. The distance from the center of the first axle to the center of the last axle in any axle group is referred to as the wheelbase length, while the wheelbase width is more commonly called the "gage."

The first restriction the *Texas Administrative Code* [1] imposes is a gross weight limit on axle groups which is determined by the number of axles in each group. These limits are shown in Table 1-2. In addition, a tire surface restriction of 149 kN/mm (850 lb/in.) of tire width is also imposed for each axle. This last restriction is primarily for the purpose of protecting the pavement. However, if either of these limits are exceeded, a permit may still be issued by the equivalent distributed load method.

Number of Axles in Group	Maximum Allowable Axle Group Weight (kN)	(kips)
1	111	25.0
2	200	45.0
3	267	60.0
4	311	70.0
5	367	81.4

Table 1-2: Axle group weight restrictions [1].

The equivalent distributed load method allows consideration of factors that may provide greater distribution of the axle group's weight. These factors are the number of tires, gage distance, and longitudinal distribution of the load by the deck. The maximum allowable permit loads are usually controlled by this method. While this method was developed for the permitting of mobile cranes and oil well equipment, TxDOT applies these rules to other loads only as a guide at this time.

The *Texas Administrative Code* [1] specifies the unmodified equivalent distributed load for any combination of axles by Eq. (1-1):

$$W_{un} = \frac{T}{WB + 1.2} \qquad \left(W_{un} = \frac{T}{WB + 4} \right) \tag{1-1}$$

where

W_{un}	=	the unmodified equivalent distributed load per linear meter (ft.),
T	=	the summation of axle loads of a group of two or more axles,
WB	=	wheelbase length meter (ft.).

Additional factors may be applied to each axle of a vehicle with eight or more tires per axle, or with a gage greater than 1.8 m (6.0 ft.). The revised equivalent axle load is calculated by rewriting equation (1-1) as:

$$W_{rev} = \frac{\sum_{i=1}^{n} \left(R_i * S_i * T_i \right)}{WB + 1.2} \qquad \left(W_{rev} = \frac{\sum_{i=1}^{n} \left(R_i * S_i * T_i \right)}{WB + 4} \right) \tag{1-2}$$

where

W_{rev}	=	revised equivalent distributed load per linearmeter (ft.),
S_i	=	reduction factor accounting for each axle with more than four tires per axle,
	=	1.0 for axles with four tires or fewer,
	=	0.96 for axles with eight or more tires,
n	=	number of axles,

R_i = reduction factor accounting for gages wider than 1.8 m (6 ft.); calculated by the following formula:

$$R_i = \frac{1.8 + G}{2G} \qquad \left(R_i = \frac{6 + G}{2G} \right) \tag{1-3}$$

where

G = gage, m (ft.).

Table 1-3 shows the calculated maximum permit weight from TxDOT. Columns 5 and 6 in Table 1-3 are group weights for 1.8 m (6 ft.) gage only. TTI Report 1266-4F gives detailed explanations as well as an example permit calculation using current TxDOT rules.

Wheelbase (m)	Wheelbase (ft.)	Distributed Load (kN/m)	Distributed Load (k/ft.)	Group Weight (kN)	Group Weight (k)
1.22	4	105.73	7.250	258.1	58.0
1.53	5	92.53	6.345	254.0	57.1
1.83	6	86.73	5.947	264.5	59.5
2.14	7	83.10	5.698	278.8	62.7
2.44	8	80.21	5.500	293.6	66.0
2.75	9	77.67	5.326	308.0	69.2
3.05	10	75.38	5.169	321.9	72.4
3.36	11	73.31	5.027	335.4	75.4
3.66	12	71.43	4.898	348.6	78.4
3.97	13	69.72	4.781	361.5	81.3
4.27	14	68.18	4.675	374.3	84.2
4.58	15	66.78	4.579	387.0	87.0
4.88	16	65.51	4.492	399.6	89.8
5.19	17	64.36	4.413	412.2	92.7
5.49	18	63.29	4.340	424.7	95.5
5.80	19	62.30	4.272	437.0	98.3
6.10	20	61.37	4.208	449.2	101.0
6.41	21	60.46	4.146	461.0	103.7
6.71	22	59.60	4.087	472.7	106.3
7.02	23	58.77	4.030	484.0	108.8
7.32	24	57.96	3.974	494.9	111.3
7.63	25	57.17	3.920	505.6	113.7
7.93	26	56.39	3.867	516.0	116.0
8.24	27	55.64	3.815	526.0	118.3
8.54	28	54.89	3.764	535.8	120.4
8.85	29	54.16	3.714	545.2	122.6
9.15	30	53.61	3.676	555.9	125.0
9.46	31	53.17	3.646	567.6	127.6
9.76	32	52.73	3.616	579.0	130.2
10.07	33	53.00	3.586	590.2	132.7
10.37	34	51.87	3.557	601.2	135.2
10.68	35	51.47	3.529	612.2	137.6
10.98	36	51.06	3.501	622.9	140.0
11.29	37	50.66	3.474	633.5	142.4
11.59	38	50.28	3.448	644.1	144.8
11.90	39	49.92	3.423	654.7	147.2
12.20	40	49.57	3.399	665.2	149.6
12.51	41	49.23	3.376	675.7	151.9

Table 1-3: TxDOT maximum permit weight table [1].

Wheelbase		Distributed Load		Group Weight	
(m)	(ft.)	(kN/m)	(k/ft.)	(kN)	(k)
12.81	42	48.91	3.354	686.3	154.3
13.12	43	48.61	3.333	696.8	156.7
13.42	44	48.32	3.313	707.3	159.0
13.73	45	48.02	3.293	717.7	161.4
14.03	46	47.75	3.274	728.1	163.7
14.34	47	47.47	3.255	738.4	166.0
14.64	48	47.19	3.236	748.5	168.3
14.95	49	46.93	3.218	758.6	170.6
15.25	50	46.67	3.200	768.6	172.8
15.56	51	46.41	3.182	778.4	175.0
15.86	52	46.14	3.164	788.1	177.2
16.17	53	45.88	3.146	797.6	179.3
16.47	54	45.62	3.128	807.0	181.4
16.78	55	45.37	3.111	816.4	183.5
17.08	56	45.12	3.094	825.7	185.6
17.39	57	44.87	3.077	834.9	187.7
17.69	58	44.64	3.061	844.2	189.8
18.00	59	44.41	3.045	853.3	191.8
18.30	60	44.19	3.030	862.6	193.9
18.61	61	43.97	3.015	871.7	196.0
18.91	62	43.75	3.000	880.7	198.0
19.22	63	43.53	2.985	889.6	200.0
19.52	64	43.33	2.971	898.6	202.0
19.83	65	43.12	2.957	907.5	204.0
20.13	66	42.92	2.943	916.3	206.0
20.44	67	42.72	2.929	925.0	208.0
20.74	68	42.51	2.915	933.5	209.9
21.05	69	42.31	2.901	942.0	211.8
21.35	70	42.10	2.887	950.3	213.6
21.66	71	41.91	2.874	958.8	215.6
21.96	72	41.72	2.861	967.2	217.4
22.27	73	41.53	2.848	975.4	219.3
22.57	74	41.34	2.835	983.6	221.1
22.88	75	41.15	2.822	991.6	222.9
23.18	76	40.97	2.809	999.6	224.7
23.49	77	40.78	2.796	1007.4	226.5
23.79	78	40.59	2.783	1015.1	228.2
24.10	79	40.41	2.771	1023.0	230.0
24.40	80	40.24	2.759	1030.9	231.8

Table 1-3: TxDOT maximum permit weight table [1] (cont.).

1.4. Example Permit Calculation

The following illustration shows the use of the current overweight permit procedures, [3]. The vehicle to be analyzed is shown in Fig. 1-4.

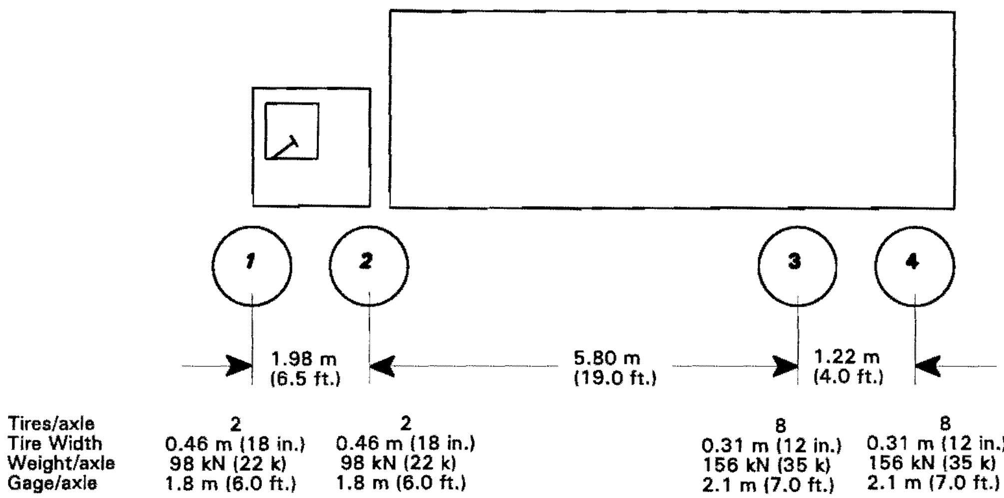

Figure 1-4: Example vehicle for permit calculation.

The group weight of the front two axles (1 and 2) falls within the single axle group restriction of 111 kN (25.0 k). Also, each of the front axles has a total tire width of 915 mm (36 in). Dividing 97.9 kN (22.2 k) by 915 mm (36 in) results in a load of 106.9 N/mm (611 lb/in.) of tire width for each of the two front axles. Hence, the two front axles also meet the individual axle restriction of 149 N/mm (850 lb/in.) of tire width.

However, the two rear axles (3 and 4) being 156 kN (35.0 k) each clearly violate the two axle group restriction of 200 kN (45.0 k) in Table 1-2. By using equations (1-2) and (1-3), an equivalent distributed load per meter (foot) can be calculated to determine if they fall within the restrictions of Table 1-3. Because the two rear axles do not have the standard gage or number of tires, 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 2.1 m (7 ft.) into equation (1-3) results in a gage reduction factor of R = 0.929. The wheelbase for the rear axle group is $WB = 1.2$ m (4 ft.). The summation of the axle loads for the rear axle group is $T = 311$ kN (70 k).

Substitution of R, S, T, and WB into equation (1-2) results in an equivalent distributed load of $W = 113.8$ kN/m (7.804 k/ft.). According to Table 1-3, the maximum allowable distributed load for an axle group with a 1.2 m (4 ft.) wheelbase is 105.7 kN/m (7.250 k/ft.).

A similar calculation can be done to determine the equivalent distributed load for axle groups 1, 2, 3, and 4. The values for R, S, and T for axles 3 and 4 will remain the same. For axles 1 and 2, $R_{1,2} = 1.0$, $S_{1,2} = 1.0$, and $T_{1,2} = 97.9$ kN (22 k). Since axles 1 through 4 are under consideration, the wheelbase is $WB = 9.00$ m (29.5 ft.). Substituting these values into equation (1-2) results in an equivalent distributed load of 46.33 kN/m (3.177 k/ft.). Analysis for axle group 2, 3, and 4 will result in equivalent distributed load of 45.60 kN/m (3.127 k/ft.). Table 1-4 summarizes the distributed loads for the example vehicle.

Because the example vehicle violates the distributed load restrictions for axles 3 and 4, the CPO will deny a permit. The Bridge Section of TxDOT will then have to perform a structural analysis of the bridges along the vehicle's route to determine if a permit may still be issued.

Axles	Wheelbase Length		Equivalent Dist. Load		TxDOT Restiction (from Table 1-3)		Issue
	(m)	(ft.)	(kN/m)	(k/ft.)	(kN/m)	(k/ft.)	
1,2	1.98	6.5	61.10	4.190	84.91	5.823	Yes
1,2,3,4	9.00	29.5	46.33	3.177	53.89	3.695	Yes
2,3,4	7.02	23.0	45.60	3.127	58.79	4.030	Yes
3,4	1.22	4.0	113.8	7.804	105.7	7.250	No

Table 1-4: Summary of distributed loads for example vehicle.

2. DEVELOPMENT OF H-TYPE AND HS-TYPE BRIDGE FORMULAE FOR SIMPLE SPANS

2.1 Procedure Overview

It is assumed that both the H-type and the HS-type bridges were originally designed by the Allowable Stress Design (ASD) or working stress method according to AASHTO specifications so that the inventory stress level would not be exceeded. The inventory stress given by AASHTO *Manual for Maintenance Inspection of Bridges* [4] is 0.55 times the yield strength of the steel ($0.55F_y$) for steel bridge members. The operating level stress is increased to $0.75F_y$ for permit loading. This increase is allowed due to the infrequency of the permitted load and the fact that only a single permitted truck is on the span. In addition, a greater amount of control may also be attained if permits are required for all vehicles which may cause stresses in excess of the inventory level.

The allowable moment capacity due to the permit vehicle is calculated by multiplying 0.75/0.55 by the bridge moment capacity, as based on strength considerations, and then subtracting the moment due to dead load. Typical dead-load to live-load-plus-impact moment ratios aid in this calculation by allowing the analysis to proceed without having to consider design parameters, other than span length, of many individual bridges. The moment ratios for typical H-type and HS-type simple span bridges can be easily calculated from the similar designs given by Seelye [5], and are compared with moment ratios given by Noel [6] and Whiteside [7].

The permit truck is then assumed to be a longitudinally distributed load positioned in the center of the bridge as shown in Fig. 2-1. TTI Report 1266-4F [3] first introduced this distributed load concept. The magnitude of the distributed load necessary to produce the allowable live load moment is then calculated. A factor, β, is later calculated to account for differences between real axle groups and the assumed distributed load. This process is repeated for wheelbases from 1.22 m (4 ft.) to 36.6 m (120 ft.) and bridge span lengths from 3.05 m (10 ft.) to 45.8 m (150 ft.).

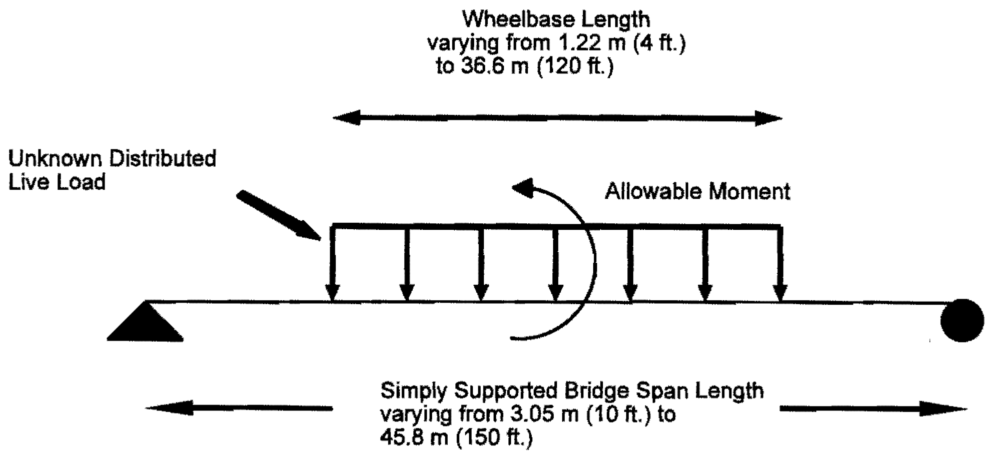

Figure 2-1: Unknown distributed load illustration [3].

AASHTO specifications state that the governing live load condition will be applied to each lane. If two standard H-type or HS-type trucks are placed side-by-side in the center of a simple span bridge, they have a 4.9 m (16 ft.) effective gage. Because only one permit truck is allowed on a bridge, a reduction factor [3], developed from finite element analyses, is used to ensure that the maximum stresses do not exceed the operational stress limit and is given as:

$$RF = 1.2 - \frac{G}{9.1} \qquad \left(RF = 1.2 - \frac{G}{30} \right) \tag{2-1}$$

The negative moment region on continuous slab bridges is also a major concern. AASHTO outlines a method for determining the moment capacity in these bridges in which a wheel load is supported by a longitudinal strip of a certain effective width. The finite element analysis performed and outlined in the TTI Report 1266-4F [3] accurately defines this effective width for several Texas bridges. Critical axle groups are then defined from an influence line analysis, and the loads are limited by the proposed general permit formula developed for simple spans. The maximum moment due to these critical axle groups is then determined and compared to the moment capacity. Using this method, the general simple span formula is validated for continuous span bridges.

2.2 Effect of Impact Factor

As it was mentioned before, the formulae developed in TTI Report 1266-4F are based upon the assumption that the bridges are designed under full impact loading capacity, and the permit trucks will pass the bridge with possible full impact effect on the bridges. However, the speed of the permit trucks passing through will always be changing, but not beyond control. The moment due to impact is found by multiplying the live load moment by an impact factor, I, which AASHTO [2] defines as:

$$I = \frac{15.24}{L + 38} \leq 0.3 \qquad \left(I = \frac{50}{L + 125} \leq 0.3 \right) \tag{2-2}$$

where

L = bridge span length, m (ft.).

TxDOT's current procedure is to use a reduced impact factor if this will allow the overload to be permitted. A 10 percent impact factor is used if the speed of the vehicle is restricted to approximate a smooth walking speed as a condition of issuance. In addition to the speed restriction, no stopping, starting, or gear changing of the pulling truck is permitted while the load is on the bridge. A zero impact allowance is used when the speed of the vehicle is restricted to less than 5 km/hr (3 mph). Equation (2-2) could be modified to reflect a maximum impact factor of 10 percent, however, a bridge span length, L, of 114.4 m (375 ft.) would be required in order to return an impact factor less than 0.10. Therefore, all bridges in the present study will be investigated using an impact factor of 10 percent.

The full impact is still assumed when calculating the design moment capacity of the bridge. The group weights of permit trucks for H15, H20, HS15, and HS20 bridges are calculated assuming the maximum 10 percent impact as well as zero percent impact. It is suggested that the formulae developed under 10 percent impact factor be used if the speed of overweight vehicles can be controlled. Otherwise, the formulae developed under 30 percent impact factor should be conservatively used. The next section discusses the effect of the impact factor on the calculated maximum permit weights for 1.8m (6 ft.) gage on H15, H20, HS15, and HS20 bridges.

2.3 Bridge Formulae as a Function of Wheelbase

By following the same procedure as in TTI Report 1266-4F [3] to calculate the maximum permit weight for both H-type and HS-type bridges, the bridge formulae as a function of only wheelbases are developed with the maximum impact factor of zero percent, 10 percent, and 30 percent, respectively. The group weights of permit trucks for H15, H20, HS15, and HS20 bridges are calculated and tabulated in Tables 2-1, 2-2, 2-3, and 2-4, respectively, assuming a 10 percent maximum impact factor and 1.8 m (6 ft.) gage truck.

Wheelbase		Distributed Load		Group Weight	
(m)	(ft.)	(kN/m)	(k/ft)	(kN)	(k)
1.2	4.0	112.80	7.73	274.9	61.8
1.8	6.0	94.27	6.46	287.4	64.6
2.4	8.0	82.30	5.64	301.1	67.7
3.1	10.0	74.13	5.08	316.3	71.1
3.7	12.0	67.71	4.64	330.1	74.2
4.3	14.0	62.90	4.31	345.2	77.6
4.9	16.0	58.96	4.04	359.4	80.8
5.5	18.0	55.31	3.79	371.0	83.4
6.1	20.0	51.95	3.56	379.9	85.4
6.7	22.0	49.03	3.36	388.8	87.4
7.3	24.0	46.55	3.19	397.2	89.3
7.9	26.0	44.51	3.05	407.0	91.5
8.5	28.0	42.76	2.93	417.2	93.8
9.2	30.0	41.30	2.83	427.9	96.2
9.8	32.0	39.98	2.74	438.6	98.6
10.4	34.0	38.82	2.66	449.7	101.1
11.0	36.0	37.94	2.60	462.6	104.0
11.6	38.0	37.07	2.54	474.6	106.7
12.2	40.0	36.34	2.49	487.5	109.6
12.8	42.0	35.61	2.44	499.1	112.2
13.4	44.0	35.02	2.40	512.4	115.2
14.0	46.0	34.59	2.37	527.1	118.5
14.6	48.0	34.15	2.34	541.3	121.7
15.3	50.0	33.71	2.31	554.7	124.7
15.9	52.0	33.27	2.28	568.0	127.7
16.5	54.0	32.98	2.26	583.2	131.1
17.1	56.0	32.69	2.24	597.8	134.4
17.7	58.0	32.25	2.21	609.4	137.0
18.3	60.0	31.96	2.19	623.6	140.2

Table 2-1: Calculated maximum permit weights for 1.8 m (6 ft.) gage on H15 bridges ($I_{max} = 10\%$).

Wheelbase		Distributed Load		Group Weight	
(m)	(ft.)	(kN/m)	(k/ft)	(kN)	(k)
18.9	62.0	31.52	2.16	634.3	142.6
19.5	64.0	31.23	2.14	647.2	145.5
20.1	66.0	31.08	2.13	663.2	149.1
20.7	68.0	30.79	2.11	675.7	151.9
21.4	70.0	30.50	2.09	688.1	154.7
22.0	72.0	30.35	2.08	703.3	158.1
22.6	74.0	30.06	2.06	714.8	160.7
23.2	76.0	29.92	2.05	729.5	164.0
23.8	78.0	29.77	2.04	744.2	167.3
24.4	80.0	29.62	2.03	758.4	170.5
25.0	82.0	29.48	2.02	772.7	173.7
25.6	84.0	29.33	2.01	786.9	176.9
26.2	86.0	29.19	2.00	800.7	180.0
26.8	88.0	29.04	1.99	814.5	183.1
27.5	90.0	28.89	1.98	827.8	186.1
28.1	92.0	28.75	1.97	841.2	189.1
28.7	94.0	28.75	1.97	858.9	193.1
29.3	96.0	28.60	1.96	871.8	196.0
29.9	98.0	28.46	1.95	884.7	198.9
30.5	100.0	28.46	1.95	902.1	202.8
31.1	102.0	28.31	1.94	914.5	205.6
31.7	104.0	28.16	1.93	927.0	208.4
32.3	106.0	28.16	1.93	944.4	212.3
32.9	108.0	28.02	1.92	956.4	215.0
33.6	110.0	27.87	1.91	968.4	217.7
34.2	112.0	27.87	1.91	985.7	221.6
34.8	114.0	27.73	1.90	997.3	224.2
35.4	116.0	27.58	1.89	1,008.9	226.8
36.0	118.0	27.58	1.89	1,025.8	230.6
36.6	120.0	27.43	1.88	1,036.9	233.1

Table 2-1: Calculated maximum permit weights for 1.8 m (6 ft.) gage on H15 bridges ($I_{max} = 10\%$) (cont.).

Wheelbase		Distributed Load		Group Weight	
(m)	(ft.)	(kN/m)	(k/ft)	(kN)	(k)
1.2	4.0	148.13	10.15	361.2	81.2
1.8	6.0	123.90	8.49	377.7	84.9
2.4	8.0	108.14	7.41	395.4	88.9
3.1	10.0	97.34	6.67	415.5	93.4
3.7	12.0	88.88	6.09	433.3	97.4
4.3	14.0	82.16	5.63	450.6	101.3
4.9	16.0	76.33	5.23	465.3	104.6
5.5	18.0	71.07	4.87	476.4	107.1
6.1	20.0	66.84	4.58	488.9	109.9
6.7	22.0	63.34	4.34	501.8	112.8
7.3	24.0	60.27	4.13	514.2	115.6
7.9	26.0	57.79	3.96	528.4	118.8
8.5	28.0	55.31	3.79	539.6	121.3
9.2	30.0	53.27	3.65	552.0	124.1
9.8	32.0	51.52	3.53	565.4	127.1
10.4	34.0	49.91	3.42	578.3	130.0
11.0	36.0	48.45	3.32	590.7	132.8
11.6	38.0	47.28	3.24	605.4	136.1
12.2	40.0	46.26	3.17	620.5	139.5
12.8	42.0	45.39	3.11	636.5	143.1
13.4	44.0	44.66	3.06	653.4	146.9
14.0	46.0	44.07	3.02	671.7	151.0
14.6	48.0	43.49	2.98	689.5	155.0
15.3	50.0	42.91	2.94	706.4	158.8
15.9	52.0	42.47	2.91	725.1	163.0
16.5	54.0	42.03	2.88	742.8	167.0
17.1	56.0	41.74	2.86	763.3	171.6
17.7	58.0	41.59	2.85	786.0	176.7
18.3	60.0	41.16	2.82	802.9	180.5

Table 2-2: Calculated maximum permit weights for 1.8 m (6 ft.) gage on H20 bridges ($I_{max} = 10\%$).

Wheelbase		Distributed Load		Group Weight	
(m)	(ft.)	(kN/m)	(k/ft)	(kN)	(k)
18.9	62.0	40.72	2.79	818.9	184.1
19.5	64.0	40.43	2.77	838.0	188.4
20.1	66.0	39.99	2.74	853.2	191.8
20.7	68.0	39.70	2.72	871.0	195.8
21.4	70.0	39.26	2.69	885.6	199.1
22.0	72.0	38.97	2.67	902.5	202.9
22.6	74.0	38.67	2.65	919.4	206.7
23.2	76.0	38.38	2.63	935.9	210.4
23.8	78.0	38.24	2.62	955.5	214.8
24.4	80.0	37.94	2.60	971.5	218.4
25.0	82.0	37.80	2.59	990.6	222.7
25.6	84.0	37.65	2.58	1,009.7	227.0
26.2	86.0	37.36	2.56	1,024.9	230.4
26.8	88.0	37.21	2.55	1,043.5	234.6
27.5	90.0	37.07	2.54	1,062.2	238.8
28.1	92.0	36.78	2.52	1,076.0	241.9
28.7	94.0	36.63	2.51	1,094.3	246.0
29.3	96.0	36.49	2.50	1,112.1	250.0
29.9	98.0	36.34	2.49	1,129.8	254.0
30.5	100.0	36.34	2.49	1,152.1	259.0
31.1	102.0	36.19	2.48	1,169.4	262.9
31.7	104.0	36.05	2.47	1,186.8	266.8
32.3	106.0	35.76	2.45	1,198.8	269.5
32.9	108.0	35.61	2.44	1,215.7	273.3
33.6	110.0	35.46	2.43	1,232.2	277.0
34.2	112.0	35.32	2.42	1,248.6	280.7
34.8	114.0	35.17	2.41	1,265.1	284.4
35.4	116.0	35.03	2.40	1,281.1	288.0
36.0	118.0	34.88	2.39	1,297.1	291.6
36.6	120.0	34.73	2.38	1,312.7	295.1

Table 2-2: Calculated maximum permit weights for 1.8 m (6 ft.) gage on H20 bridges ($I_{max} = 10\%$) (cont.).

Wheelbase		Distributed Load		Group Weight	
(m)	(ft.)	(kN/m)	(k/ft)	(kN)	(k)
1.2	4.0	112.08	7.68	273.1	61.4
1.8	6	94.86	6.5	289.1	65.0
2.4	8	83.62	5.73	306.0	68.8
3.1	10	75.31	5.16	321.2	72.2
3.7	12	69.32	4.75	338.1	76.0
4.3	14	65.09	4.46	357.2	80.3
4.9	16	62.02	4.25	378.1	85.0
5.5	18	59.84	4.1	401.2	90.2
6.1	20	58.52	4.01	427.9	96.2
6.7	22	57.94	3.97	459.1	103.2
7.3	24	57.35	3.93	489.3	110.0
7.9	26	56.62	3.88	517.8	116.4
8.5	28	55.60	3.81	542.2	121.9
9.2	30	55.02	3.77	570.3	128.2
9.8	32	54.00	3.7	592.5	133.2
10.4	34	52.54	3.6	608.5	136.8
11	36	50.93	3.49	621.0	139.6
11.6	38	49.33	3.38	631.6	142.0
12.2	40	47.87	3.28	641.9	144.3
12.8	42	46.55	3.19	652.6	146.7
13.4	44	45.24	3.1	661.9	148.8
14	46	44.07	3.02	671.7	151.0
14.6	48	43.05	2.95	682.4	153.4
15.3	50	42.03	2.88	691.7	155.5
15.9	52	41.01	2.81	700.1	157.4
16.5	54	40.13	2.75	709.5	159.5
17.1	56	39.40	2.7	720.6	162.0
17.7	58	38.53	2.64	728.2	163.7
18.3	60	37.80	2.59	737.5	165.8

Table 2-3: Calculated maximum permit weights for 1.8 m (6 ft.) gage on HS15 bridges ($I_{max} = 10\%$).

Wheelbase		Distributed Load		Group Weight	
(m)	(ft.)	(kN/m)	(k/ft)	(kN)	(k)
18.9	62.0	37.07	2.54	745.5	167.6
19.5	64.0	36.34	2.49	753.1	169.3
20.1	66.0	35.76	2.45	762.9	171.5
20.7	68.0	35.03	2.40	768.6	172.8
21.4	70.0	34.44	2.36	776.7	174.6
22.0	72.0	34.00	2.33	787.8	177.1
22.6	74.0	33.42	2.29	794.4	178.6
23.2	76.0	32.98	2.26	804.2	180.8
23.8	78.0	32.54	2.23	813.6	182.9
24.4	80.0	31.96	2.19	818.5	184.0
25.0	82.0	31.52	2.16	826.5	185.8
25.6	84.0	31.09	2.13	833.6	187.4
26.2	86.0	30.79	2.11	844.7	189.9
26.8	88.0	30.36	2.08	851.4	191.4
27.5	90.0	30.06	2.06	861.2	193.6
28.1	92.0	29.77	2.04	871.0	195.8
28.7	94.0	29.33	2.01	876.3	197.0
29.3	96.0	29.04	1.99	885.2	199.0
29.9	98.0	28.90	1.98	898.5	202.0
30.5	100.0	28.60	1.96	906.5	203.8
31.1	102.0	28.31	1.94	914.5	205.6
31.7	104.0	28.17	1.93	927.0	208.4
32.3	106.0	27.87	1.91	934.6	210.1
32.9	108.0	27.73	1.90	946.6	212.8
33.6	110.0	27.58	1.89	958.6	215.5
34.2	112.0	27.29	1.87	964.8	216.9
34.8	114.0	27.14	1.86	976.4	219.5
35.4	116.0	27.00	1.85	987.5	222.0
36.0	118.0	26.85	1.84	998.6	224.5
36.6	120.0	26.85	1.84	1,015.1	228.2

Table 2-3: Calculated maximum permit weights for 1.8 m (6 ft.) gage on HS15 bridges ($I_{max} = 10\%$) (cont.).

Wheelbase		Distributed Load		Group Weight	
(m)	(ft.)	(kN/m)	(k/ft)	(kN)	(k)
1.2	4.0	147.69	10.12	360.3	81.0
1.8	6.0	125.07	8.57	381.2	85.7
2.4	8.0	110.33	7.56	403.5	90.7
3.1	10.0	99.39	6.81	423.9	95.3
3.7	12.0	91.50	6.27	446.2	100.3
4.3	14.0	85.81	5.88	470.6	105.8
4.9	16.0	81.87	5.61	499.1	112.2
5.5	18.0	79.10	5.42	530.2	119.2
6.1	20.0	77.35	5.30	565.8	127.2
6.7	22.0	76.47	5.24	605.8	136.2
7.3	24.0	75.60	5.18	645.0	145.0
7.9	26.0	74.58	5.11	681.9	153.3
8.5	28.0	73.26	5.02	714.4	160.6
9.2	30.0	72.39	4.96	750.0	168.6
9.8	32.0	71.22	4.88	781.5	175.7
10.4	34.0	68.74	4.71	796.2	179.0
11.0	36.0	66.40	4.55	809.6	182.0
11.6	38.0	64.36	4.41	823.8	185.2
12.2	40.0	62.32	4.27	835.8	187.9
12.8	42.0	60.57	4.15	849.2	190.9
13.4	44.0	58.81	4.03	860.3	193.4
14.0	46.0	57.35	3.93	874.1	196.5
14.6	48.0	55.75	3.82	883.4	198.6
15.3	50.0	54.44	3.73	895.9	201.4
15.9	52.0	53.27	3.65	909.2	204.4
16.5	54.0	52.10	3.57	921.2	207.1
17.1	56.0	50.93	3.49	931.5	209.4
17.7	58.0	49.91	3.42	943.0	212.0
18.3	60.0	48.74	3.34	951.0	213.8

Table 2-4: Calculated maximum permit weights for 1.8 m (6 ft.) gage on HS20 bridges ($I_{max} = 10\%$).

Wheelbase		Distributed Load		Group Weight	
(m)	(ft.)	(kN/m)	(k/ft)	(kN)	(k)
18.9	62.0	47.72	3.27	959.9	215.8
19.5	64.0	46.84	3.21	971.0	218.3
20.1	66.0	45.97	3.15	980.8	220.5
20.7	68.0	45.09	3.09	989.7	222.5
21.4	70.0	44.36	3.04	1,000.8	225.0
22.0	72.0	43.63	2.99	1,010.6	227.2
22.6	74.0	43.05	2.95	1,023.5	230.1
23.2	76.0	42.32	2.90	1,032.0	232.0
23.8	78.0	41.74	2.86	1,043.1	234.5
24.4	80.0	41.01	2.81	1,049.8	236.0
25.0	82.0	40.42	2.77	1,059.6	238.2
25.6	84.0	39.98	2.74	1,072.5	241.1
26.2	86.0	39.40	2.70	1,080.9	243.0
26.8	88.0	38.96	2.67	1,092.5	245.6
27.5	90.0	38.53	2.64	1,104.0	248.2
28.1	92.0	38.09	2.61	1,114.7	250.6
28.7	94.0	37.65	2.58	1,124.5	252.8
29.3	96.0	37.36	2.56	1,138.7	256.0
29.9	98.0	36.92	2.53	1,148.1	258.1
30.5	100.0	36.63	2.51	1,161.0	261.0
31.1	102.0	36.34	2.49	1,173.9	263.9
31.7	104.0	36.04	2.47	1,186.8	266.8
32.3	106.0	35.75	2.45	1,198.8	269.5
32.9	108.0	35.46	2.43	1,210.8	272.2
33.6	110.0	35.32	2.42	1,227.3	275.9
34.2	112.0	35.02	2.40	1,238.4	278.4
34.8	114.0	34.88	2.39	1,254.4	282.0
35.4	116.0	34.73	2.38	1,270.4	285.6
36.0	118.0	34.59	2.37	1,286.0	289.1
36.6	120.0	34.29	2.35	1,296.2	291.4

Table 2-4: Calculated maximum permit weights for 1.8 m (6 ft.) gage on HS20 bridges ($I_{max} = 10\%$) (cont.).

The group weights of permit trucks for H15, H20, HS15, and HS20 bridges are calculated assuming the maximum 10 percent as well as a zero percent impact factor. The calculated maximum permit weights are then linearly fitted in terms of wheelbases for both 10 and zero percent impact factors. Along with the results available in TTI Report 1266-4F (3) for 30 percent maximum impact factor, the general formulae of allowable gross weight of a truck axle group as a function of wheelbase for four types of bridges can be expressed as follows:

$$GW = a + b * WB \tag{2-2}$$

where

GW = group weight, kN (k),

WB = wheelbase, m (ft.).

a, b = constants for different types of bridges as in Table 2-5.

Bridge Type	WB m (ft.)	a I=0%	a I=10%	a I=30%	b I=0%	b I=10%	b I=30%
H15	0<WB<36.6	260	236	186	25.4	23.1	21.8
	(0<WB<120)	(58.4)	(53.1)	(41.9)	(1.63)	(1.48)	(1.40)
H20	0<WB<36.6	346	312	246	32.0	29.0	27.6
	(0<WB<120)	(77.1)	(70.1)	(55.2)	(2.05)	(1.86)	(1.77)
HS15	0<WB<11.6	239	217	182	43.0	39.1	33.4
	(0<WB<38)	(53.7)	(48.8)	(40.9)	(2.76)	(2.51)	(2.14)
HS15	11.6<WB<36.6	514	467	378	17.2	15.6	16.4
	(38<WB<120)	(116)	(105)	(84.9)	(1.10)	(1.00)	(1.05)
HS20	0<WB<11.6	318	290	236	55.8	50.8	45.2
	(0<WB<38)	(71.6)	(65.1)	(53.1)	(3.58)	(3.26)	(2.90)
HS20	11.6<WB<36.6	677	615	507	21.4	19.3	20.3
	(38<WB<120)	(152)	(138)	(114)	(1.37)	(1.24)	(1.30)

Table 2-5: Constants for general bridge formulae as a function of wheelbase.

The calculated maximum permit weights for 1.8 m (6 ft.) gage on H15, H20, HS15, and HS20 bridges with 10 percent maximum impact factor are shown in Figures 2-2, 2-3, 2-4, and 2-5, respectively. Also shown in these figures are the current TxDOT permit weights for mobile cranes and oil well service vehicles as well as the linear curve fittings in terms of zero percent, 10 percent, and 30 percent maximum impact factors.

It can be seen from these figures that the maximum permit weights are increased due to the lower impact factor assumption. The average increase for four types of bridges is about 12 percent from 30 percent impact to 10 percent impact. The average increase for four types of bridge is about 10 percent from 10 percent impact to zero percent impact. However, as far as H15 bridges are concerned in Fig. 2-2, the permit weights of TxDOT are still higher than the proposed permit weight for all wheelbases, even for zero percent impact. In the case of H20 bridges in Fig. 2-3, the proposed permit weights with 10 percent impact are very close to the permit weights of TxDOT. For HS15 bridges in Fig. 2-4, the proposed permit weights with 10 percent impact are very close to the permit weights of TxDOT for wheelbases less than 12.2 m (40 ft.) The proposed permit weights with 10 percent impact become smaller than that of TxDOT as wheelbase increases. But in the case of HS20 bridges in Fig. 2-5, the proposed permit weights with 10 percent impact are larger than the permit weights of TxDOT for all wheelbases. This indicates that current TxDOT permit rules are restrictive in some cases and conservative in other cases. Therefore, more flexible rules considering more factors such as the rules and formulae proposed in this study should be adopted as they can benefit both TxDOT and the permit applicant. In some cases, it needs to be restrictive in permit issuing for the bridge safety, while in other cases, more capacity of the bridge can be explored using proposed permit weight rules.

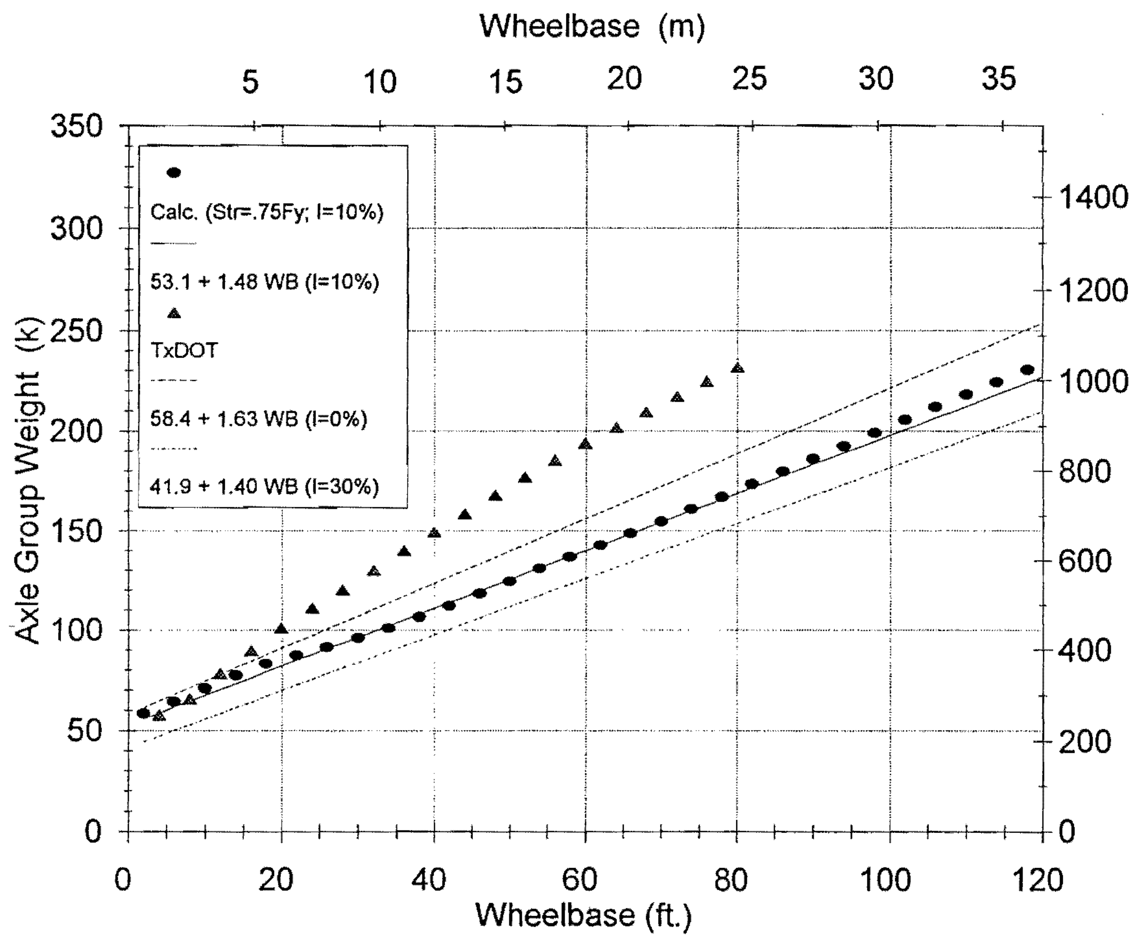

Figure 2-2: Calculated maximum permit weights for 1.8 m (6 ft.) gage on H15 bridges.

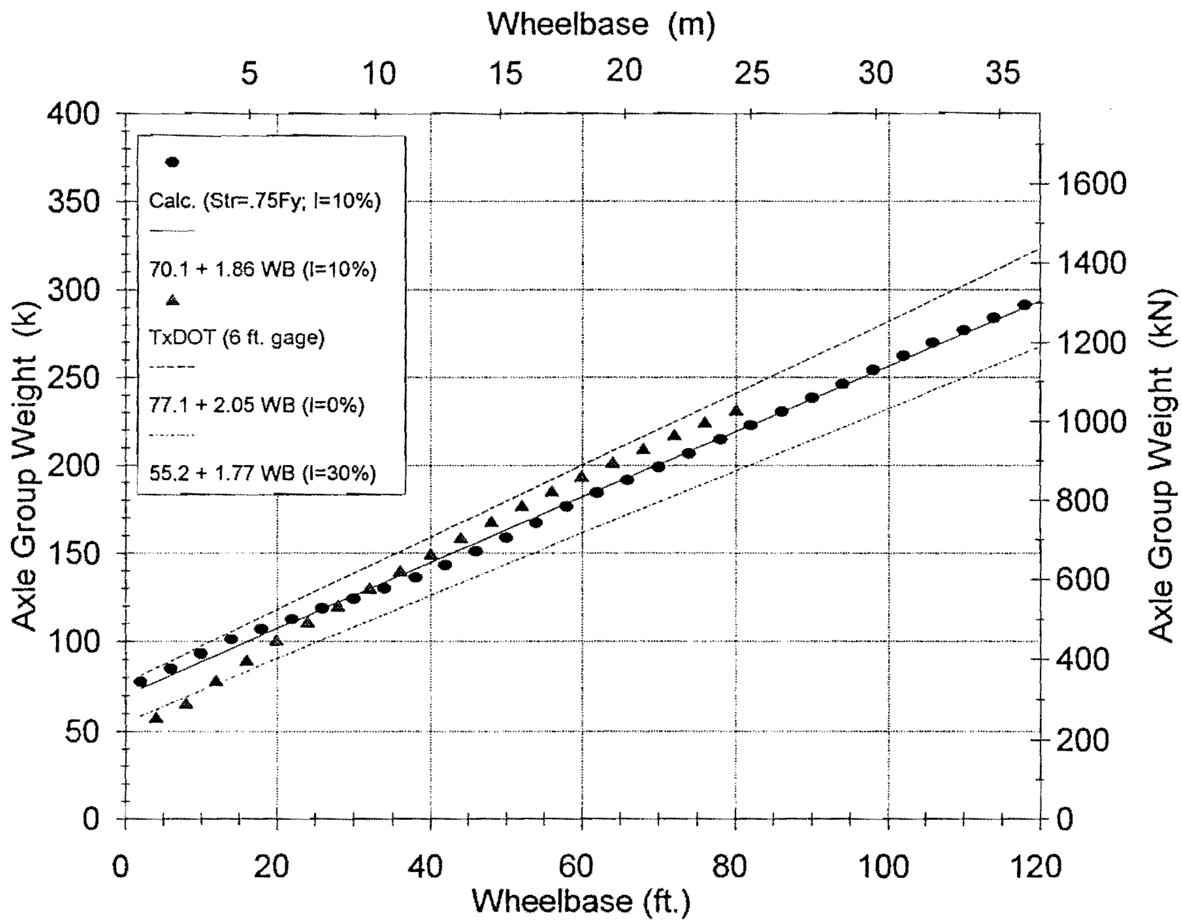

Figure 2-3: Calculated maximum permit weights for 1.8 m (6 ft.) gage on H20 bridges.

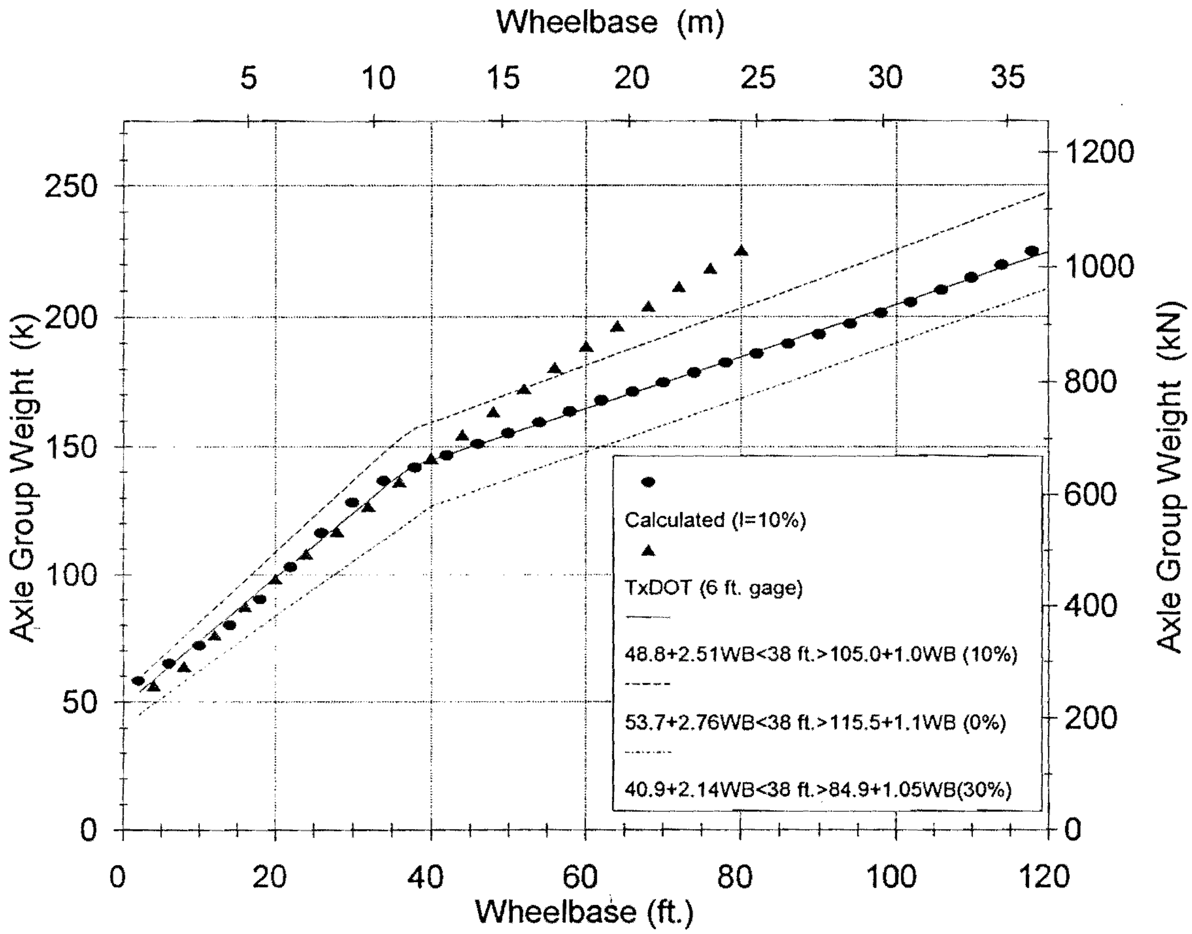

Figure 2-4: Calculated maximum permit weights for 1.8 m (6 ft.) gage on HS15 bridges.

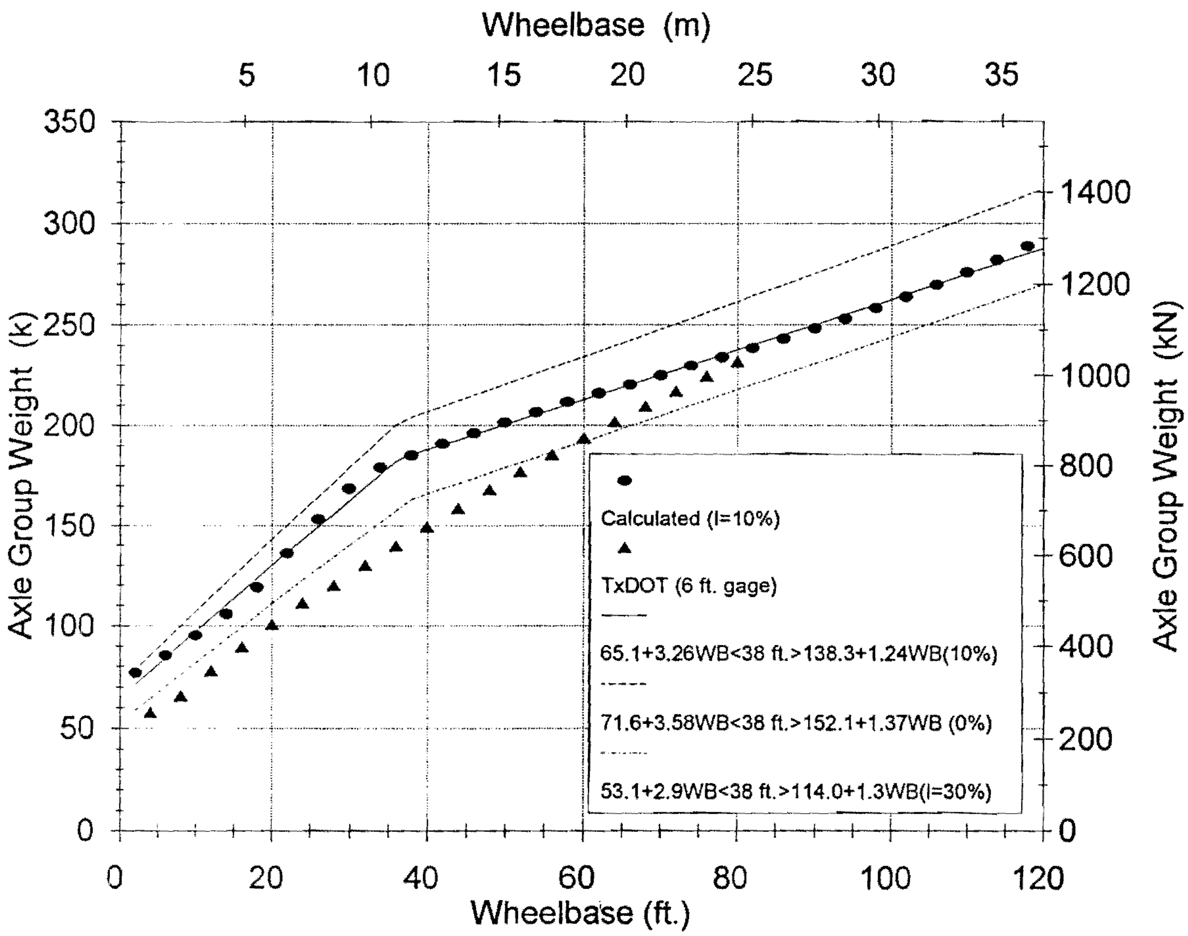

Figure 2-5: Calculated maximum permit weights for 1.8 m (6 ft.) gage on HS20 bridges.

2.4 Bridge Formulae Considering Span Length

The previous proposed weight restrictions in equation (2-2) is only a function of the wheelbase of a truck's axle groups. This is done to ensure that the vehicle can safely pass over a bridge of any span length. Therefore, the CPO may issue a permit for a given truck without knowing the specifics of the bridge to be crossed. In some cases, these weight restrictions limit the permit weights significantly more than necessary. When the route of the permit vehicle is known, a greater weight may be allowed. This is due to the fact that if a particular route is specified, the span lengths of bridges encountered can also be determined. It is of interest, therefore, to develop a formula that is a function of both wheelbase and bridge span length. In the future, if TxDOT uses computers to assist in the permitting of trucks, such a formula can be used to assist in specifying the best route for a particular load. With this additional information, heavier loads can be safely granted permits.

By using the Marquardt-Levenberg algorithm, a formula for the allowable distributed load as a function of wheelbase and bridge span length is determined. With this method, the coefficients to a pre-determined characteristic equation are determined by minimizing the sum of the squares of the residuals. Because this is an iterative process, the *SigmaPlot* [9] program is used to perform this task. The formula is based upon a truck with a gage of 1.8 m (6 ft.) and axles with less than 8 tires per axle. This may be modified for different gages by applying the factors in equations (1-2) and (1-3) or (1-4). The general form is identified as:

$$w = \frac{k_1 \cdot L^2 + k_2 \cdot L + \dfrac{k_3}{L} + k_4}{WBL(2L - WBL)} \tag{2-3}$$

where

k_1, k_2, k_3, k_4 = constants for different types of bridges in Table 2-6,

L = span length, m (ft.),

WBL	=	WB, wheelbase, m (ft.) when $WB < L$,
	=	L, span length, m (ft.) when $WB > L$,
w	=	allowable distributed load, kN/m (k/ft.).

The computer algorithm is then used to calculate values of the constants which will produce the best fit to the data. The cases where the wheelbase exceeds the span length are not included in the curve fitting process. For these cases, the maximum wheelbase on the bridge is limited to the span length.

The maximum impact factor is again changed from zero percent to 10 percent to 30 percent. The allowable distributed load, w, is then calculated according to a different impact factor, which results in different constants k_1, k_2, k_3, k_4 as shown in Table 2-6.

As an example, graphs portraying group weights as a function of wheelbase and bridge span length for H15 bridges are in Figures 2-6, 2-7, and 2-8 which are corresponding to impact factors of 0, 10, and 30 percent, respectively. The group weight is determined by multiplying the distributed load determined in equation (2-3) by the wheelbase length.

$$GW = w * WB \tag{2-4}$$

where

GW	=	group weight, kN (k),
w	=	allowable distributed load from equation (2-3), kN/m (k/ft.),
WB	=	wheelbase, m (ft.).

It can be seen in Figures 2-6, 2-7, and 2-8, when the bridge span length is considered, significantly higher weights may be allowed for various span lengths. Although the formulae are determined using data from span lengths of 3.1 to 45.7 m

(10 to 150 ft.) and wheelbases of 1.2 to 36.6 m (4 to 120 ft.), they converge at larger values. Therefore, these formulae may be used for larger wheelbases and span lengths. It is also expected that the higher the assumed maximum impact factor, the less the allowed permit weight, comparatively.

Bridge	Impact	k_1	k_2	k_3	k_4
	0%	29.19	0.0	-7,199	3,606
		(2.0)	(0.0)	(-17,420)	(2,660)
H15	10%	10.03	0.0	-6,529	3,254
		(0.688)	(0.0)	(-15,800)	(2,400)
	30%	23.32	0.0	-4,446	2,440
		(1.67)	(0.0)	(-11,000)	(1,800)
	0%	36.91	0.0	-9,736	4,908
		(2.53)	(0.0)	(-23,560)	(3,620)
H20	10%	32.11	0.0	-8,657	4,433
		(2.20)	(0.0)	(-20,950)	(3,270)
	30%	30.65	0.0	-6,198	3,389
		(2.10)	(0.0)	(-15,000)	(2,500)
	0%	2.780	1,254	14,670	-7,796
		(0.191)	(282)	(35,500)	(-5,750)
HS15	10%	8.339	934	9,422	-4,501
		(0.571)	(210)	(22,800)	(-3,320)
	30%	10.94	712	7,025	-3,389
		(0.750)	(160)	(17,000)	(-2,500)
	0%	9.966	1,388	14,270	-6,847
		(0.683)	(312)	(34,540)	(-5,050)
HS20	10%	9.121	1,268	12,980	-6,236
		(0.625)	(285)	(31,400)	(-4,600)
	30%	14.59	890	8,265	-4,067
		(1.0)	(200)	(20,000)	(-3,000)

Table 2-6: Constants in bridge specific formulae for different bridges.

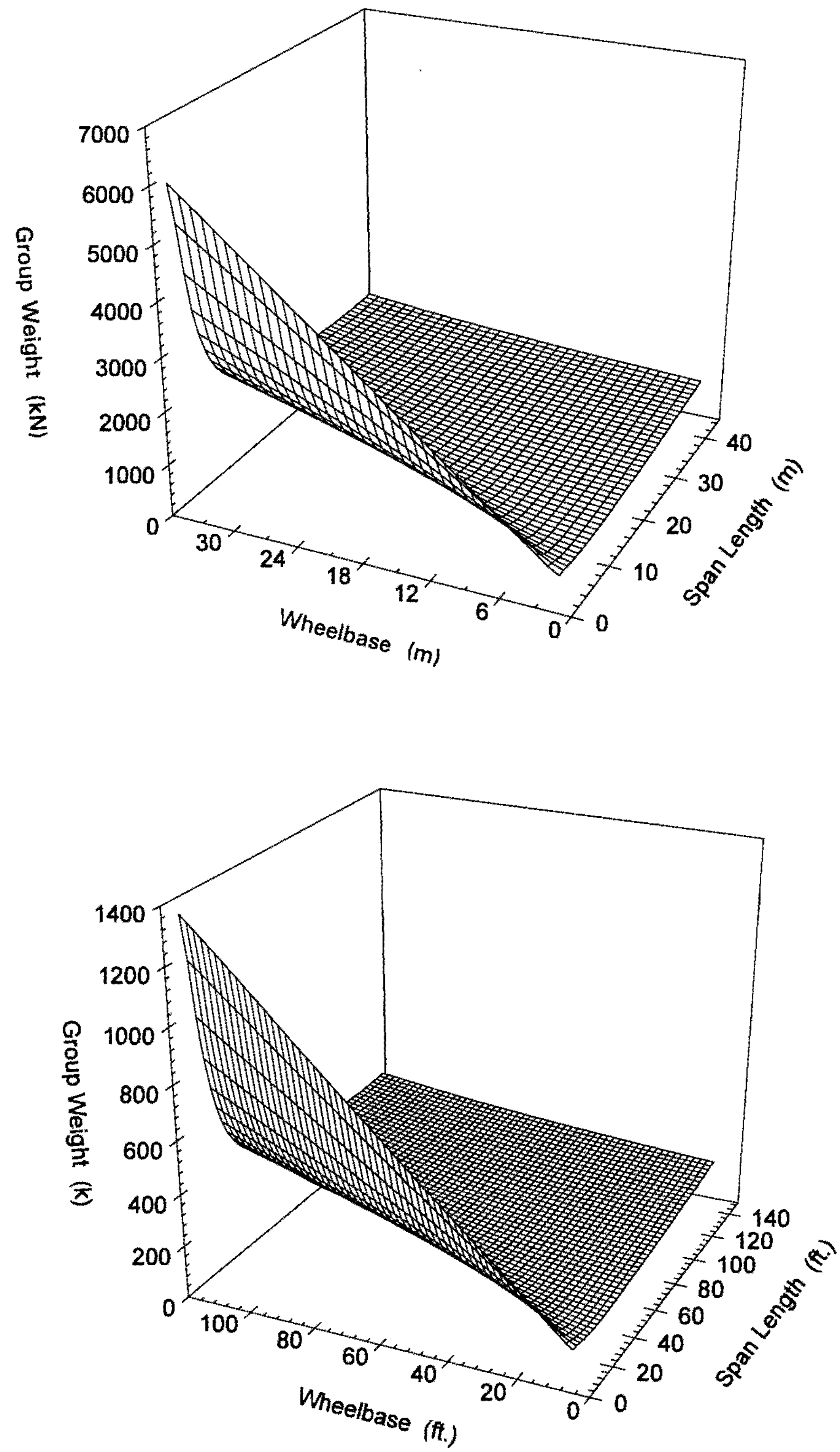

Figure 2-6: Calculated group weight versus wheelbase and bridge span length for a 1.8 m (6 ft.) gage on H15 bridges (I_{max} = 0%).

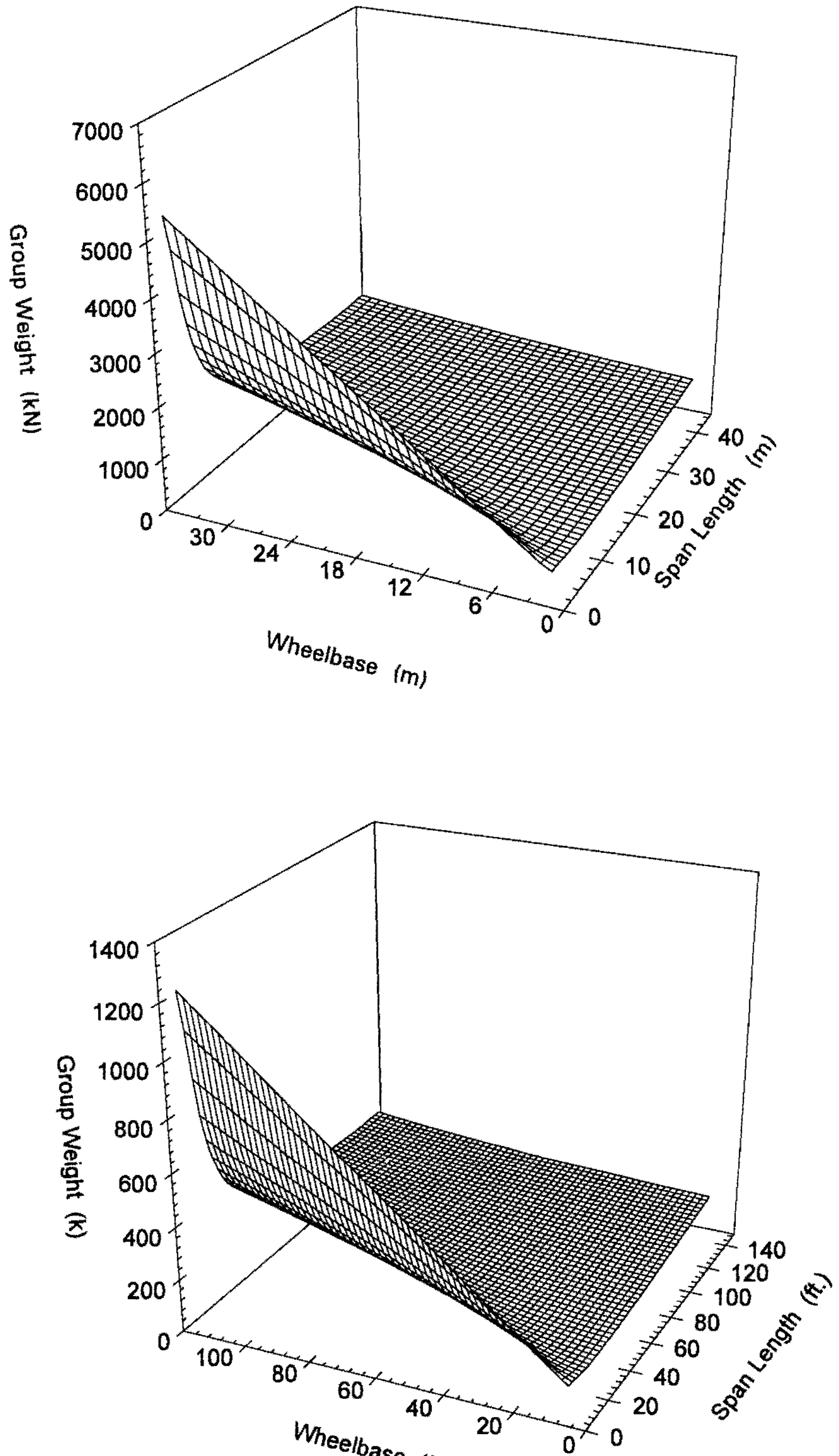

Figure 2-7: Calculated group weight versus wheelbase and bridge span length for a 1.8 m (6 ft.) gage on H15 bridges (I_{max} = 10%).

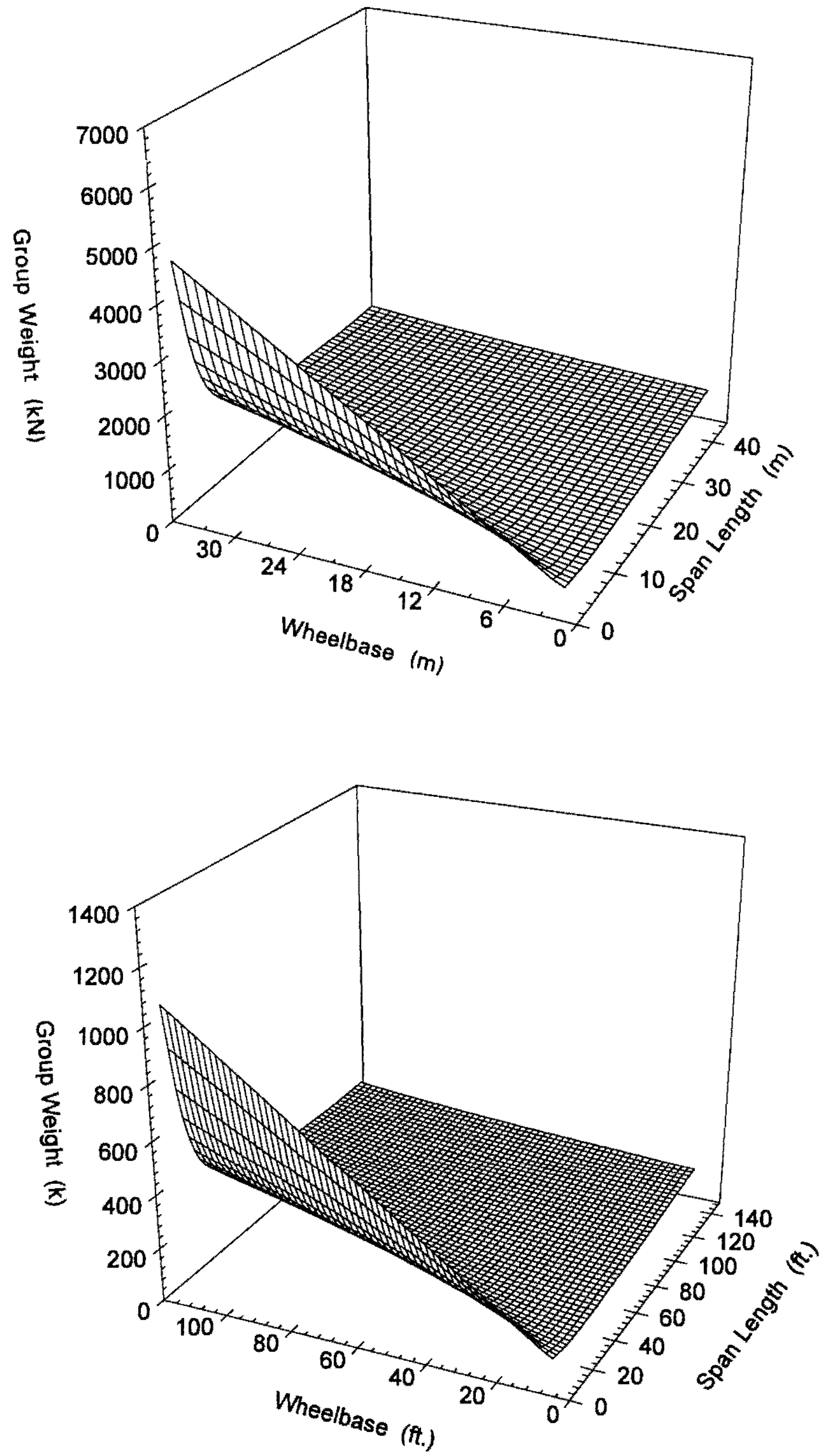

Figure 2-8: Calculated group weight versus wheelbase and bridge span length for a 1.8 m (6 ft.) gage on H15 bridges (I_{max} = 30%).

3. DEVELOPMENT OF H*X* AND HS*X* FORMULAE

3.1 Determining H*X* and HS*X* Formulae for an Unknown Span Length

In the previous sections, two different types of formulae have been developed. The first type does not depend on the bridge span length and can be rearranged into the general form of equation (3-1).

$$GW = (\, a + b\,WB \,)\, X \tag{3-1}$$

where

GW	=	group weight, kN (k),
WB	=	wheelbase, m (ft.),
X	=	design rating of the bridge,
a,b	=	constants depending on the design rating.

Because the general form is linear, it is reasonable to assume that a linear relationship can be developed for the constants *a* and *b* for both the H-type and HS-type formulae. It is important to note that while this relationship is linear, it is not a direct ratio to the design rating. While the average ratio of the *b* terms from equation (2-2) and Table 2-5 for H15 and H20 bridges is 0.792 and is close to the ratio of the design ratings of 0.750, it should not be considered exact. The difference between the two ratios is because the incremental change in the live load is greater than the incremental change in the dead load. The ratio of the total moments, and thus, the ratio of allowable moments, will be closer to unity than the ratio of the live loads.

The equations for the constants *a* and *b* are determined by solving for them in each of the design ratings for both the H-type and the HS-type bridge formulae. Then a linear equation is formed between the two values. For example, in solving for *b* in the general HX formula for 30 percent impact, $15b = 1.4$, or $b = 0.093$, and $20b = 1.77$, or $b = 0.088$ reduces to $b = 0.108 - 0.001*X$. Table 3-1 summarizes the equations for the constants.

Bridge	Impact	*a*	*b*
	0%	$17.83 - 0.034X$	$1.973 - 0.0119X$
		$(4.009 - 0.0077X)$	$(0.1265 - 0.0012X)$
HX	10%	$16.21 - 0.031X$	$1.804 - 0.0172X$
		$(3.645 - 0.007X)$	$(0.1157 - 0.0011X)$
	30%	$12.81 - 0.027X$	$1.684 - 0.016X$
		$(2.88 - 0.006X)$	$(0.108 - 0.001X)$
HSX	0%	0.0	$3.103 - 0.016X$
		(0.0)	$(0.199 - 0.001X)$
$WB < 11.6$ m	10%	$14.45 + 0.0013X$	$2.791 - 0.013X$
($WB < 38$ ft.)		$(3.249 + 0.0003X)$	$(0.179 - 0.0008X)$
	30%	$13.08 - 0.0623X$	$2.136 + 0.0062X$
		$(2.94 - 0.014X)$	$(0.137 + 0.0004X)$
HSX	0%	$35.52 - 0.0342X$	$1.380 - 0.016X$
		$(7.985 - 0.0077X)$	$(0.0885 - 0.001X)$
$WB > 11.6$ m	10%	$32.27 - 0.0756X$	$1.258 - 0.014X$
($WB > 38$ ft.)		$(7.255 - 0.017X)$	$(0.0807 - 0.0009X)$
	30%	$24.64 + 0.0356X$	$1.325 - 0.016X$
		$(5.54 + 0.008X)$	$(0.085 - 0.001X)$

Table 3-1: Linear equations defining constants for general formulae.

Because the constants *a* and *b* are dependent upon the design rating, *X*, solving equation (3-1) for *X* becomes an iterative process. However, this dependency is very slight and substituting either 15 or 20 for *X* in these formulae will result in a design rating within 3 percent of the actual rating.

3.2 Determining H*X* and HS*X* Formulae for a Known Span Length

The process to develop the bridge specific formulae is the same as the general formulae. First of all, equation (2-3) and those constants in Table 2-6 can be simplified into the form shown in equation (3-2).

$$w = \left(\frac{aL^2 + bL + \frac{c}{L} + d}{WBL(2L - WBL)} \right) X \tag{3-2}$$

where

L	=	span length, m (ft.),
WBL	=	WB, wheelbase, m (ft.) when $WB < L$,
	=	L, span length, m (ft.) when $WB > L$,
w	=	allowable distributed load, kN/m (k/ft.),
X	=	design rating of the bridge,
a,b,c,d	=	constants depending on the design rating.

A linear relationship is then created between the two design ratings for each of the constants. Table 3-2 lists the constants of the equations.

Formula	Impact	*a*	*b*	*c*	*d*
	0%	2.262 - 0.0204*X*	0.0	-459.1 - 1.376*X*	225.5 + 0.994*X*
		(0.155 - 0.0014*X*)	(0.0)	(-1111 - 3.33*X*)	(166.3+.733*X*)
H*X*	10%	2.320 - 0.0350*X*	0.0	-442.5 + 0.4835*X*	202.7 - 0.949*X*
		(0.159 - 0.0024*X*)	(0.0)	(-1071+1.17*X*)	(149.5+0.7*X*)
	30%	1.883 - 0.0175*X*	0.0	-281.8 - 1.405*X*	143.4 - 1.36*X*
		(0.129 - 0.0012*X*)	(0.0)	(-682 - 3.4*X*)	(105 + *X*)
	0%	0.7588 - 0.0623*X*	126.3 - 2.85*X*	1771- 52.89*X*	-835.1+24.63*X*
		(0.052 - 0.0043*X*)	(28.4 - 0.64*X*)	(4287 - 128*X*)	(-616+18.17*X*)
HS*X*	10%	0.861 - 0.0204*X*	59.16 + 0.22*X*	556.1+ 4.13*X*	-264.4 - 2.350*X*
		(0.059 - 0.0014*X*)	(13.3 + 0.05*X*)	(1370 + 10*X*)	(-195 - 1.733*X*)
	30%	0.730	56.49 - 0.5916*X*	633.1- 10.99*X*	-295.6 + 4.61*X*
		(0.05)	(12.7 - 0.133*X*)	(1532 - 26.6*X*)	(- 218 + 3.4*X*)

Table 3-2: Linear equations defining constants for bridge-specific formulae.

There are three main uses for the new H*X* and HS*X* formulae. First, the general formulae will give an allowable group axle load for a given wheelbase and bridge design rating on an unknown bridge span length. The H15 formula may be used if the bridge design ratings are also unknown. The "bridge-specific" H*X* and HS*X* formulae will result in the greatest allowable load for a specific bridge with known span length and design rating. These formulae are still applicable if a bridge's deteriorated state warrants a change in its design rating. Equation (3-1) can also be used to reclassify any real vehicle as an equivalent H*X* or HS*X* permit truck. This concept would greatly simplify the procedure of determining which bridges a particular truck may cross.

3.3 Extrapolation of H*X* and HS*X* Formulae for Additional Design Ratings

Because both the H*X* and HS*X* formulae have been derived by forming a linear relationship of the constants between the design ratings of 15 and 20, they should be checked for design ratings outside of this range. Even though the relationships between the constants are linear, the relationships between the axle group weights are not. A check for other design ratings becomes necessary if either the HS25 design loading or the deteriorated design rating system are to be used in the state of Texas. Therefore, both formulae have been evaluated at design ratings of 10 and 25.

Since the only Texas bridges with design ratings other than 15 or 20 are a few H10 bridges designed by a system very different from that in the *Standard Specifications for Highway Bridges* [2], it is not possible to check the new formulae for actual bridges. However, it is possible to evaluate these formulae based on a comparison between the live load ratios and formula ratios. Figure 3-1 compares these ratios to a design rating of 20 for the H*X* general formula. It is noted that proposed permit weights with a maximum 10 percent impact factor is used in this extrapolation analysis.

In Fig. 3-1, live load ratios are merely the weight of the design truck in question divided by the weight of the H20 truck. The formula ratios are the ratios of the group weights for that design rating to the formula weight for a design rating of 20. The interest in this graph is not necessarily how similar the formula ratios are to the live load ratios, but that the behavior between the two are nearly identical for design ratings of 10 and 25 as for 15.

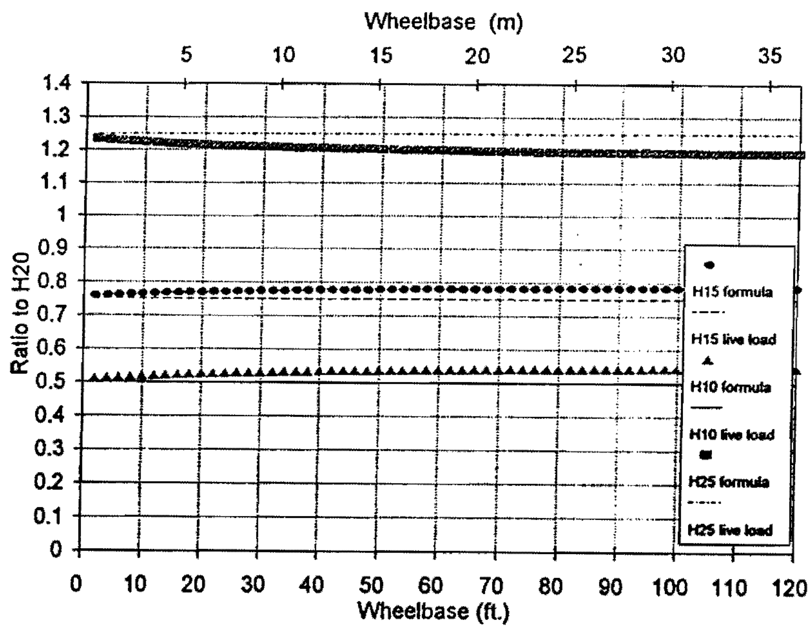

Figure 3-1: Extrapolation of H*X* general formula for additional design ratings.

For all three design ratings, the live load ratio and the formula ratio are approximately the same value for small wheelbases. As the wheelbase increases, the formula ratio gradually drifts toward unity. This phenomena may be explained because the incremental change in the bridge dead load gradually decreases while the incremental change in the live load remains constant. Because the comparison of the formula ratios for design ratings of 10 and 25 are very close to those for the design rating of 15, the H*X* general formula should be valid for the extrapolated design ratings and 25. Figure 3-2 shows that the HSX general formula may also be extrapolated for other design ratings.

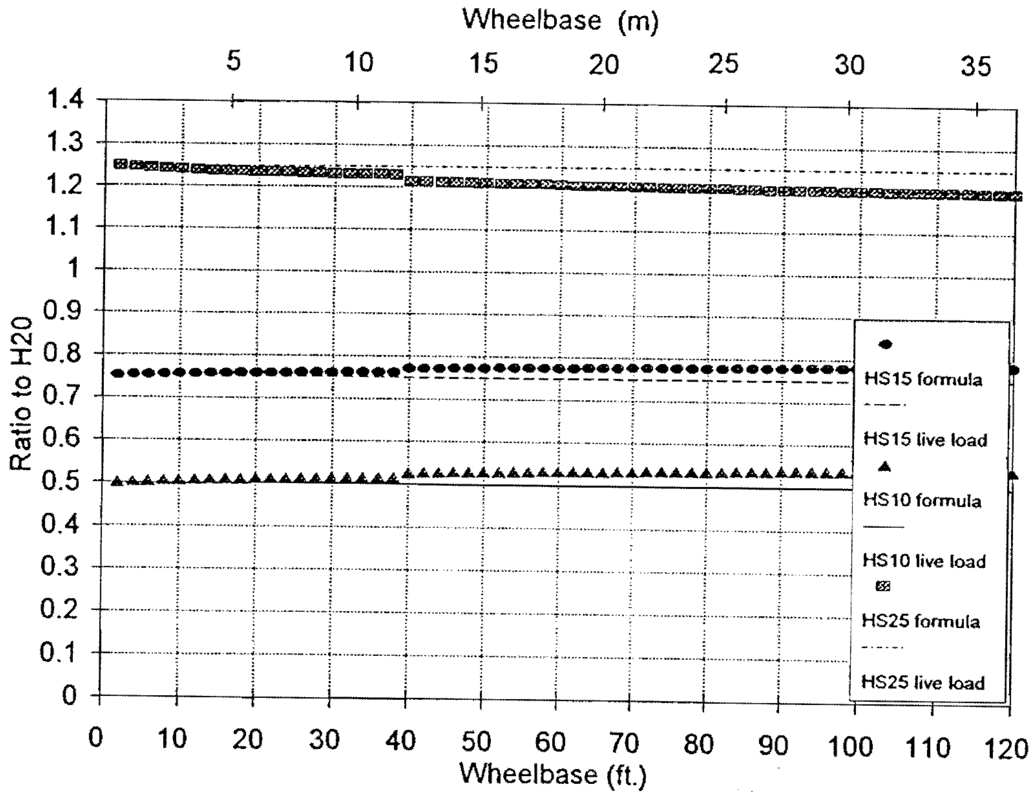

Figure 3-2: Extrapolation of HSX general formula for additional design ratings.

The abrupt change of equations at 11.6 m (38 ft.) has a more profound effect on design ratings of 10 and 25 than for a design rating of 15, but exhibits the same behavior and deviations. This comparison for the extrapolated design ratings may also be done for the bridge-specific HX and HSX formulae. Because the denominator in equation (3-2) cancels out all ratios, the comparisons may be made by varying only the span length. Figures 3-3 and 3-4 show the result of the comparisons for the HX and HSX bridge-specific formulae. Although both of these graphs portray a more asymptotic behavior, the comparisons between the live load ratio and the formula ratio are very similar for the three design ratings. Therefore, it is reasonable to assume that the general and bridge-specific formulae for both the H*X* and HS*X* design types may be extrapolated for design ratings between 10 and 25.

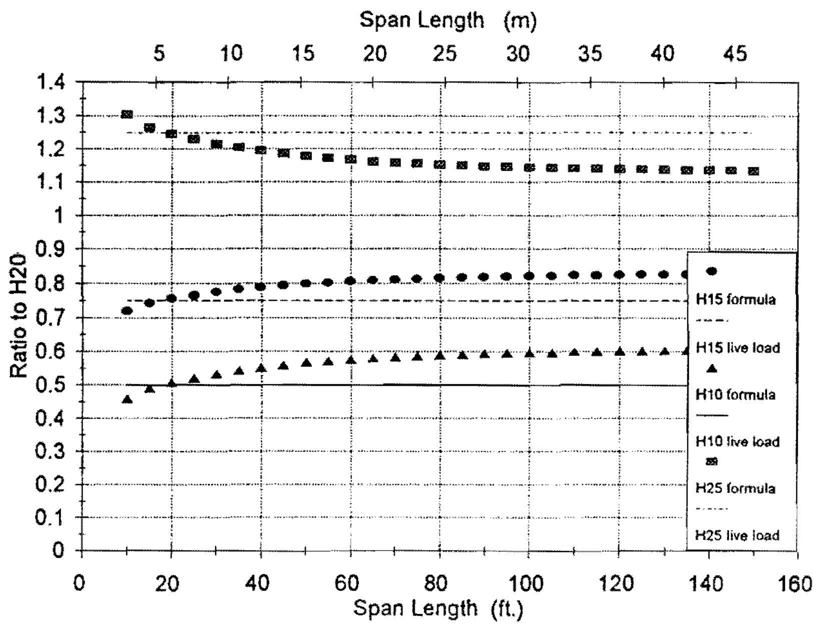

Figure 3-3: Extrapolation of H*X* bridge-specific formula for additional design ratings.

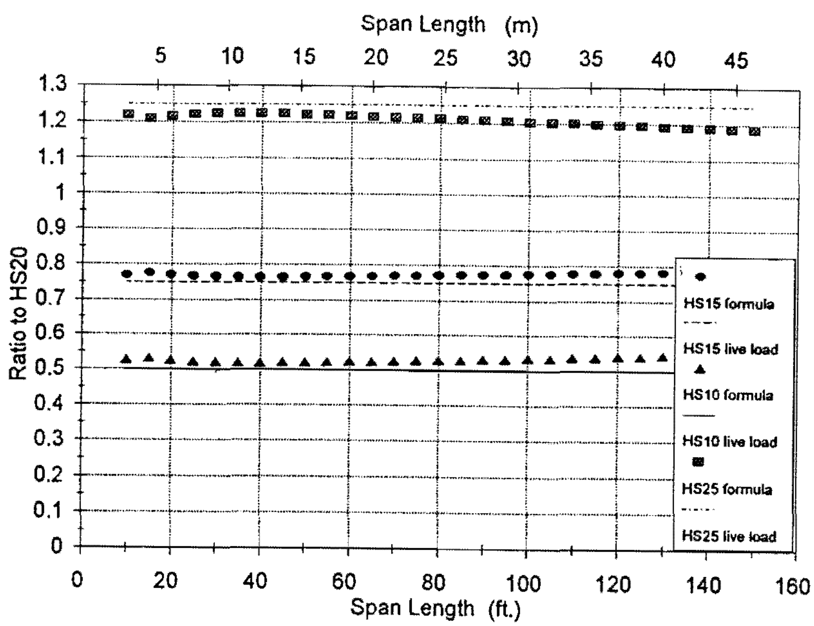

Figure 3-4: Extrapolation of HS*X* bridge-specific formula for additional design ratings.

3.4 H*X* and HS*X* Formulae Including Correction Factors

Because the H*X* and HS*X* formulae already derived in equations (3-1) and (3-2) are based on a distributed load, the effect of β on simple span bridges will be to increase the wheelbase of the permit truck by a factor $1/\beta$ [3]. For simplification, this will be summarized into the term WB_{rev} defined in equation (3-3).

$$WB_{rev} = \frac{WB}{\beta} \tag{3-3}$$

where

WB_{rev} = revised wheelbase, m (ft.),

WB = vehicle wheelbase, m (ft.),

β = correction factor for concentrated loadings
= defined in TTI Report 1266-4F [3] for simple span bridges,
= 1.0 for continuous span bridges.

When β is combined with the correction factors in equation (1-2), a complete form of the H*X* and HS*X* formulae can be determined. The revised general formula is shown in equation (3-4).

$$GW_{rev} = \frac{GW}{n} \sum_{i=1}^{n} \frac{1}{R_i * S_i} \tag{3-4}$$

where

GW_{rev} = revised axle group weight, kN (k),

S_i = reduction factor accounting for each axle with more than four tires per axle,
= 1.0 for axles with four tires or fewer,
= 0.96 for axles with eight or more tires,

n = number of axles,

RF = reduction factor accounting for gages wider than 1.8 m (6 ft.);

GW = axle group weight, kN (k);
calculated from either equations (3-5) or (3-6) as:

$$GW = \left(a + b * WB_{rev} \right) X \quad \text{(All Bridges)} \tag{3-5}$$

$$GW = w * WB_{rev} \quad \text{(Bridge Specific Formula)} \tag{3-6}$$

where

w = allowable distributed load, kN/m (k/ft.);
= calculated from equation (3-7) as:

$$w = \left(\frac{aL^2 + bL + \frac{c}{L} + d}{WBL(2L - WBL)} \right) X \tag{3-7}$$

where

L = span length, m (ft.),

WBL = WB_{rev}, revised wheelbase, m (ft.) when $WB < L$,
= L, span length, m (ft.) when $WB > L$,

X = design rating of the bridge,

a,b,c,d = constants defined in Tables 3-1 and 3-2.

The factor β was developed in TTI Report 1266-F [3] to provide a better estimate of the distributed loading to that of the actual wheel group concentrated loads. The current TxDOT procedure is to add 1.2 m (4 ft.) to the wheelbase to compensate for the difference in the resulting bending moments between concentrated wheel loads and distributed loading.

3.5 Comparisons Between Proposed Formulae and Current Regulations

One major concern of the proposed formulae is how they compare to the current TxDOT overweight permit regulations. Fig. 2-2 shows that the proposed formula (assuming $I_{max} = 30\%$) evaluated for the H15 design without any additional correction factors is approximately 0.67 times the current regulations for nearly all wheelbases. This is the same value evaluated from the reduction factor equation (equation (2-1)) for reducing the 4.9 m (16 ft.) gage to a 1.8 m (6 ft.) gage. Nearly the same result could be obtained by using the *S*/7 distribution factor for two loaded lanes to calculate the allowable permit loads.

If the axle configurations used to derive the current criteria are evaluated according the revised general H*X* and H*SX* formulae with the correction factor for concentrated loadings, the results compare more favorably with current TxDOT overweight permit regulations. The legal 2, 3, and 4 axle groups evaluated by the proposed formula provide heavier axle group weights than the current restrictions (see Figs. 3-5 to 3-7). As the wheelbase increases to 5 and 6 axle groups, the current restrictions start exceeding the load allowed by the proposed formulae, as shown in Figs. 3-8 and 3-9. However, the current restrictions only exceed the proposed formula for H15 bridges. At the legal 6 axle groups, this exceedance is 16 percent.

While both procedures can be used to obtain similar results, the proposed formulae are based on a generalized analysis of bridge loading and member strength. The current restrictions are based on a compilation of past permitted vehicles.

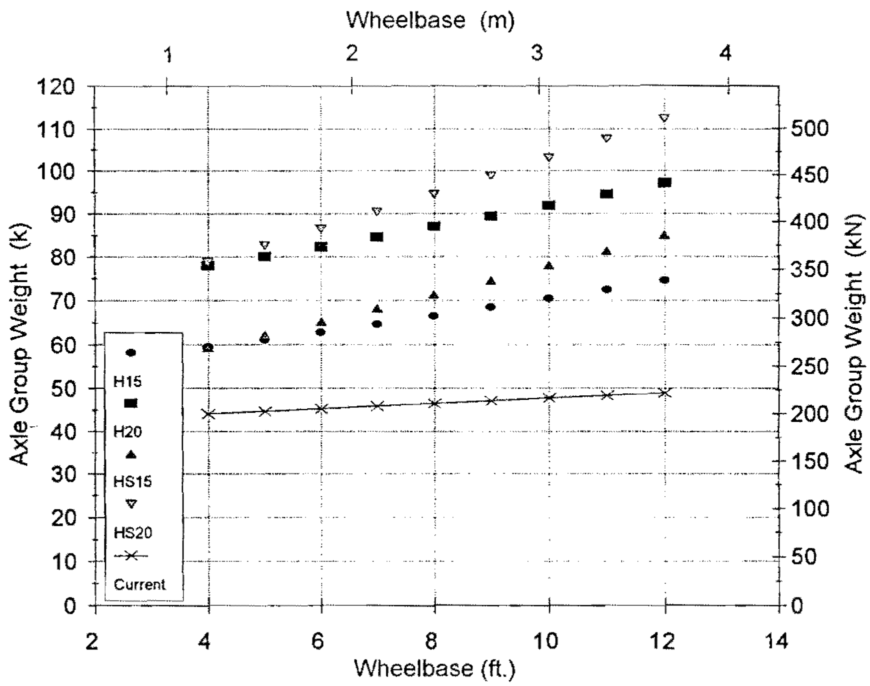

Figure 3-5: Legal 2 axle groups evaluated by proposed general formula.

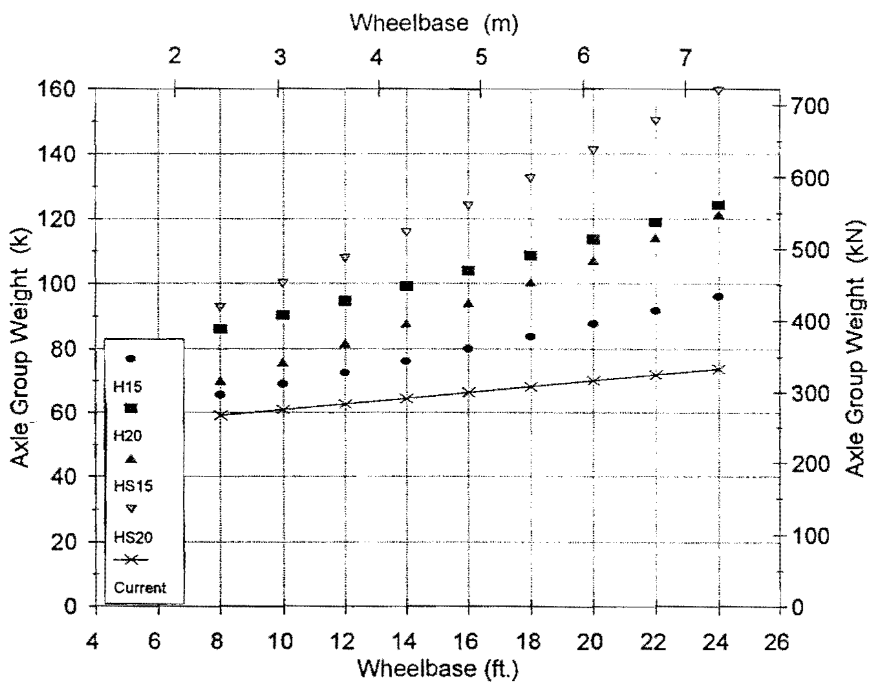

Figure 3-6: Legal 3 axle groups evaluated by proposed general formula.

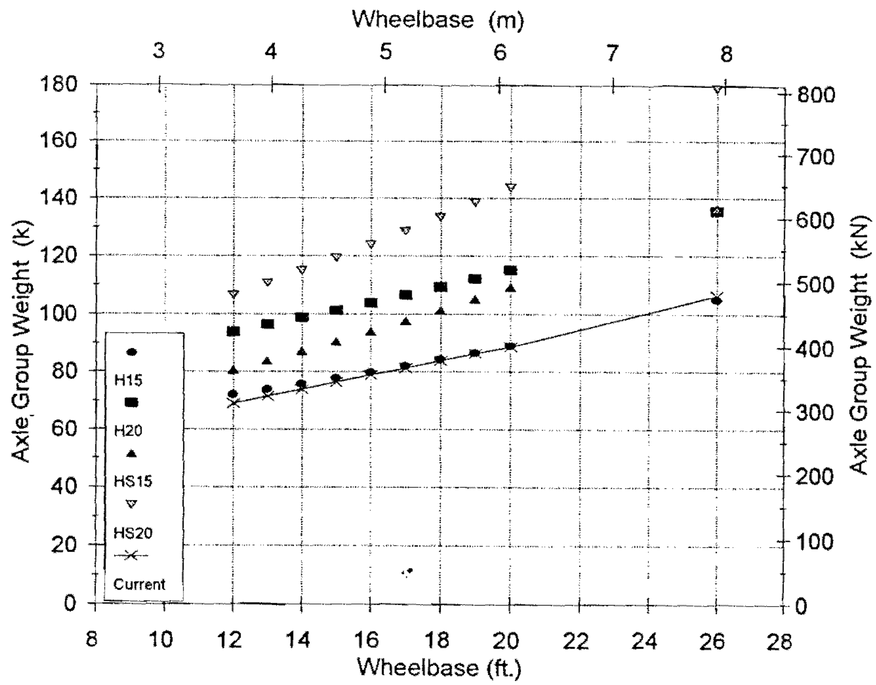

Figure 3-7: Legal 4 axle groups evaluated by proposed general formula.

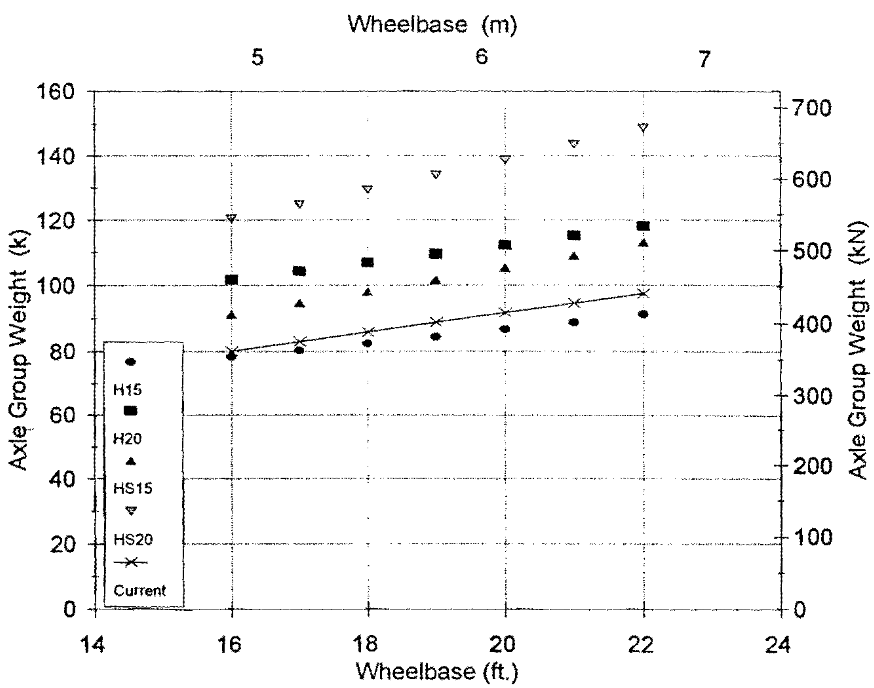

Figure 3-8: Legal 5 axle groups evaluated by proposed general formula.

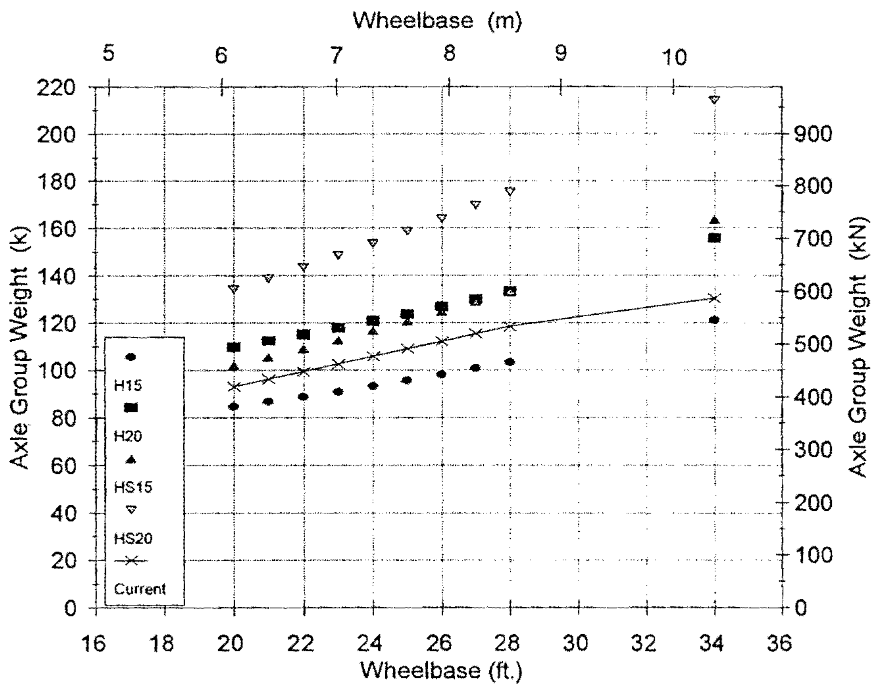

Figure 3-9: Legal 6 axle groups evaluated by proposed general formula.

4. EVALUATION OF CONTINUOUS SPAN BRIDGES

4.1 Introduction to Continuous Span Bridges

The H*X* and HS*X* formulae have been derived for simple span bridges. Continuous span bridges contain interior supports where negative bending occurs. A vehicle with axles near the center of the spans adjacent to the support will maximize the stress at this point. Therefore, the assumption that the critical axle configurations may be approximated by a distributed load is not valid for continuous span bridges. In order to ensure that the proposed restrictions provide adequate protection for continuous spans, these formulae have been checked with nine reinforced concrete slab bridges.

Many reinforced concrete slab bridges were designed for H-type rather than HS-type live loads. The lighter H-type loading results in lower negative bending moment capacity over the interior supports. Consequently, the design strength of these types of bridges can be greatly exceeded by vehicles with wheel patterns that generate more negative moment. These slab bridges, which are typical of those designed by TxDOT in the 1940's, 1950's, and 1960's, are thought to be critical because of their short spans and narrow widths. A schematic drawing of the Cameron 50 bridge is shown in Fig. 4-1, while Tables 4-1 and 4-2 contain geometrical information for all nine bridges.

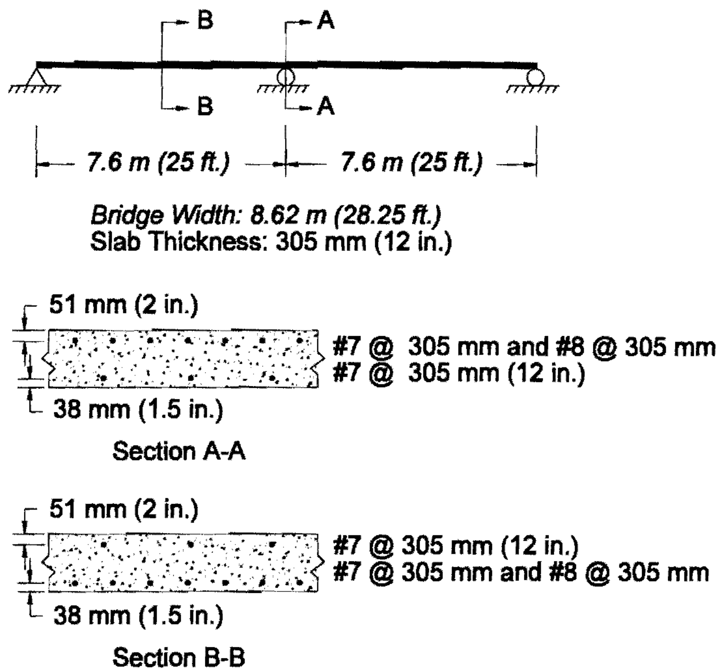

Figure 4-1: Cameron 50 bridge.

Bridge	Design Type	Span Lengths (m)	Span Lengths (ft.)	Date Const.
Cameron 50	H15	7.6-7.6	25-25	1965
Cameron 80	H15	7.6-9.2-7.6	25-30-25	1965
San Saba	H15	7.9-7.9-7.9-7.9	26-26-26-26	1963
CS 0-38-50	H20	7.6-7.6	25-25	1944
CS 18-28-80	H20	7.6-9.2-7.6	25-30-25	1944
CS	H20	7.6-9.2-9.2-7.6	25-30-30-25	1944
CS-0-50	HS20	7.6-7.6	25-25	1949
CS-0-80	HS20	7.6-9.2-7.6	25-30-25	1949
CS-0-110	HS20	7.6-9.2-9.2-7.6	25-30-30-25	1949

Table 4-1: Specifications for continuous span bridges.

Bridge	Deck Width		Roadway Width		Slab Thickness	
	(m)	(ft.)	(m)	(ft.)	(mm)	(in.)
Cameron 50	8.62	28.25	7.93	26.00	304.8	12.00
Cameron 80	8.62	28.25	7.93	26.00	304.8	12.00
San Saba	8.03	26.33	7.32	24.00	304.8	12.00
CS 0-38-50	12.20	40.00	11.59	38.00	362.0	14.25
CS 18-28-80	9.50	31.17	8.54	28.00	368.3	14.5
CS	9.50	31.17	8.54	28.00	368.3	14.5
CS-0-50	12.20	40.00	11.59	38.00	355.6	14
CS-0-80	12.20	40.00	11.59	38.00	355.6	14
CS-0-110	12.20	40.00	11.59	38.00	355.6	14

Table 4-2: Widths and thicknesses for continuous span bridges.

4.2 H*X* and HS*X* General Formulae Applied to Continuous Span Bridges

In order to apply the H*X* and HS*X* general formulae to the bridges in Tables 4-1 and 4-2, the operating level moment capacities need to first be calculated at critical bridge locations. These critical locations are places where the positive and negative maximum moments occur, which is near midspans and at interior supports. The working stress method used in *The Manual for Maintenance Inspection of Bridges* [4] is based on the maximum allowable stress of either the concrete or steel reinforcement. The lower moment from equations (4-1) and (4-2) controls the capacity at that location.

$$M_{fs} = A_s f_s j d \tag{4-2}$$

$$M_{fc} = 0.5 f_c j k b d^2 \tag{4-1}$$

where

M_{fc} = moment capacity of concrete, kN-m/m (k-ft/ft),

M_{fs} = moment capacity of steel, kN-m/m (k-ft/ft),

f_c = allowable stress for concrete;
= 13.1 kPa (1.8 ksi),

f_s = allowable stress for steel;
= 193 kPai (28 ks),

j, k = concrete section factors,

b = width of cross-section, mm (in.),

A_s = area of steel, mm^2 (in^2),

d = depth to tension steel, mm (in.).

The operating level moment capacity, M_{OL}, is determined by multiplying the lesser of M_{fc} and M_{fs} by the effective width. The effective width is used to approximate a nonuniform longitudinal bending stress distribution in the slab by a constant distribution. This effective width may be a value for a single line of wheels, designated as E, or for a lane loading, E_L. Continuous span bridges are analyzed using E_L which is two times E. Because the effective width is a linear approximation of the actual stress distribution, E_L will be calculated by three different methods. Extensive finite element studies have been performed by the Texas Transportation Institute [3] to more accurately define E_L for the bridges in question. Values resulting from this study have been used as a comparison to the effective widths given by the AASHTO [2] and the Load Resistance Factor Design method [10]. AASHTO's current formula for a lane loading is shown again in equation (4-3) as:

$$E_L = 2(1.2 + 0.018L) < 4.3 \quad \left(E_L = 2(4.0 + 0.06L) < 14.0\right) \qquad (4\text{-}3)$$

where

L = span length, m (ft.),

E_L = effective width, m (ft.).

The Load Resistance Factor Design (LRFD) formula, based on finite element studies of typical slab bridges in the United States, is given in equation (4-4) as:

$$E_L = 0.305 + 0.152\sqrt{LW} \quad \left(E_L = 1.00 + 0.50\sqrt{LW}\right) \tag{4-4}$$

where

L = bridge span length, m (ft.),

W = bridge width, m (ft.).

Although effective widths given by equation (4-4) are much closer to those found by TTI [3], researchers found that this formula substantially underestimates E_L for many wider bridges. TTI found the effective width to be a function of the position of the truck within its lane, the number of axles, vehicle gage, span length, and width of the bridge. The finite element effective widths used to evaluate the HX and HSX general formulae have been determined at the center of the lane.

The influence line for the bridge location in question is then determined. An influence line is a graphical representation of the resulting forces at a particular point when an applied unit load is moved across the bridge. The example in Fig. 4-2 shows the influence line for the moment at the first interior support. Superposed on this influence line is a critical axle configuration consisting of two 5-axle groups. This theoretical truck is positioned so that each of the 5-axle groups is centrally located on the point which will create the largest moment at the interior support. Considering that each axle must be at least 1.2 m (4 ft.) apart, the total wheelbase can be calculated by adding 4.9 m (16 ft.) to 6.45 m (21.14 ft.).

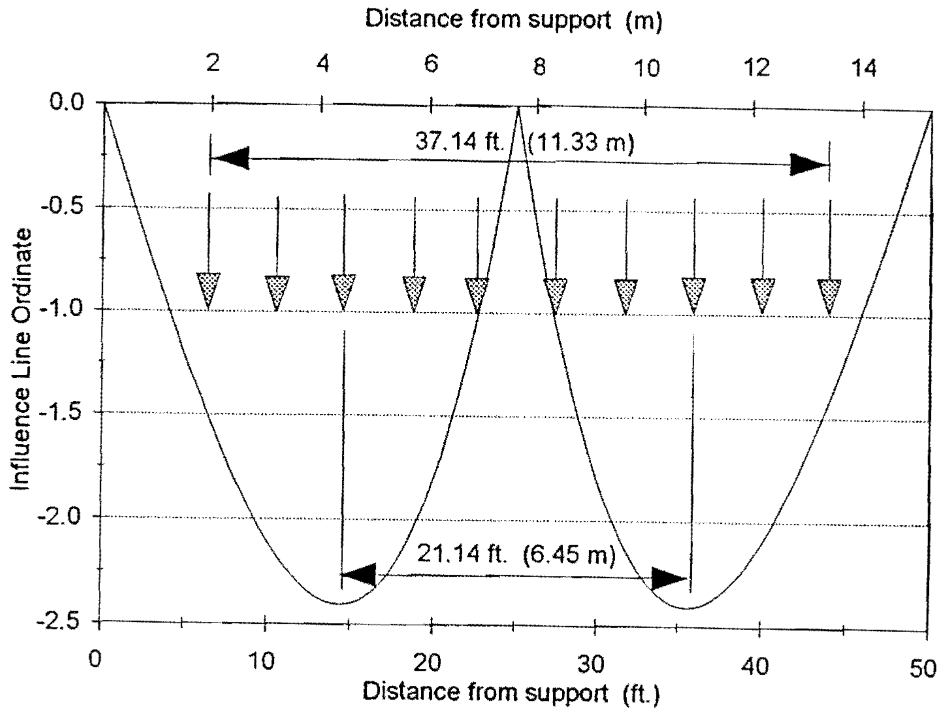

Figure 4-2: Influence line at interior support and critical axle configuration for Cameron 50.

When the total wheelbase of each critical axle configuration is determined, it is then substituted into the general simple span formula, equation (3-5), for the appropriate design type. For the example in Fig. 4-2, the proposed group weight is 417.7 kN (93.90 kips). This weight is distributed evenly to the ten axles.

Several axle configurations with different wheelbases are developed for each critical point. Each configuration is analyzed using an influence line program, and a maximum moment for each case is determined. This maximum moment due to the influence line analysis (M_{IL}) is then compared to the operating level moment capacities using the AASHTO ($M_{OL\ AASHTO}$), LRFD ($M_{OL\ LRFD}$), and finite element center-of-lane ($M_{OL\ FEM}$) effective widths. Overstress ratios (*OSR*'s) are then determined for the AASHTO, LRFD, and FEM cases using equation (4-5). *OSR*'s over 1.00 indicate that the proposed load will cause stresses which exceed the maximum allowable.

$$OSR = \frac{M_{IL}}{M_{OL}} \tag{4-5}$$

The group axle weight is then adjusted by dividing it by the overstress ratio. If unsymmetrical groups of axles are expected to cause the maximum moment, the influence line analysis is repeated until the maximum M_{IL} is found. The final adjusted group axle weight is plotted against the general simple span formula. The H15 plots for the AASHTO, LRFD, and FEM methods are shown in Figs. 4-3 to 4-6. Figures 4-7 to 4-14 show similar plots for H20 and HS20 bridges. Fig. 4-6 shows that the positive moment within the span controls for wheelbases less than 6.1 m (20 ft.), while greater wheelbases cause the negative moment at the interior support to control.

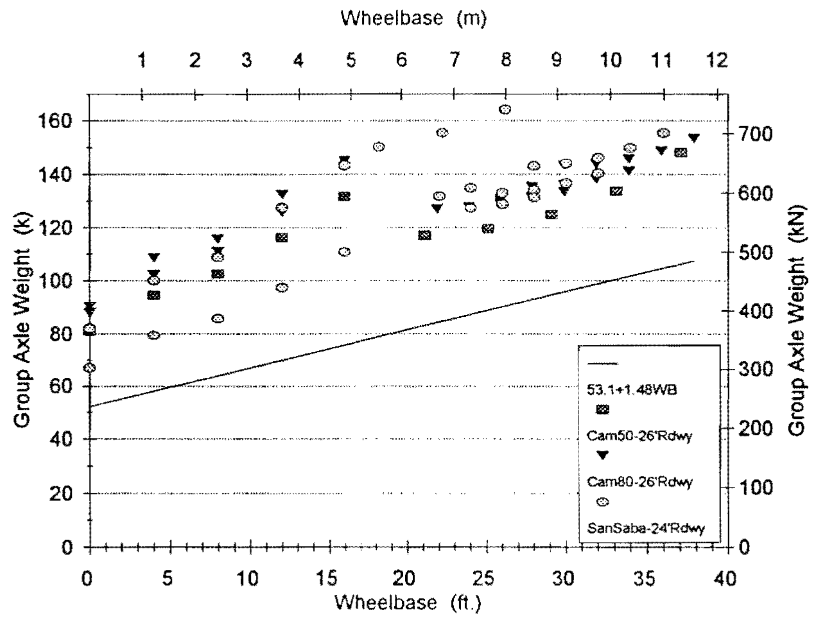

Figure 4-5: Group axle weight versus wheelbase for FEM effective widths on H15 bridges.

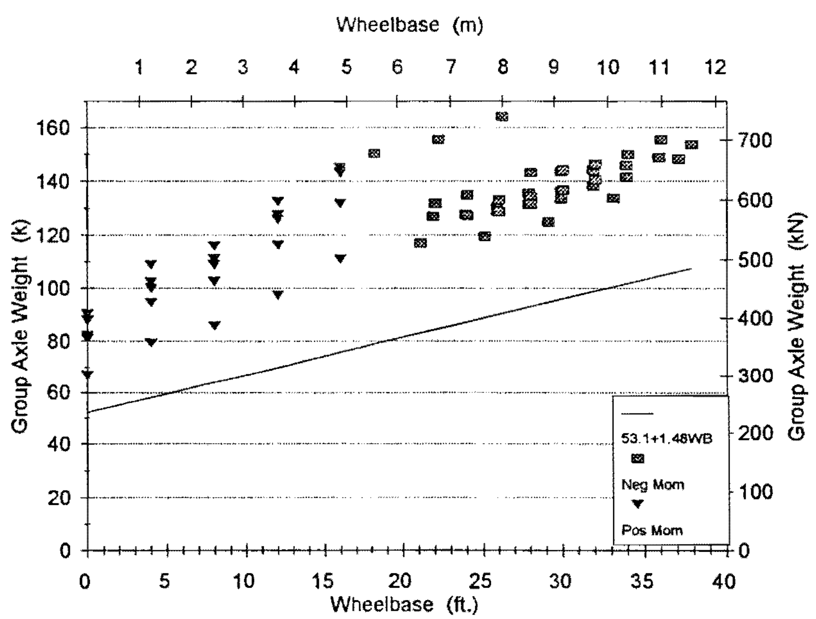

Figure 4-6: Group axle weight versus wheelbase for positive and negative moments on H15 bridges.

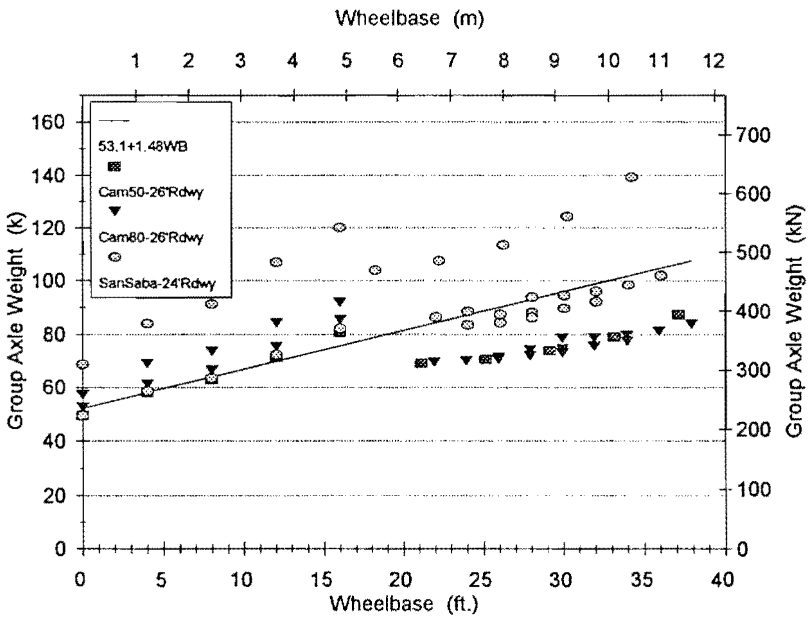

Figure 4-3: Group axle weight versus wheelbase for AASHTO effective widths on H15 bridges.

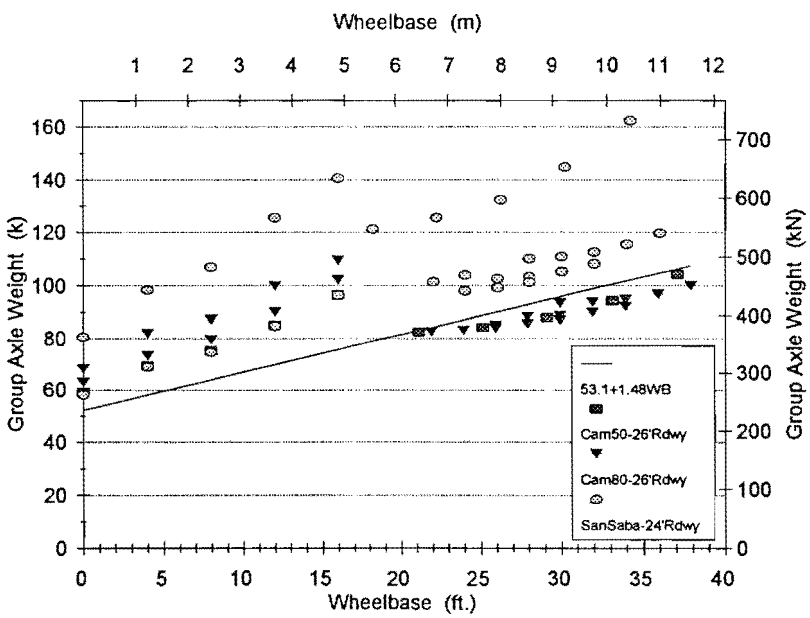

Figure 4-4: Group axle weight versus wheelbase for LRFD effective widths on H15 bridges.

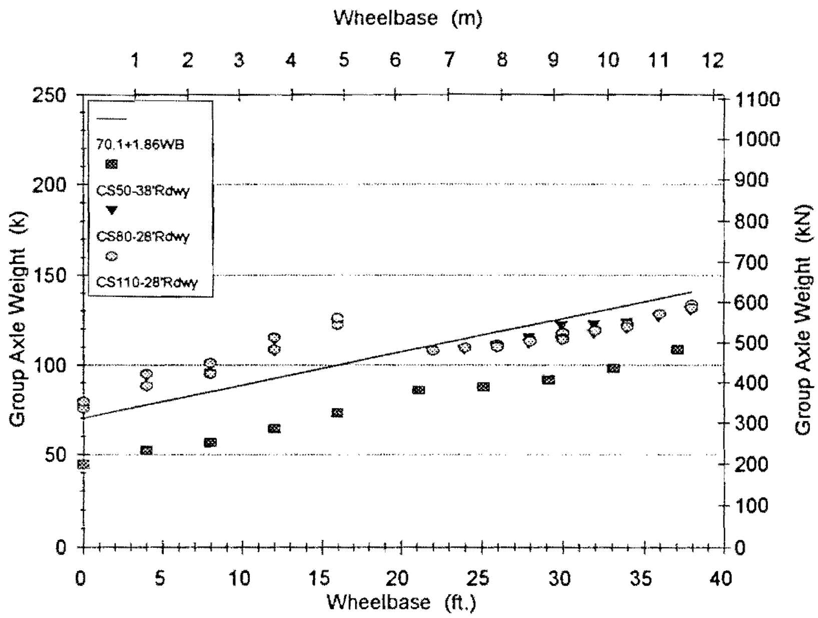

Figure 4-7: Group axle weight versus wheelbase for AASHTO effective widths on H20 bridges.

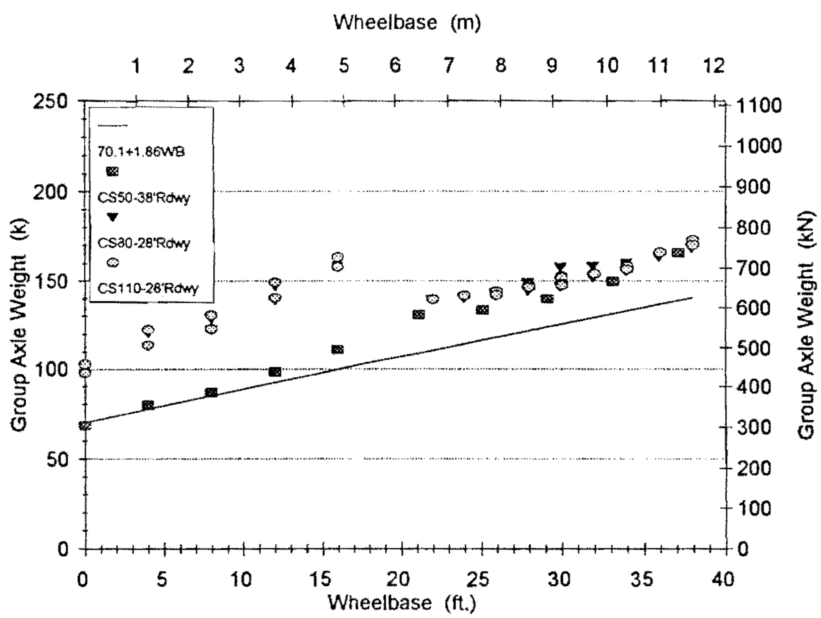

Figure 4-8: Group axle weight versus wheelbase for LRFD effective widths on H20 bridges.

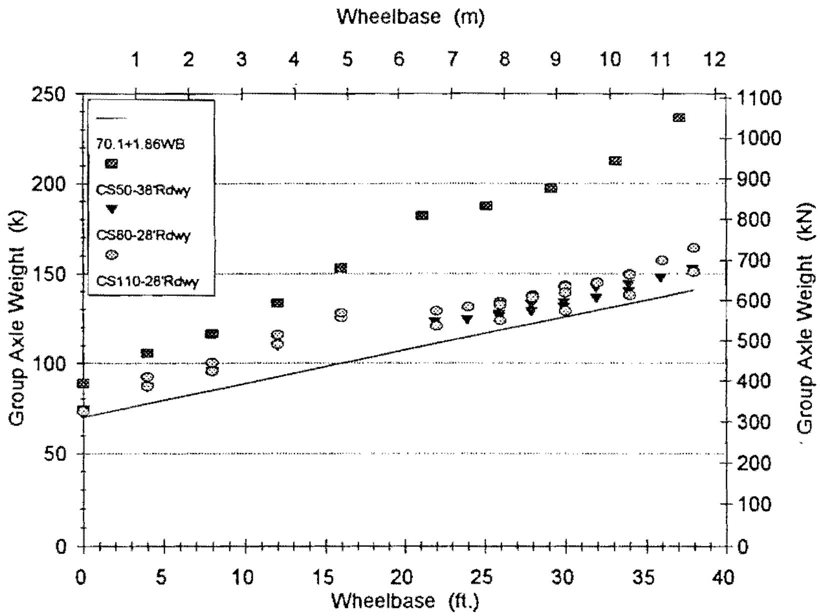

Figure 4-9: Group axle weight versus wheelbase for FEM effective widths on H20 bridges.

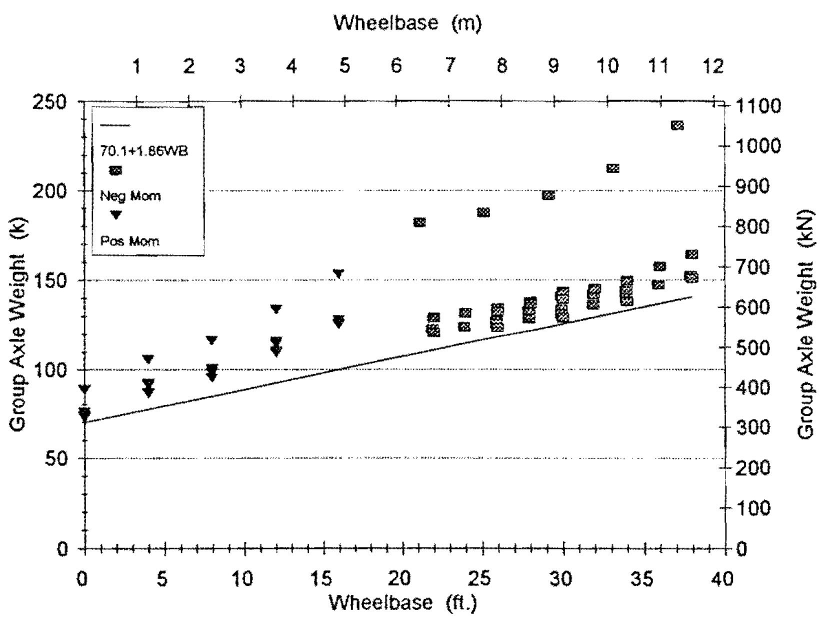

Figure 4-10: Group axle weight versus wheelbase for positive and negative moments on H20 bridges.

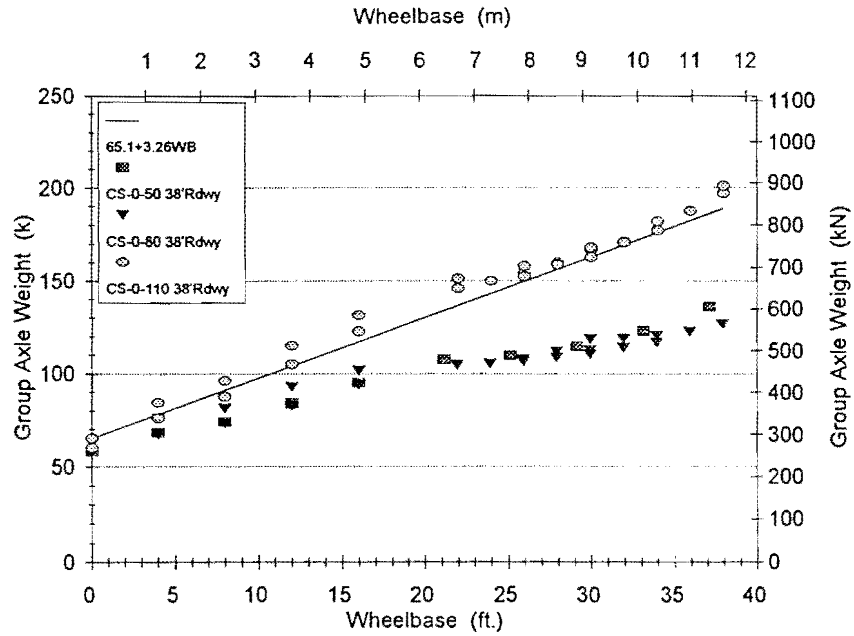

Figure 4-11: Group axle weight versus wheelbase for AASHTO effective widths on HS20 bridges.

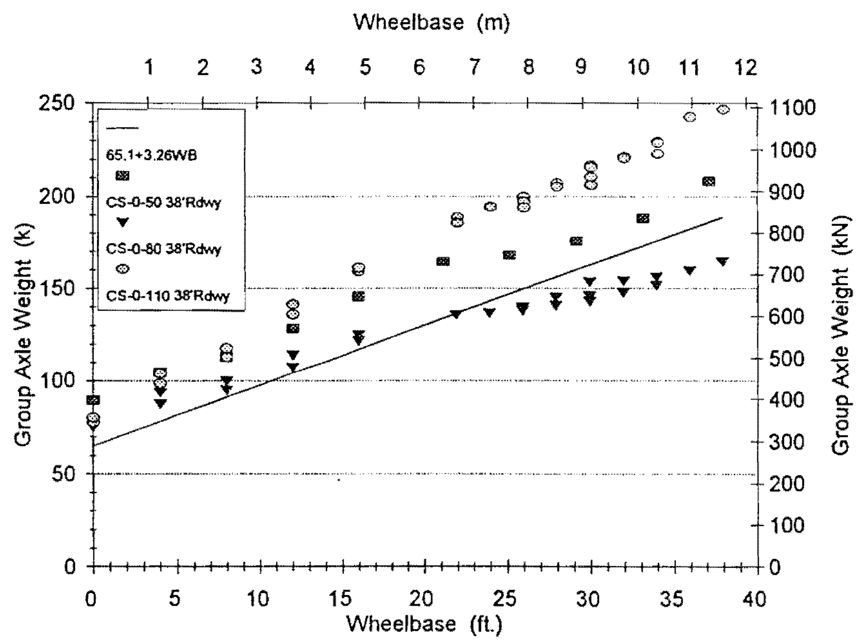

Figure 4-12: Group axle weight versus wheelbase for LRFD effective widths on HS20 bridges.

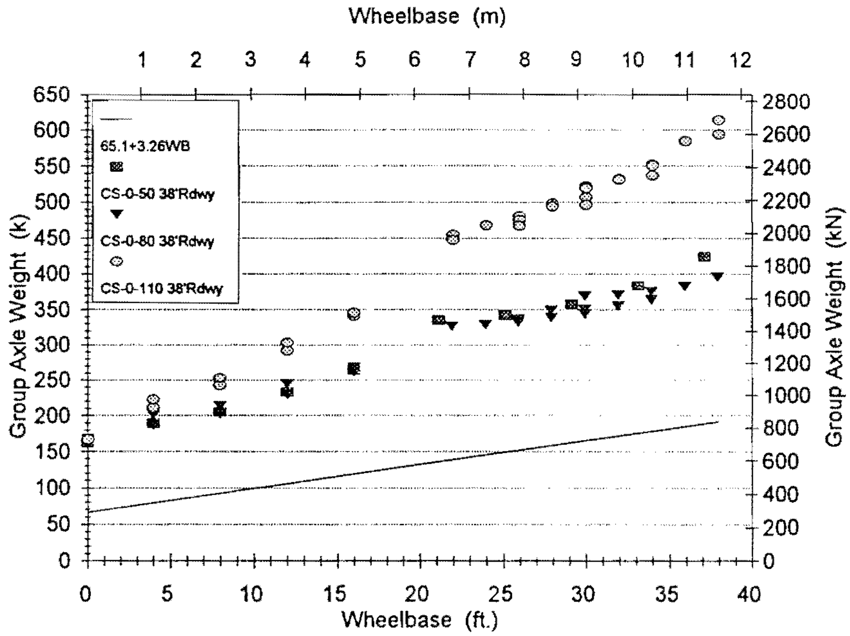

Figure 4-13: Group axle weight versus wheelbase for FEM effective widths on HS20 bridges.

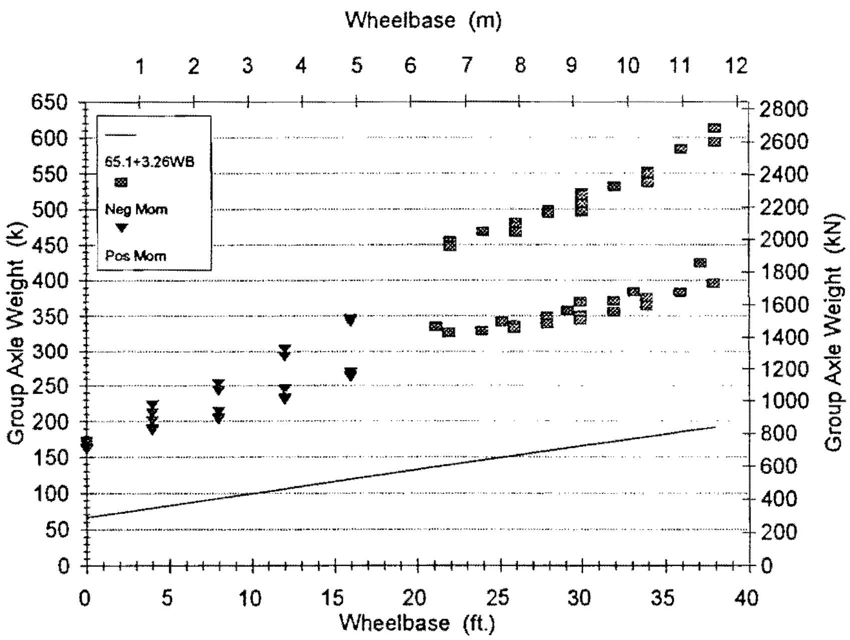

Figure 4-14: Group axle weight versus wheelbase for positive and negative moments on HS20 bridges.

4.3 Summary of Results for Continuous Span Bridges

Figures 4-3 to 4-14 show that the proposed H*X* and HS*X* formulae are adequate for use with continuous span bridges. These figures also show that these bridges actually have a greater moment capacity than the calculated AASHTO or LRFD capacities. There are many negative moment points below the proposed formula line on each of the AASHTO graphs (Figs. 4-3,4-7,4-11). Analysis with this method alone would indicate that the proposed formulae should be reduced by as much as 33 percent for these bridges. The LRFD Figs. 4-4, 4-8, and 4-12 are similar to the AASHTO graphs but show 15-20 percent more capacity. The overstress ratios for the negative moment regions with the LRFD effective widths do not exceed 1.12. However, the finite element effective widths show that the actual moment capacities at the lane center line are at least 25 percent higher than those allowed by the general H*X* and HS*X* formulae. Although the effective widths, and thus the moment capacities, drop considerably as the load moves toward the edge of the bridge [3], the proposed formulae should be adequate for all reinforced concrete continuous span bridges. The much greater capacities of the CS 0-38-50 bridge and HS20 bridges are due to the fact that the effective widths are much greater for wider bridges.

5. APPLICATION EXAMPLES USING PROPOSED FORMULAE

To better demonstrate how to use the proposed H*X* and HS*X* formulae, the truck in Fig. 1-4 will be reexamined in the following example problems. The first assumes the truck will cross many bridges of unknown type and span length. The second problem shows how the specific formula may be applied to a specific bridge. In the last problem, the truck is converted to an equivalent *X* rating for both H and HS-type bridges.

5.1 Example 1: Use of General Formula

Recall that under the current TxDOT restrictions, the truck in Fig. 1-4 was denied a permit because axles 3 and 4 exceeded the maximum allowable. Now, assume that this truck is going to cross several bridges of unknown type and span length. Because H15 bridges are the most critical, the truck should be evaluated according to the revised group weight formula and the general H*X* formula, equations (3-4) and (3-5), for the H15 design rating. The truck may cross continuous span bridges so the factor for concentrated loadings should not be applied. By inserting the proper constants from Table 3-1 (assuming the maximum 10 percent impact factor), the unrevised allowable group axle weight equation (3-5) reduces to the following form.

$$GW = \left(15.75 + 1.55\,WB\right)15 \tag{5-1}$$

$$\left(GW = \left(3.54 + 0.0992\,WB\right)15\right)$$

where

GW = unrevised group axle weight, kN (k),

WB = vehicle wheelbase, m (ft.).

The unrevised allowable weights for each of the axle configurations are shown in Table 5-1. When compared to the actual axle group weights given in Fig. 1-4 and Table 5-1, groups containing axles 1,2,3,4; 2,3,4; and 3,4 do not pass.

Axle Groups	Axle Group Weight		Wheelbase Length		Allowable Group Weight (unrevised)	
	(kN)	(k)	(m)	(ft.)	(kN)	(k)
1,2	196	44	1.98	6.5	279.3	62.8
1,2,3,4	508	114	9.00	29.5	431.5	97.0
2,3,4	410	92	7.02	23.0	388.3	87.3
3,4	312	70	1.22	4.0	262.9	59.1

Table 5-1: Unrevised axle group weights for Example 1.

The allowable group weights are now recalculated incorporating the revision factors from equation (3-4) as summarized in Table 5-2. When the revised allowable group weights are compared with the actual group weights given in Table 5-1, the group containing axles 1,2,3,4 and 3,4 do not pass.

Axle Groups	n	$1/n\ (\Sigma 1/R_iS_i)$	Allowable Group Weight (revised)	
			(kN)	(k)
1,2	2	1.000	4.4	62.8
1,2,3,4	4	1.039	448.4	100.8
2,3,4	3	1.052	408.3	91.8
3,4	2	1.078	283.4	63.7

Table 5-2: Revision factors and revised axle group weights for Example 1.

Note that use of this formula in this manner results in a different restrictive policy for H15 bridges. With the current restrictions, the group containing axles 3 and 4 failed to qualify for a permit. But with the proposed restrictions, the two groups containing axles 1, 2,3,4 and 3,4 fail to qualify for a permit.

5.2 Example 2: Use of Bridge-Specific Formula

With this example, assume that the same truck in Fig. 1-4 will cross a single HS20 simple span bridge with a 13.7 m (45 ft.) span length. Equations (3-3) and (3-6) will be used along with the factor for concentrated loadings. The analysis for β is summarized in Table 5-3 and is based on the simpler, more conservative formula for β as given by TTI Report 1266-4F:

$$\beta = 1 - \frac{GD}{21.3} \leq 0.92 \quad \left(\beta = 1 - \frac{GD}{70} \leq 0.92 \right) \tag{5-2}$$

	Max Axle Distance		β	WB_{rev}	
Axles	(m)	(ft.)		(m)	(ft.)
1,2	1.98	6.5	0.907	2.19	7.2
1,2,3,4	5.80	19.0	0.729	12.3	40.5
2,3,4	5.80	19.0	0.729	9.62	31.6
3,4	1.22	4.0	0.920	1.29	4.3

Table 5-3: Correction factors for vehicle in Example 2.

Now the bridge specific constants from Table 3-2 reduce equation (3-7) to the following form in equation (5-3).

$$w = \left(\frac{0.381 L^2 + 63.6 L + \frac{639}{L} - 311}{WBL(2L - WBL)} \right) 20$$

$$\left(w = \left(\frac{0.031 L^2 + 14.3 L + \frac{1570}{L} - 230}{WBL(2L - WBL)} \right) 20 \right) \tag{5-3}$$

where

L = span length, m (ft.),

WBL = WB_{rev}, wheelbase, m (ft.),

w = allowable distributed load, kN/m (k/ft.),

The correction factors listed in Table 5-2 and equation (5-3) may then be combined in the revised group weight. The results are shown in Table 5-4.

	Distributed Weight w		Unrevised Group Weight		Revised Group Weight	
Axles	(kN/m)	(k/ft.)	(kN)	(k)	(kN)	(k)
1,2	76.3	17.15	549.3	123.5	549.3	123.5
1,2,3,4	22.7	5.10	918.6	206.5	974.6	219.1
2,3,4	24.6	5.54	778.9	175.1	842.0	189.3
3,4	126.2	28.37	530.2	119.2	594.7	133.7

Table 5-4: Revised axle group weights for Example 2.

This truck is well within the allowable limits to cross this particular bridge.

5.3 Example 3: Reclassifying Trucks with H-Type and HS-Type Ratings

The following example will use equation (3-5) to solve for the H-type and HS-type ratings. After these ratings are calculated, anyone will be able to quickly determine whether this truck may be able to cross a particular bridge. However, if the truck rating is higher than the bridge rating, it does not necessarily mean the truck should not be allowed to cross the bridge. This is because the more restrictive general formulae will be used to calculate this rating.

A permit may still be issued for a particular bridge if the bridge specific formula is used. Therefore, this method is the most useful when the truck route and the bridges to be crossed are unknown.

Since it is already known that the axle group containing axles 3 and 4 is one of the critical groups for the truck in Fig. 1-4, the analysis will be applied to this group. Ordinarily, if the critical axle group is not obvious, all groups should be examined with the lowest rating controlling.

The entire process consists of solving equation (3-5) for X. Because the constants in Table 3-1 are in terms of X, this is technically an iterative process. However, the formulae for the constants are relatively stable, and many situations will require only one iteration. In addition, the unrevised rating may be divided by the 1.078, the revision factor to give a revised rating. The unrevised and revised H-type and HS-type ratings for the truck in Fig. 1-4 are given in Table 5-5.

Design Type	Rating 1st it.	Rating 2nd it.	Revised Rating
H-type	22.14	22.65	21.01
HS-type	21.20	21.70	20.13

Table 5-5: Unrevised and revised H-type and HS-type ratings for example vehicle.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

In summary, HX and HSX formulae have been determined which limit the maximum allowable load of permit vehicles. A general formula limits the weight of any group of axles by the bridge design type and the group wheelbase while the bridge specific formula also takes into account the span length of the bridge. Additional factors which aid in distributing the load may then be applied to both formulae. Equation (6-1) defines the revised axle group weight.

$$GW_{rev} = \frac{GW}{n} \sum_{i=1}^{n} \frac{1}{R_i * S_i} \qquad (6\text{-}1)$$

where

GW_{rev} = revised axle group weight, kN (k),

S_i = reduction factor accounting for each axle with more than four tires per axle,
= 1.0 for axles with four tires or fewer,
= 0.96 for axles with eight or more tires,

n = number of axles,

R_i = reduction factor accounting for gages wider than 1.8 m (6 ft.) (or RF)

GW = axle group weight, kN (k), calculated from either equations (6-2) or (6-3) and (6-4) as:

$$GW = \left(a + b\,WB_{rev} \right) X \qquad (6\text{-}2)$$

$$GW = w * WB_{rev} \qquad (6\text{-}3)$$

$$w = \left(\frac{aL^2 + bL + \frac{c}{L} + d}{WBL(2L - WBL)} \right) X \tag{6-4}$$

where

w	=	allowable distributed load, kN/m (k/ft.),
L	=	span length, m (ft.),
WBL	=	WB_{rev}, revised wheelbase, m (ft.) when $WB < L$,
	=	L, span length, m (ft.) when $WB > L$,
X	=	design rating of the bridge,
a,b,c,d	=	constants defined in Tables 6-1 and 6-2.
Wb_{rev}	=	revised wheelbase, m (ft.).

The revised wheelbase is defined in equation (6-5) as:

$$WB_{rev} = \frac{WB}{\beta} \tag{6-5}$$

where

WB	=	vehicle wheelbase, m (ft.),
β	=	correction factor for concentrated loadings;
	=	1.0 for continuous span bridges,
	=	defined by equations (6-6) or (6-7) for simple span bridges as:

$$\beta = 0.97 - \frac{D}{12.2} \le 0.92 \quad \left(\beta = 0.97 - \frac{D}{40} \le 0.92 \right) \tag{6-6}$$

$$\beta = 1 - \frac{GD}{21.3} \le 0.92 \quad \left(\beta = 1 - \frac{GD}{70} \le 0.92 \right) \tag{6-7}$$

where

D	=	distance between the center of gravity and nearest axle, m (ft.),
GD	=	greatest distance between any two axles, m (ft.).

Bridge	Impact	*a*	*b*
	0%	17.83 - 0.034*X*	1.973 - 0.0119*X*
		(4.009 - 0.0077*X*)	(0.1265 - 0.0012*X*)
H*X*	10%	16.21 - 0.031*X*	1.804 - 0.0172*X*
		(3.645 - 0.007*X*)	(0.1157 - 0.0011*X*)
	30%	12.81 - 0.027*X*	1.684 - 0.016*X*
		(2.88 - 0.006*X*)	(0.108 - 0.001*X*)
HS*X*	0%	0.0	3.103 - 0.016*X*
		(0.0)	(0.199 - 0.001*X*)
WB < 11.6 m	10%	14.45 + 0.0013*X*	2.791 - 0.013*X*
(*WB* < 38 ft.)		(3.249 + 0.0003*X*)	(0.179 - 0.0008*X*)
	30%	13.08 - 0.0623*X*	2.136 + 0.0062*X*
		(2.94 - 0.014*X*)	(0.137 + 0.0004*X*)
HS*X*	0%	35.52 - 0.0342*X*	1.380 - 0.016*X*
		(7.985 - 0.0077*X*)	(0.0885 - 0.001*X*)
WB > 11.6 m	10%	32.27 - 0.0756*X*	1.258 - 0.014*X*
(*WB* > 38 ft.)		(7.255 - 0.017*X*)	(0.0807 - 0.0009*X*)
	30%	24.64 + 0.0356*X*	1.325 - 0.016*X*
		(5.54 + 0.008*X*)	(0.085 - 0.001*X*)

Table 6-1: Constants for general formula (repeated from Table 3-1).

Formula	Impact	*a*	*b*	*c*	*d*
	0%	2.262 - 0.0204*X*	0.0	-459.1 - 1.376*X*	225.5 + 0.994*X*
		(0.155 - 0.0014*X*)	(0.0)	(-1111 - 3.33*X*)	(166.3+.733*X*)
H*X*	10%	2.320 - 0.0350*X*	0.0	-442.5 + 0.4835*X*	202.7 - 0.949*X*
		(0.159 - 0.0024*X*)	(0.0)	(-1071+1.17*X*)	(149.5+0.7*X*)
	30%	1.883 - 0.0175*X*	0.0	-281.8 - 1.405*X*	143.4 - 1.36*X*
		(0.129 - 0.0012*X*)	(0.0)	(-682 - 3.4*X*)	(105 + *X*)
	0%	0.7588 - 0.0623*X*	126.3 - 2.85*X*	1771- 52.89*X*	-835.1+24.63*X*
		(0.052 - 0.0043*X*)	(28.4 - 0.64*X*)	(4287 - 128*X*)	(-616+18.17*X*)
HS*X*	10%	0.861 - 0.0204*X*	59.16 + 0.22*X*	556.1+ 4.13*X*	-264.4 - 2.350*X*
		(0.059 - 0.0014*X*)	(13.3 + 0.05*X*)	(1370 + 10*X*)	(-195 - 1.733*X*)
	30%	0.730	56.49 - 0.5916*X*	633.1- 10.99*X*	-295.6 + 4.61*X*
		(0.05)	(12.7 - 0.133*X*)	(1532 - 26.6*X*)	(- 218 + 3.4*X*)

Table 6-2: Constants for bridge-specific formula (repeated from Table 3-2).

The proposed formulae have several advantages over the current permit restrictions. One benefit is that the proposed formulae are much more versatile than those currently used. By taking the bridge design type and the span length into account, greater loads may be allowed without a detailed engineering analysis. The proposed HX and HSX formulae may also be used for damaged bridges which have been given a new X rating to reflect their deteriorated state. This reclassification process must include the original dead load of the bridge. Application example three illustrates another valuable asset of these formulae. In this problem, a given axle configuration is reclassified into both a H-type and HS-type permit truck. This new X rating will greatly reduce the permitting time necessary to evaluate this truck crossing any bridge.

The finite element analysis (FEM) results show that no overstressing occurs in the continuous span reinforced concrete structures when the proposed permit restrictions are imposed. However, when the same comparisons are made using AASHTO and LRFD effective widths, some bridges reflect overstress.

According to the proposed restrictions, the current TxDOT permit criteria allow loads which may exceed operational stress levels in certain bridges. Therefore, it should be asked why more problems have not arisen. Probably the biggest reason is that in this analysis, the worst type of loading was applied to the most critical section of the weakest bridges. The probability of this happening in reality is small. Additionally, because speed is difficult to enforce, the permit vehicle has been assumed to cause full impact on the bridge, i.e., the maximum 30 percent impact factor. If speed is adequately monitored, the impact of the vehicle may be reduced or neglected which will increase the allowable loads by as much as 30 percent. In this case, the proposed formulae corresponding to a 10 percent maximum impact factor should be used. Other factors also serve to protect most bridges from the overloaded vehicles. One of these factors is that as-built bridges always have some inherent composite action associated with the friction between the deck and the steel stringers.

Due to the difficulty in measuring this additional capacity, it has not been included in this analysis.

Another factor which has not been considered is the use of diaphragm members to transmit forces laterally. Although it is commonly thought that cross members aid significantly in the distribution of stresses for overweight vehicles, they are not considered in the development of the gage reduction factor. A final factor which may increase the overall moment capacity of bridges is the longitudinal transmission of forces by the deck. The addition of 1.2 m (4 ft.) in the unmodified equivalent loading equation in the current permit restrictions is probably quite conservative. This fact is very difficult to quantify, and enough research has not been done in this area to warrant its addition to the proposed restrictions.

6.2 Additional Research

Additional research in the overweight permit restrictions should focus on three major areas. First, the factors determining composite action in non-composite bridges, distribution of forces by diaphragm members, and longitudinal transmission of forces by the deck should be quantified. Additional analysis in these areas will safely allow for greater permit loads on all bridges.

Second, researchers should study and implement the automation of issuing permits. Even though the proposed restrictions should reduce the time required to evaluate an overweight vehicle, analysis of all the axle configurations and their associated factors can be quite tedious. The highly repetitive use of the proposed formulae are ideally suited for a computer system. With access to BRINSAP, a computer system could perform the necessary analysis for the bridge-specific formula and allow greater loads for specific routes without the additional time.

Thirdly, the dynamic effects of superheavy vehicles need to be better understood so that impact factors can be better quantified. Since these vehicles have axle and truck configurations that are nontypical it may not be appropriate to assume the same impact factors are those used with more traditional vehicles.

7. REFERENCES

1. *Texas Administrative Code*, 1992 Supplement. West Publishing Co., St. Paul, MN (1992).

2. *Standard Specifications for Highway Bridges*. 14th edition, American Association of State Highway and Transportation Officials, Washington, DC, (1989).

3. P. B. Keating, et al., Overweight Permit Procedures. FHWA-TX-93-1266-4F, Texas A&M University, College Station, TX (1993).

4. *Manual For Maintenance Inspection of Bridges*, 1983 edition. American Association of State Highway and Transportation Officials, Washington, DC, (1983).

5. E. E. Seelye, *Data Book for Civil Engineers*, Volume 1, 3rd edition. Wiley London, Chapman and Hall, New York, 1302-1317 (1957).

6. J. S. Noel, et al., Bridge Formula Development. FHWA-RD-85-088, Texas A&M University, College Station, TX (1985).

7. R. E. Whiteside, et. al., Changes in Legal Vehicle Weights and Dimensions; Some Economic Effects on Highways. NCHRP 141, Washington, DC, 123-27 (1973).

8. B. Bakht and L. Jaeger, *Bridge Analysis Simplified*. McGraw Hill, St. Louis, MO, 279-82 (1985).

9. *SigmaPlot User's Manual*, Version 5.0. Jandel Scientific, Corte Madera, CA (1991).

10. T. Zoakaie, Distribution of Wheel Loads on Highway Bridges. NCHRP 12-26, Washington, DC (1992).