AXLE GROUP SPACING: INFLUENCE ON INFRASTRUCTURE DAMAGE

J.J. Hajek, Senior Research Engineer
Ministry of Transportation of Ontario
Downsview, Ontario, Canada M3M 1J8
Telephone: (416)-235-4681

And

A.C. Agarwal, Senior Research Engineer
Ministry of Transportation of Ontario
Downsview, Ontario, Canada M3M 1J8
Telephone: (416)-235-4672

ABSTRACT

In some jurisdictions, such as the Province of Ontario, variation in legal loads with axle spacing on a dual or triple axle are generally based on the load carrying capacity of the bridge components. Current pavement design guides do not consider any effects of axle spacings. A recent RTAC study has also been inconclusive regarding the influence of axle spacings on pavement damage.

In this report, damage effects of dual and triple axles on flexible pavements as a function of axle spacing are examined using various analytical methods, and permissible loads on dual and triple axles are determined for suggested acceptable pavement damage criteria. The results indicate a significant influence of the axle spacing on pavement damage which should be taken into account when determining legal load limits on the axles.

The report also examines the effects of the dual and triple axles of various practical axle spacings on bridge components. Operational load limits are determined for bridges designed by the American and Canadian Bridge Design Codes, and compared with the existing legal limits in Ontario and the proposed RTAC limits for interprovincial transportation in Canada. The results indicate that the bridges designed by the Ontario Highway Bridge Design Code and the new CSA-S6 Code will adequately carry the current legal axle loads in Canada. However, bridges designed by the AASHTO Specifications for the HS 20 loading may be somewhat deficient for the current levels of legal axle loads.

INTRODUCTION

Structural damage to pavements and bridges caused by heavy vehicles depends on many loading characteristics including gross vehicle weight, axle loads, axle group configuration and spacing, load contact pressure, and dynamic loading effects. This paper addresses the effect of one of these load characteristics - axle group configuration and spacing.

Many jurisdictions, for example the Province of Ontario, regulate permissible axle group weights according to axle group spacing, while others, for example France and Sweden, do not [1]. The Highway Traffic Act in Ontario prescribes the permissible load limit for a dual axle (also known as tandem) and a triple axle (also known as tridem) which varies with the axle spacing. Axle spacing is defined as the spacing between the two individual axles in a dual axle, and the distance between the first and the third axles in a triple axle. The variation in permissible loads with the axle spacing is mainly based on the load carrying capacity of the bridge components. The permissible load on a single axle, however, is based on pavement considerations.

AASHTO Pavement Design Guide [2] distinguishes between the damaging effect of dual and triple axle combinations, but assumes that these combinations have the same damaging effect regardless of the axle spacing within the combination. Considering flexible pavements, a triple axle carrying 8 170 kg (18 000 lb) on each axle has the AASHTO load equivalency factor of 1.66 regardless of the actual spacing between the individual axles. However, if the spacing between the axles exceeds an unspecified distance so that the three axles can be considered to be independent, the corresponding AASHTO load equivalency factor is 3.00. Based on a recent literature survey, the effect of axle spacing on infrastructure damage has not been systematically examined before.

The objective of this study is to evaluate the influence of axle spacing on damage caused to flexible pavements, and to determine operational load limits on dual and triple axles of various axle spacings as governed by the bridge components, by (a) reviewing measured pavement response data, particularly the results of RTAC study [3,4], (b) calculating pavement responses using elastic layer theory and, (c) evaluating various bridge loading scenarios.

PAVEMENT DAMAGE

Quantification of the Damaging Effect of Different Loads

The effect of heavy loads on pavement structural damage, such as fatigue (alligator) cracking and rutting, has been traditionally expressed using the concept of load equivalency factors (*LEF*'s). For convenience, the *LEF*'s have been related to a standard axle load of 8 170 kg (18 000 lb) imposed on a single axle with dual tires. This load is called Equivalent Single Axle Load (*ESAL*). Load equivalency factors can be obtained in two ways; by a field experiment or by an evaluation of pavement responses to individual loads.

LEF's Obtained by Field Experiments

A number of axle loads of a given magnitude and type required to cause a certain level of pavement deterioration, N_i, is determined and compared with the number of *ESAL*'s required to cause the same amount of pavement deterioration on the identical pavement structure, N_{ESAL}:

$$LEF = \frac{N_{ESAL}}{N_i} \qquad (1)$$

The resulting *LEF*'s depend on the definition of pavement deterioration and on its level, and on the type and strength of the pavement structure. Thus, for the same N_i, there may be different *LEF*'s for different pavement types, thicknesses, subgrades, and pavement distresses. The best known example of a field experiment is AASHTO Road Test in the early 1960s [5]. The Test encompassed a number of different pavement structures, but on a uniform subgrade, and the *LEF*'s were mainly related to pavement damage in terms of roughness, which is directly related to the way the pavement serves the travelling public. This approach to obtaining load equivalency factors is extremely expensive and time consuming and cannot be used to evaluate load configurations and pavement structures which physically do not exist.

LEF's Obtained by Evaluating Pavement Responses to Individual Loads

Measured or calculated pavement responses to individual load configurations are used to calculate load equivalency factors as follows:

$$LEF_r = \left(\frac{R_i}{R_{ESAL}}\right)^n \qquad (2)$$

where:

LEF_r = Load Equivalency Factor based on pavement response r

R_{ESAL} = pavement response r to one *ESAL*

R_i = pavement response r to the load of a defined magnitude and type designated as i

n = exponent to ensure that *LEF* (from Equation 1) is equal to LEF_r (from Equation 2) for pavement response r

This approach, used in this study, requires the identification of pavement responses, such as strains and stresses, which cause specific pavement structural distresses. These distresses should be related to pavement deterioration that affect the way pavements serve the travelling public. As a corollary, it is assumed that increased strains and stresses in the pavement structure increase pavement distresses (and reduce the pavement serviceability). Furthermore, the approach is faced with two main complications. Firstly, load equivalency factors depend on the type and amount of pavement distresses, and there are many possible combinations. Secondly, according to Equation 2, it is assumed that the pavement response to an axle group load, which can be rather complex, can be characterized and summarized by one number. In the absence of a universally accepted procedure to summarize pavement responses in terms of one encompassing number, the use of different procedures may yield different results.

Response Parameters Used

Load equivalency factors used by an agency should be based on the pavement distress or distresses which trigger the local need for pavement rehabilitation. For example, Hallin et al [6] developed LEF's for Washington state based on fatigue cracking because "cracking is the principal form of asphalt pavement distress in Washington state". A statistical examination of Ontario pavement distress data [7] revealed that practically all 15 routinely evaluated pavement surface distresses occur at the critical levels of severity and density requiring rehabilitation, and that fatigue cracking is not a predominant distress. For this reason, the following three traditional generic pavement responses linked to the formation of pavement distress have been used in this study:

a) *Pavement surface deflection:* This response has been linked to pavement life-span measured mainly in terms of roughness. Several pavement distresses, such as cracking, distortion, and rutting can contribute to pavement roughness.

b) *Interfacial strain:* Strain at the bottom of asphalt concrete layer which is related to fatigue (alligator) cracking.

c) *Vertical strain on the top of the subgrade:* This response has been related both to rutting in the pavement structure and to pavement life-span.

A typical history of these three responses for a flexible pavement subjected to a dual axle load is shown in Figure 1.

Summation of Pavement Responses to Axle Loads

The comparison of damage caused by different loads requires quantification and summation of pavement response curves (Figure 1) resulting from the passage of these loads. Two approaches can be used: discrete methods and integration methods. Discrete methods use only discrete values at the peaks and valleys of the response curves, while integration methods attempt to use the whole response curve. The basic difference between the two methods is illustrated in Figure 2.

Discrete Methods

The discrete methods used in this study are outlined in Figure 3. *LEF*'s are calculated by accumulating peak responses by modifying Equation (2) as follows:

$$LEF_{r,m} = \frac{\sum_{i=1}^{p} (r_i)^n}{(r_{ESAL})^n} \tag{3}$$

where:

$LEF_{r,m}$ = load equivalency factor for pavement response r, and method m

r_i = discrete pavement response for cycle i identified by method m

n = as defined before, adopted to be 3.8 based on an extensive review by Christison [4]

p = number load cycles (axles)

RTAC Method

This method was originally used for the analysis of measured pavement responses as part of Canadian Vehicle Weights and Dimensions Study [3,4]. For surface deflections (and in this study also for strains on the top of the subgrade) the peak under the lead (first) axle is extracted first, followed by the through to peak differences in the response curve for the subsequent axles (Figure 3). For interfacial strains, only the peak tensile strains measured from the rest position are used.

University of Waterloo Method

This method was developed by Hutchinson et al [8] for isolating and counting surface deflection cycles. In this study, it was also used for summation of subgrade strains. The method follows an ASTM Standard Practice [9] which recommends that the highest peak and lowest valley is used first, followed by the second largest cycle, etc., until all peak counts are used (Figure 3).

Peak Method

For surface deflections and subgrade strains, Peak method uses the total response under each axle from the rest position. For interfacial strains, Peak method uses the peak to through rise and falls in the strain history (Figure 3), a procedure which is identical to that recommended by ASTM Standard Practice for Cycle Counting in Fatigue Analysis [9] and which appears to be an improvement over the RTAC method.

Regarding surface deflections, proponents of this method [10] argue that even though the surface deflections between two subsequent axles do not reach a rest position, asphalt concrete layer at this location reverses its curvature (tensile strain to compressive strain, Figure 1) so that the inclusion of the total deflection best models the overall pavement response. Another argument in support of this method may be advanced by considering how different response curves, such as those shown for cases a and b in Figure 2, are accounted for by Peak method. Peak method uses responses D_1 and D_3 and thus distinguishes between the damaging effects of the two cases while the other two methods, RTAC and Waterloo, do not (they are based on responses D_1 and D_2).

Integration Methods

Flexible pavements respond to loads as visco-elastic systems with resulting permanent and elastic strains. The permanent strains are influenced by both the amount and the duration of load. Integration methods take both these parameters into account by integrating the response curve expressed as a function of time or distance. Referring to Figure 2, integration methods distinguish between response curves of not only cases a and b, but also between cases a and c which have similar peaks. The formula used in this study for calculating *LEF's* by integration is shown in Figure 2. Conceptually, it resembles the formulation used by Govind and Walton [11].

Measured Pavement Responses

Measured pavement responses used in this study were taken from the Canadian Vehicle Weights and Dimensions Study [3]. This 1985 study provides a comprehensive set of measured pavement responses in terms of surface deflections and interfacial strains measured at 14 sites for a variety of loading conditions. The results based on these measurements are referred to in this study as "RTAC measurements".

Modeling of Pavement Response

Computational Method

The flexible pavement was modeled as an idealized elastic layered system and its responses to loads were calculated by the ELSYM5 computer program [12]. The use of elastic layer theory to obtain load equivalency factors has been successfully used before [6,13,14,15].

Pavement Structure

Calculations were done for thin and thick flexible pavement structures shown in Figure 4. The thin section has a structural number (*SN*) of 3.0, and represents a low-volume road; the thick section has a *SN* of 5.7 and represents a typical structure for a high-volume facility. It may be noted that the average *SN* for the 14 sections used in the RTAC study was 5.0.

Pavement Loadings

Analyses were done for single, dual, and triple axle groups. All axles had dual tires spaced about 350 mm (14 in.) apart. The tire footprints were assumed to be circular with a pressure of 690 kPa (100 psi). Axle loads on individual axles ranged from 5 450 kg (12 000 lb) to 11 800 kg (26 000 lb).

As the load increased, the tire contact area increased because the tire pressure was held constant.

Location of Maximum Deflections and Strains

When comparing pavement response to different axle loads, it is important to use the maximum responses in all cases as a common denominator. Analysis showed that the maximum responses for deflection and strains occur on the line at the midpoint between the dual tires, regardless of axle spacing. The responses along this line were calculated in sufficient detail to identify all relevant features of the response curves required for analysis.

Effect of Axle Spacing on Pavement Damage

Load equivalency factors are plotted as a function of axle spacing for double and triple axles in Figures 5 and 6 respectively. For easier comparisons, the axle loading in Figures 5 and 6 is kept constant at a standard design load of 8 170 kg (18 000 lb) per axle. The results are briefly interpreted in the following.

Summation of Pavement Responses to Axle Loads

Summation methods have a large influence on *LEF*'s, notably on *LEF*'s based on surface deflections and subgrade strain. Based on available information and data, it is not possible to unequivocally recommend any particular summation method. (Reader can only be referred to previous description of the methods and the assumptions on which they are based.)

Measured Versus Calculated Pavement Responses

It appears that the summation methods have a larger influence on the resulting *LEF*'s than whether the original pavement responses (on which the methods operate) were measured or calculated. For example, considering *LEF*'s for dual axles based on surface deflections (top of Figure 5), the results can be grouped according to the summation method rather than whether the pavement responses were measured or calculated. Future efforts should be directed towards a better understanding of the influence the response curves have on pavement damage.

Pavement Response Parameters

For a sufficiently large axle spacing, all *LEF*'s tend to approach 2.0 for dual axles and 3.0 for triple axles. Overall, regardless of the summation method used, *LEF*'s based on deflections are larger than those based on strains (interfacial, and subgrade) and decrease (monotonously) with increasing axle spacing. The *LEF*'s based on interfacial strains and RTAC method increase (rather than decrease) with larger axle spacing. The same also roughly applies to subgrade strains. When single axles are close together, the compressive strain caused by one axle can offset the tensile strain caused by another axle (Figure 1), effectively reducing the net tensile pavement strain. It may be recalled that RTAC method does not work with the total strain cycle. It excludes compressive strain from the *LEF*'s calculation but not from the strain response calculation (or from the measurement).

Pavement Structure

The influence of axle spacing on *LEF*'s decreases with pavement structural strength. Thin, structurally weak pavements do not distribute axle loads effectively. Consequently, their responses are governed mainly by individual axles. For example, regardless of axle spacing or the summation method used, *LEF*'s for dual axles based on interfacial strains are equal to 2.0.

Comparison With AASHTO Factors

Typical *LEF*'s recommended by the AASHTO Guide [2] for dual and triple axles are given in Table 1. The AASHTO *LEF*'s do not change with axle group spacing. Also shown in Table 1 are *LEF*'s for zero spacing and for spacing large enough so that axles can be considered to act independently. This spacing is not defined by the AASHTO Guide. Considering that the spacing between the consecutive axles can be quite variable, particularly for axle groups which do not equalize loadings, the results suggest that AASHTO *LEF*'s would benefit from including the influence of axle spacing.

Damage Comparisons

The maximum allowable axle weight for single axles with dual tires in Ontario is 10 000 kg (22 050 lb). This represents 2.0 or 2.1 *LEF*'s, depending on the pavement response used. If the single axle can be allowed to have a maximum of 2.0 *LEF*'s, then, based on the principle that any axle can cause identical damage, a dual axle can be allowed to have 4.0 *LEF*'s and a triple axle can be allowed to have 6.0 *LEF*'s. Based on this principle, what are the maximum weights for dual and triple axles corresponding to the single axle weight of 10 000 kg? This question is addressed in this section.

Figures 7 and 8 show the influence of axle spacing and axle group weights for dual and triple axles. Also shown are lines indicating damage levels for a corresponding number of single axles. For example, considering dual axle and surface deflections (top of Figure 7), based on Peak summation method and 1.0 m spacing, the total dual axle group weight causing the same damage (having the same *LEF*) as two single axles with maximum allowable weight, is 14 900 kg.

The results of Figures 7 and 8 are summarized, together with AASHTO [2] data and Ontario allowable limits [16] in Table 2. The following two basic observations can be made, based on Table 2:

(1) Ontario permissible weights for dual and triple axles are lower than those established by any computational scenario with the exception of deflection-based Peak method. The greatest difference (3 400 kg) exists for triple axles on the largest spacing (4.8 m). Ontario regulations allow 28 600 kg, while the deflection-based Peak method would allow only 25 200 kg.

(2) AASHTO-based weights are higher than the weights based on deflections and interfacial strains regardless of the summation method used. They happen to be roughly similar to the allowable weights based on subgrade strain response evaluated by RTAC method. It appears that the AASHTO Guide may underestimate the damaging effects of dual and triple axles in comparison with the single axles.

BRIDGE CONSIDERATIONS

Bridge Design Codes and Live Load Models

Most of the existing bridges on the provincial highways in Ontario were designed by the AASHTO Specification for H 20 or HS 20 load [17]. H 20 Truck is a part of the HS 20 Truck, shown in Figure 9(a), obtained by deleting the third axle. For short span bridges and for the local structural components, H 20 and HS 20 loadings both give the same force effects. A small number of existing bridges was designed only for the H 15 load which is 75 percent of the H 20 load. New bridges on the provincial highways constructed during the 1980s generally have been designed according to the Ontario Highway Bridge Design Code (OHBDC) for the OHBD loading [18]. The OHBD Truck configuration is shown in Figure 9(b).

On the municipal system, however, the existing bridges were generally designed for the H 15, H 20 or HS 20 loading. A large number of these municipal bridges are currently posted for a reduced load limit.

The AASHTO HS 20 loading represents the operational load level that may be permitted on a bridge, designed for this loading using AASHTO Specifications, without overstressing the bridge or compromising safety in accordance with sound engineering principles. In the United States, AASHTO loading serves as the basis of legal load limits in most jurisdictions. It must be noted that the legal limit on a single axle is governed by the pavement considerations and not by the bridges. The permissible legal loads on the axles can be determined by direct comparison of the load effects of the dual and triple axles in the structural components with those caused by the HS 20 Truck.

The OHBD loading includes the observed overloads in everyday trucking, to the extent of 10 000 kg on an axle unit (single, dual or triple axle), a group of consecutive axle units, or on the total vehicle [19]. Thus it represents a load level of 10 000 kg above the legal loads where 10 000 kg can be considered as the operational overload allowance in the bridge design. Maximum permissible load on a dual or triple axle is obtained by deducting the operational overload allowance from the theoretical load carrying capacity of the structural components determined by a direct comparison of the load effects due to the dual or triple axle with those caused by the OHBD loading.

Until recently, the National Standard of Canada, CSA CAN3-S6, for design of highway bridges included a live loading model similar to the AASHTO Standard. The revised edition of the CSA-S6 Code [20], based on the limit states philosophy, now has a new live load model which is expected to be used all across Canada outside of Ontario. The new truck model, CS-600 is shown in Figure 9(c).

The CS-600 Loading [21] is representative of the operational load level that can be permitted on a bridge designed by the CSA-S6 Standard while maintaining the required margin of safety from sound engineering principles. The permissible limit on a dual or triple axle can, therefore, be determined by a direct comparison of the force effects in a structural component due to the axle with those caused by the CS-600 Truck.

Critical Bridge Components

A dual axle or a triple axle would govern the load effects in the following bridge components:

(a) short span longitudinal components such as beams in short span bridges, stringers, etc.,

(b) transverse floor beams and truss verticals, and

(c) deck slab.

Short Span Longitudinal Components

In the short span longitudinal components, force effects to be considered are moments and shears in simple spans. Tables 3 to 5 list the maximum moments and shears in simple spans of 3.0 to 8.0 m due to the HS 20 Truck, the OHBD Truck and the CS-600 Truck models.

Figure 10 shows positions of a dual axle for maximum force effects in a simple span. In a recent study conducted by the Roads and Transportation Association of Canada (RTAC), a minimum interaxle spacing of 3.0 m was proposed between a single axle and a dual axle [23]. Therefore possibility of a single axle along with the dual axle on the short span components is also considered, as shown in Figure 10. Proposed interaxle spacings between two dual axles, or between a dual and a triple axle are large enough so that their combined effect would not be critical for the short span components. In Ontario, existing vehicle configurations include axle units at much shorter interaxle spacings compared to the RTAC Proposal. This would have a more severe effect on the bridge components.

For a dual axle, three axle spacings, 1.2 m, 1.5 m and 1.8 m, are considered. These are the most common dual axle spacings in use in Ontario. The moments and shears in simple spans of 3.0 to 8.0 m due to a unit value of load P on the dual axle, or due to a combination of the dual and single axles are given in Table 6. For the bridges designed by the AASHTO specifications for HS 20 loading, the permissible load, W_p, is given by,

$$W_p = L_{HS20}/L_{dual} \tag{4}$$

Where

L_{HS20} = force effect due to the HS 20 Truck from Table 3

L_{dual} = force effect due to a unit load on the dual axle alone or in combination with a single axle at 3.0 m distance carrying a load equal to half of the load on the dual axle, whichever is more critical; given by Table 6

Similarly, the permissible load on bridges designed in accordance with the CSA-S6 Code for CS-600 loading is given by,

$$W_p = L_{CS600}/L_{dual} \tag{5}$$

where,

L_{CS600} = force effect due to the CS-600 Truck from Table 5

Where maximum load effect is caused by the dual axle alone, permissible load on the dual axles for the bridges designed in accordance with the OHBDC is given by the following for reasons explained earlier,

$$W_p = L_{OHBD}/L_{dual} - 10\ 000 \text{ kg} \tag{6}$$

where,

L_{OHBD} = force effect due to the OHBD Truck from Table 4

Where maximum load effect is caused by a combination of the dual axle and the single axle at 3.0 m distance from the dual axle, the overload allowance of 10.0 t is split between the dual axle and the single axle. Thus in this case,

$$W_p = L_{OHBD}/L_{dual} - 6\ 670 \text{ kg} \tag{7}$$

Based on the above relationships, permissible operational load limits on dual axles for moments and shears in short span longitudinal components were determined and plotted in Figures 12 to 14 for the three axle spacings. It is clear that the permissible loads would increase with an increase in axle spacing.

Figure 11(b) shows positions of a triple axle for maximum force effect in a simple span. Moments and shears due to the unit value of total load, P, on the triple axle are given in Table 7 for axle spacings 2.4, 3.6 and 4.8 m. Again, the permissible load on triple axles is given by,

$$W_p = \begin{cases} L_{HS20}/L_{triple} & \text{- for HS 20 bridges} \\ L_{OHBD}/L_{triple} - 10\ 000 \text{ kg} & \text{- for OHBDC bridges} \\ L_{CS600}/L_{triple} & \text{- for CS-600 bridges} \end{cases} \tag{8}$$

where,

L_{triple} = force effect due to unit load on the triple axle given by Table 7

Permissible load on the triple axles for three axle spacings based on moments and shears in short span longitudinal components are shown in Figures 15 to 17. Once again, permissible load increases with an increase in axle spacing.

Floor Beams And Truss Verticals

Floor beams are the transverse components supporting longitudinal stringers in truss or girder bridges. The load transferred to the floor beams or the vertical components of relevance in the trusses would be equivalent to the reaction at the floor beams to the loads carried by the stringers as shown in Figures 10 and 11. The load transferred to the floor beam due to one design truck in a traffic lane are given in Table 8 for the floor beam spacings of 3.0 to 8.0 m.

The placing of the dual axle together with a single axle at an interaxle spacing of 3.0 m is shown in Figure 10(d) for maximum force effects in the floor beams. Table 9 gives the maximum load transferred to the floor beam due to the load, with unit value of total load, *P*, on the dual axle. Once again using the approach given earlier for the short span longitudinal components, the permissible load on the dual axles for bridges designed by the three codes were determined and plotted in Figures 12 to 14. In comparison to the short span longitudinal components, floor beams and the truss verticals appear to be more critical, giving a lower value of the permissible dual axle load.

Figure 11(c) shows the position of a triple axle to obtain maximum load on a transverse floor beam. Table 10 gives the maximum load transferred to the floor beam due to the unit value of total load, *P*, on the triple axle for three axle spacings of 2.4, 3.6 and 4.8 m. Permissible loads on the triple axles determined using these values are also shown in Figures 15 to 17. For triple axles also, floor beams and the truss verticals appear to be more critical.

Deck Slabs

Concrete deck slabs have been found to have a significantly high capacity to carry wheel loads through the membrane action [22], and hence would not be critical.

Comparison of the Permissible Loads

A comparison of the lowest value of the permissible operational loads from Figures 12 to 17 with the legal limits in Ontario [16] and legal limits proposed by RTAC [23] are given in Tables 11 and 12 for the dual and triple axles, respectively. Generally, the permissible loads on the HS 20 bridges are lower as compared to the bridges designed by the Ontario Highway Bridge Design Code and the new CSA-S6 Code. The difference is due, in part, to the somewhat conservative design approach in the AASHTO Specifications, as well as a difference in the design load models. For the Ontario and CSA Codes, the design model represents modern truck traffic reflecting current legal load levels. However, it is clear that the legal load limits in Ontario are at a reasonable level and reflect increasing load carrying capacity of bridge components with increases in axle spacings.

CONCLUSIONS

1. Contrary to the inference from the pavement design guides, axle spacings have significant effect on pavement damage which should be accounted for in determining the permissible load limits on dual and triple axle units. AASHTO Guide appears to underestimate the damaging effect of dual and triple axles in comparison with the single axles.

2. Permissible weights on dual and triple axles in Ontario are generally lower than those determined by various computational methods used to analyse pavement damage with the exception of the Peak method which gives up to 12 percent lower permissible weights for larger axle spacings compared to the Ontario legal limits.

3. Permissible loads for the axle units based on force effects on the bridge components increase with an increase in the axle spacing.

4. Bridges designed in accordance with the Ontario Highway Bridge Design Code or the new limit states design based CSA-S6 Code are adequate to carry the current levels of legal axle loads in Canada. Bridges designed in accordance with the AASHTO Standard for HS 20 loading may be somewhat deficient.

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23. "The Memorandum of Understanding on Interprovincial Vehicle Weights and Dimensions", Roads and Transportation Association of Canada, Ottawa, Ontario, 1988.

Table 1/ AASHTO Load Equivalency Factors for Dual and Triple Axles

Axle Type	**Zero Spacing** (only one axle)	**Typical Spacing**[1)]	**Large Spacing**[1)] (independent axles)
Tandem	13.9	1.38	2.0
Triple	above 53	1.68	3.0

Note 1: Spacing between consecutive axles. The actual spacing is not defined.

Conditions: Flexible pavement, SN = 5, $p_t = 2.5$.
Load on each individual axle is 8170 kg (18000 lb).

Source: Reference 2.

Table 2/ Equivalent Damage Loads for Dual and Triple Axles[1)]

The total weight of dual (or triple) axles in kg which causes the same amount of damage as 2 (or 3) single axles with the maximum allowable weight of 10000 kg.

How Determined	**Dual Axle**, Spacing **1.2 m**	**1.8 m**	**Triple Axle**, Spacing **2.0 m**	**4.8 m**
Deflections				
Peak Method	14 900	16 700	20 300	25 200
Waterloo Method	18 000	19 600	26 200	28 300
Strains, A.C.				
Peak Method	18 300	18 900	28 300	29 900
RTAC Method	19 000	19 700	32 100	29 900
Strains, Subgrade				
Peak Method	17 100	18 600	26 100	30 100
RTAC Method	20 600	22 000	31 000	35 500
AASHTO[2)]	21 600	21 600	34 300	34 300
Ontario Weight Limits[3)]	15 400	19 100	19 500	28 600

Notes:
1) Flexible pavement, SN = 5. See Figure 4.
2) Source: Reference 2.
3) Source: Reference 16.

Table 3/ Force Effects in Simple Spans due to the HS 20 Truck

Span Length (m)	Moment (t.m)	Shear (t)
3.0	107	142
4.0	142	142
5.0	178	163
6.0	214	183
7.0	249	198
8.0	285	209

Table 4/ Force Effects in Simple Spans due to the OHBD Truck

Span Length (m)	Moment (kN.m)	Shear (kN)
3.0	150	224
4.0	202	238
5.0	271	249
6.0	340	264
7.0	410	275
8.0	488	283

Table 5/ Force Effects in Simple Spans due to the CS-600 Truck

Span Length (m)	Moment (kN.m)	Shear (kN)
3.0	135	180
4.0	180	180
5.0	225	192
6.0	270	200
7.0	315	206
8.0	367	225

Table 6/ Force Effects in Simple Spans due to a Unit Load on a Dual Axle and a Single Axle at 3.0 m Carrying Half of the Dual Axle Load

Span Length (m)	Axle Spacing 1.2 m		Axle Spacing 1.5 m		Axle Spacing 1.8 m	
	Moment	Shear	Moment	Shear	Moment	Shear
3.0	0.480	0.800	0.422	0.750	0.368	0.700
4.0	0.723	0.850	0.660	0.813	0.601	0.775
5.0	0.968	0.960	0.903	0.900	0.841	0.840
6.0	1.223	1.050	1.148	1.000	1.084	0.950
7.0	1.594	1.114	1.150	1.513	1.434	1.029
8.0	1.967	1.163	1.887	1.125	1.808	1.088

Table 7/ Force Effects in Simple Spans due to a Unit Load on a Triple Axle

Span Length (m)	Axle Spacing 2.4 m		Axle Spacing 3.6 m		Axle Spacing 4.8 m	
	Moment	Shear	Moment	Shear	Moment	Shear
3.0	0.350	0.600	0.250	0.467	0.250	0.400
4.0	0.600	0.700	0.400	0.550	0.333	0.467
5.0	0.850	0.760	0.650	0.640	0.481	0.520
6.0	1.100	0.800	0.900	0.700	0.700	0.600
7.0	1.350	0.829	1.150	0.743	0.950	0.657
8.0	1.600	0.850	1.400	0.775	1.200	0.700

Table 8/ Load on a Transverse Floor Beam due to one Design Truck

Floor Beam Spacing(m)	HS 20 Truck (kN)	OHBD Truck (kN)	CS-600 Truck (kN)
3.0	142	224	180
4.0	142	244	180
5.0	168	263	192
6.0	194	276	200
7.0	212	303	231
8.0	225	333	270

Table 9/ Load on Transverse Floor Beams due to Unit Load on a Dual Axle and a Single Axle at 3.0 m Carrying Half of the Dual Axle Load

Floor Beam Spacing(m)	Axle Spacing 1.2 m	Axle Spacing 1.5 m	Axle Spacing 1.8 m
3.0	0.800	0.750	0.700
4.0	0.975	0.938	0.900
5.0	1.080	1.050	1.020
6.2	1.150	1.125	1.100
7.0	1.200	1.179	1.157
8.0	1.238	1.219	1.200

Table 10/ Load on transverse floor beams due to unit load on a triple axle

Floor Beam Spacing(m)	Axle Spacing 2.4 m	Axle Spacing 3.6 m	Axle Spacing 4.8 m
3.0	0.733	0.600	0.467
4.0	0.800	0.700	0.600
5.0	0.840	0.760	0.680
6.0	0.867	0.800	0.733
7.0	0.886	0.829	0.771
8.0	0.900	0.850	0.800

Table 11/ Comparison of Permissible Loads on a Dual Axle

Axle Spacing (m)	HS 20 Bridges (kg)	OHBD Bridges (kg)	CS-600 Bridges (kg)	Ontario Limit (kg)	RTAC Proposal (kg)
1.2	14 890	17 800	17 740	16 800	17 000
1.5	15 480	18 350	18 130	17 900	17 000
1.8	16 130	18 920	18 540	19 100	17 000

Table 12/ Comparison of Permissible Loads on a Triple Axle

Axle Spacing (m)	HS 20 Bridges (kg)	OHBD Bridges (kg)	CS-600 Bridges (kg)	Ontario Limit (kg)	RTAC Proposal (kg)
2.4	18 140	20 970	22 950	21 300	21 000
3.6	20 740	25 320	25 500	24 400	24 000
4.8	24 190	28 380	27 820	28 600	Not Spec.

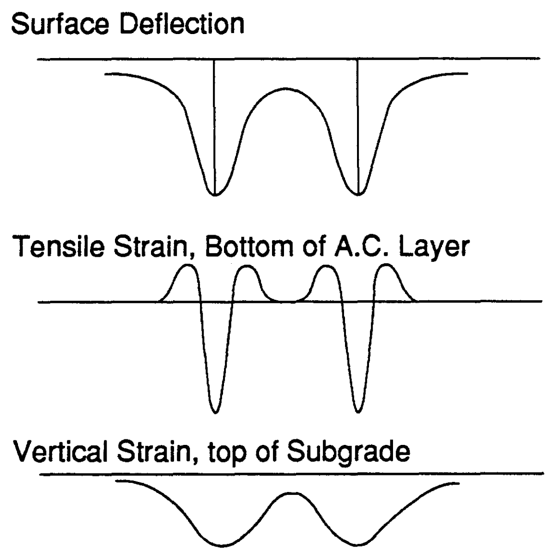
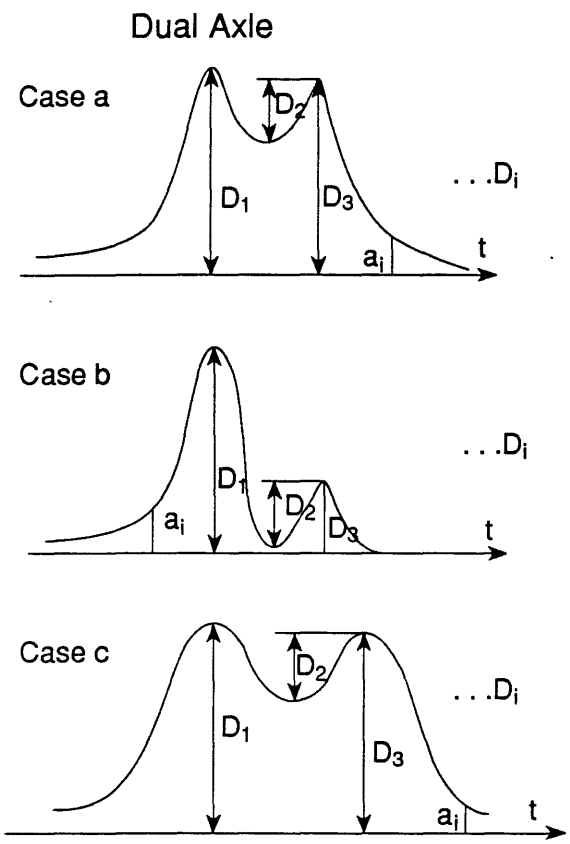
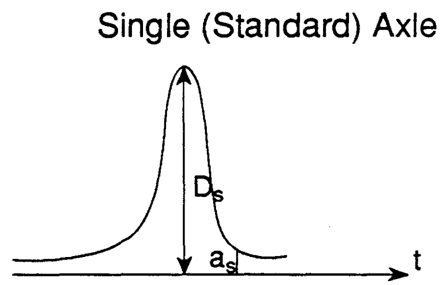


Figure 1/ Typical Response of Flexible Pavement to Dual Axle Load





Calculation of Load Equivalency Factors

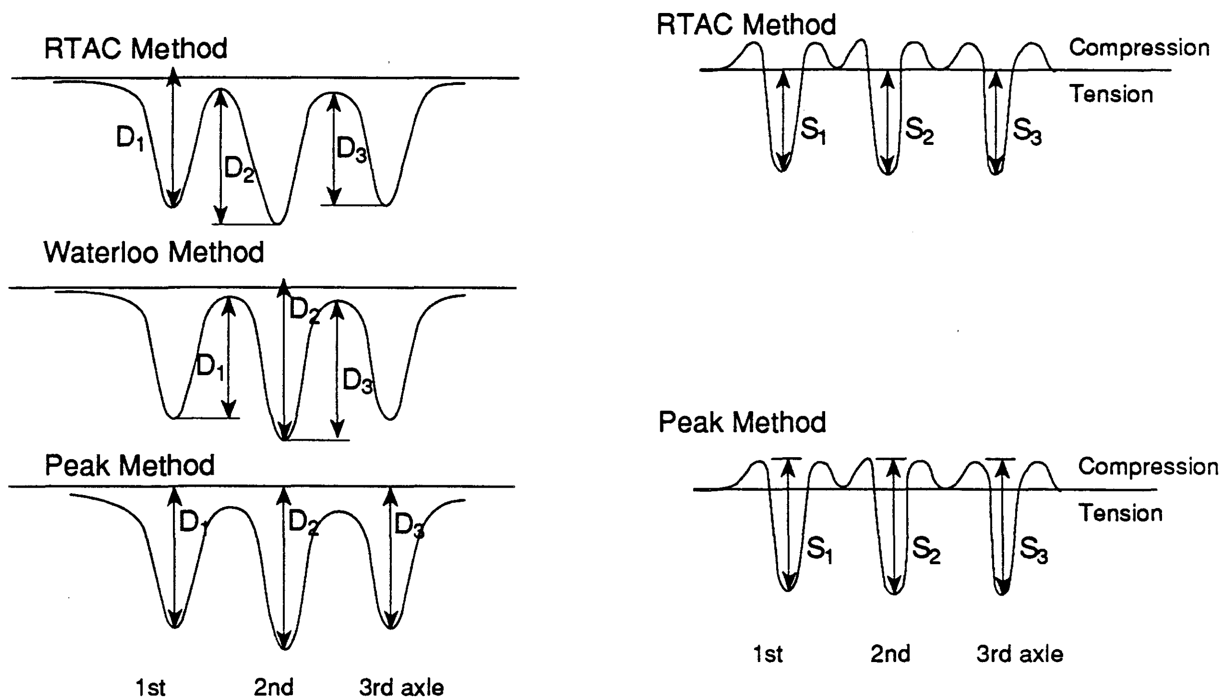
1. Discrete Method

$$LEF = \left(\frac{\Sigma\ D_i}{D_s}\right)^{\alpha}$$

2. Integration Method

$$LEF = \frac{\int_o^t a_i^{\alpha-1}\ dt}{\int_o^t a_s^{\alpha-1}\ dt}$$

Figure 2/ Discrete and Integration Methods for Calculation of Load Equivalency Factors



Surface Deflections, or Vertical Strain on the Top of Subgrade

Tensile Strain at the bottom of Asphalt Concrete Layer

Figure 3/ Different Discrete Methods Used to Estimate the Effect of Multiple Axle Groups

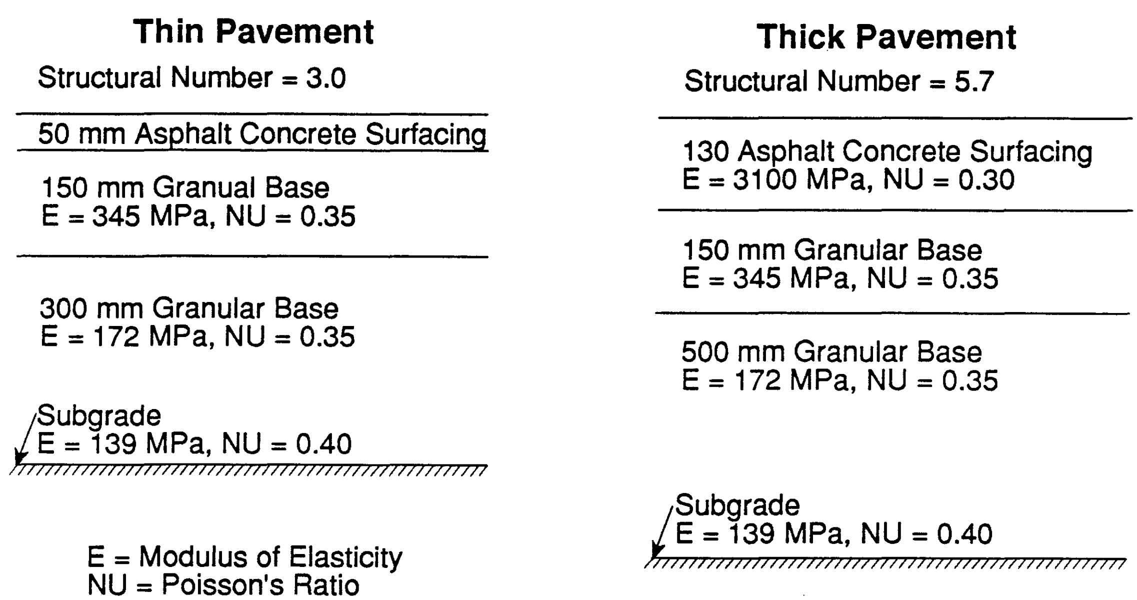


Figure 4/ Flexible Pavement Structures Used in Analysis

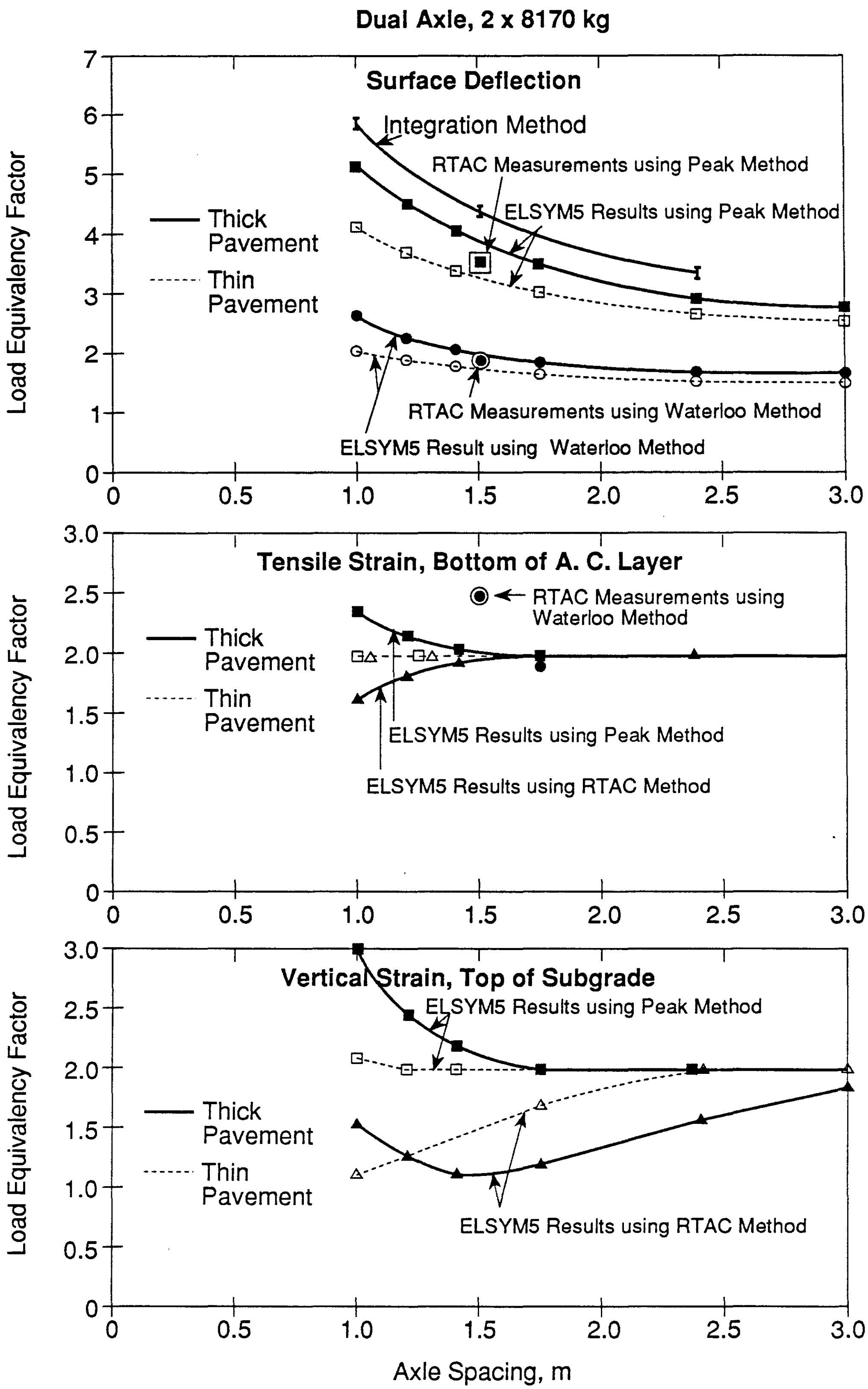


Figure 5/ Influence of Axle Spacing on Load Equivalency Factor for Dual Axles

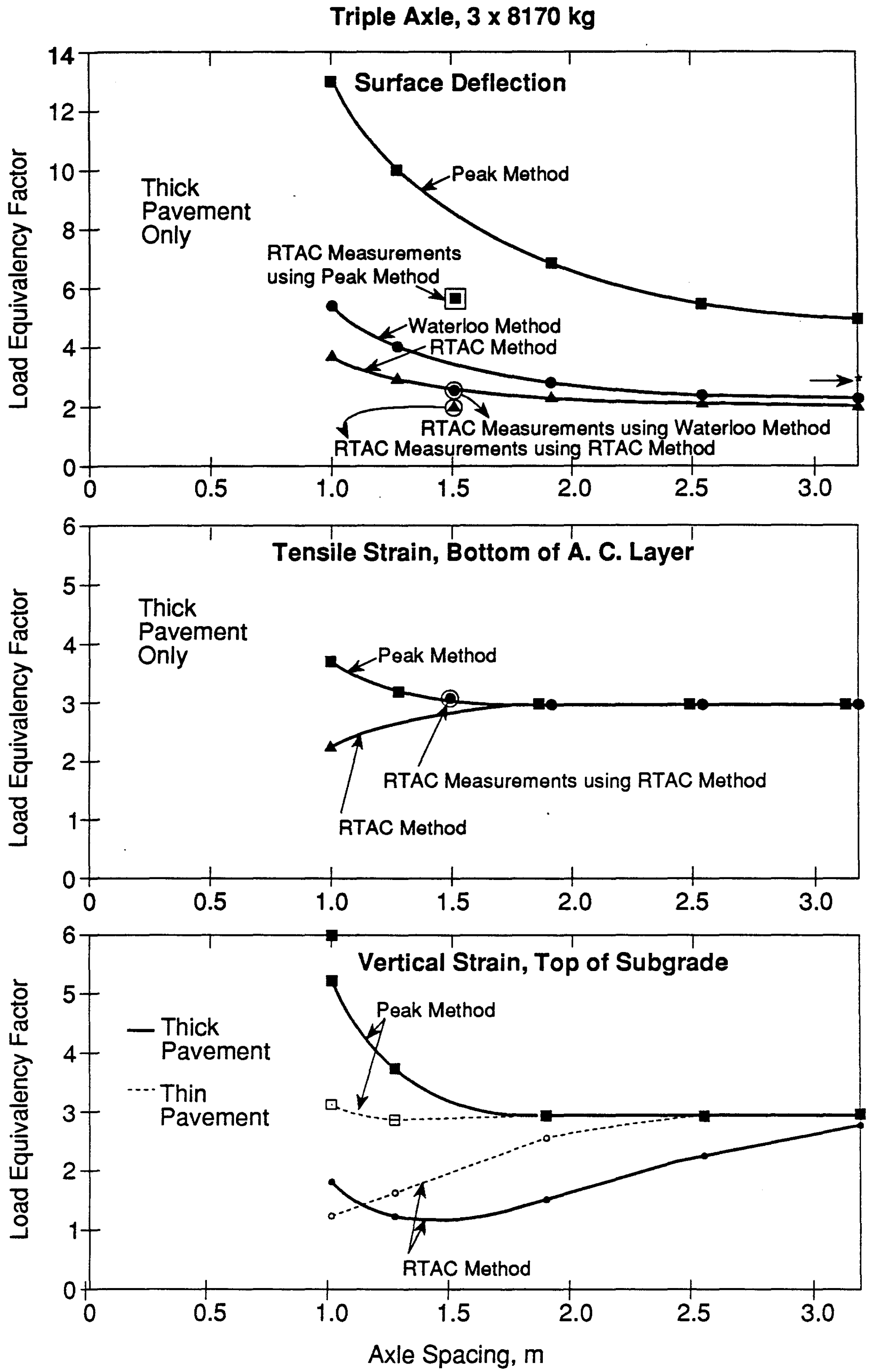


Figure 6/ Influence of Axle Spacing on Load Equivalency Factor for Triple Axles

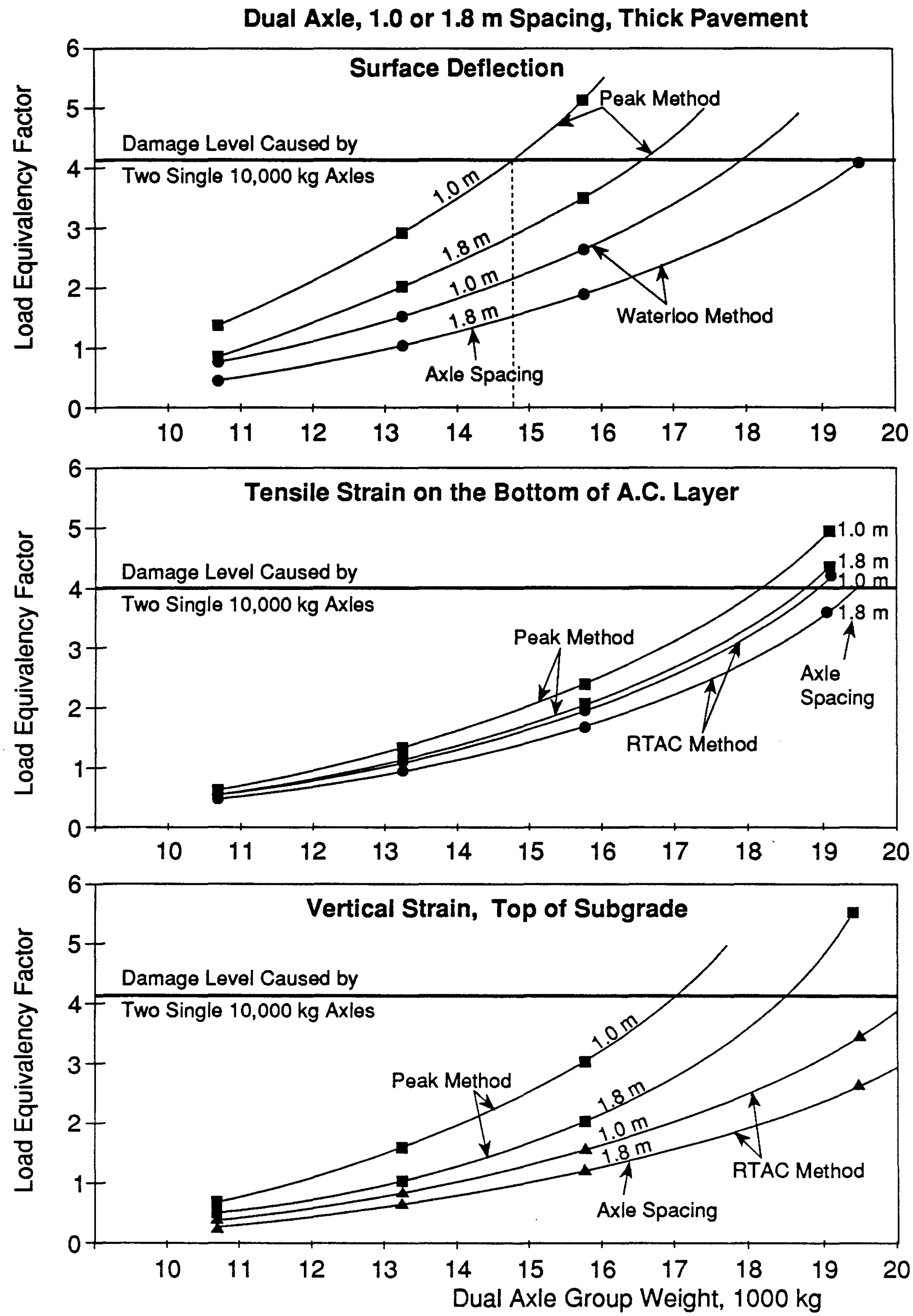


Figure 7/ Comparison of Damage Caused by Two Single Axles Versus One Double Axle Group (Thick Pavement)

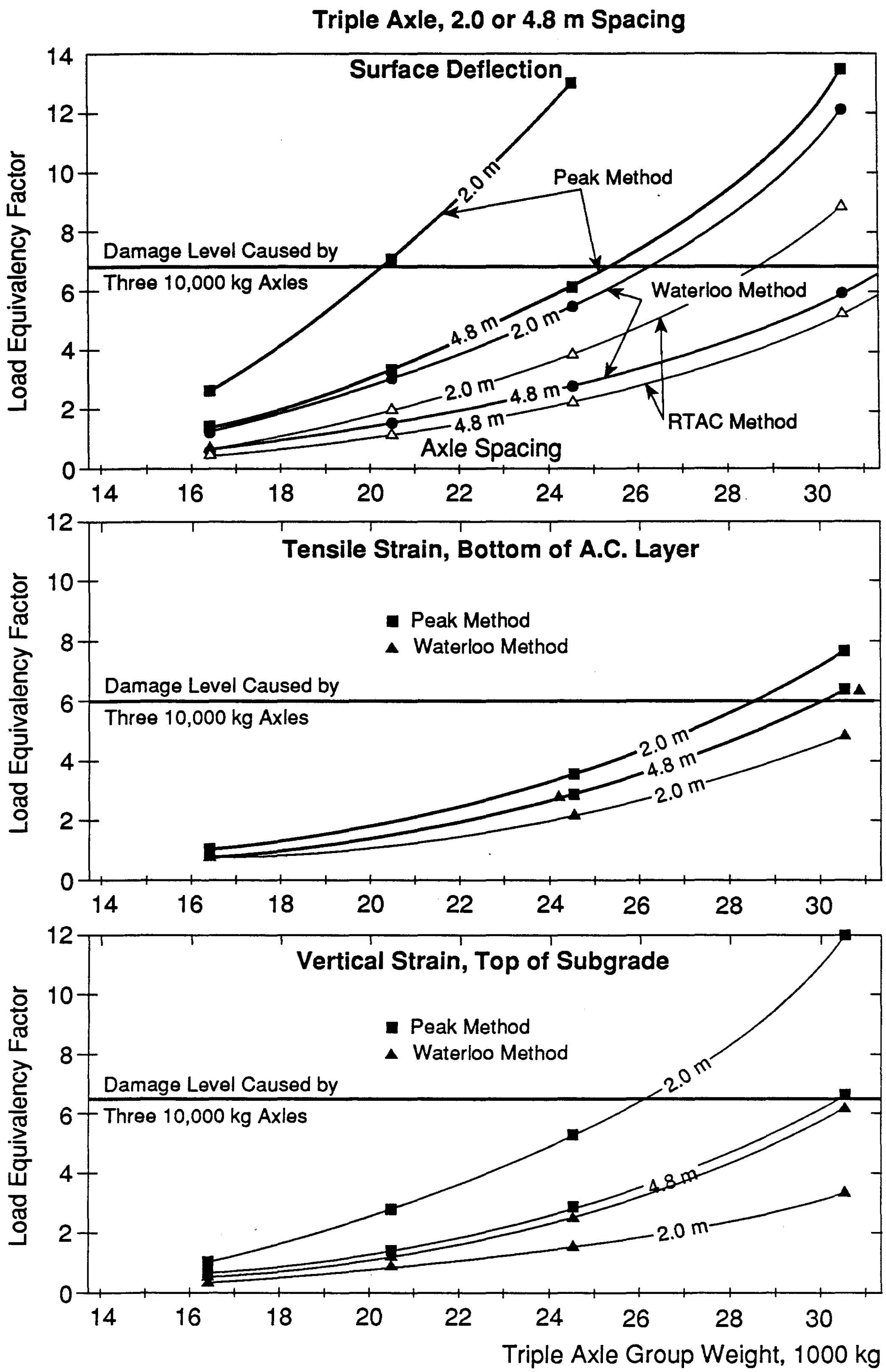
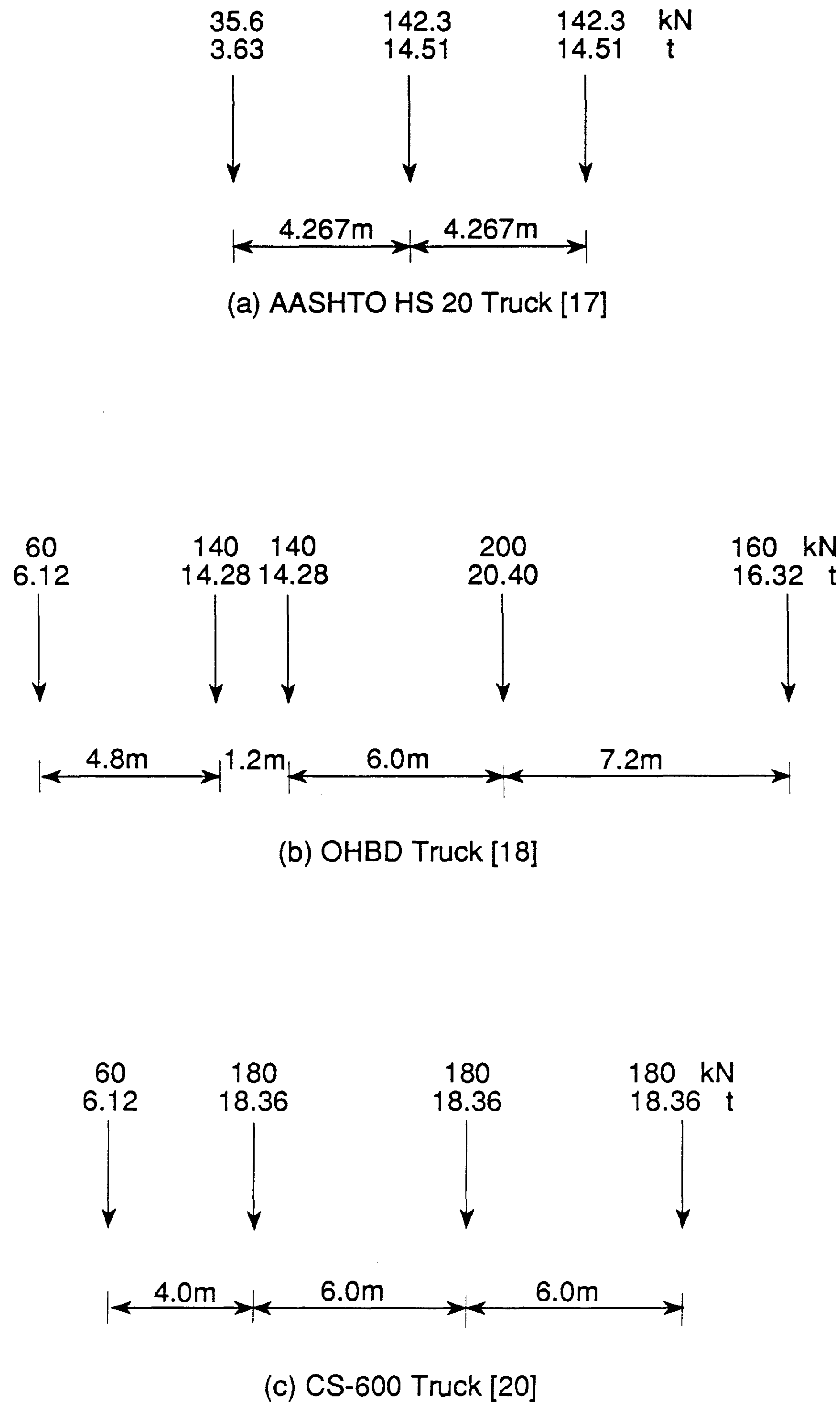


Figure 8/ Comparison of Damage Caused by Three Single Axles Versus One Triple Axle Group (Thick Pavement)

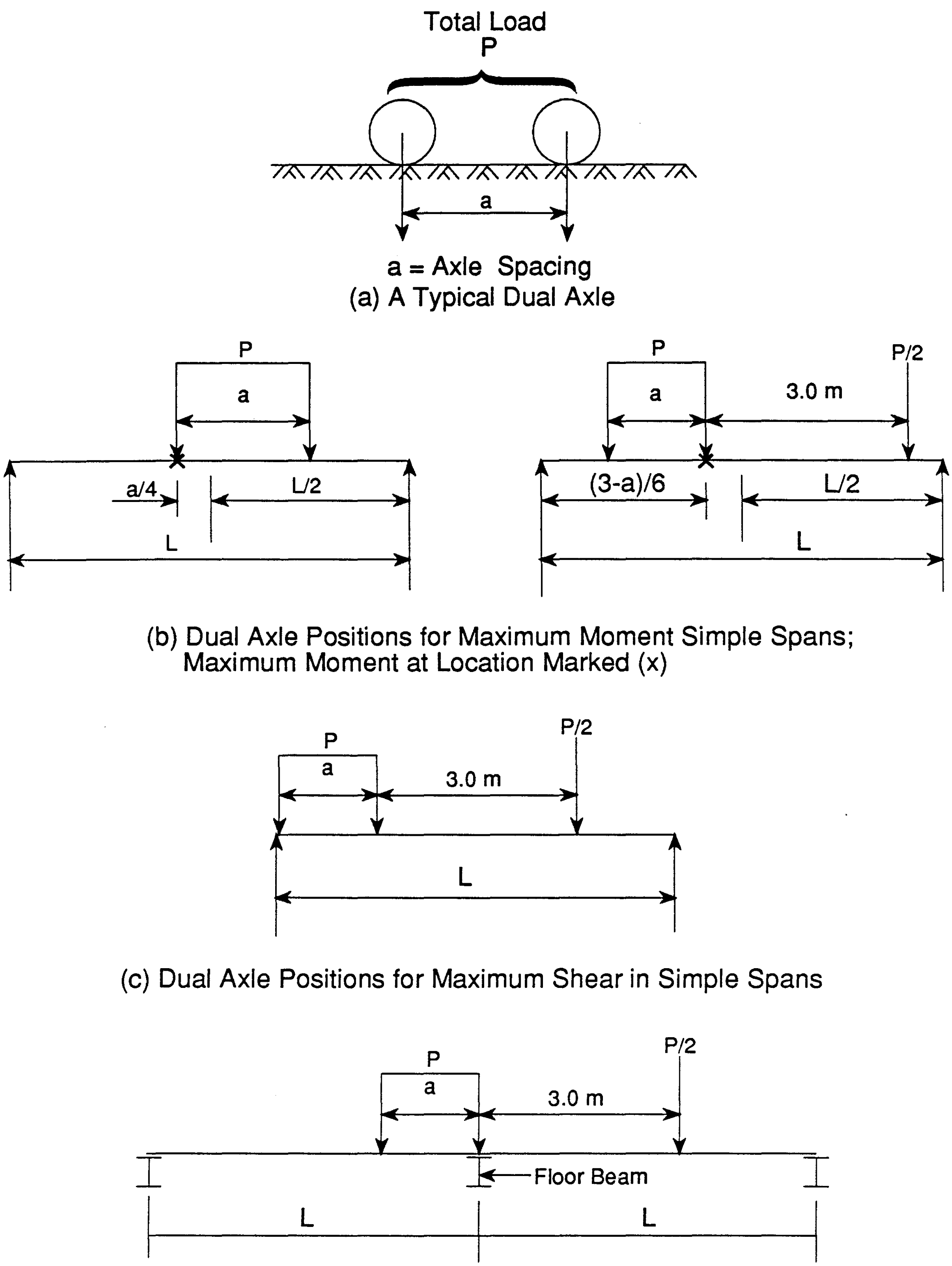


(a) AASHTO HS 20 Truck [17]

(b) OHBD Truck [18]

(c) CS-600 Truck [20]

Figure 9/ Bridge Design Truck Models in Canadian and American Codes



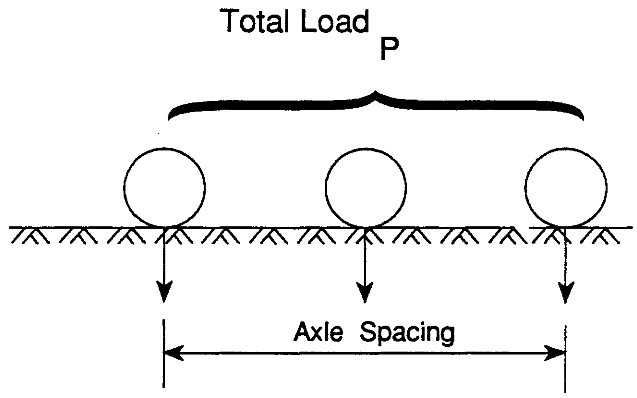
(a) A Typical Dual Axle

(b) Dual Axle Positions for Maximum Moment Simple Spans; Maximum Moment at Location Marked (x)

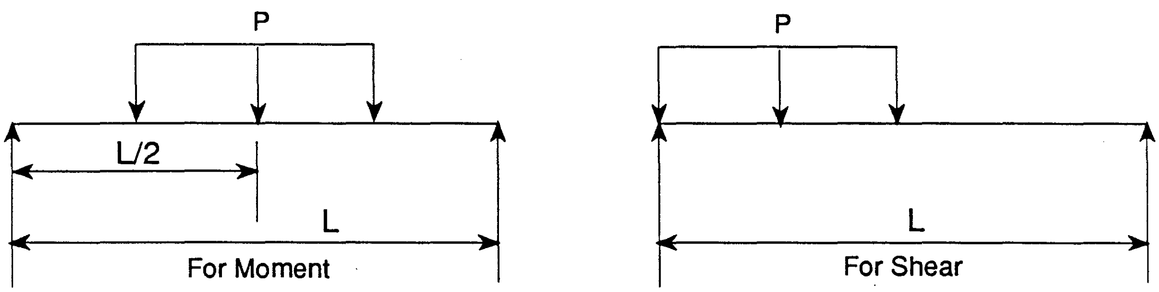
(c) Dual Axle Positions for Maximum Shear in Simple Spans

(d) Dual Axle Positions for Maximum Force Effects in a Floor Beam

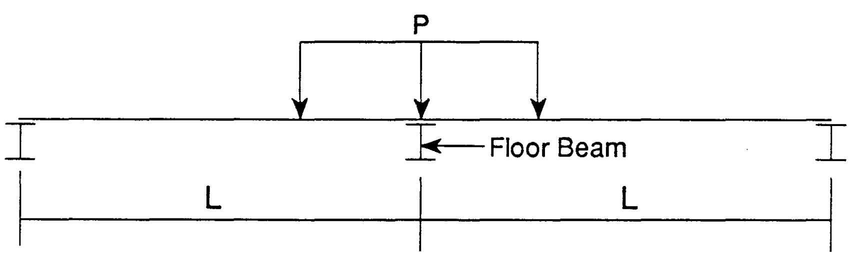
Figure 10/ Positions of a Dual Axle for Maximum Force Effects



(a) A Typical Triple Axle



(b) Triple Axle Positions for Maximum Force Effects in Simple Spans



(c) Triple Axle Positions for Maximum Force Effects in a Floor Beam

Figure 11/ Positions of a Triple Axle for Maximum Force Effects

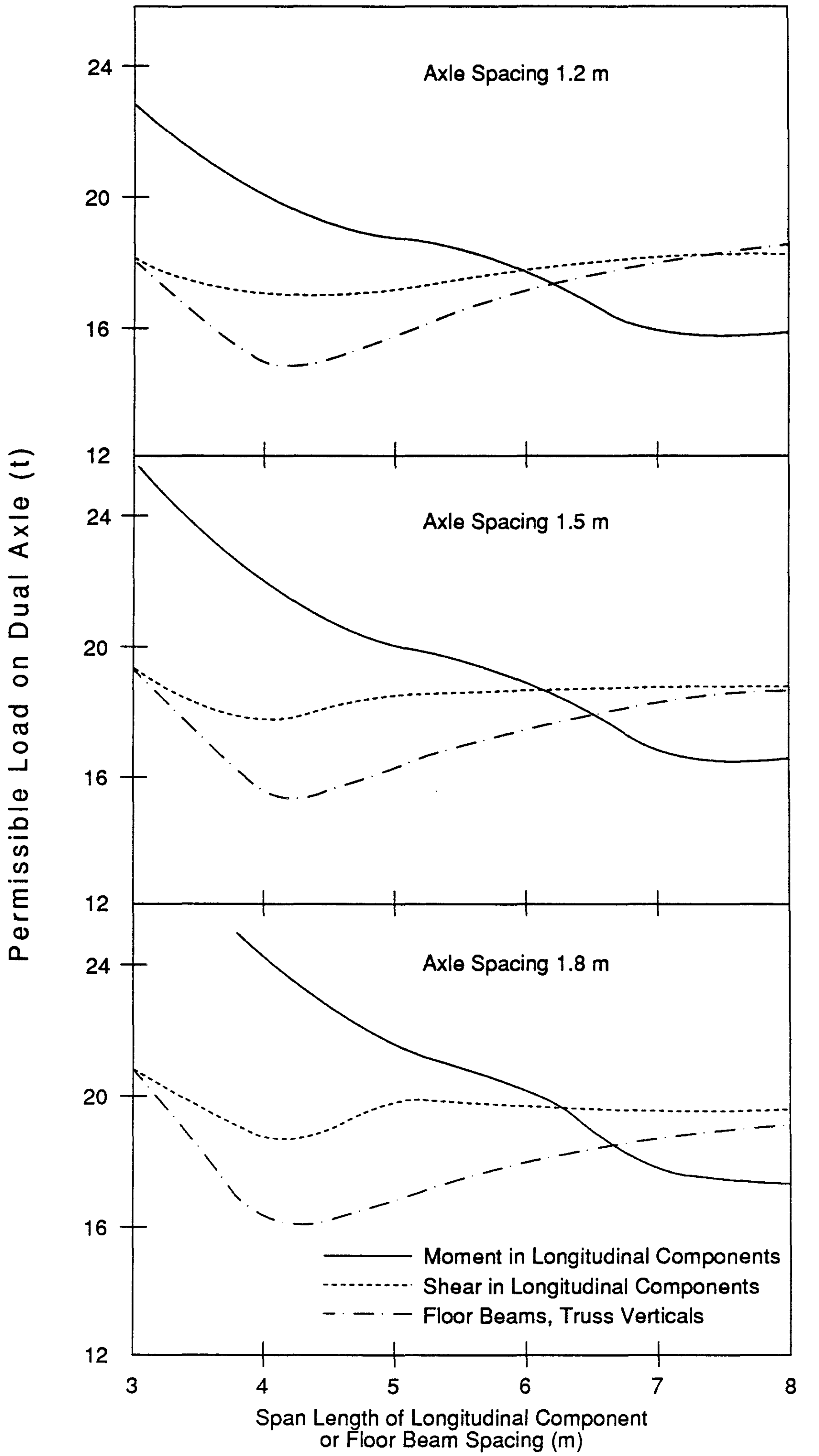


Figure 12/ Permissible Load on Dual Axles for
Bridges Designed for AASHTO HS 20 Loading

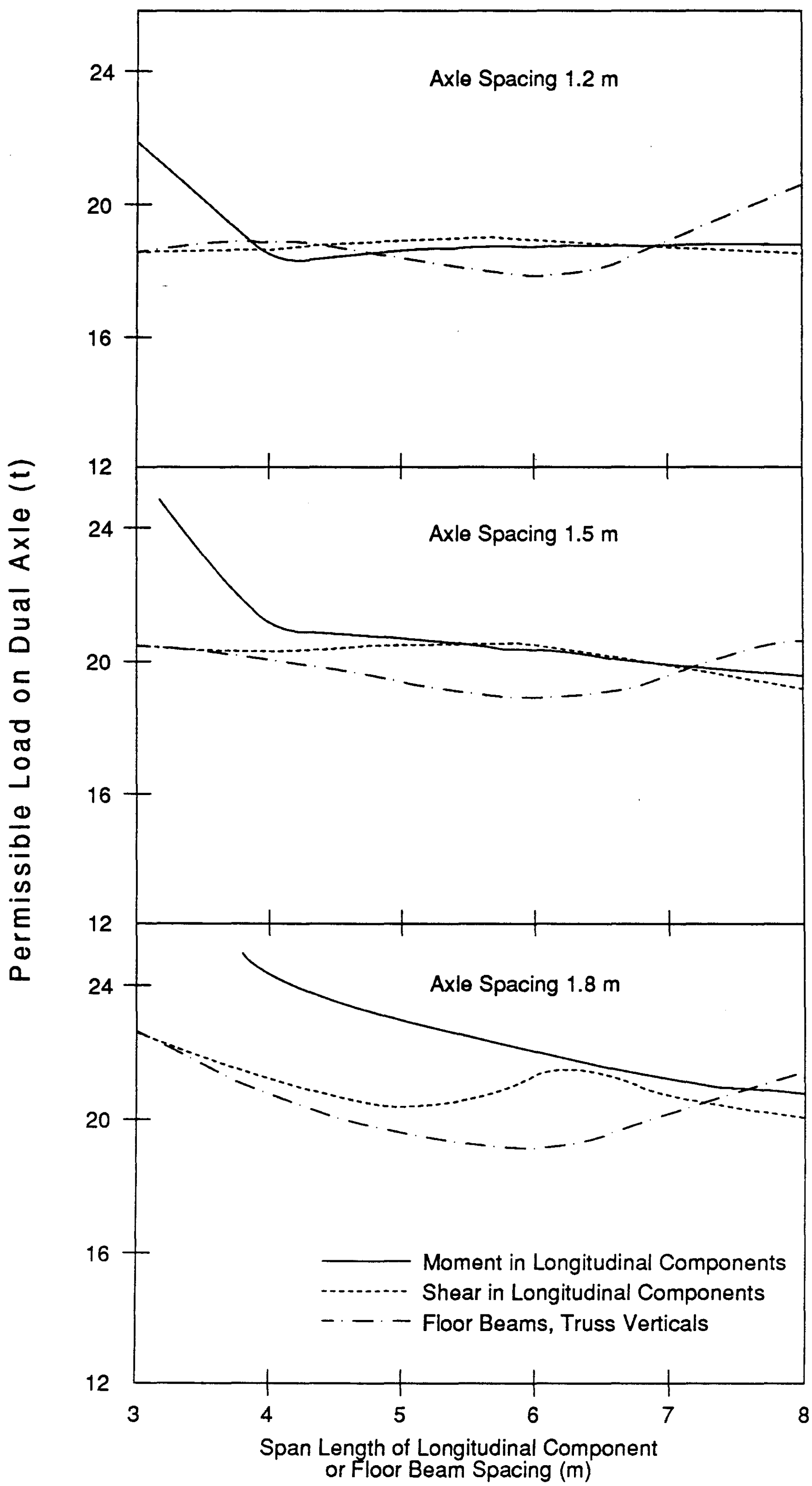


Figure 13/ Permissible Load on Dual Axles for Bridges Designed by OHBDC

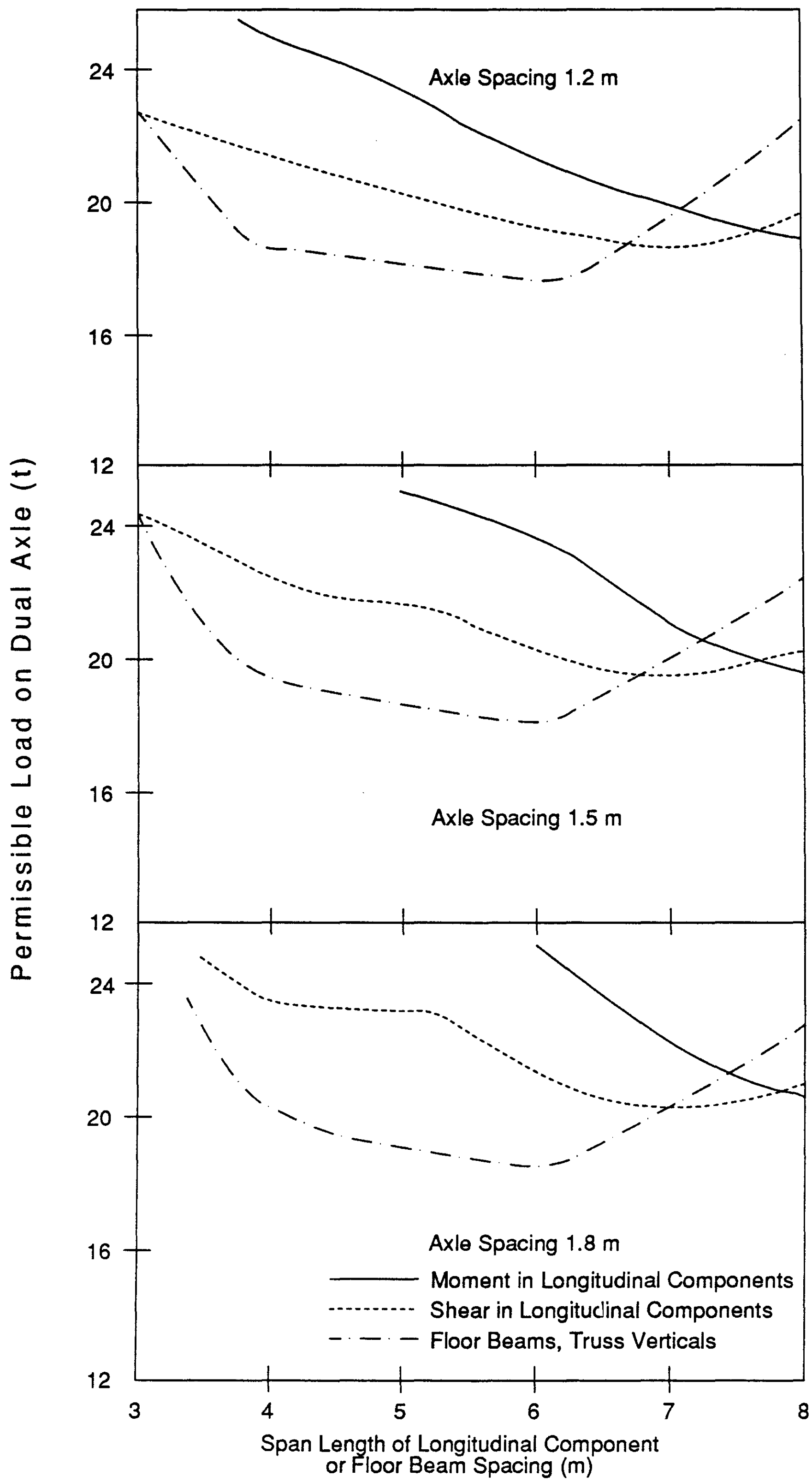


Figure 14/ Permissible Load on Dual Axles for Bridges Designed for CS-600 Loading

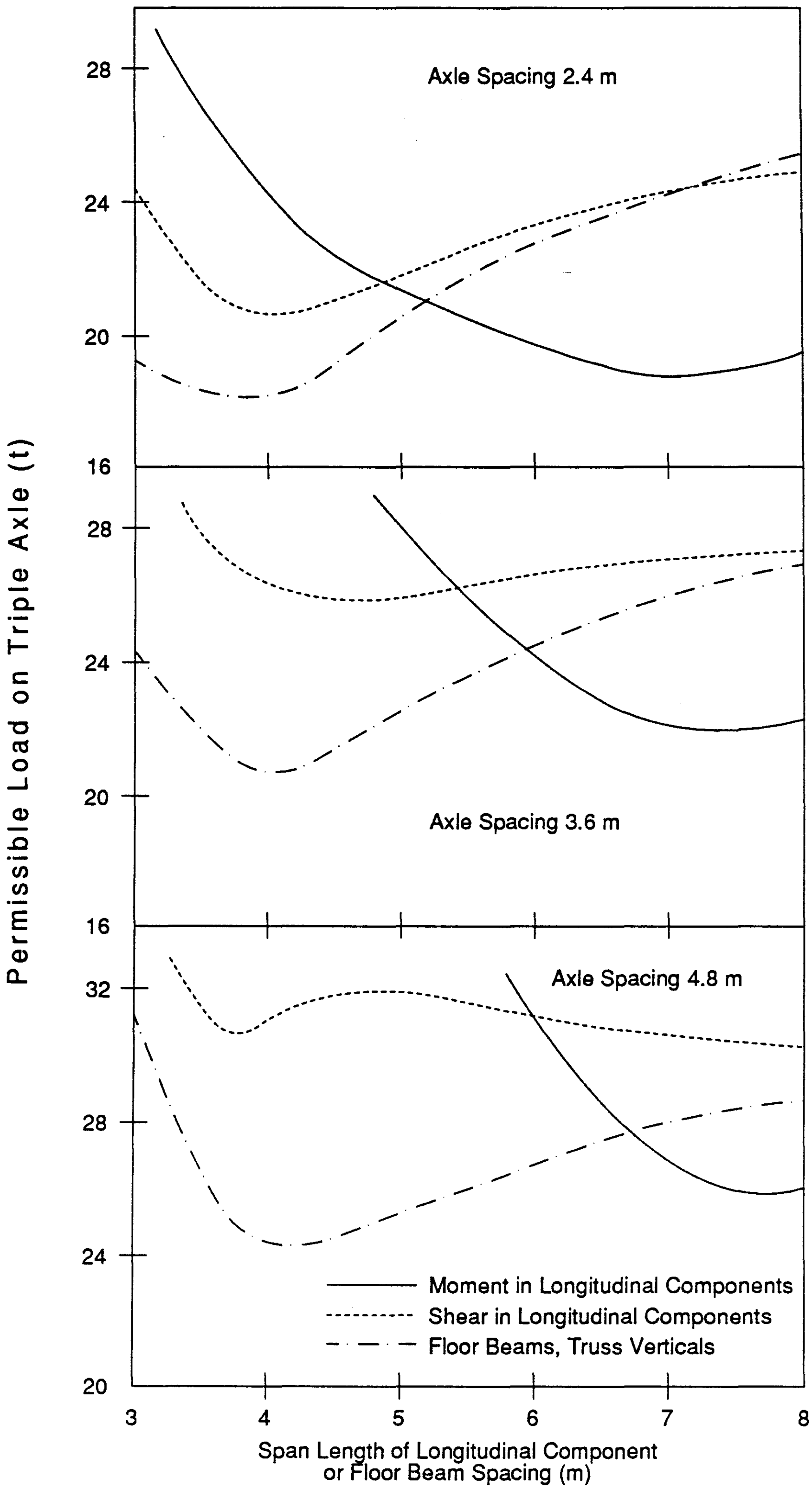


Figure 15/ Permissible Load on Triple Axles for Bridges Designed for AASHTO HS 20 Loading

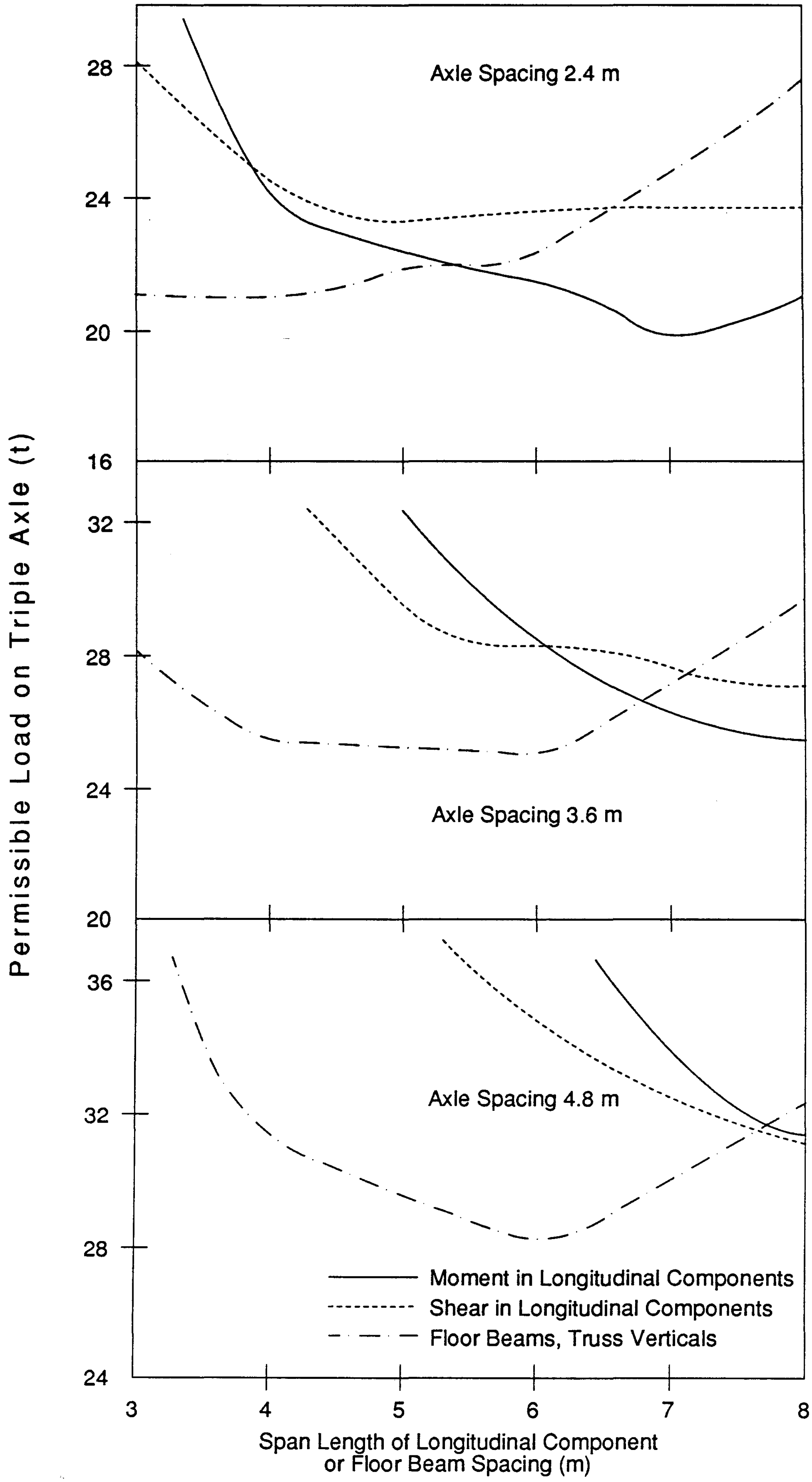


Figure 16/ Permissible Load on Triple Axles for Bridges Designed for OHBDC

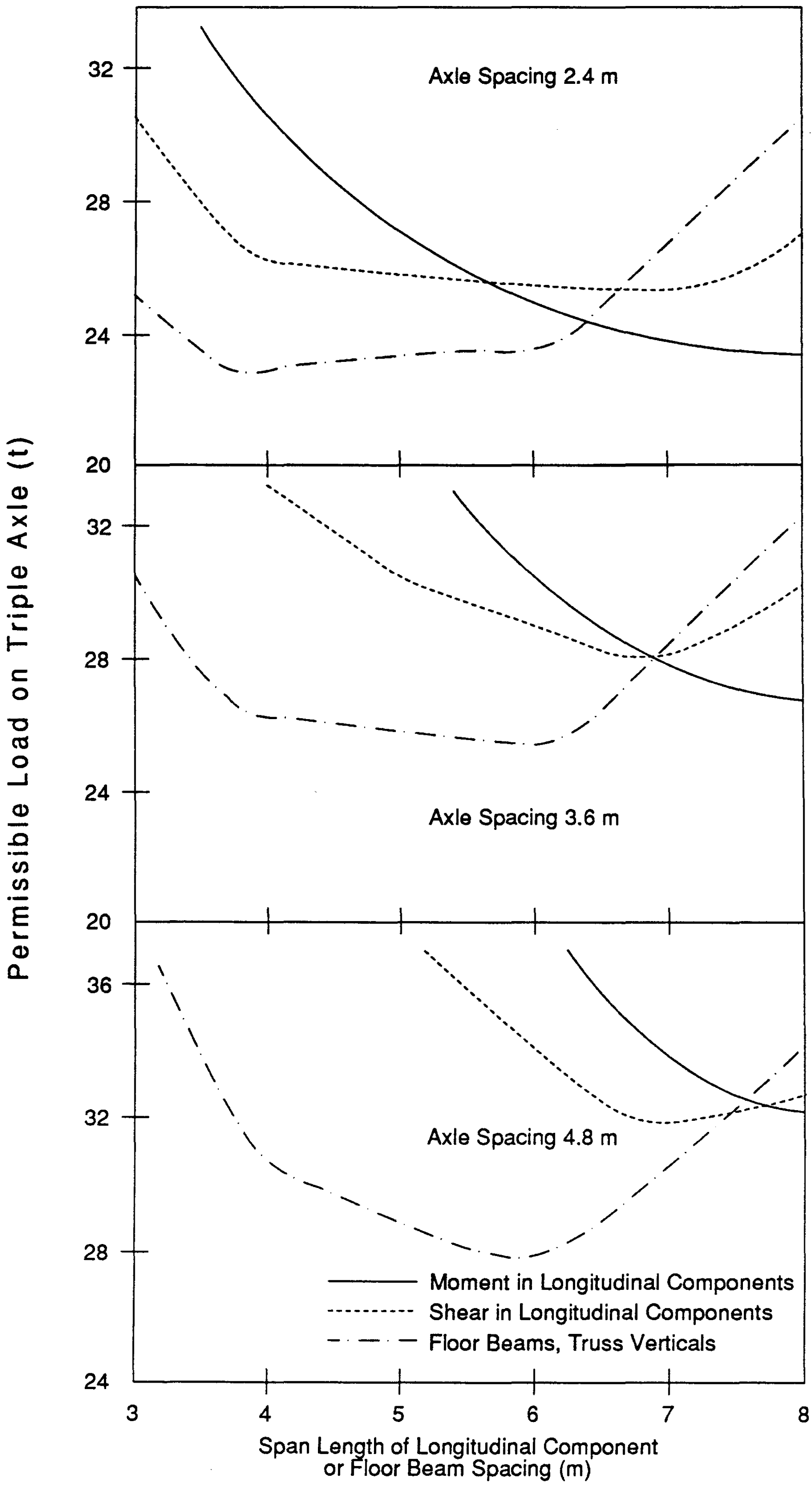


Figure 17/ Permissible Load on Triple Axles for Bridges Designed for CS-600 Loading