gestions made by his colleagues R.H. Pion, G.S. Hibbert, and H.I. Nadler and expresses thanks to J.C. Beauchamp, Chief Bridge Engineer of Public Works Canada, for his permission to apply the rating procedure in the strengthening programs of many federal bridges.

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A Rational Procedure for Overweight Permits

BAIDAR BAKHT and LESLIE G. JAEGER

ABSTRACT

A rational procedure for calculating safe permit loads for vehicles as governed by the bridges on the route without having to analytically evaluate all the bridges is given. The basis of the procedure is the worst combination of maximum vehicle weights that a bridge is likely to have sustained during its lifetime. With the severest load combination as the datum, maximum increases over legal loads for normal traffic are calculated for control vehicles. Expressions for calculating the modification factors corresponding to two- and three-lane loadings are also provided.

Applications are quite often made for permission to let a much heavier vehicle cross a bridge than that legally permitted for normal traffic. The maximum safe weight for such a vehicle can be obtained by an analytical evaluation of the bridge. Alternatively, according to the procedures developed in this paper, the maximum safe weight of a special-permit vehicle can be obtained from the heavy vehicle traffic that a bridge is known to have carried during its lifetime.

The design capacity of a highway bridge carrying normal traffic safely implicitly takes account of the following factors:

1. Legally permitted normal vehicle weights as represented by the design vehicle and possibly a portion of the live-load factor,
2. Bridge type,
3. Number of lanes on a bridge,
4. Length of span,
5. Accidental and deliberate exceedance of legally permitted weights,
6. Transverse vehicle position,
7. Simultaneous presence of more than one vehicle in the transverse direction,
8. Simultaneous presence of more than one vehicle in a lane,
9. Vehicle width, and
10. Vehicle speed as represented by the dynamic load allowance or impact factor.

In the case of a special-permit vehicle, factors 5-10 are either known beforehand with some degree of certainty or can be prescribed as a condition for the permit. More reliable knowledge of these factors can be used to advantage to permit a substantially heavy special-permit vehicle without compromising the safety of the structure.

A safe estimate of the maximum load of a special-permit vehicle for a bridge can be obtained by the procedure given here without analytical evaluation of the structure. This procedure requires the knowledge of one of the following:

1. The maximum vehicle weights corresponding to the code-specified factors 1-10, given previously, that a bridge is capable of sustaining and
2. The worst combination of maximum vehicle weights that the bridge is likely to have sustained in its lifetime.

The former can be obtained from the design calculations but only if the design vehicle has a direct relationship with the actual vehicle weights, as it does, for example, in the case of the Ontario Highway Bridge Design Code (1,2). As pointed out by Buckland and Sexsmith (3), the AASHTO (4) design loads are not in close correspondence with actual traffic. Therefore, the knowledge that a bridge has been designed to AASHTO specifications is not always sufficient to establish the maximum vehicle weights that the bridge can sustain.

The determination of the maximum loads that a bridge is likely to have sustained in the past requires a probabilistic analysis, which is given in the following section.

PROBABILISTIC ANALYSIS

Factors 5-10 listed earlier are probabilistic in nature. For example, it is not known beforehand what fraction of the total vehicle population will exceed legally permitted weights nor what the probability is that such overweight vehicles will travel in extremely eccentric positions. It is usual to include the most unfavorable of these factors in design or evaluation calculations. However, calculations based on estimating the probability of individual occurrence of these unfavorable factors rapidly leads to the conclusion that certain combinations of them are extremely unlikely even once in the lifetime of a bridge. Hence it is suggested that it is unrealistic to design a new bridge on the basis that all of these factors are present simultaneously and to the maximum extent possible.

Similarly, in assessing the load-carrying capacity of an existing bridge it should not be assumed that the bridge has already safely carried a load combination in which all of the factors were simultaneously present at their worst. Instead it is recommended that a technique be used in which a realistic estimate is made of the worst combinations of normal traffic that have already been experienced and that this estimate be used as the basis for calculating the permissible size of an overweight vehicle.

The probability of the simultaneous presence of vehicles in two adjacent lanes is studied in the following simple example.

Example

Consider a two-lane bridge with a span 300 ft long that carries a low density of heavy trucks, estimated at 300 per day. Let it be supposed that the bridge is 20 years old and that the coefficient of variation of the heavy truck weights is 20 percent. The statistical distribution of heavy truck weights is assumed to be as shown in Figure 1. An estimate is sought of the probability that two trucks at least 60 percent heavier than the mean will be simultaneously situated on the middle third of the bridge, one in each lane, at least once during the 20-year life of the bridge.

It is first necessary to establish a time interval within which any given truck may be on the central 100 ft of the bridge. Taking a slow vehicle speed of 20 ft/sec, a time interval of 5 sec is postulated. From the properties of the normal distribution, it is found that 0.00135 of the total heavy vehicle population has weights of 60 percent or more above the mean. This fraction is shaded in Figure 1. Thus, the expected number of heavy vehicles per day that cross the bridge and are this heavy is 300 x 0.00135 = 0.405 vehicle per day.

In one day there are 1,728 time intervals of 5 sec each. Hence, the probability that a vehicle 60 percent or more heavier than the mean will be on the middle third of the span during any single 5-sec interval is 0.405 (1/17,280) = 2.34×10^{-5}. The probability that two such vehicles will be present during any given 5-sec interval is the square of this quantity:

$$P = 5.47 \times 10^{-10} \qquad (1)$$

The combination of a low probability that such an event will happen in a given 5-sec interval together with the large number of intervals that represent the lifetime of a bridge is ideally suited to the use of the Poisson distribution of probability of occurrence. With the customary notation, the probability that the stated occurrences will happen in any given 5-sec interval is denoted by p and the total number of time intervals by N. The product Np is the Poisson parameter α_1:

$$Np = \alpha_1 \qquad (2)$$

If the probability of m occurrences is defined as P(m), the following equations are obtained by using Poisson distribution theory:

$$P(0) = \exp(-\alpha_1)$$

$$P(1) = \alpha_1 \exp(-\alpha_1)$$

$$P(2) = (\alpha_1^2/2!)\exp(-\alpha_1) \qquad (3)$$

During the 20-year life of the bridge the total number of 5-sec intervals (N) is equal to 1.26×10^8. Hence α_1 is found to be 0.069, and Equations 3 then give the following:

$$P(0) = 0.933$$

$$P(1) = 0.063 \qquad (4)$$

It is concluded that on the basis of the assumptions made it is unlikely (only a 6.7 percent chance) that the two vehicles, each at least 60 percent heavier than the mean, will be on the middle third of the bridge simultaneously.

However, the situation changes swiftly if the simultaneous presence of somewhat lighter vehicles is investigated. For example, for vehicles 40 percent heavier than the mean, α_1 is found to equal 19.65 and P(0) to be approximately 10^{-8}. Thus it is virtually certain that there will be two vehicles, each at least 40 percent heavier than the mean, simultaneously present on the middle third of the bridge at least once during its lifetime.

The large probability change associated with the reduction of the vehicle weight is at the basis of the generalized treatment given in the following. What is demonstrated in the foregoing is that

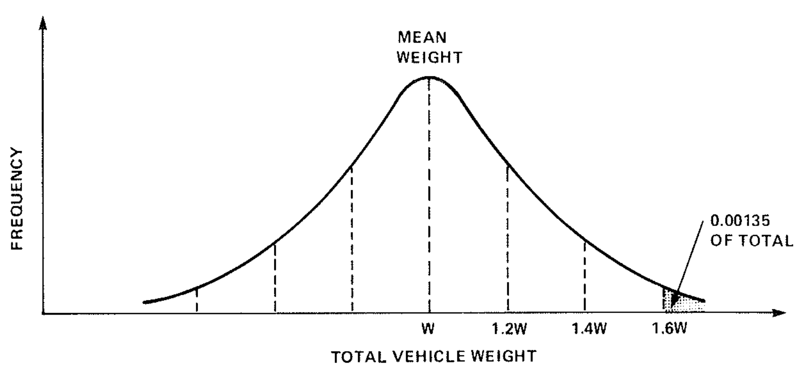

FIGURE 1 Assumed distribution of heavy vehicle weights.

changes in weight levels considered have effects many times larger than the values of other parameters. For example, if the time interval was estimated as 2.5 sec instead of 5 sec, the value of N would double. However, the same value of α_1 would be obtained by halving the value of P, and this would affect the load level only slightly.

Simultaneous Presence of Two Vehicles

For the multiple presence of vehicles, it is recommended that the Poisson approach be used. A value of the parameter α_1 is first selected, the value being so chosen as to make it likely that the event has already occurred in the life of the bridge. It is recommended that $\alpha_1 = 3.0$ on the basis that this corresponds to a 95 percent probability of at least one previous occurrence of the event. The weights of vehicles corresponding to this value of α_1 are then ascertained.

Let the two simultaneously present vehicles be r standard deviations above the mean of the vehicle weights, the number of vehicles per day crossing each lane of the bridge be n, the time interval for presence on the middle third of the bridge be T sec, and the previous life of the bridge be B years.

Figure 2 defines the fraction [q(r)] of all heavy vehicles that are at least r standard deviations higher than the mean. Values of q(r) for various r-values can be obtained from Figure 3.

The expected number of heavy vehicles per day that cross each lane of the bridge and have weights at least r standard deviations above the mean is nq(r). The probability of having such a vehicle present during any one time interval of T sec is Tnq(r)/86,400. Hence the probability (P) that two such vehicles will be simultaneously on the middle third of a span during a given interval of T sec is as follows:

$$P = T^2 n^2 [q(r)]^2/(86{,}400)^2 \qquad (5)$$

The number of T-sec time intervals (N) in B years is given by

$$N = (B \times 365 \times 86{,}400)/T \qquad (6)$$

Hence

$$\alpha_1 = \{B \times 365 \times T \times n^2 [q(r)]^2\}/86{,}400 \qquad (7)$$

Adopting $\alpha_1 = 3.0$ as the criterion, Equation 7 gives

$$q(r) = (26.65/n)[1/BT]^{0.5} \qquad (8)$$

From known values of B, T, and n, q(r) can be obtained from Equation 8, and values of r can be obtained from known properties of the normal distribution shown in Figure 3. The value of r thus obtained corresponds with 95 percent probability to the number of standard deviations above the mean for the weights of two trucks that will be simultaneously present on the middle third of the bridge during its previous history.

Using the procedure just described, values of r for B = 20 and 50 years and values of T = 3 and 5 sec were calculated for various values of n. These values are plotted in Figure 4 with respect to n.

Physical Significance of r

From the results of a vehicle weight survey, Csagoly and Knobel (5) confirmed that the distribution of heavy vehicles is indeed nearly normal. They also showed that the maximum observed loads corresponding to various base lengths (6) are approximately equal to $\mu + 3.5\sigma$, where μ is the mean heavy vehicle weight corresponding to a particular base length and σ is the relevant standard deviation. Values of r obtained from Equation 8 and plotted in Figure 4 imply that two vehicles of weight $\mu + r\sigma$ are most likely to be simultaneously present within the middle third of a span at least once during the time period considered. Thus the familiar modification factor F_2 for simultaneous loading of two lanes is given by

$$F_2 = (\mu + r\sigma)/(\mu + 3.5\sigma) \qquad (9)$$

Substituting σ in terms of μ and the coefficient of variation (COV), Equation 9 can be rewritten as follows:

$$F_2 = [1 + r(\mathrm{COV})]/[1 + 3.5(\mathrm{COV})] \qquad (10)$$

Values of F_2 for various time intervals T for B = 50 and 20 years and 20 percent COV are plotted in Figure 5. Thus if the value of F_2 is, for example, 0.90, it can be assumed that the bridge has at least once in its lifetime sustained in the middle third of its span two side-by-side vehicles, each of which weighed 0.90 times the maximum expected weight for a single vehicle.

Simultaneous Presence of Three Vehicles

The same approach that has been developed for two vehicles can be applied to investigate what the size of the vehicle will be such that three such vehicles will be simultaneously present with 95 percent probability during a given time period. Using the same

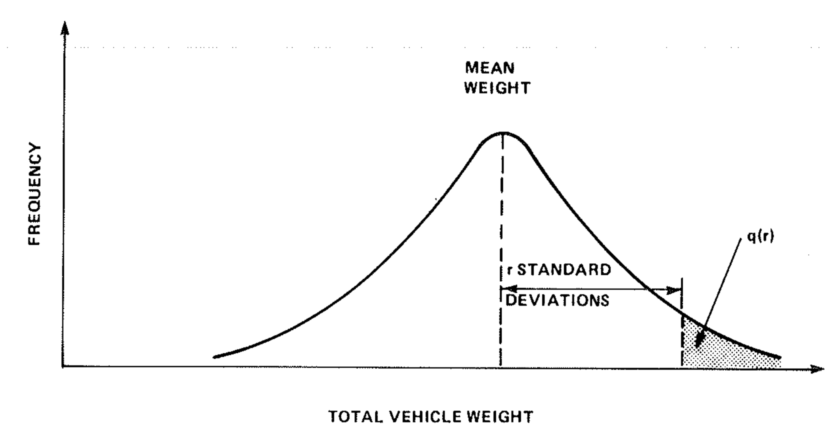

FIGURE 2 Vehicle weight distribution.

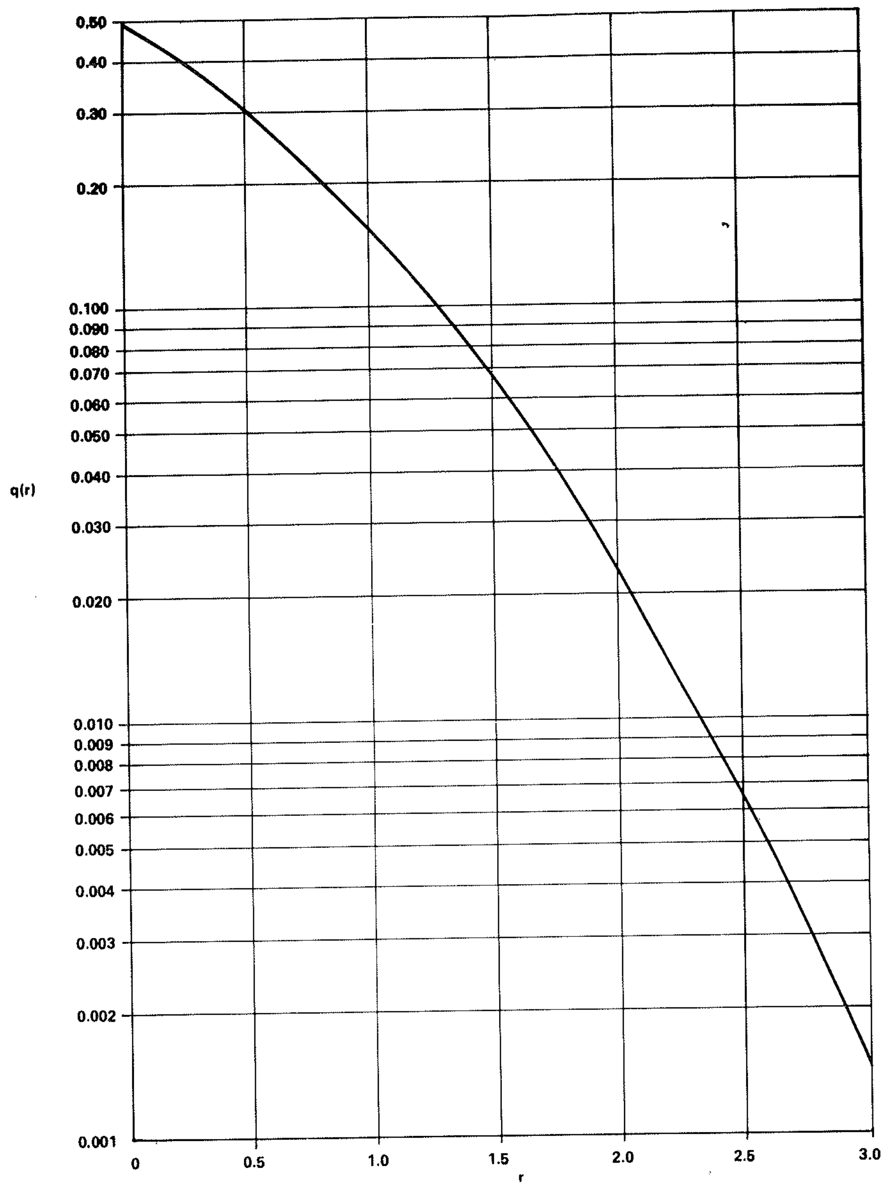

FIGURE 3 Variation of q(r) with r.

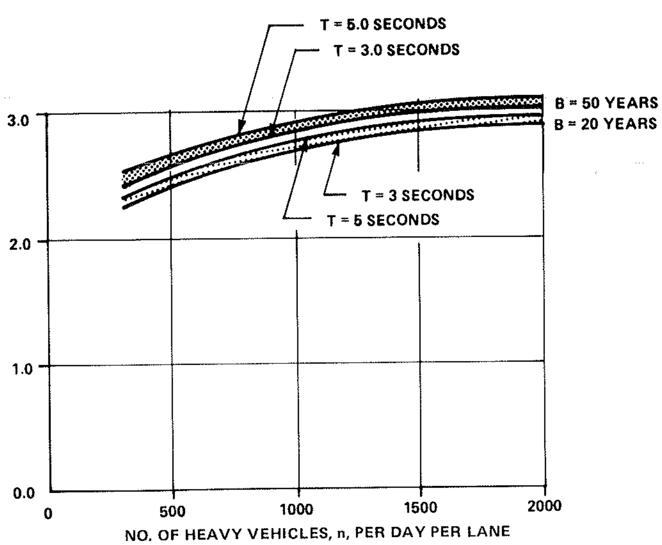

FIGURE 4 Values of r for simultaneous presence of two vehicles.

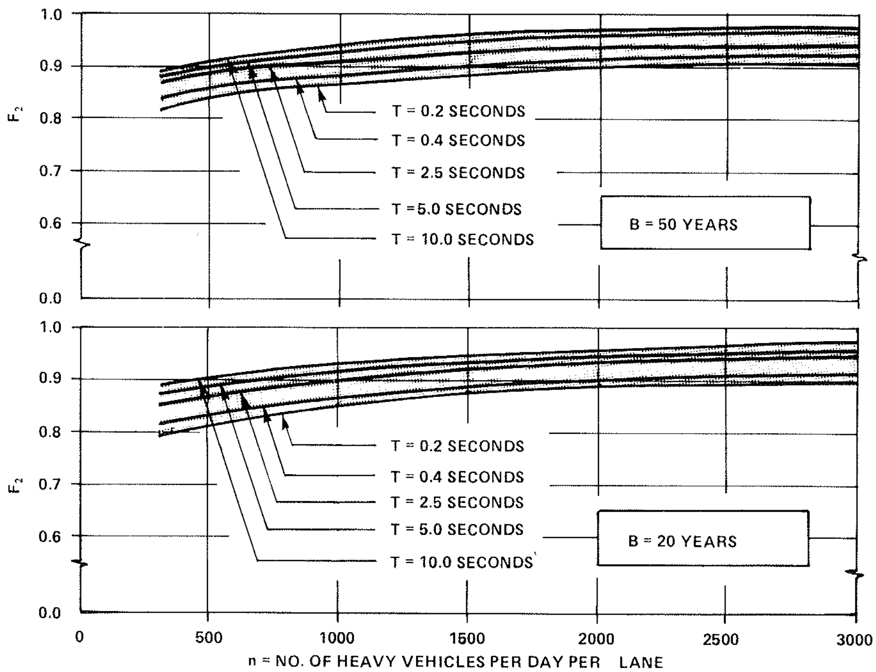

FIGURE 5 Two-lane loading modification factors for evaluation of safe overweights.

notation as before, the probability (P) that three such vehicles will be simultaneously present on the middle third of a span during a given time interval of T sec is as follows:

$$P = T^3 n^3 [q(r)]^3/(86{,}400)^3 \quad (11)$$

Substituting the value of N from Equation 6, α_1 is given by the following:

$$\alpha_1 = \{B \times 365 \times T^2 \times n^3 [q(r)]^3\} / (86{,}400)^2 \quad (12)$$

Again adopting $\alpha = 3.0$ as the criterion, Equation 12 gives

$$q(r) = (394.41/n)[1/BT^2]^{0.33} \quad (13)$$

From Equation 13 the value of r can be obtained as for the case of two vehicles. The modification factor F_3 corresponding to the simultaneous presence of three vehicles is given by

$$F_3 = [1 + r(COV)]/[1 + 3.5(COV)] \quad (14)$$

Comments on Assumptions Made

In connection with the development of a design vehicle for the forthcoming bridge design code of the Canadian Standards Association (CSA), vehicle survey data from seven Canadian provinces were processed (Cheung and Agarwal, unpublished data). The COVs for heavy vehicles with different numbers of axles obtained from this study are plotted in Figure 6, which also defines the weight limit below which the vehicles were not included in the COV calculations. It can be seen that the value of COV generally decreases with the increase in the number of axles, suggesting a smaller spread of total weight for heavier vehicles.

Overall moments and shears in a bridge are usually governed by vehicles with five or more axles. For such vehicles, as shown in Figure 6, the COV for total weight rarely exceeds 20 percent, thus justifying the 20 percent value assumed earlier. A higher value of COV was not chosen because it would have resulted in a more benign value of the modification factor. The effect of changes in the COV values on F_2 is shown in Figure 7. It can be seen that a smaller COV leads to a more conservative value of F_2.

Derivation of the Poisson parameter α_1 has been based on finding two vehicles on the middle third of a bridge span. This fraction of the span is chosen because, in arriving at the longitudinal bending moments, a vehicle placed anywhere in the middle third of a span produces moments that are quite close to the maximum moments in the span. Similarly for longitudinal shears, the same kind of reasoning applies to the simultaneous presence of two vehicles in the end third of the span, and indeed the same reasoning can be applied to moments in continuous bridges.

The assumption that the maximum expected vehicle weight is equal to $(\mu + 3.5\sigma)$ is a safe one if the modification factors based on this assumption are used to assess the most severe load combination that a bridge may have experienced. However, if modification factors thus obtained are to be used for design purposes, it may be prudent to assume that the maximum expected vehicle weight is equal to $(\mu + 3.0\sigma)$. This revision of the assumption will lead to slightly larger, hence safer, values of the modification factors. With this assumption, modification factors for the simultaneous loading of two and three lanes, that is, F_2 and F_3, respectively, are calculated for various cases by the preceding procedure. Values thus obtained are plotted in Figure 8.

It is interesting to compare the previously calculated modification factors with those specified in the AASHTO (4) and Ontario (1) codes. The AASHTO specified values of F_2 and F_3 are 1.0 and 0.90, respectively. As can be seen in Figure 8, these values are safe upper limits for all cases shown, some of which are quite unrealistic. A high volume of heavy vehicle traffic, say, more than 1,000 vehicles per day per lane, is possible only when the speed of the vehicles is 55 mph or more. For such a

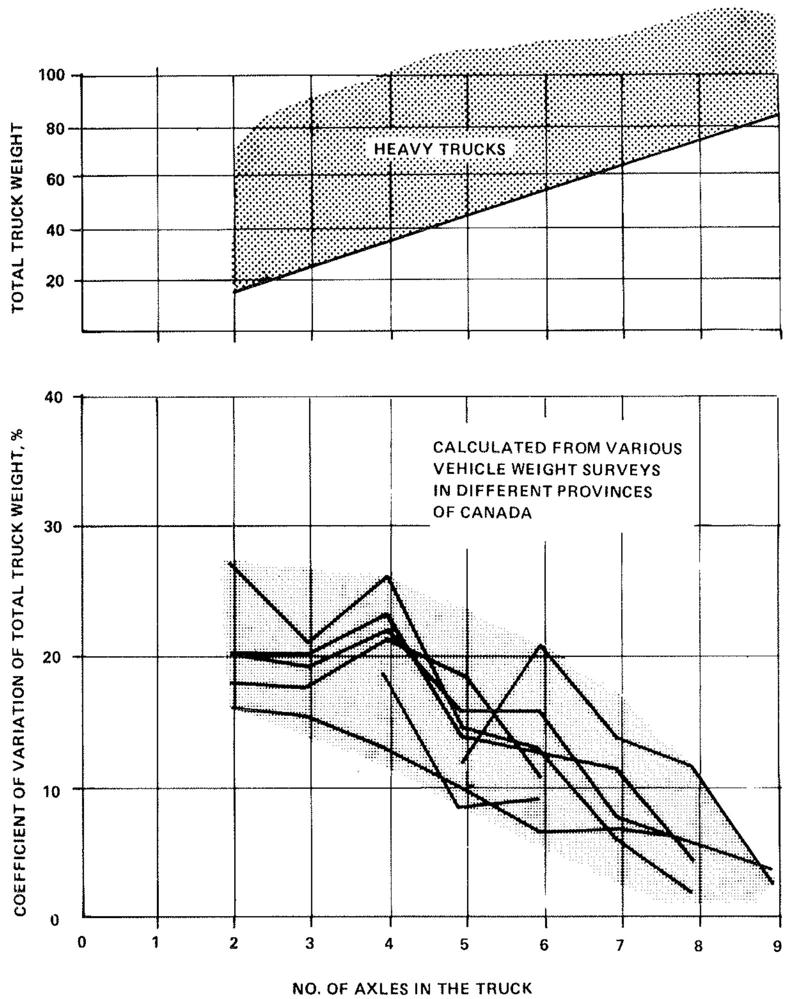

FIGURE 6 COVs of heavy truck weights.

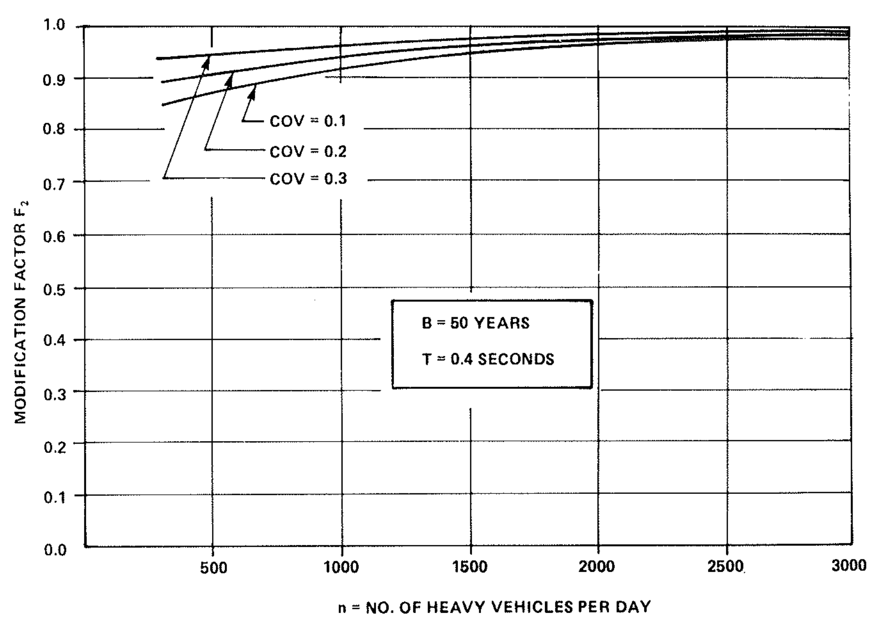

FIGURE 7 Effect of COV on F_2.

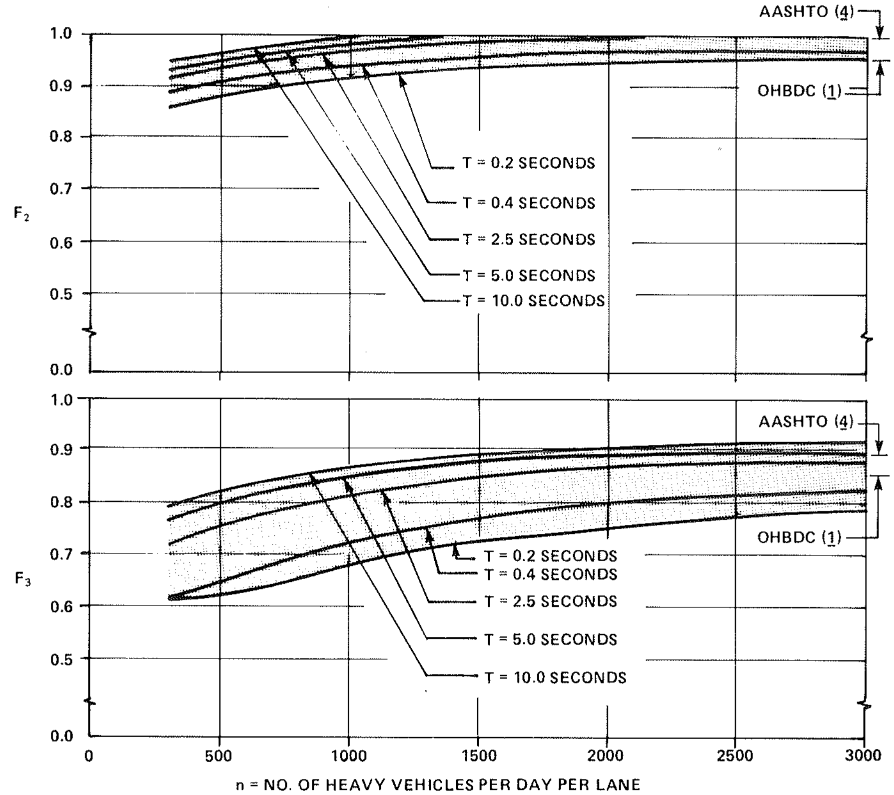

FIGURE 8 Modification factor for multilane loading for 50-year bridge life.

speed and a large span of 300 ft, T is equal to about 1.20 sec. For this case, the procedure gives F_2 and F_3 values of 0.96 and 0.79, respectively. Compared with these values, the AASHTO values of F_2 and F_3 appear quite conservative. The Ontario code (1) values of F_2 and F_3 are 0.95 and 0.85, respectively. These values compare well with the previously calculated values. It should be noted that in the second edition of the Ontario code (2) F_2 and F_3 values have been reduced to 0.90 and 0.80, respectively. These reductions result from the amalgamation of the modification factors for vehicle weights and dynamic load allowances, that is, impact factors (7), which were given different values in the first edition (1).

Application of Modification Factors

Although most bridges in North America are designed to the same specifications, legally permitted weights for normal traffic vary widely in different jurisdictions. These variations may represent conscious decisions to maintain certain margins of safety required by the various jurisdictions. Besides the vehicle weight regulations for normal traffic, maximum vehicle weights are the result of the accidental exceedance of legal weights and deliberate violations of weight laws. An estimate of the former can be obtained from the load factors given in the Ontario code (2) for normal traffic vehicles and for special-permit vehicles for which the loads can be closely obtained: The former is 1.40 and the latter, 1.25. Thus if it is ensured that the permitted weights will not be deliberately exceeded, normal traffic weights increased by a factor (R_1) of 1.40/1.25 = 1.12 may be permitted for special-permit vehicles flowing without restrictions with normal traffic.

It should be noted that the factor of 1.12 is applicable to Ontario and other jurisdictions with similar weight enforcement measures. In jurisdictions with strict enforcement, the factor should be reduced. It may, however, be prudent not to increase the factor beyond 1.15 even if the degree of enforcement is lenient. For most jurisdictions a factor of 1.10 may be appropriate.

An implicit assumption in the previous calculations is that during the bridge's lifetime, its load-carrying capacity remains unaffected. It is often argued that the condition of a bridge may change in such a way that at a given time it may not be able to safely carry the worst load combination that it once did. Such an argument may be convincing for unusually heavy test loads but not for normal traffic. If the condition of a bridge has changed so much that it is unable to carry the most severe load combination that it once did, the bridge is unsafe for normal traffic because such a load combination, being the result of chance, may occur again. In this paper only those bridges that are expected to carry normal traffic safely are involved.

If it is ensured that on a multilane bridge, a special-permit vehicle is the only one present, a single vehicle weight that would produce the same maximum effects as those produced by the most severe load combination can be calculated by procedures given in the following section.

DETERMINISTIC ANALYSIS

The permissible safe weight of a vehicle for a multilane bridge can be substantially increased above normal traffic weights if the vehicle is prescribed to travel along the bridge centerline and during its passage other vehicles are excluded from the bridge. An accurate assessment of this permissible increase requires a knowledge of the load distribution properties of the bridge. In this section a general method, which does not explicitly require the load distribution characteristics of the bridge, is developed for two-lane bridges. The conclusion could also be applied to bridges with more than two lanes with the knowledge that the results will be more conservative than those for two-lane bridges.

One Central Vehicle on the Bridge

It has been established (8; Jaeger et al., unpublished data) that for the purpose of load distribution, right single-span bridges can be idealized by two dimensionless parameters (α and θ). The former parameter establishes the relationship of the torsional properties of the bridge with its flexural properties; the latter is based on a relationship between the flexural properties of the bridge and its ratio of span to width. Expressions for the two parameters are given in the Ontario Highway Bridge Design Code (2) and by Jaeger et al., who show that bridges of different types have different but distinctly identifiable ranges of α. As shown in Figure 9, for timber bridges α is between 0.001 and 0.02; for slab-on-girder bridges with steel or concrete girders α is between 0.06 and 0.2; for slab, voided-slab, and cellular bridges α is equal to 1.0; and for box-girder bridges α is between about 1.5 and 2.0. For all bridges θ is between 0.5 and 2.5.

A bridge with the smallest α-value and the largest θ-value has the worst, that is, the most unfavorable, transverse load distribution pattern. Conversely, a bridge with the largest α-value and smallest θ-value is one in which the load distribution is as uniform across its width as is possible for bridges of its type.

Eight two-lane bridges representing those with the best and worst load distribution characteristics in the four categories mentioned earlier were analyzed by the orthotropic plate theory (9) for different load cases. The values of the characterizing parameters are shown in Table 1, and the width of the bridge together with the details of the various load combinations are given in Figure 10. The load case shown in Figure 10a corresponds to the most severe load combination according to the AASHTO specifications (4). The transverse vehicle positions also correspond to the most severe load combination according to the Ontario code (2). Normal transverse positions for vehicles in the two lanes are shown in Figure 10b. Figures 10c-f show centrally placed single vehicles with wheel spreads (distance between the two lines of wheels) of 1.83, 2.00, 2.50, and 3.0 m, respectively. The contact area for each concentrated load was the same as that used by Bakht et al. (8) and shown in Figure 10h.

If the intensity of longitudinal moments (M_x) is taken as the basis for equivalence, two load combinations are equivalent to each other if each of the two cases produces the same maximum intensity of longitudinal moments. Using this criterion and taking the load case shown in Figure 10a as the datum, the percentage of increase (K) in vehicle weight for the other five load cases was calculated from the orthotropic plate analyses. It is noted

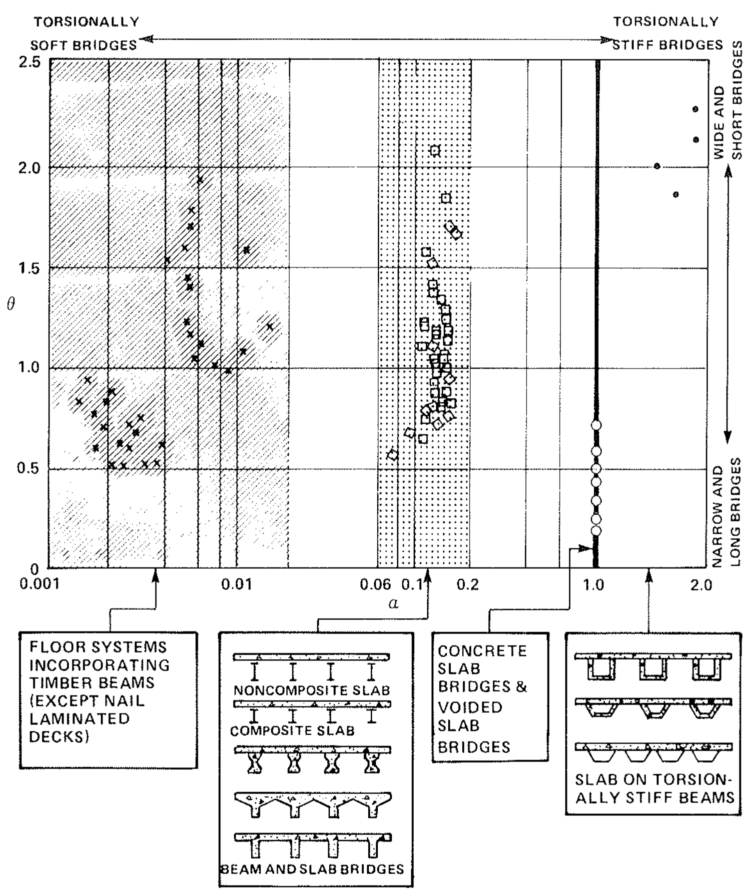

FIGURE 9 Values of α and θ for practical bridges.

that the datum load case corresponds to the most severe transverse position as specified by the AASHTO and Ontario codes. The percentage of increase depended on the bridge type and also on whether the bridge had the worst load distribution characteristics or the best. For each bridge type, the governing value of K was taken to be the smaller of the two for the best and the worst load distributions. It was found that going from the datum load case to the case with two vehicles in normal traveling positions (Figure 10b) did not result in any significant advantage for slab bridges; for box-girder bridges, the weights could be increased 3 percent, for slab-on-girder bridges 5 percent, and for timber bridges 8 percent. K is significant for single vehicles and depends on the bridge type and the wheel spread. Values of K for the datum case under consideration are plotted as case I in Figure 11 with respect to different values of wheel spread (W). As shown in Figure 11, values of K are the same for timber bridges and slab-on-girder bridges.

The preceding exercise was repeated by using the load case shown in Figure 10b as the datum. The resulting values of K are plotted as case II in Figure 11. This datum load case corresponds to the normal traveling positions of two vehicles. Thus case II in Figure 11 can be used when the only information available about the bridge is that it has been carrying normal traffic safely.

The curves for K-values given in Figure 11 for a specific bridge type can be used to calculate the permissible weight if the special-permit vehicle is known to traverse only one bridge or one type of bridge. Alternatively, when the vehicle has to pass over several bridges, the curves for timber bridges, which give the smallest values of K, can be used.

The curves given in Figure 11 are based on the assumption that each axle has two concentrated loads for which the effective contact region is as shown in Figure 10h. Axles with wider wheel spreads may each have four sets of double tires, resulting in four concentrated loads instead of the two assumed in the analysis. For such cases, the actual values of K will be higher than those given in Figure 11.

K-values given in Figure 11 correspond to two normal vehicles with full loads, that is, vehicles with maximum weights. As discussed earlier, a bridge may not have experienced two such vehicles in its

TABLE 1 Values of α and θ Used in the Analyses

	Load Distribution by Type of Bridge							
	Timber		Slab on Girder		Slab and Voided Slab		Box Girder	
Parameter	Worst	Best	Worst	Best	Worst	Best	Worst	Best
α	0.06	0.06	0.06	0.30	1.00	1.00	1.50	2.00
θ	2.0	0.5	2.0	0.5	1.5	0.5	2.5	1.5

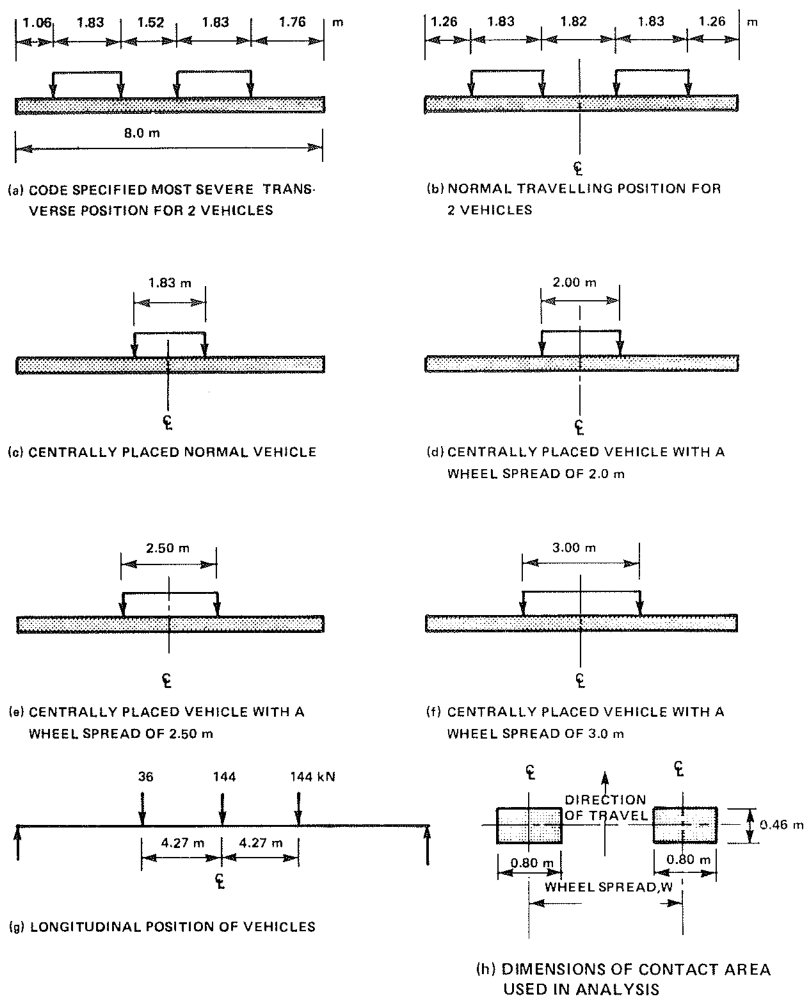

FIGURE 10 Details of loads.

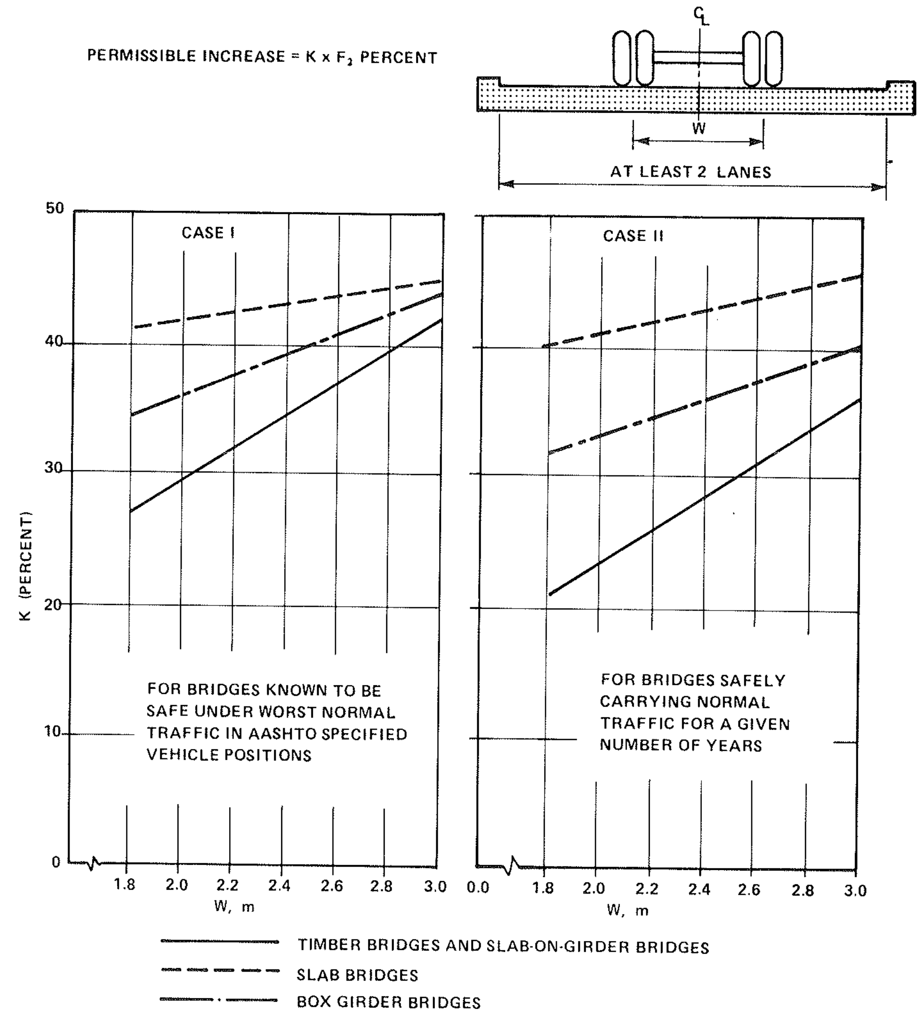

FIGURE 11 Values of K for single vehicles.

lifetime. However, it is certain that the bridge would have sustained two vehicles of weights equal to F_2 times full weight. Thus the factor R_2 by which a single vehicle weight can be increased to account for the presence of two vehicles is given by

$$R_2 = F_2[(1 + K)/100] \qquad (15)$$

Long Spans

For the AASHTO HS20 loading, a single design vehicle governs moments for spans of up to 140 ft, and for the Ontario code this limit is about 80 ft. For both codes the design loadings beyond these limits incorporate a uniformly distributed load to account for the effect of more than one vehicle in a lane. The ratio of simple-span moments due to a single design vehicle and the corresponding moment due to the design loading incorporating the lane load is denoted by R_3. Values of R_3 corresponding to various simple spans are plotted in Figure 12 for both the AASHTO and Ontario loads. These curves can be used to estimate a permissible increase beyond the normal for the special-permit vehicle weight on longer spans if other vehicles are excluded from the bridge. The curves corresponding to the Ontario loads may be found to be more appropriate because, unlike the AASHTO loads, they are based on modern traffic conditions.

Dynamic Load Allowance

By restricting the speed of the permit vehicle on a bridge, the dynamic load allowance (DLA), which is usually known as the impact factor, can be reduced. Advantage can be taken of this reduced DLA in permitting a proportionally higher weight for the permit vehicle.

The Ontario code (2) specifies that the DLA for a single controlled vehicle crossing a bridge at restricted speed can be multiplied by the following reduction factors:

1. For a speed limit of 6 mph or less, 0.30;
2. For speed limits between 6 and 15 mph, 0.50; and
3. For speeds in excess of 15 mph, 1.0.

In arriving at an increase of the static vehicle weight that is equivalent to the reduction of the DLA, it should be remembered that the total load effects due to a single control vehicle are being compared with those due to two normal vehicles. The increase of vehicle weight to account for the reduction of the DLA is denoted by R_4. Taking 0.25 as the smallest DLA value specified by the Ontario code (2) and 0.7 as the modification factor for DLA due to two vehicles (1), R_4 for vehicle speeds of less than 6 mph is obtained as follows:

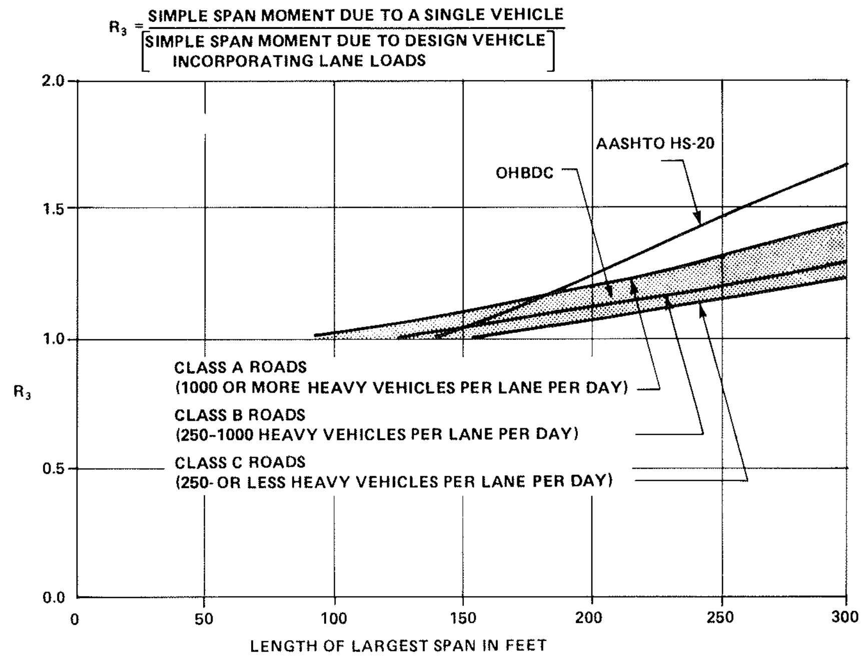

FIGURE 12 Values of R_3 for AASHTO and Ontario loads.

$$R_4 = (1 + 0.25 \times 0.7)/(1 + 0.25 \times 0.3) = 1.09 \quad (16)$$

R_4 for speeds between 6 and 15 mph works out to 1.04. It is noted that taking higher values of DLA would have resulted in larger and therefore less conservative values of R_4.

SUMMARY OF PROCEDURE

For a special-permit vehicle traveling along the centerline of a bridge with at least two lanes and with other traffic excluded from the bridge, an estimate of permissible vehicle weight increase above legal loads for normal traffic can be obtained as follows:

1. Allowance for confidence in weight: The factor R_1 accounts for the confidence that the weight of the permit vehicle will not be deliberately exceeded. Assume that R_1 is equal to at least 1.10.
2. Allowance for multilane loading: Using Equations 8 and 10 and assuming that COV = 0.2, calculate the value of F_2. Use T = 0.2 sec if there are many bridges involved in the trip. If the life of the bridge is 20 or 50 years, read F_2 directly from Figure 5 corresponding to the number of heavy vehicles per lane per day on the highway under consideration. Remember that the smaller values of F_2 lead to more conservative results. Using the value of F_2, calculate R_2 from Equation 15.
3. Allowance for presence of more than one vehicle in one lane: If all bridge spans are larger than, say, 80 ft, corresponding to the smallest span, read R_3 from the relevant curve for the Ontario code, $R_3 = 1.0$.
4. Allowance of speed restrictions: For vehicle speeds less than 6 mph assume R_4 equal to 1.09; for speeds between 6 and 15 mph, take R_4 equal to 1.04. For higher speeds R_4 is equal to 1.0.
5. Total weight increase: The final factor, R, which combines all allowances, is equal to $1 + (R_1 - 1.0) + (R_2 - 1.0) + (R_3 - 1.0) + (R_4 - 1.0)$, or $(R_1 + R_2 + R_3 + R_4 - 3.0)$.

EXAMPLE

To illustrate the use of the method, the example of a two-lane slab-on-girder bridge having three spans of 131, 164, and 131 ft is considered. The bridge is 20 years old and safely carries an average of 1,000 vehicles per lane per day at an average speed of 55 mph. According to the vehicle weight regulations of the jurisdiction a certain five-axle truck in normal traffic is allowed to carry only 49 tons. The maximum weight that this vehicle can carry as a special-permit vehicle is derived as shown in the following paragraphs. These restrictions were imposed:

1. Strict control of weight,
2. Travel along bridge centerline,
3. Travel speed on bridge less than 6 mph,
4. Other traffic excluded from the bridge, and
5. Wheel spread increased to 9 ft.

Because of weight control, R_1 is taken as 1.10.

For the middle third of the 131-ft span, T = 0.53 sec. From Figure 5 for B = 20 years, n = 1,000, and T = 0.53 sec, F_2 is approximately equal to 0.86. From case II of Figure 11, K is equal to about 33 percent for slab-on-girder bridges and W = 9 ft. Thus from Equation 15,

$$R_2 = 0.86\ [1 + (33/100)] = 1.17.$$

From Figure 12, for a span of 131 ft, R_3 is nearly equal to 1.0.

For vehicle speeds of less than 6 mph, $R_4 = 1.09$. Therefore,

$$R = 1.10 + 1.17 + 1.00 + 1.09 - 3 = 1.36.$$

The maximum permitted vehicle weight is therefore equal to 1.36 x 49 = 66.64 tons.

If an evaluation had shown the bridge to be safe for normal traffic corresponding to the most severe transverse load positions and the modification factor F_2 according to the AASHTO specifications (4), F_2 would be equal to 1.0, and case I of Figure 11 would give K = 40 percent. Then

$$R_2 = [1 + (40/100)] = 1.40.$$

In this case R would be equal to 1.59 and the permitted vehicle weight would be equal to 78.84 tons.

CONCLUSIONS

It has been shown that the modification factors for multilane loading can be obtained statistically and that these factors depend on the life of the bridge, the number of vehicles per day, the length of the span, and the speed of the vehicles. Expressions are developed for calculating this factor for two- and three-lane loadings. The corresponding AASHTO (4) factors are found to be quite conservative, whereas those of the Ontario code (1), although calculated by a different procedure, were found to be more realistic. A method is developed by which safe maximum weights for special-permit vehicles can be obtained without analytically evaluating the bridges on the route.

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Bridge Weight-Limit Posting Practice in the United States

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ABSTRACT

Bridge weight limits allow the continued, limited use of a weak bridge that would otherwise present a significant safety hazard while protecting the legal and economic interests of the bridge owner. For weight limits to be effective, however, bridges must be posted for the proper weight limit, and the posting must be observed and enforced. The federal government became involved in bridge weight-limit posting in 1968 with the creation of National Bridge Inspection Standards (NBIS), which required states to inspect, inventory, and evaluate bridges on federal-aid routes. Weight-limit posting was required for bridges found to have insufficient structural capacity. The results of a survey of state posting practices are presented and the findings of a study on weight-limit posting in the United States are summarized. NBIS provides some engineering guidance for inspecting, evaluating, and posting highway bridges, but considerable engineering judgment is still required to fill the gaps. As a result, even within the limits set by NBIS, engineering practices vary among the states, which leads to differences in posting methods. This is evident from the results of the survey of the states. Development of a simple, uniform posting criterion by which the legitimate differences that exist between states can be rationally considered is recommended.

The United States is currently faced with a massive bridge replacement and rehabilitation problem. FHWA has estimated that there are currently more than 126,600 structurally deficient bridges within the United States (1). Many of these bridges should be rehabilitated or replaced, but they must compete for funding with an equally large number of bridges that have become functionally obsolete because of narrow widths and poor alignments. Because of the cost of modernizing all bridges on the U.S. highway system, it is necessary to delay improvements on many of these bridges for several years. In the meantime, it will be necessary to regulate the traffic on these bridges. This is normally done by establishing weight limits for vehicles using the bridge.

The weight and axle configuration of vehicles allowed to use the highways without special permits is governed by statutory law. In most states, this