Impact of Adopting Canadian Interprovincial and Canamex Limits on Vehicle Size and Weight on the Montana State Highway System

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16. Abstract
The impact on the Montana state highway system of adopting Canadian Interprovincial, Canamex, or Canamex Short limits on vehicle size and weight was determined. Canadian Interprovincial vehicle configurations were developed based on an investigation of vehicle safety, economy, and damage to the highway system. Canamex and Canamex Short limits are hybrid size and weight systems that retain existing Montana axle weight limits coupled with Canadian gross vehicle weights. All scenarios allow vehicles to operate at higher gross weights than are presently allowed in Montana. The impact of these vehicles on the highway system was determined by a) developing traffic streams that included these vehicles, b) determining the engineering impact these traffic streams would have on existing bridges and pavements and on the future designs required to support these vehicles, and c) assigning a cost to these impacts based on the current cost of equivalent work. These analyses found that 16 to 20 percent of the bridges system-wide are deficient to carry Canadian Interprovincial vehicles (above and beyond the bridges already deficient under HS20 loads). Incremental deficiencies under Canamex and Canamex Short vehicles are between 1 and 3 percent of the bridges system-wide. The results were found to be sensitive to the assumed level of bridge capacity and the specific segment of the system being considered (i.e., interstate, primary, etc.). Long term pavement demands under all scenarios considered increase by less than 5 percent compared to demands under the existing traffic stream. Based on these impacts, an increase in equivalent uniform annual cost (EUAC) for bridges and pavements of 12 to 42 million dollars was calculated for Canadian Interprovincial limits. These costs represent a 11 to 36 percent increase in cost over that projected for the same activities under the existing traffic stream. An increase in EUAC of 4 to 7 million dollars was calculated for Canamex limits, which represents a 4 to 6 percent increase in cost over that projected for the same activities under the existing traffic stream. Similar but slightly higher costs were determined for the Canamex Short scenario relative to the Canamex scenario. These costs are for the interstate and primary systems. These costs represent an increased cost of 0.01 to 018, 0.02 to 0.08, and 0.02 to 0.15 dollars per mile driven on the interstate system by the new configurations for Canadian Interprovincial, Canamex, and Canamex Short limits. Similar costs per mile driven on the primary system were 1.3 to 10 times higher than these costs.

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EXECUTIVE SUMMARY

The objective of this study was to determine the impact on the Montana state highway system if Canadian Interprovincial or Canamex limits on vehicle size and weight are adopted on a regional or national level. The overall cost of transporting goods on the highway system is influenced by several factors, including the costs of the vehicle and driver, fuel costs, highway user fees, and vehicle capacity and efficiency. To a certain extent, these costs are interdependent, with a reduction in costs in one area possibly resulting in higher costs in another area. Operating costs, for example, may be reduced by hauling more freight on each trip using larger and/or heavier vehicles. Such vehicles, however, will cause increased damage to the highway system and thus require the operator to pay increased highway user fees to cover the cost of this damage. The operator, and thus ultimately the consumer, may still realize a net cost savings in this instance, however, if reductions in operating costs are larger than the costs of the increased damage to the highway system. This specific issue, that is, the relationship between truck size and weight and overall highway transportation costs, was the focus of a recent Transportation Research Board study entitled *Truck Weight Limits: Issues and Options*. In this study, seven vehicle size and weight scenarios were investigated. In the six of the seven scenarios in which vehicle size and weight limits were increased, reductions in overall transportation costs were predicted.

The greatest cost savings were predicted in the TRB study for the adoption of Canadican Interprovinicial limits on truck size and weight. Transportation officials in Canada developed a set of new vehicle size and weight limits, referred to as the Canadian Interprovincial limits, in an effort to improve the efficiency and safety of truck transportation while simultaneously limiting damage to pavement and bridges. These limits allow heavier and shorter combination vehicles to operate on the highway system with higher axle group weights than are currently allowed under Montana limits. With the passage of the North American Free Trade Agreement, the juxtaposition of Montana and Canada, the possible cost savings to be realized, and the possible safety benefits to be gained, it was judged to be worthwhile to investigate the impact on Montana highways of adopting the Canadian Interprovincial Limits on truck weight and size. The increase in weights allowed under Canadian Interprovincial vehicles is substantial (an 8 axle combination unit, for example, can operate at up to 138,000 pounds, which is approximately 20 percent greater then currently allowed), so two hybrid size and weight scenarios were also investigated in which vehicles would be allowed to operate at Canadian gross vehicle weights within the constraints of meeting current Montana axle weight limits. Thus, some of the cost savings and improved safety associated with Canadian Interprovincial limits may be realized under these scenarios while infrastructure impacts may be reduced. One hybrid scenario, referred to as Canamex, involves operating large combination vehicles similar in configuration to existing vehicles in Montana at weights up to 128,000 pounds. The second hybrid scenario, referred to as Canamex Short, involves operating vehicles similar in configuration to Canadian Interprovincial vehicles at weights up to 126,000 pounds.

The impact that the introduction of these various vehicles would have on the Montana highway system was determined in several steps: a) projections were made of the compositions of the new traffic streams that would evolve under the revised weight limits, b) the physical impacts of these traffic streams on the existing highway system were assessed, and c) the costs of these physical impacts on the highway system were calculated. While the focus of this investigation was on the bridges and pavements on the highway system, limited consideration was also given to other related highway features and activities that will be impacted by adoption of these new size and weight limits (e.g., roadway geometry, roadway maintenance, bridge inspection, etc.).

The composition of the traffic stream on the highways around the state will change if new size and weight limits are adopted, as operators move to take advantage of any economic benefit offered by these vehicles. The new vehicle size and weight limits considered in this study generally offer the ability to transport greater weight (but not volume) than present limits, so weight limited operators are expected to migrate to the new configurations. In general, the absolute number of heavy vehicles in the traffic stream is expected to remain fairly constant. Intervehicle diversions are expected to reduce the number of vehicles (shifting of freight to fewer heavier vehicles) while intermodal diversions are expected to increase the number of vehicles (simply adding vehicles). Under Canadian Interprovincial limits, operators are expected to migrate to the 6 axle tractor, semi-trailer and 8 axle B-train combination unit. These configurations will increase from being 4 and 1 percent of the existing vehicle fleet, respectively to each being 14 percent of the fleet. Correspondingly, a significant reduction in the use of traditional 5 axle tractor semi-trailer units is expected (from being 66 percent to being 44 percent of the vehicle fleet). Under both Canamex and Canamex Short limits, less sweeping changes in the traffic stream are expected. Under these scenarios, operators are expected to migrate predominantly to the 8 axle C-train, which will increase from being 2 percent to as much as 12 percent of the vehicle fleet. Under all the scenarios, diversion of freight from rail to truck is expected, and an allowance was made for this occurrence as the new traffic streams were developed.

The new vehicles in the Canadian Interprovincial, Canamex, and Canamex Short traffic streams will have an impact on the highway system. Bridges and pavements on the system will feel the primary impact if new limits are adopted. Assessing the impact of these vehicles on the bridge system is a complex problem, in that all of the bridges on the interstate system and the majority of the bridges on the primary system, for example, are not expected to sustain immediate damage. While the demands these vehicles place on the bridges exceed the demands used in their original design, bridges have traditionally been conservatively designed. Many of these structures may possess adequate reserve capacity to offer an acceptable level of safety under the new demands.

The analyses performed in this study found that 16 to 20 percent of all the bridges on the state highway system are deficient under Canadian Interprovincial vehicles (above and beyond the bridges currently deficient under HS20 design loads, the design standard used in Montana for most bridges). This range of deficiencies was calculated using different representations for the capacity of the bridge system. The higher figure for deficiencies reflects an average bridge capacity approximately midway between the two capacities (Inventory and Operating) typically used in the Allowable Stress based rating system. This intermediate level of capacity may better reflect the useable as-built, as-performing, and as-load rated capacity of existing structures on the Montana highway system than their original design capacity. The lower estimate of bridge deficiencies is based on full Allowable Stress based Operating ratings, calculated again from the original design capacity of the bridges. These Operating ratings were believed to reflect an upper boundary on the maximum useable capacity of existing structures on the highway system. Useable load ratings at this level may be obtained for the specific conditions in Montana (low traffic, good structural conditions), as verified using the results of a new load rating procedure (Load and Resistance Factor approach) developed to help facilitate the attainment of a uniform level of safety for bridges across the variety of conditions encountered in service.

Significantly fewer bridges are deficient (above and beyond those bridges already deficient to carry the HS20 design vehicle) under Canamex and Canamex Short limits compared to Canadian Interprovincial limits, as might be expected based on the lower axle weights and gross vehicle weights allowed under these scenarios. Between 1 and 3 percent of all the bridges on the state highway system are deficient under Canamex and Canamex Short limits, above and beyond those currently deficient under the HS20 design vehicle. Once again, the latter failure rate was calculated using a bridge capacity midway between Allowable Stress based Inventory and Operating ratings; the former, using full Allowable Stress based Operating ratings.

Predicted bridge impacts are sensitive to the element of the highway system under consideration, in addition to the size and weight scenario and assumed level of bridge capacity. The lowest percentages of deficient bridges are consistently found on the interstate system. Thirty-two, two, and six percent, respectively, of the bridges on the interstate system, for example, are deficient under Canadian Interprovincial, Canamex, and Canamex Short loads, assuming a bridge capacity midway between Allowable Stress based Inventory and Operating ratings. Corresponding deficiency rates on the primary and secondary systems range around 70 percent for all scenarios. These results are not unexpected, in that the interstate system in Montana is relatively young (average bridge age of 25 years) and most of the bridges on the system were designed using the HS20-44 vehicle. The primary and secondary systems are older than the interstate system (average bridge ages of 42 and 36 years, respectively) and both systems include many bridges built for lower design loads than used on the interstate system.

While strength is of primary importance in evaluating bridge performance, durability is an important consideration from a practical perspective. A limited experimental and analytical investigation of bridge behavior at Canadian Interprovincial load levels indicated that long term durability and performance will not be compromised under these loads. This study focused on possible accelerated deterioration of concrete decks, prestressed concrete beams, and steel stringers (fatigue).

With regard to decks, Canadian Interprovincial limits will place increased demands on bridge decks as wheel loads are carried into the stringer systems. An experimental and analytical investigation of these demands indicated, however, that they will not lead to accelerated deck deterioration. A limited experimental and analytical investigation of the response of prestressed concrete beams under Canadian B-trains also found that long term integrity of the beams will not be compromised under these loads. A network analysis of fatigue response in steel bridges indicated that less than 20 percent of the bridges on the system will have less than a 75 year life under the new vehicles considered herein, although fatigue demands are predicted to increase by up to approximately 30 and 10 percent under Canadian Interprovincial and under Canamex and Canamex Short limits, respectively.

Vehicle demands on the pavement will increase under Canadian Interprovincial, Canamex, and Canamex Short size and weight limits. Canadian Interprovincial limits allow tandem and tridem axles to be loaded 10 and 25 percent heavier, respectively, than is permitted under existing Montana weight limits. Catastrophic pavement failure is not expected to occur in a single passage (or even a few passages) of these loads, but long term pavement deterioration will be accelerated. While Canamex and Canamex Short vehicles are restricted to operate at existing maximum axle weight limits, the weight carried by the axles on these vehicles is expected to increase compared to current practice. Axle weights on large combination units are presently limited to less than their allowable maximum values by Bridge Formula B axle group constraints. If Formula B were negated (as is proposed for specific configurations in the Canamex and Canamex Short scenarios), these axle weights will increase.

Long term pavement demands, as measured in ESALs, are projected to increase approximately 3 and 4 percent, respectively, for the Canadian Interprovincial and for the two Canamex scenarios as compared to projected demands of the current traffic stream. These demands will result in a nominal reduction in the life of existing pavements (typically less than 1 year) and a nominal increase in the thickness of future overlays (typically less than 2 percent), based on calculations performed using an AASHTO ESAL based pavement performance model.

Costs were assessed for the impacts identified above by calculating costs for equivalent work at current prices, projecting these costs into the future as necessary, and determining equivalent uniform annual costs for the resulting cash flow. These cost increases are specifically associated with (a) replacing currently adequate bridges on the system that are found to be inadequate under the new vehicle loads and (b) overlaying roads earlier than expected using pavements nominally thicker than would be required under the existing traffic stream to accommodate the new vehicles. In most cases, the majority of these costs are associated with bridge impacts. In all cases, the cost impacts for the primary system significantly exceed those for the interstate system.

If Canadian Interprovincial limits are adopted, the incremental increase in combined bridge and pavement costs on the interstate and primary systems is projected to be between 12 and 42 million dollars per year, which represent increases of 12 and 36 percent, respectively, relative to comparable costs under the present traffic stream. The impacts of adopting Canamex and Canamex Short limits are projected to be significantly less than those for Canadian Interprovincial limits, which would be expected based on the relative magnitude of the allowable loads under the two systems. If Canamex limits are adopted, the incremental increase in pavement and bridge costs on the interstate and primary systems is projected to be between 4 and 7 million dollars per year, which represent increases of 4 and 6 percent, respectively, over comparable costs projected under the current traffic stream. If Canamex Short Limits are adopted, the incremental increase in pavement and bridge costs on the interstate and primary systems is projected to be between 5 and 10 million dollars per year, which represent increases of 4 and 9 percent, respectively, over comparable costs projected under the current traffic stream.

The increase in user cost responsibility associated with adopting Canadian Interprovincial, Canamex, or Canamex Short limits was estimated based on the increased costs for the highway system as identified above and the projected use of the system by the new vehicles. The per unit cost responsibilities for Canadian Interprovincial vehicles were found to be lower than might be expected based on the total cost impacts stated above. Adoption of Canadian Interprovincial limits would affect the greatest number of vehicles in the resulting traffic stream, thereby reducing cost responsibility per vehicle mile driven.

The estimated incremental cost responsibility for Canadian Interprovincial vehicles operating on the interstate system ranges from 0.01 and 0.18 dollars per mile driven by the new vehicles. Cost responsibilities ranging from 0.02 to 0.08 and from 0.02 to 0.15 dollars per mile driven are estimated for Canamex and Canamex Short vehicles, respectively, operating on the interstate system. In each instance, the first figure was calculated using full Allowable Stress based Operating ratings to represent bridge capacity; the second figure, using an intermediate bridge capacity between Allowable Stress based Inventory and Operating levels. Actual cost responsibilities are expected to fall within these ranges. The sensitivity of these estimates of cost responsibility to the assumed level of bridge capacity is obvious.

Lower cost responsibilities were consistently calculated for vehicles operating on the interstate relative to the primary system. Calculated cost responsibilities on the primary system are from 1.3 to 10 times greater than cost responsibilities estimated for the interstate system. Per unit cost responsibilities were not calculated for the secondary system. These costs, however, are expected to be higher than those for the interstate and primary system. The lighter pavements and bridges on the secondary system are expected to be less tolerant of the increases in load under Canadian Interprovincial, Canamex, and Canamex Short vehicles than the more substantial pavements on the primary and interstate systems, and the lower truck volumes would further inflate the per unit costs.

Overall (and assuming geographically widespread implementation of the scenario), Canadian Interprovincial limits will result in significantly higher demands on the highway system than Canamex or Canamex Short limits, as would be expected based on the difference in loads allowed under the three systems. Demands under Canamex Short limits, in turn, are nominally higher than the demands under Canamex Limits. These differentials in demand are associated primarily with the bridge system, where Canadian Interprovincial vehicles stress more structures closer to their ultimate capacity than Canamex Short and Canamex vehicles. In general, fewer bridges were found to be deficient on the interstate compared to other systems.

Based on these various results, it may be practical to focus the operation of the new vehicles on designated routes within the state, notably the interstate routes (or some portion of them). The interstate system should be able to handle either Canadian Interprovincial, Canamex, or Canamex Short vehicles without substantial modification. It will be possible, however, to open more of the system to Canamex vehicles than to either Canamex Short or, particularly, Canadian Interprovincial vehicles. Collector routes along the interstate (primary, secondary, and urban routes) may also be able to better handle Canamex vehicles than Canamex Short and Canadian Interprovincial vehicles. In almost all cases, the majority of the incremental uniform annual cost is bridge related. Thus, costs associated with specific routes could be significantly lower than the average costs presented above, if these routes contain only a few (or no) deficient bridges.

1. INTRODUCTION

1.1 GENERAL REMARKS

The cost of transporting goods by truck is influenced by several factors, including the costs of the driver and the truck, the capacity of the truck and its efficiency, the cost of fuel, and the cost of the highway. To some extent, these costs are interdependent, with a reduction of costs in one or more areas resulting in a cost increase in another area. Operating costs, for example, may be reduced by transporting the same amount of goods in fewer trips by hauling heavier loads. Heavier loads, however, are more damaging to the highway system, resulting in increased highway costs. Heavier loads may still afford an overall cost advantage to the consumer, if the savings in operating costs are more than the increases in highway expenses. The Transportation Research Board (TRB) published a study in 1990 that specifically investigated the impact on overall truck transportation costs of increasing truck weight limits (TRB, 1990a). The results of this study clearly show that total truck transportation costs will be reduced by increasing weight limits. TRB considered seven different weight limit scenarios in their study, six scenarios that involved increasing weight limits and one scenario that involved decreasing weight limits. While every proposal that involved increasing weight limits resulted in an associated increase in pavement and bridge costs, these cost increases were more than offset by savings in operating costs. In the single scenario they considered in which more restrictive weight limits were imposed on trucks, lower pavement and bridge costs did result. Overall truck transportation costs, however, increased significantly.

The greatest net savings in truck transportation costs (approximately 8 percent) were realized in the TRB study by the adoption of Canadian Interprovincial Limits on truck weights. The greatest increases in pavement and bridge costs were also observed for this scenario. The Canadian Interprovincial Limits on weights of both individual axle groups and combinations of axles groups exceed those allowed in the United States. Correspondingly, these vehicles will place greater demands on U.S. highways than they were initially designed to resist. The TRB study also found that traffic accidents and fatalities would nominally decrease with the adoption of Canadian Interprovincial Limits. These results are not surprising, in that the Canadian

Interprovincial Limits are based on an extensive research program, and they were specifically established to improve overall economy and safety of truck transport (Roads and Transportation Association of Canada (RTAC), 1987). The authors of the TRB study concluded that adoption of Canadian Interprovincial Limits in the U.S. would be impractical, primarily due to the large number of bridges that would have to be replaced.

With the passage of the North American Free Trade Agreement, the juxtaposition of Montana and Canada, the possible cost savings to be realized, and possible safety benefits to be gained, it is worthwhile to investigate the impact on Montana highways of adopting the Canadian Interprovincial Limits on truck weight and size.

1.2 OBJECTIVES AND SCOPE

The objective of this study was to determine the impact on the Montana state highway system of the adoption of Canadian Interprovincial Limits on vehicle size and weight at a regional and/or national level. Three scenarios were investigated. In the first scenario, adoption of full Canadian Interprovincial limits on both vehicle size and weight was considered. Two additional scenarios were considered, in which various aspects of the Canadian Interprovincial Limits would be adopted while restricting axle loads to existing Montana weight limits. These second two scenarios were believed to possibly be less damaging to the highway system than full Canadian Interprovincial limits, while still offering some of the reported overall economic advantages of those limits. The first of these hybrid scenarios, referred to as Canamex, involves allowing specific vehicles similar in geometry to large combination vehicles currently used in Montana to operate on Montana's highways at gross weights up to Canadian Interprovincial gross vehicle weights, within the constraints of Montana axle load limits. This scenario has been labeled Canamex (Alberta Transport and Utilities, 1994). The second hybrid scenario, referred to as Canamex Short in this study, involves allowing additional weight on only those vehicles which meet Canadian Interprovincial vehicle geometries. These vehicles would be allowed to operate up to Canadian Interprovincial gross vehicle weights within the constraints of Montana axle load limits.

The impact that the introduction of these vehicles would have on the Montana highway system was determined in several steps:

a) Projections were made of the compositions of the new traffic streams that might evolve under the revised size and weight limits. This evolution will naturally occur as vehicle owners modify their operations to realize any cost savings available under the new increased weight limits. The composition of the traffic stream was also modified to include diversion of freight from rail to truck by imposing a simple percentage increase in the amount of freight carried by truck.

b) The physical impact of this new traffic stream on the existing highway system was assessed. This assessment was accomplished using engineering analyses to determine the response of existing bridges and pavements under the new traffic loads. These calculations were performed at the network level using simplified analysis techniques. Calculations were performed to identify bridges that are inadequate to carry Canadian Interprovincial and Canamex vehicles and to investigate any fatigue and durability problems that may develop in bridges under these increased loads. Limited detailed analyses and field studies were performed to further evaluate the possible effects of the new vehicles on the pavement and bridge systems. Calculations were also performed to determine any reduction in the remaining life of existing pavements under the new traffic streams and to determine any increase in design requirements for the future overlays necessary under these streams.

c) The costs of these physical impacts on the highway system were calculated. Costs were figured for all activities (bridge replacement and future overlay) in terms of the present cost of similar activities. These costs were then adjusted to their actual time of occurrence and re-expressed as an equivalent uniform annual cost to allow for comparison of the various scenarios. A gross estimate was made regarding the allocation of the incremental costs associated with adopting these new weight limits to the new vehicles that occasioned them.

While the focus of this investigation was on the direct impact Canadian Interprovincial and Canamex vehicles will have on the pavements and bridges on the highway system, consideration was also given to other related highway features and activities that will be impacted by such a step (e.g., roadway geometry, roadway maintenance, bridge inspection, etc.).

The analyses performed in this study were compared, as possible and appropriate, with the analyses and results of other investigators and with the experience in various Canadian provinces since their adoption of Canadian Interprovincial limits.

2. DESCRIPTION OF THE HIGHWAY SYSTEM AND ITS USERS

2.1 GENERAL REMARKS

This study is concerned with the highway infrastructure in the state of Montana and the vehicles that use it. Interest is specifically focused on 1) the roadways and 2) the bridges in the state for which the Montana Department of Transportation (MDT) assumes responsibility. While the function of each of these components is the same, that is, to carry vehicles between two points, they accomplish this function in very different fashions, and they will be treated separately in the following analyses. The vehicles that use the roadways and bridges can also be divided into distinct groups based on their axle configurations. Specific vehicle configurations and traffic patterns have evolved in Montana in response to social/economic needs and the constraints of motor vehicle size and weight regulations. Demands on the roadways and bridges in the state are integrally related to these vehicle configurations and traffic patterns.

2.2 MONTANA STATE HIGHWAY SYSTEM

2.2.1 Roadways - In 1993, approximately 11,753 miles of highway made up the interstate, primary, secondary, and urban systems in the state of Montana (MDT, 1993b; Cloud, 1995). By virtue of being designated to one of these systems, a highway is eligible for one or more types of federal aid funding. A summary of these highways is presented in Table 2.2.1-1. This summary is presented in terms of the federal aid classification system used prior to the establishment of the National Highway System (NHS) in 1993 and its adoption in final form in 1995. Since most of the data provided to this study from MDT were organized in terms of the old classification system, this study was conducted in terms of the old federal aid system. The interstate system is identical under both highway systems, and approximately one-half of the old primary system was incorporated into the NHS. The majority of the remainder of the old primary, secondary, and urban systems are functionally classified as major collectors or above.

Road surfaces on the Montana state highway system are constructed of asphalt (flexible), concrete (rigid), treated gravel, and gravel. The percent of each system paved with each type of material is reported in Table 2.2.1-1. Asphalt is the most commonly used material on state highways, comprising 79 percent of the roads on the total state highway system. Only on the interstate system is concrete used to any major extent (12 percent), and most of this pavement is on a single interstate route (Interstate 90). The overall condition of the interstate system, as represented by the present serviceability index (PSI), was judged to be fair to good in 1991, with a length weighted average PSI value of 3.6 (MDT, 1991). The interstate system did exhibit nominal rutting damage. Eight percent of the system lane mileage had ruts with a depth of ½-to-¾ inch. One percent of the system had ruts with a depth greater than 3/4 inch (MDT, 1991). The primary system was judged to be in fair to good condition in 1992, with a length weighted average PSI of 3.3. The conclusion was reached in 1992 that system deterioration was beginning to proceed at a rate faster than repair (MDT, 1993a). This system also exhibited rutting distress. Twenty-seven percent of the primary system had ruts with a depth of ½-to-¾ inch. Five percent of the system had ruts with a depth greater than 3/4 inch. The median remaining life of the roadways that comprise the primary system was estimated in 1992 to be 7 years. Only limited data appears to be collected on the secondary, urban, and off- system roads. Information on the general condition of the pavements on these systems is unavailable.

Table 2.2.1-1 State Highway System Mileage by Federal Aid System (MDT, 1993b; Cloud, 1995)

System	Mileage	Percent of mileage within each system by surface type		
		% Flexible	% Rigid	% Other[a]
Interstate	1191	89	12	0
Primary	5452	96	1	3
Secondary	4757	56	0	44
Urban	353	87	1	12
Off system	(1139)	-[b]	-[b]	-[b]
Total	11753	79	2	19

[a] bituminous surface treatment, gravel, or primitive
[b] data unavailable

2.2.2 Bridges - A summary of the bridges in the state inventory is presented in Table 2.2.2-1. Bridges on the state highway system are constructed using three types of structural systems, namely, stringer, truss, and flat plate systems. Stringer systems are the most common bridge type in the state, comprising 95 percent of the inventory by length. This type of bridge consists of a series of parallel beams (stringers) oriented in the direction of the span. The beams support the deck and are in turn supported by the abutments and piers. Loads are carried through transverse shear forces and bending moments in the beams. The beams are either simply supported on each end, or they can be continuous across any internal supports. Simply supported stringer bridges compromise 70 percent of all spans (by length) on the state highway system. Continuous stringer bridges compromise only 25 percent of the bridges on the system.

Flat plate bridges and truss bridges comprise only 5 percent of the bridges on the state highway system. To a large extent, flat plate bridges carry loads through the same mechanisms as stringer bridges, but their strength is distributed across the width of the structure rather than being focused at a few locations in a few beams. Truss bridges carry loads through axial forces in their members. Only 3 percent of all bridges in the state inventory are truss structures.

With respect to materials, bridges in Montana are constructed with prestressed concrete, concrete, steel, and wood. The most common bridge on the system is the simply supported, prestressed concrete stringer bridge. These bridges comprise 46 percent of all the bridges on the system (based on length), and they represent even higher proportions of the bridges on the interstate system (65 percent). Prestressed concrete bridges reportedly offer better long-term performance compared to other bridge systems (Dunker and Raubat, undated), and most new and replacement bridges are being constructed using this material (Murphy, 1995). Standard prestressed bridge designs have been developed by MDT based on span length and roadway width. Continuous steel stringer bridges are the second most common bridge on the system, comprising 24 percent of all bridges (by length). Timber bridges comprise a significant part of the inventory (11 percent). Most of the timber bridges are on the primary and secondary systems.

Table 2.2.2-1 Characteristics of Bridges on the State Highway System (MDT, 1994)

Structural System	No. of Spans	Average Length (ft)	% (by length) of all spans
Stringer			
Simply supported			
Prestress	3005	59	46
Steel	571	56	8
Wood	2152	20	11
Concrete	437	42	5
Continuous			
Prestress	3	103	0
Steel	886	104	24
Concrete	160	22	1
Total	**7214**	**51**	**95**
Flat Plate			
Simply supported			
Concrete	79	20	0
Continuous			
Concrete	442	20	2
Total	**521**	**20**	**2**
Truss			
Steel	85	130	3
Total	**85**	**130**	**3**
Total	**7820**	**50**	**100**

All the bridges on the interstate and primary systems have overall structural ratings of at least good, as this rating is calculated for the National Bridge Inventory System (FHWA, 1988). These good conditions may reflect in part the relative young age of many of the bridges, the relatively light traffic they experience, and the favorable environmental conditions (relatively low relative humidity and only modest use of de-icers) in Montana. Average age and daily traffic on the bridges on each system are summarized in Table 2.2.2-2. The average age of all the bridges in the Inventory is 37 years (Meyer, 1996).

The Inventory load rating on every bridge on the interstate system is at least HS20-44 (MDT 1994), the current standard vehicle used by most states for bridge design. The HS20-44 design vehicle is a three axle tractor, semi-trailer with a gross weight of 72,000 pounds and an over-all wheel base of 28 to 44 feet (AASHTO, 1990). This vehicle is not intended to represent any specific vehicle that operates on the highway system. The HS20-44 vehicle was developed as a bridge design tool in 1944 to provide a single vehicle to be used in the design process that analytically generates the maximum stresses caused in bridges by a collection of actual truck configurations (Ritter, 1990; Tonias, 1995). The HS20-44 design loading also includes a uniformly distributed lane load developed to model a train of trucks crossing a bridge. The Inventory load rating on approximately 60 percent of the bridges on the primary system is H15 or lower. The H15 design vehicle is a two axle truck with a gross weight of 30,000 pounds and a wheel base of 14 feet (AASHTO, 1990). This design vehicle generally places lower demands on bridges than the HS20-44 vehicle, and it is used on secondary and local roads when a lesser loading may be appropriate (Ritter, 1990). Eighty percent of the bridges on the primary system with a load rating of H15 or less are short span timber structures. Most of the bridges on the secondary system have Inventory load ratings of H15 or less (66 percent). The majority of these bridges are short span timber structures, as was observed for the primary system.

In almost all cases, the reported Inventory load ratings for bridges across all systems appear to be the vehicles used for the bridge designs (e.g., HS20-44, H15, etc.). Specific load ratings were not done to obtain the majority of these values (Murphy, 1996).

Table 2.2.2-2 Average Age and Daily Traffic on State Highway Bridges by System (based on information provided by Meyer, 1996)

System	Number of bridges	Average age (yrs)	Average daily traffic
Interstate	843	25	5582
Primary	1193	42	1922
Secondary	556	36	700[a]
Urban	66	35	10429

[a] high uncertainty on exact value, order of magnitude reasonable

2.3 TRAFFIC

2.3.1 Vehicle Configurations - Vehicle configurations in Montana are controlled by legal limits that include requirements on load per inch of tire width, maximum axle group weights, maximum gross vehicle weights, maximum vehicle lengths, and maximum vehicle widths (MCA, 1995). Various truck configurations that have evolved under these limits are shown in Figure 2.3.1-1. While vehicle size and weight limits in Montana are generally consistent with regulations around the country, some features of Montana's laws are specific to the western United States and more particularly to the state of Montana. Specific regulations of interest include:

1) maximum gross vehicle weights are determined by the Federal Bridge Formula B,
2) long combination vehicles (LCVs) are allowed to operate, and
3) triple trailers are allowed to operate on the interstate system.

With regard to maximum gross vehicle weights, Montana has elected not to adopt the 80,000 pound maximum gross vehicle weight endorsed by the federal government, but rather to control demands placed on bridges using Federal Bridge Formula B. This formula gives the allowable weight on any group of two or more axles in terms of the number and spacing of the axles,

$$W = 500\,[LN/(N-1) + 12N + 36]$$

where,

W = allowable weight on the collection of axles under consideration

L = length between extreme axles in collection of axles under consideration

N = number of axles under consideration

Within the constraints of the Bridge Formula B and maximum axle weights, Montana allows double trailer units up to 100 feet long to operate on the state's highways with a special permit. Double trailer units up to 75 feet long can operate without a permit. A popular double trailer vehicle configuration, referred to as the Rocky Mountain double, has either 7 or 8 axles and can operate at gross vehicle weights up to 113,000 and 117,000 pounds, respectively. These vehicles often run with two trailers with lengths of 45 and 28 feet. Typical legal limits on various vehicle configurations are presented in Table 2.3.1-1. Axle loads in Montana are limited to 20,000, 34,000, and 42,500 on singles, tandems, and tridems, with tridems controlled by the Bridge Formula. Loads on axles with single tires (except the steering axle) are limited to 500 pounds per inch of width (MCA, 1995).

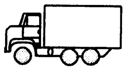
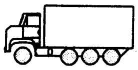
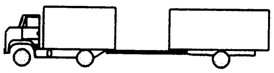
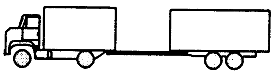
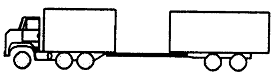
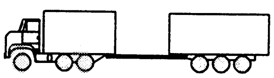
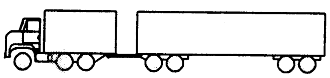
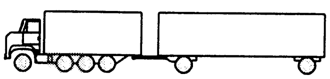
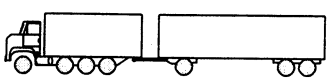
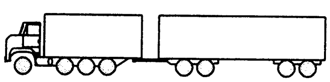
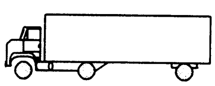
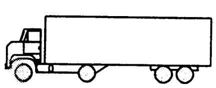
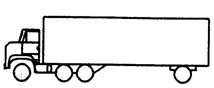
	2SU	2 AXLE SINGLE UNIT
	3SU	3 AXLE SINGLE UNIT
	4SU	4 AXLE SINGLE UNIT
	2-1	TRUCK AND TRAILER
	2-2	TRUCK AND TRAILER
	3-2	TRUCK AND TRAILER
	3-3	TRUCK AND TRAILER
	3-4	TRUCK AND TRAILER
	4-2	TRUCK AND TRAILER
	4-3	TRUCK AND TRAILER
	4-4	TRUCK AND TRAILER
	2S1	3 AXLE TRACTOR SEMI TRAILER
	2S2	4 AXLE TRACTOR SEMI TRAILER
	3S1	4 AXLE TRACTOR SEMI TRAILER

Figure 2.3.1-1 Typical Vehicle Configurations (page 1 of 2)

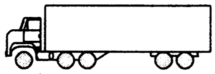
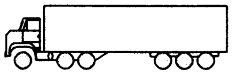
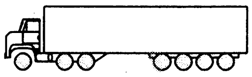
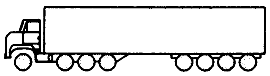
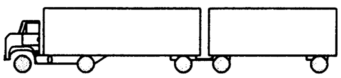
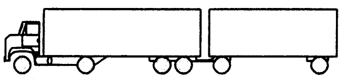
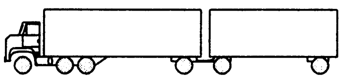
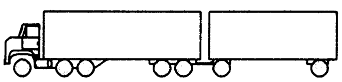
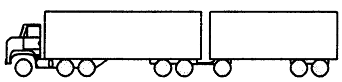
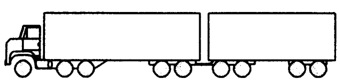
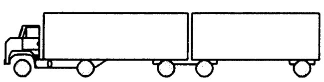
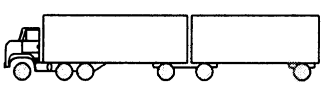
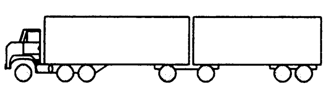
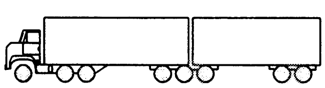
	3S2	5 AXLE TRACTOR SEMI TRAILER
	3S3	6 AXLE TRACTOR SEMI TRAILER
	3S4	7 AXLE TRACTOR SEMI TRAILER
	4S4	8 AXLE TRACTOR SEMI TRAILER
	2S1-2	5 AX A/C TRAIN COMBINATION
	2S2-2	6 AX A/C TRAIN COMBINATION
	3S1-2	6 AX A/C TRAIN COMBINATION
	3S2-2	7 AX A/C TRAIN COMBINATION
	3S2-3	8 AX A/C TRAIN COMBINATION
	3S2-4	9 AX A/C TRAIN COMBINATION
	5 AX BT	B TRAIN COMBINATION
	6 AX BT	B TRAIN COMBINATION
	7 AX BT	B TRAIN COMBINATION
	8 AX BT	B TRAIN COMBINATION

Figure 2.3.1-1 Typical Vehicle Configurations (page 2 of 2)

Table 2.3.1-1 Maximum Gross Vehicle Weights, Widths, and Lengths, Without a Permit, Current Montana Limits, Compiled from Montana Code Annotated (MCA, 1995)

Configuration	GVW (kips)	Length[a] (ft)	Width (ft)
Single Units			
2SU	36.0	45	8.5
3SU	50.0	45	8.5
4SU	58.0	45	8.5
Truck and Full Trailers			
2-1	56.0	75	8.5
2-2	70.0	75	8.5
3-2	84.0	75	8.5
3-3	92.0	75	8.5
3-4	103.8	75	8.5
Tractor, Semi-trailers			
2S1	52.0	75	8.5
2S2	66.0	75	8.5
3S2	80.0	75	8.5
3S3	88.0	75	8.5
3 Unit Combinations			
5 AX A Train, 2S1-2	92.0	75	8.5
6 AX A Train, 2S2-2	106.0	75	8.5
7 AX A Train, 3S2-2	112.5	75	8.5
8 AX A Train, 3S2-3	117.4	75	8.5
9 AX A Train, 3S3-3	122.6	75	8.5

[a] large combination vehicles can operate up to 95 feet long with a permit

2.3.2 Existing Traffic Distributions by Vehicle Configuration and Weight - Information on the specific vehicle configurations operating around the state is collected by the Data Collection/Analysis Section of MDT. This information consists of visual classification counts, automatic vehicle classification counts, and weight and classification data collected at static weigh stations. These data collection activities are focused on the interstate and primary systems, where much of the vehicle activity in the state is focused. With regard to determining the composition of the traffic stream, reliance was placed upon the data collected from the automatic vehicle classifiers. Classifications from machine counts are insensitive to any temporal variations in the composition of the traffic stream, as this information is collected continuously, 24 hours per day, 365 days per year. Currently, however, automatic classifiers in Montana are configured to sort recognizable vehicles into the 13 vehicle categories established by the Federal Highway Administration. For the purposes of this study, a more refined picture of the traffic stream was required, so this data was further disaggregated into the vehicle configurations listed in Table 2.3.2-1. Refinement of the classifications was done by MDT using information obtained from cross correlations between visual classification counts (performed using the Montana vehicle configurations listed in Table 2.3.2-1) and machine counts (performed using the FHWA vehicle categories).

Information on the composition of the traffic stream was provided by MDT for every mile of interstate and primary highway in the state. A typical record of this information from a segment of interstate highway is presented in Table 2.3.2-1. All routes were parsed into segments within which the composition of the traffic stream was expected to remain constant. The composition of this traffic stream was then established using data available from any automatic classifiers (and/or visual classification counts) in that area. Classification data collected from a single year, 1994, was used for this purpose. MDT was confident of the completeness and quality of the data collected in 1994; some concerns were expressed by MDT over the accuracy of the information available from previous years (Hult, 1995). While using only the 1994 data eliminated the problem of distorting the study results by using inaccurate data, it introduced the problem of skewing the study results due to any irregularities in vehicle operations specific to 1994. A qualitative review of the data found no major anomalies in traffic patterns for 1994 compared to other years.

Table 2.3.2-1 Typical Composition of the Traffic Stream on an Interstate in Montana

Route # I-90
Segment MP 154, Drummond to Deer Lodge
Length 31.3 miles
AADT 6666

Vehicle FHWA Class	Configuration Montana Designation	% of Traffic Stream FHWA	% of Traffic Stream Montana	No of Vehicles Montana Designation (AADT)
1	Motorcycle	0.1	0.1	9
2	Pass. Car	57.6	57.6	3842
3	PICKUP	19.7	19.7	1314
	2A-4T RV		0	0
	2A-4T SU		0	0
4	SCHOOL BUSES	0.8	0.8	53
	2A-COM. BUSES		0	0
	3A-COM. BUSES		0	0
5	2A-6T RV	1.5	0	0
	2A-6T SU		1.5	99
6	3A-RV	0.8	0	0
	3A-SU		0.8	53
7	4A-RV	0.1	0	0
	4A-SU		0.1	6
8	2-1 3A-TR	1.6	0.1	3
	2-2 4A-TR		1.1	72
	2S1 3A-TR		0.2	15
	2S2 4A-ST		0.2	15
9	3S2 5A-ST	11.9	11.6	776
	3-2 5A-TR		0.3	20
10	3S3 6A-ST	2.9	1.9	127
	3S4 7A-ST		0	0
	4S4 8A-ST		0	0
	3-3 6A-TR		1.0	67
	3-4 7A-TR		0	0
	3-5 8A-TR		0	0
	3-6 9A-TR		0	0
	4-6 10A-TR		0	0
11	2S1-2 5A-TU	0.2	0.2	13
12	3S1-2 6A-TU	0.2	0.2	13
	2S2-2 6A-TU		0	0
13	3S2-2 7A-TU	2.5	1.4	97
	3S2-3 8A-TU		1.1	70
	3S2-4 9A-TU		0	0
	3S1-2-1 7 A-MT		0	0
	2S1-2-2 7 A-MT		0	0
	3S1-2-2 8 A-MT		0	0

Information on vehicle operating weights by configuration was also obtained from MDT. All of the data collected from 32 static weigh station sites around the state in 1994 were used. The state has only recently begun to install weigh-in-motion (WIM) equipment, and no data is presently available from this source. Static weights of all vehicles passing the weigh stations over eight hour sampling periods are collected throughout the year. The time of the sampling period is purposefully varied with respect to time of day (note that mostly daylight hours are sampled), day of week, and day of month to capture all temporal variations in vehicle activity (Galt, 1996). Weights of 12,000 vehicles from this data collection program in 1994 were used in this study. While temporal variations in vehicle operation may be adequately represented in this sample, overweight vehicle operation is not, due to the manner in which the data was collected. The decision was made to do this analysis without correcting the static weight data for overweight vehicles believed to be in the traffic stream. The state of Montana has only limited information on the percentage of overweight vehicles that operate on the highways.

The composition of the heavy vehicle traffic operating on Montana's highways (considering 3 axle single units and larger vehicles), as represented in the weigh station data set, is presented in Figure 2.3.2-1. The overwhelming majority of the heavy vehicles using the system are 3S2 units. These vehicles compromise over 60 percent of the heavy vehicles on the system (out of the total of 66 percent of all 5 axle combinations). The second most frequent vehicle class on the system is single trucks (20 percent) followed by 7 axle combinations (6 percent). Thus, these three vehicle categories account for 92 percent of the heavy vehicle traffic, with the remaining 8 percent split primarily between 3S3s and 5, 6, and 8 axle combinations.

A histogram of the vehicle weights measured for the most common truck configuration on the system, the 3S2, is presented in Figure 2.3.2-2. Average empty and average operating weights were determined for all heavy vehicle configurations using data of this type (see Table 2.3.2-2). Operating weights for a few vehicle types were very different on the interstate and primary systems, and every effort was made to account for these differences as appropriate.

The volume of average daily truck traffic (1994 data) along the interstate system and along a sampling of primary routes around the state are summarized in Table 2.3.2-3. These values are for 3 axle single unit and larger vehicles. These values were obtained using the data provided by MDT in the format presented in Table 2.3.2-1.

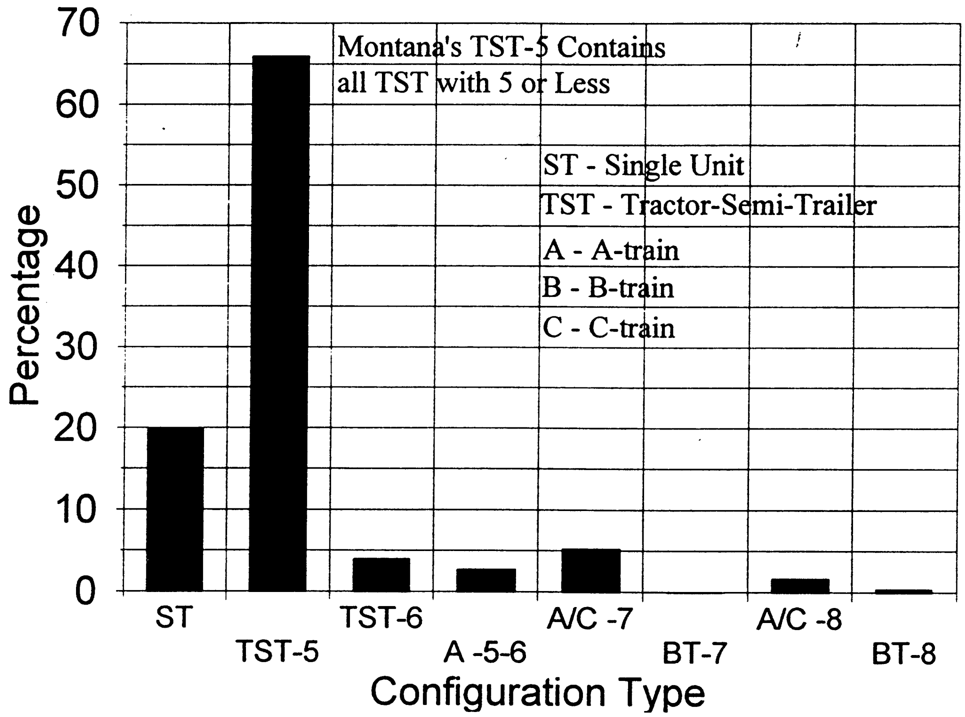


Figure 2.3.2-1 Present Composition of the Heavy Vehicle Traffic on the Montana Highway System

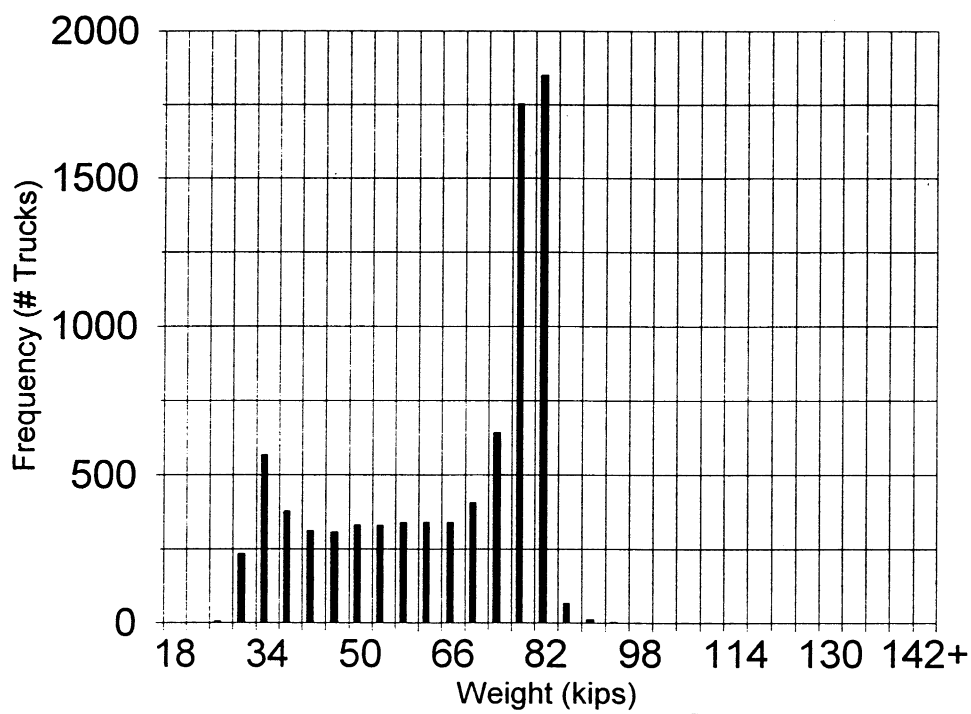


Figure 2.3.2-2 Weight Distribution for 3S2 Vehicles as Reported by MDT for 1994

Table 2.3.2-2 Average Empty Weight and Operating Weight of Vehicles on the State Highway System

Configuration	Average GVW Weights From Weigh Station Data, kips	Average Empty Weight from Weigh Station Data, kips	Average Pay Load, kips
Single Unit			
2SU	16.0	14.5	1.5
3SU	30.2	24.0	6.2
4SU	52.1	28.0	24.1
Truck and Trailer			
2-1	19.5	13.5	6.0
2-2	23.2	14.5	8.7
3-2	69.0	31.0	38.0
3-3[a]	71.9	36.0	35.9
3-4[a]	71.9	36.0	35.9
Tractor, Semi-Trailer			
2S1	28.1	23.5	4.6
2S2	32.3	25.5	6.8
3S2	63.4	32.0	31.4
3S3	68.8	35.0	33.8
3 Unit Combination			
5 Ax A-train	64.5	35.0	29.5
6 Ax A-train	62.6	37.0	25.6
7 Ax A-train	78.7	38.0	40.7
8 Ax A-train	91.4	40.0	51.4
9 Ax A-train	86.0	42.0	44.0

[a] vehicles grouped as a single configuration in data

Table 2.3.2-3 Average Daily Truck Traffic (3 SU and larger vehicles, 1994 data)

Route[a]	Length (miles)	Average Daily Truck Traffic
I-15	396	561
I-90	546	1247
I-94	249	746
All Interstate	**1191**	**916**
P-1	666	213
P-2	95	82
P-4	58	269
P-5	186	483
P-7	95	469
P-10	112	183
P-14	271	79
P-16	48	203
P-22	89	98
P-23	140	131
P-24	140	247
P-29	90	140
P-32	66	64
P-37	104	230
P-42	76	35
P-44	28	70
P-45	44	111
P-57	328	196
P-59	57	88
P-61	157	84
P-66	50	51
Selected Primaries	**2900**	**189**
Interstate and All Primaries	**6643**	**320**

[a] route locations are shown on Figure 2.3.2-3

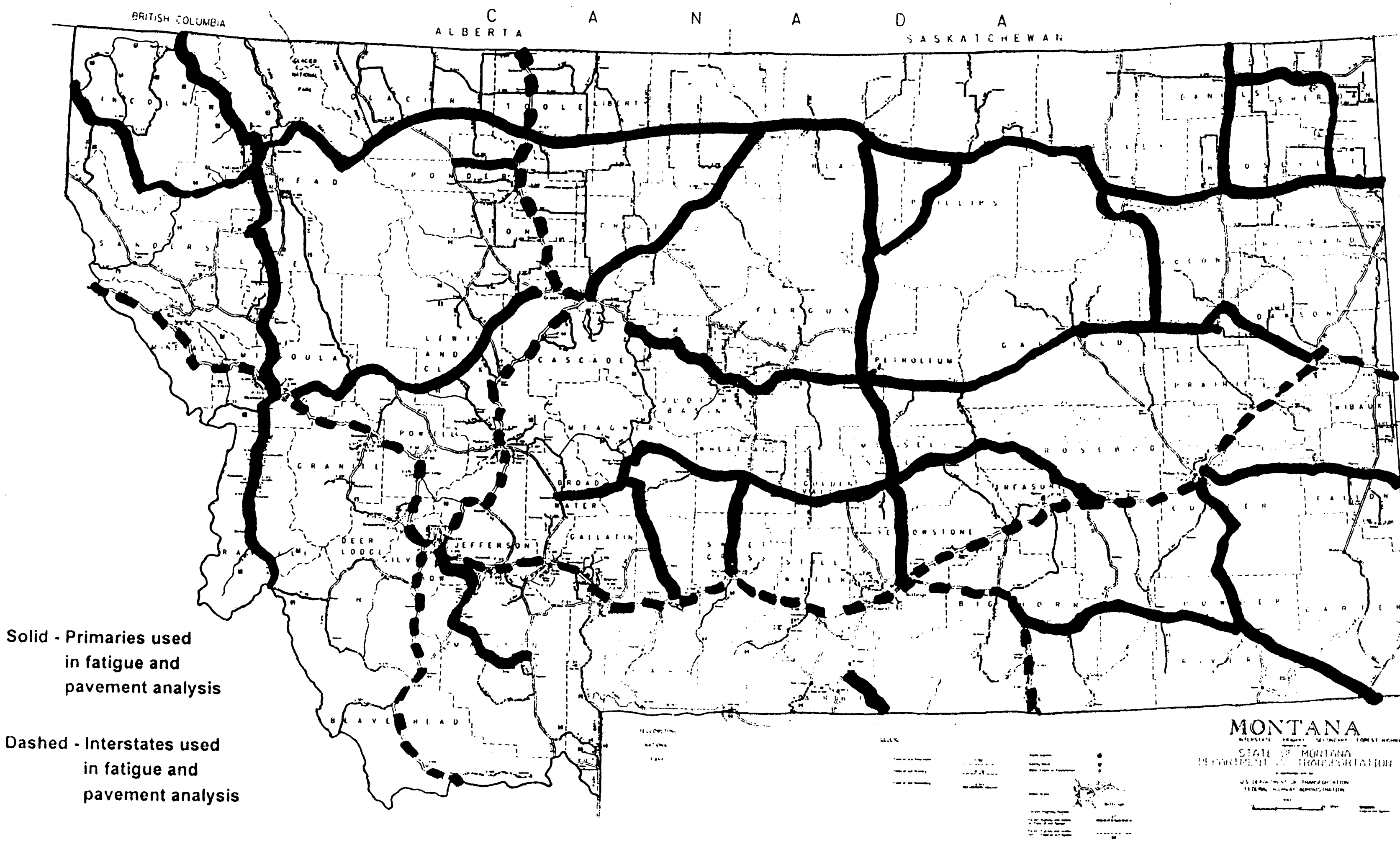


Figure 2.3.2-3 Location of Interstate and Primary Routes for which Truck Traffic Data are Presented in Table 2.3.2-3 (these same routes were used for subsequent fatigue, pavement, and cost analyses)

3. NEW VEHICLE CONFIGURATIONS AND TRAFFIC STREAMS

3.1 GENERAL REMARKS

Three alternate vehicle size and weight regulatory situations were considered in this investigation, namely, the adoption of full Canadian Interprovincial limits and the adoption of two hybrid systems of limits that incorporate aspects of both the Canadian Interprovincial and existing Montana limits (referred to as Canamex and Canamex Short limits). It was assumed that these limits would be adopted at least on a regional scale, so that an operator's choice of vehicle configuration would not be significantly restricted by differences in size and weight limits in adjacent states. For all three size and weight limit scenarios, the assumption was made that all configurations currently allowed under Montana law would still be able to operate in the future, and that additionally either the Canadian Interprovincial, Canamex, or Canamex Short vehicles would also be allowed to operate. Note that in all cases, acceptable vehicle configurations had to meet all the requirements of either the existing system or the alternate system under consideration. Thus, for example, in considering the adoption of Canadian Interprovincial limits, Canadian axle load limits were not universally and unconditionally extended across all U.S. vehicle configurations. Vehicles loaded to full Canadian axle weight limits had to adhere to Canadian axle configurations. This approach was followed under the presumption that the configurations used for the heavy Canadian vehicles were specifically established based on operational safety considerations, and that this level of safety might not be realized by Montana configurations operating at these same axle weights.

For each of the three regulatory situations under consideration, two future scenarios were investigated. These scenarios consisted of:

1) a short term vision of the future traffic stream, in which operators will take advantage of any increased weight allowed on their existing equipment as well as perform modest modifications of their existing equipment if a large weight gain is to be realized by such modifications. The further assumption was made that negligible changes will occur in choices of transportation modes for various purposes and commodities.
2) a long term vision of the future traffic stream, in which operators will purchase new equipment consistent with their needs and the new regulatory situation, and some changes

will occur in choices of transportation modes for various purposes and commodities (notably, freight will be diverted from rail to truck).

The process of predicting the composition of the new traffic streams consisted of assigning all of the present freight carried on the highway system, plus any new freight diverted from other modes (rail), to a vehicle fleet consisting of all the old configurations and the new Canadian, Canamex, or Canamex Short configurations.

3.2 CANADIAN INTERPROVINCIAL AND CANAMEX LIMITS

3.2.1 Canadian Interprovincial Limits - The Canadian Interprovincial limits on truck weight and size generally allow:

1) higher axle weights for tandem and tridem axle groups than are presently allowed in Montana, and
2) shorter and heavier combination vehicles than are presently allowed in Montana.

The Canadian regulations are specifically directed toward vehicles engaged in interprovincial transport, which were assumed to consist of semi-trailers and other combinations (RTAC, 1987). The regulations were established based on results of an extensive research program, with due consideration given to highway safety and transport economy (RTAC, 1987).

The Canadian Interprovincial limits include restrictions on weight by axle group type and the spacing between axle groups. Minimum and maximum values are also specified for the length of the components of combination vehicles and their overall length. The intent of these restrictions is to insure a minimum level of safety with respect to vehicle operation based on length, weight, and coupling mechanisms; to limit pavement damage by restricting axle weight by group type; and to limit bridge damage by enforcing minimum spacings between individual axles within groups and between axle groups in combination vehicles. Considered collectively, the various regulations result in a narrow range of acceptable vehicle configurations compared to current Montana practice.

Montana has attempted to achieve the same objectives regarding pavement and bridge damage as the Canadian system by implementing broad rules that allow wide latitude to commercial vehicle operators in meeting their transportation needs. Thus, rather than dictating specific axle group spacings, for example, Montana simply requires that whatever spacings are selected must meet the Bridge Formula. One consequence of the system used in Montana is that determining if a vehicle is legal may be more difficult than under the prescriptive Canadian system. Another consequence of Montana's approach to size and weight limits is that very specific vehicle configurations (notably for larger and heavier vehicles) that place acceptable demands on the infrastructure may be excluded from use by the general formulas used to establish legal vehicles. While the Canadian system overcomes some of these problems, it limits the options available to the vehicle operator in meeting varied transportation needs. Under Montana's purposefully broad system, however, configurations can evolve that meet the letter of the law, but that violate the intent of the law to protect the highway infrastructure. Configurations are closely enough specified in the Canadian system to generally preclude this possibility. Thus, advantages and disadvantages are associated with both the system used in Montana and that used in Canada (under the Interprovincial Limits). Note that Montana's regulations have apparently been driven by controlling demands on the highway infrastructure, with little rigorous study of safety issues.

Canadian Interprovincial axle weight limits, summarized in Table 3.2.1-1, are up to 26 percent greater than the corresponding Montana axle load limits. The Canadian system explicitly enforces a single steering axle weight limit of 12,100 pounds; Montana does not have a weight limit explicitly for steering axles, although the limit set by tire manufacturers generally restricts the weight of such axles to around 14,000 pounds (Galt, 1996).

Table 3.2.1-1 Maximum Axle Weights, Canadian Interprovincial Limits vs. Current Montana Limits

Axle Type	Canadian Interprovincial Limit[a]	Montana limit	Ratio Canadian/Montana
Steering	12.1	None[b]	-
Single	20.1	20.0	1.005
Tandem	37.5	34.0	1.103
Tridem	52.9	42.5[c]	1.260

[a] based on information from Alberta Motor Transport Services, 1992
[b] limited to approximately 14,000 pounds by manufacturer's rated capacity
[c] limited by bridge formula

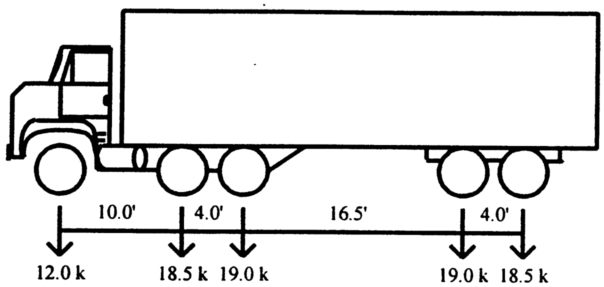
Typical Canadian Interprovincial weight and size limits by vehicle type are presented in Figure 3.2.1-1. The maximum allowable gross weight of a vehicle is generally calculated by simply adding up the maximum allowable weights of the axle groups of which it is comprised. The maximum gross vehicle weights (and other characteristics) of specific double trailer configurations (A-, B-, and C-trains) are restricted to a lower value than would be obtained using the above mentioned procedure, due to vehicle handling and safety considerations. Note that A-, B-, and C-trains are equipped with different coupling mechanisms between the trailer units, as shown in Figure 3.2.1-2. Typical allowable gross vehicle weights under existing Montana limits and under the Canadian Interprovincial limits are compared in Table 3.2.1-2. The maximum gross vehicle weights under Canadian Interprovincial limits are generally higher than under Montana limits (as determined by Bridge Formula B), with the greatest absolute differences in allowable weights occurring for the largest trucks. Weight increases of significance include a 9 percent increase for a 3S2, a 16 percent increase for a 3S3, and a 17 percent increase for an 8 axle B-train compared to an existing 8 axle A-train.

Length restriction comparisons between Montana and Canadian Interprovincial limits are difficult to formulate, in that the regulations in the two countries are based on differing philosophies, as previously discussed. The maximum length of combination vehicles under the Canadian Interprovincial limits is 82 feet. In Montana, combinations such as the Rocky Mountain double can operate at up to 95 feet with a permit (75 feet, without a permit). The maximum vehicle width under both Montana and Canadian Interprovincial limits is 8.5 feet. Length and width restrictions do influence pavement and bridge demand levels, in addition to geometric layout requirements.

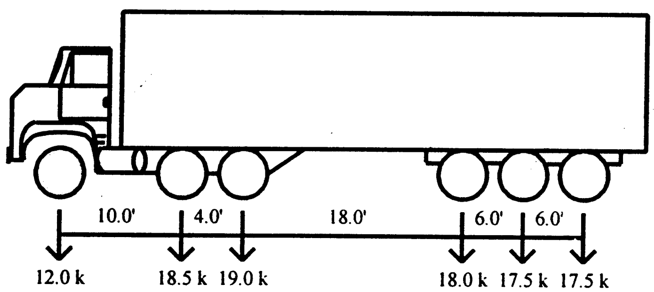
Comparisons of weight and volumetric capacity of typical vehicles under the present and the Canadian Interprovincial systems are presented in Table 3.2.1-3. A general comparison of "equivalent" vehicles under the two systems, the Rocky Mountain double and the Canadian C-train, is presented in Figure 3.2.1-3 (long configuration of each is shown). Canadian Interprovincial Limits generally offer the opportunity to haul more weight on large vehicles in a single trip than Montana limits, but the Canadian limits can restrict volumetric capacity compared to Montana limits.

Complete Canadian Interprovincial Limits on truck size and weight for single units, single units with trailers, and combination units are presented in Appendix A.

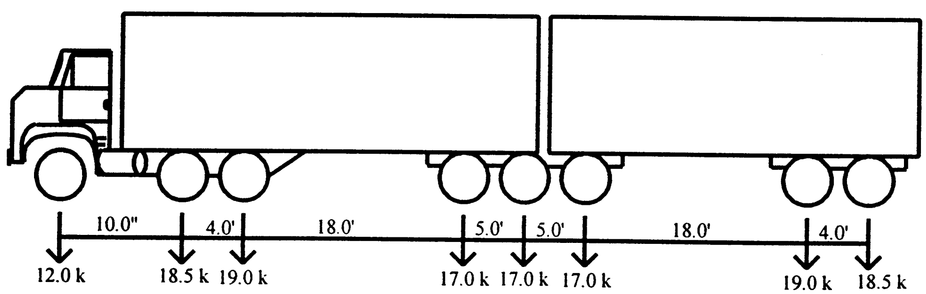
3S2



3S3



8 AXLE B TRAIN



8 AXLE C TRAIN

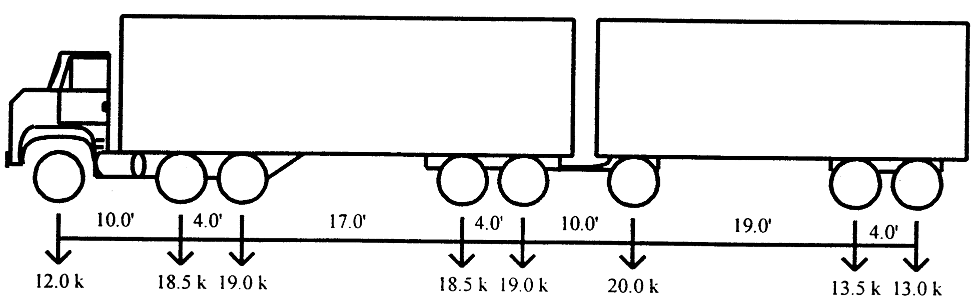
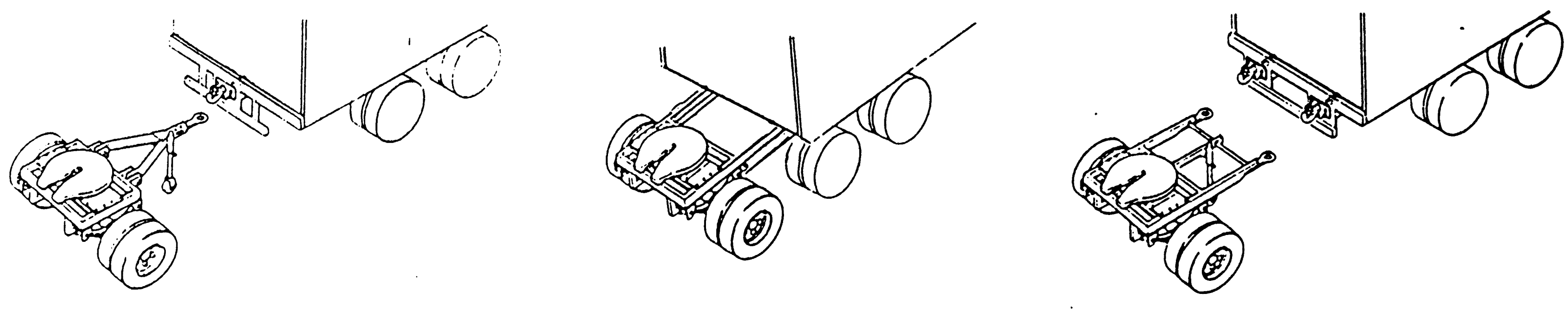


Figure 3.2.1-1 Typical Canadian Interprovincial Limits on Vehicle Size and Weight



a) A-train dolly b) B-train solid carriage c) C-train dolly (stabilized A-dolly)

Figure 3.2.1-2 Canadian Coupling Mechanisms for Combination Vehicles (adapted from RTAC, 1987)

Table 3.2.1-2 Typical Maximum Gross Vehicle Weights, Canadian Interprovincial, Canamex, and Canamex Short Vehicles vs. Current Montana Vehicles

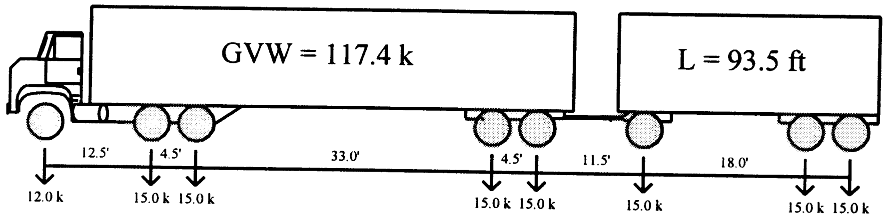
Configuration Type	Maximum Gross Vehicle Weights Under Various Systems (kips)			
	Montana	Canadian Interprovincial	Canamex	Canamex Short
Single Unit				
2SU	36.0	36.2	-[a]	36.0
3SU	50.0	53.6	-[a]	50.0
4SU	58.0	-[a]	-[a]	58.0
Truck and Trailer				
2-1	56.0	45.2	-[a]	56.0
2-2	70.0	76.3	-[a]	76.0
3-2	84.0	93.7	-[a]	90.0
3-3	92.0	111.1	-[a]	104.0
3-4	103.8	118.0	-[a]	118.0
Tractor, Semi-Trailer				
2S1	52.0	52.2	-[a]	52.0
2S2	66.0	69.7	-[a]	66.0
3S2	80.0	87.1	-[a]	80.0
3S3	88.0	102.5	-[a]	88.0
3 Unit Combination[d]				
5 Ax A-train	92.0	87.5[b]	92.0	87.3[b]
6 Ax A-train	106.0	104.9[b]	106.0	101.3[b]
7 Ax A-train	112.5	118.0[b]	118.0	115.3[b]
8 Ax A-train	117.4	118.0[b]	118.0	115.3[b]
9 Ax A-train	122.6	-[a]	-[a]	-[a]
5 Ax B-train	86.0	89.7	-[a]	86.0
6 Ax B-train	100.0	107.1	-[a]	100.0
7 Ax B-train	104.9	124.6	-[a]	108.0
8 Ax B-train	111.0	137.8	-[a]	122.0
5 Ax C-train	92.0	93.4	92.0	92.0
6 Ax C-train	100.2	109.8	106.0	106.0
7 Ax C-train	104.9	127.2	120.0	120.0
8 Ax C-train	111.0	133.4[c]	128.0	126.3[c]

[a] not part of system
[b] A-train back trailer limited to 35.3 kips
[c] C-train back trailer limited to 46.3 kips
[d] C-train under existing limits calculated using vehicle with Canadian Interprovincial geometrics

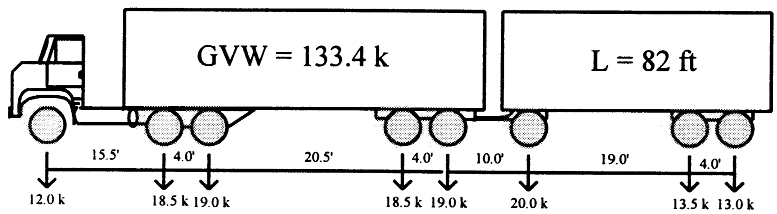
Table 3.2.1-3 Comparison of Weights and Volumetric Capacities, Montana and Canadian Interprovincial Limits

	Montana Limits		Canadian Limits		Canadian/Montana	
	GVW (lbs)	Length (ft)	GVW (lbs)	Length (ft)	GVW Ratio	Volume Ratio
Tractor, Semi-trailer						
2S1	52000	75	52200	82	1.00	1.00
2S2	66000	75	69700	82	1.06	1.00
3S2	80000	75	87100	82	1.09	1.00
3S3	88000	75	102500	82	1.16	1.00
3 Unit Combination						
5 Ax A-train	92000	95	87500	82	0.95	0.72
6 Ax A-train	106000	95	104900	82	0.99	0.72
7 Ax A-train	112500	95	118000	82	1.05	0.72
8 Ax A-train	117400	95	118000	82	1.01	0.72
9 Ax A-train	122600	95	-[a]	-[a]	-[a]	-[a]
5 Ax B-train	86000	95	89700	82	1.04	0.80
6 Ax B-train	100000	95	107100	82	1.07	0.80
7 Ax B-train	104900	95	124600	82	1.19	0.80
8 Ax B-train	111000	95	137800	82	1.24	0.80
5 Ax C-train	92000	95	93400	82	1.02	0.80
6 Ax C-train	100200	95	109800	82	1.10	0.80
7 Ax C-train	104900	95	127200	82	1.21	0.80
8 Ax C-train	111000	95	133400	82	1.20	0.80

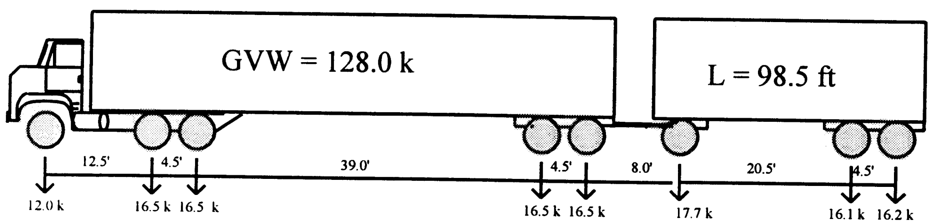
[a] Canadian Interprovincial Limits do not address a 9 Axle Double



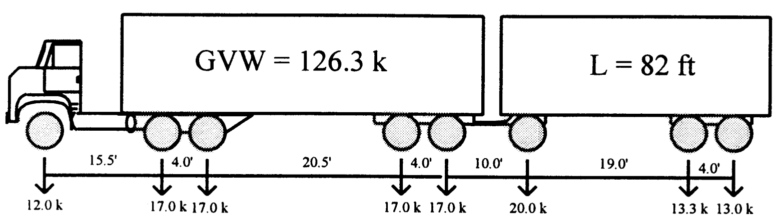
a) Rocky Mountain Double (existing 8 Axle A-train)



b) Canadian Interprovincial Limits, 8 Axle C-train



c) Canamex, 8 Axle C-train



d) Canamex Short, 8 Axle C-train

Figure 3.2.1-3 General Comparison of the Rocky Mountain Double and "Equivalent" Vehicles from Each Scenario

3.2.2 Canamex Limits - The Canamex size and weight limits specifically address 5, 6, 7 and 8 axle combination units (Alberta Transport and Utilities, 1994). These limits generally permit

1) nominally longer combination units than are presently allowed to operate in Montana (under permit), and

2) heavier 7 and 8 axle combination units than are presently allowed in Montana.

Canamex vehicles, which geometrically resemble existing Montana combination vehicles, are required to adhere to current Montana axle weight limits, but they are allowed to operate at Canadian Interprovincial gross vehicle weights. At such weights, these vehicles violate Bridge Formula B. A general comparison of an 8 axle Canamex C-train and an 8 axle Rocky Mountain double is presented in Figure 3.2.1-3. The maximum Canamex vehicle length is 98.5 feet compared to the current length of 95 feet allowed in Montana (with a permit). The gross weight limits for 5 and 6 axle combination vehicles under Canamex are identical to the existing weight limits for 5 and 6 axle combinations (see Table 3.2.1-2). Seven and 8 axle C-trains, however, can carry 7 to 9 percent more weight than the corresponding Montana A-trains.

The Canamex limits are presented in a format similar to that of the Canadian Interprovincial limits. Weight limits are determined based on axle group type, axle group length, and spacings between axle groups. Minimum and maximum values are specified for (a) the lengths of various components of the vehicle and (b) its overall length. A complete description of the Canamex size and weight limits is presented in Appendix A.

The recent TRB study of truck size and weight (TRB, 1990a), found that vehicles operating at Canadian Interprovincial Limits place high demands on the infrastructure in the United States, as previously mentioned. The Canamex vehicles, with lower axle loads, lower gross vehicle weights, and longer wheelbases than the Canadian Interprovincial vehicles, may offer a compromise that allows some of the economic and safety benefits of the Canadian system to be realized without placing such high demands on the highway infrastructure.

3.2.3 Canamex Short Limits - The Canamex Short vehicle size and weight scenario is similar to the Canamex scenario, in that while vehicles are required to adhere to Montana axle weight limits, they are allowed to operate up to Canadian Interprovincial gross vehicle weights. In the

Canamex Short scenario, however, operators taking advantage of the increased gross vehicle weights must satisfy the geometrics of the Canadian Interprovincial limits. Thus, the Canamex Short scenario generally allows shorter and heavier combination vehicles to operate on the highway than are presently permitted in Montana. The particular vehicles of interest in the Canamex Short scenario are C-trains. Following Canamex Short limits, these vehicles can operate at gross weights up to 8 percent higher than are presently permitted in Montana on a similar A-train (see Table 3.2.1-2). An 8 axle Canamex Short C-train is shown in Figure 3.2.1-3.

Similar to the Canamex vehicles, the Canamex Short vehicles have lower allowable axle weights and gross vehicle weights than the Canadian Interprovincial vehicles, and therefore they were expected to place lower demands on the highway infrastructure than the Canadian Interprovincial vehicles. Canamex and Canamex Short vehicles were expected to have similar impacts on the highway infrastructure, in that the two systems enforce the same axle weight limits and similar maximum gross vehicle weights. The Canamex Short vehicles are, however, significantly shorter than the Canamex vehicles (maximum lengths of 82 and 98 ½ feet, respectively). This length difference was expected to result in some differences in the bridge impacts for the two scenarios.

3.3 GENERATION OF NEW TRAFFIC STREAMS

3.3.1 General Remarks - If the Canadian Interprovincial limits or either of the Canamex limits described above are adopted in Montana, a gradual change will occur in the composition of the traffic stream and the characteristics of the vehicle fleet. Shifts are expected to occur in both the total load carried by particular vehicle configurations as well as in the relative populations of each vehicle. The total amount of goods carried by truck may also increase to some degree, as some shipments are shifted from rail to truck transport (TRB, 1990a). In developing new traffic streams for the various scenarios described above, the decision was made to allow all weight limited carriers the option of switching to vehicle configurations that allow higher payloads. This approach is consistent with assuming that interstate carriers that operate in Montana would not be restricted in their choice of operating weights, volumes, and vehicles by more restrictive laws in other states or provinces. Thus, adoption of the new limits was assumed to occur at least at a regional (multi-state) level. This approach should affect the greatest change in the predicted traffic streams under each scenario compared to the present situation.

The process of predicting the composition of the new traffic streams consisted of assigning all of the present freight carried on the highway system, plus any new freight diverted from other modes (rail), to a vehicle fleet consisting of all the old configurations and the new Canadian, Canamex, or Canamex Short configurations. Diversion of freight from existing configurations and its assignment to new configurations was decided after review of the factors that affect an operator's decision to convert to a different vehicle or operating at a heavier weight; the present distribution of vehicles in the traffic stream and the distribution by weight of vehicles within each configuration; the diversions used in the TRB truck size and weight study (TRB, 1990a); and changes that occurred in the composition of the vehicle fleet in Canada after the adoption of Interprovincial limits.

<u>3.3.2 Diversions Between Vehicles</u> - The assumption was made in this study that only weight limited vehicles would consider shifting to new configurations and operating weights if Canadian Interprovincial, Canamex, or Canamex Short limits on truck size and weight were adopted. Weight limited vehicles are vehicles which have space for additional cargo when loaded at their maximum gross vehicle weight. Thus, while such vehicles have space for additional cargo, they are prohibited from carrying it. The new configurations offer advantages in such situations over existing vehicles through their increased weight limit. Note that the Canadian Interprovincial and Canamex Short configurations offer little advantage to volumetrically limited vehicles (see Table 3.2.1-3). Volumetrically, the Canadian Interprovincial and Canamex Short configurations generally provide the same capacity as existing vehicles (the situation for tractor with semi-trailer and smaller vehicles) or less capacity than existing configurations (the situation for multi-trailer combinations).

Vehicles that appeared to be weight limited under current truck size and weight limits were identified from vehicle weight data provided by MDT. The manner in which this data was collected has been previously described. A typical distribution obtained from the data (in this case for 3S2) is presented in Figure 3.3.2-1. Those vehicles typically observed to be clustered within 10 percent of the maximum allowable gross vehicle weight for a given configuration were judged to possibly be weight limited. Naturally, some of these vehicles were both volume and weight limited. RTAC implies that from 33 percent to 66 percent of weight limited vehicles are also volume limited (TAC/CTRI, 1994). In this study, the number of vehicles assumed to be volume and weight limited within each configuration was generally kept within this range. The specific percentage assumed was established based on trends observed in the weight distribution data. In the absence of any such trends, the percentage of both weight and volume limited vehicles was set at 50 percent. The number of possibly weight limited trucks determined above was reduced by this figure to obtain an estimate of the weight limited number of vehicles in each configuration.

Naturally, not all weight limited vehicles in all configurations will change their mode of operation (either by operating at a heavier weight and/or changing equipment) if new truck size and weight limits are instituted. Many of the smaller and lighter vehicle configurations are expected to be unaffected by the adoption of the Canadian Interprovincial limits or either of the Canamex Limits. The philosophy followed in this regard was that operators of many of these configurations would have already switched to existing larger configurations, if increased weight capacity was important. Configurations judged to be in this category include: single units (2SU, 3SU, and 4SU); light tractor, semi-trailers (2S1 and 2S2); light truck and trailers (2-1 and 2-2); and light combination vehicles (2S1-2, 2S2-2). Furthermore, the reduction in volumetric capacity of some Canadian configurations relative to comparable Montana configurations will make them less attractive to some weight limited operators.

Presuming that some attractive alternate configurations were available for a particular existing vehicle, the decision was made to move the freight carried by all weight limited vehicles of that kind (as identified above) to the alternate configuration(s). This action was generally accomplished by moving 33 to 66 percent of the total freight carried on vehicles within 10 percent of the current maximum gross vehicle weight to alternate configurations. Alternate configurations were broadly defined to be either the same vehicle operating at heavier weights or an entirely different configuration (generally, one of the new, heavier vehicles). In reality, the

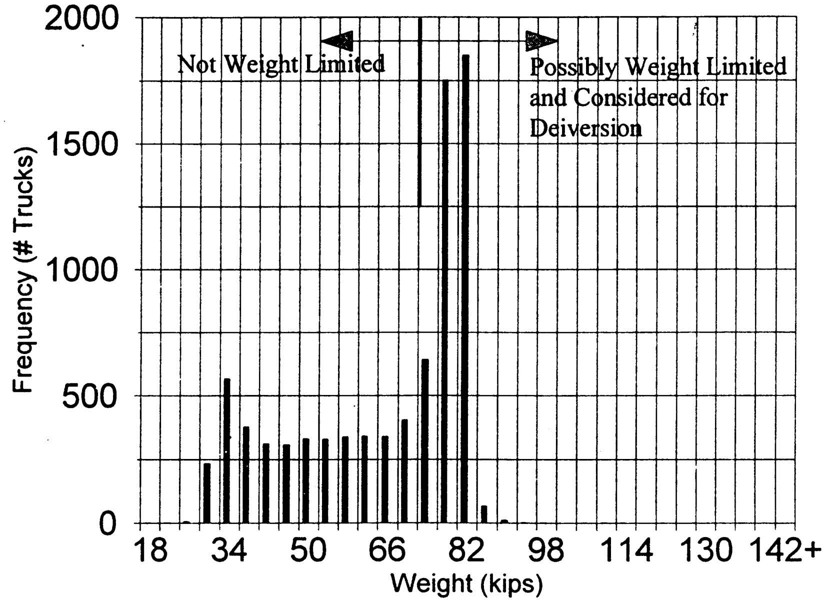


Figure 3.3.2-1 Weight Distribution and Freight Diversion, 3S2 Vehicles

availability of proper shipping/receiving facilities, cost of new equipment, maneuverability requirements, type of haul, etc. will influence decisions of this kind, and some weight limited operators will choose to continue to use their existing configurations.

3.3.3 Rail Diversion - Diversion of freight from rail to truck reportedly can be significantly influenced by changes in truck transport costs, which are in turn directly affected by changes in truck size and weight limits (TRB, 1990a). In general, increases in truck size and weight are expected to result in some diversion of freight from rail to truck (TRB, 1990a). Of the various scenarios studied by TRB in their truck size and weight study (TRB, 1990a), the highest diversion of freight from rail to truck was estimated for the adoption of Canadian Interprovincial Limits. TRB estimated that if Canadian Interprovincial Limits are adopted the freight diverted from rail to truck would increase the ton-mile freight movements on the highway system by approximately 3 3/4 percent. This percentage increase in freight hauled by truck was used in this study in creating new vehicle weight distributions. Intermodal freight diversion is affected by many parameters and can vary substantially between regions. Therefore, this figure may merit further refinement in future work. The freight diverted off of rail was assumed to move to B-trains (Canadian Interprovincial scenario) and C-trains (both Canamex scenarios). The same amount of freight was assumed to be diverted under both Canamex options as under the Canadian Interprovincial option, although the lower weight allowed on Canamex vehicles versus Canadian Interprovincial vehicles might result in less diversion of freight from rail to truck.

3.3.4 Implementation - The diversion factors established above, and the expected operating characteristics of each vehicle as determined from the new weight distributions, were used to generate new traffic streams at any location of interest from knowledge of the composition of the existing traffic stream at that location. The composition of the current traffic stream on every segment of the interstate and primary system was obtained from the vehicle classification data provided by MDT, as discussed in Section 2.2.2. On any given section of highway, the freight hauled by each vehicle configuration was calculated by multiplying the number of vehicles from the classification by the average payload for the vehicle. This amount of freight was then reassigned to the vehicle configurations in the new traffic stream following the methodology described in Section 3.3.2. Additional freight was assigned to each new vehicle configuration as necessary to accommodate any freight being diverted to the configuration from rail. The number of vehicles required in the new traffic streams to carry this freight was calculated by dividing the total freight assigned to the vehicle configuration by the new average payload for the vehicle. Average payload for each vehicle and traffic scenario were calculated as the average operating weight of the vehicle minus the average empty weight of the vehicle. Average operating weights were calculated for each configuration under each scenario using the weight/frequency distributions derived above.

3.4 NEW TRAFFIC STREAMS

3.4.1 General Remarks - In all cases, new weight distributions were created for all configurations following the broad philosophies on vehicle-to-vehicle and rail-to-vehicle freight diversions discussed above. Specific selections of vehicles to both lose and receive freight were made based on the specific scenario under consideration. Attention generally focused on the treatment of the 3S2 configuration, in that this configuration presently accounts for 60 percent of the heavy vehicles on the system. New weight distributions were generated for each configuration after the diversions discussed above were completed. The shape of the new weight distributions were established to match, as appropriate, various aspects of the shapes of existing distributions, and the magnitudes were established based on the weight of freight to be carried. The new weight distributions were used to calculate average operating weights and average payloads for each type of vehicle under each scenario. These new operating characteristics were then used in conjunction with the freight diversion factors to determine the composition of a new traffic stream from the vehicles and freight carried by the old stream.

The specific scenarios considered in this study consisted of short and long term predictions of the traffic stream if Canadian Interprovincial, Canamex, or Canamex Short limits (total of 6 scenarios) were adopted. In the short term, it was generally assumed that only simple and inexpensive changes would occur in commercial vehicle operations; in the long term, that major equipment investments would be made as well as decisions on shifting freight to/from rail.

3.4.2 Canadian Interprovincial Limits, Short and Long Term Changes - The first Canadian Interprovincial scenario considered short term (1 to 3 years) changes in the characteristics of the vehicle fleet if full Canadian Interprovincial limits were to be adopted. The TRB study on truck size and weight (TRB, 1990a) indicated that equipment changes will be substantially accomplished within 3 years of the change in weight regulations. Decisions on what will happen to each configuration (existing and new) under this scenario are summarized in Table 3.4.2-1. The composition of the heavy vehicle traffic for this scenario is summarized in Figure 3.4.2-1. Over the short term, operators were assumed to move to take advantage of increased weights allowed on existing configurations, but not to significantly invest in new equipment. Thus, for example, weight limited operators of 3S2 vehicles were assumed to shift to operating "Canadian" 3S2 vehicles at weights approaching the Canadian 88,000 pound limit. Distributions of both present and estimated future weights for 3S2 vehicles are shown in Figure 3.4.2-2. It was also assumed that operators of long combinations (e.g., 3S2-2, 3S2-3) would use two short trailers and adapt their dolly configurations to operate as Canadian C-Trains (increasing their

Table 3.4.2-1 Summary of Vehicle Diversion Decisions, Canadian Interprovincial Limits

Configuration Type	Short Term		Long Term	
	% Freight Diverted[a]	Comments	% Freight Diverted[a]	Comments
SINGLE UNIT				
2SU, 3SU	0	Assumed that they would have already switched to 4SU if needed extra capacity	0	Assumed that they would have already switched to 4SU if needed extra capacity
4SU	0	Canada has no 4SU to divert to	0	Canada has no 4SU to divert to
TRUCK AND TRAILER				
2-2	0	Assumed that they would have already switched to 3-2 if needed extra capacity	0	Assumed that they would have already switched to 3-2 if needed extra capacity
3-2	35	Would use same configuration at higher Canadian weights	35	Would use same configuration at higher Canadian weights
3-4	40	Would use same configuration at higher Canadian weights	40	Would use same configuration at higher Canadian weights
4-4	0	Canada has no 4-4 to divert to in the short term	90	Would divert to Canadian 3-4 for higher capacities and lower operating costs
3-5, 3-6, 2-1 3-3, 4-2, 4-3	mixed	Very few vehicles operating, will divert like similar configurations	mixed	Very few vehicles operating, will divert like similar configurations
TRACTOR SEMI TRAILER				
2S2	0	Assumed that they would have already switched to 3S2 if needed extra capacity	0	Assumed that they would have already switched to 3S2 if needed extra capacity
3S2	35	35 % same configuration at higher weight	50	15 % same configuration at higher weight, 25 % to 3S3, 10 % to 8 AX B-train
3S3	50	50 % same configuration at higher Canadian weights	50	50 % same configuration at higher weight, will receive freight from 3S2
4S4	0	No Canadian 4S4 vehicle to divert to	100	Would divert all to Canadian 3S3 for higher capacities and lower operating costs
4S3,4S2,2S1, 2S4, 2S3,3S1	mixed	Very few vehicles operating, will divert like similar configurations	mixed	Very few vehicles operating, will divert like similar configurations
3 UNIT COMBINATION				
5,6 AX A T	0	Assumed that they would have already switched to 7AX A T if needed extra capacity	0	Assumed that they would have already switched to 7AX A-train if needed extra capacity
7 AX A T	40	Would divert to 7 AX C-train	40	Would divert to 8 AX B-train
8 AX A T	45	Would divert to 8 AX C-train	45	Would divert to 8 AX B-train
9 AX A T	0	No Canadian 9 axle vehicle to divert to	35	Would divert to 8 AX B-train
8 AX B T	0	Current 8 AX B T will operate at Canadian Limits	-	Will receive freight from 7,8,9 AX A T
7,8 AX C T	-	Will receive freight from 7,8 AX A T	0	No advantage over other configurations
5,6,7 AX B T 5,6 AX C T	0	No advantage over other configurations	0	No advantage over other configurations

[a] percentage of total freight carried by the configuration diverted to a different configuration

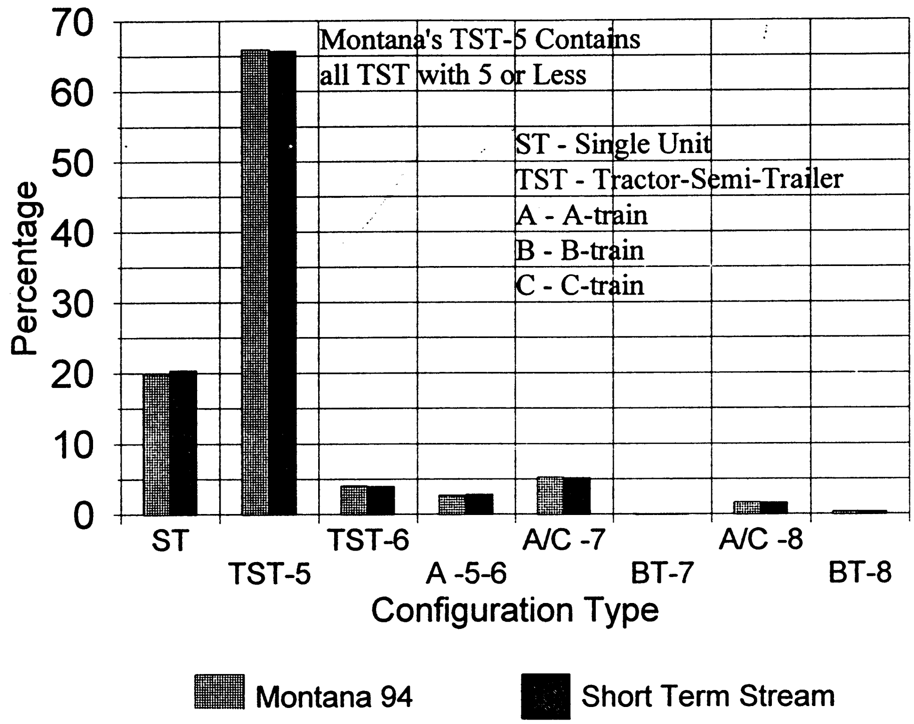


Figure 3.4.2-1 Composition of the Traffic Stream, Canadian Interprovincial Limits, Short Term Estimate

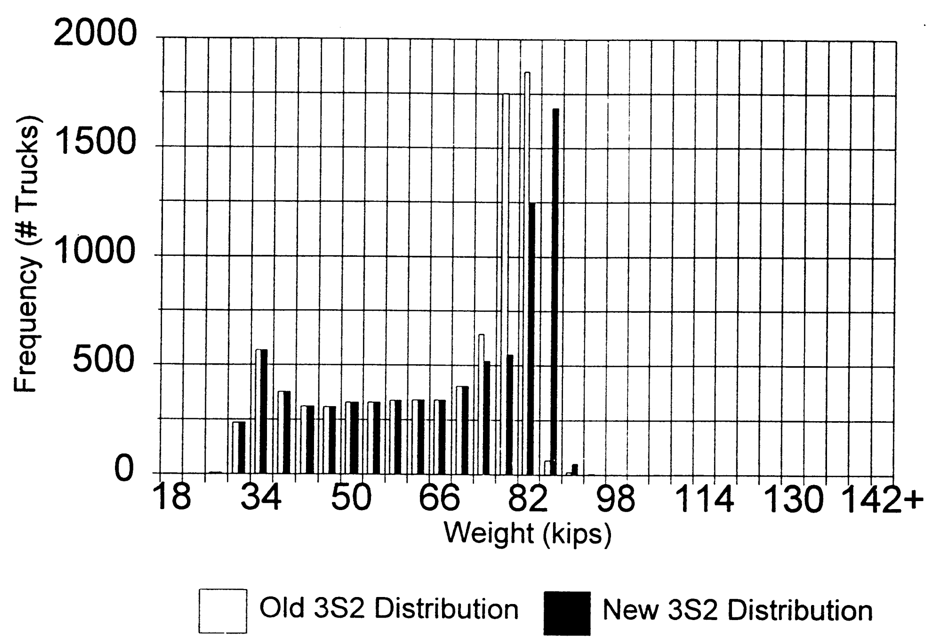


Figure 3.4.2-2 Distribution of Vehicle Weights, 3S2 Vehicles, Existing Limits and Canadian Interprovincial Limits, Short Term Estimate

allowable gross vehicle weights, for example, from 113,000 to up to 127,000 pounds on a 7 axle vehicle). Distributions of both present and estimated future weights of 7 axle combination vehicles (3S2-2) are shown in Figure 3.4.2-3. The peak in the future distribution terminating at 113,000 pounds is associated with volumetrically and weight limited operators that continue to use Montana 7 axle configurations; the next peak at 127,000 pounds, to weight limited operators that switched to 7 axle Canadian C-trains.

The second Canadian scenario considered changes in the traffic stream over the long term (over 3 years) if full Canadian Interprovincial Limits were adopted. Decisions on what will happen to each configuration (existing and new) under this scenario are summarized in Table 3.4.2-1. The resulting composition of the heavy vehicle traffic by major configuration is shown in Figure 3.4.2-4. Under this scenario it was assumed that operators would invest in new equipment to take advantage of the increased weights allowed on 3S3 and larger vehicles under the Canadian system. Weight limited operators were specifically assumed to move away from the 3S2 configuration in favor of the 3S3, and to also move away from 7 and 8 axle A- and C-trains in favor of the B-train. Some freight was also shifted from the 3S2 configuration to the B-train in response to the large increase in payload offered by the B-train compared to other existing configurations. Thus, the percentage of 3S2 vehicles decreased substantially from being 66 to being 44 percent of the heavy vehicle traffic. The percentage of 3S3 and B-trains in the traffic stream increased dramatically. The 3S3 configuration increased from being 4 percent of the heavy vehicle traffic to being 14 percent of this traffic. The weight distribution for 3S2 and 3S3 vehicles under present conditions and for this new scenario are shown in Figures 3.4.2-5 and 3.4.2-6, respectively. The Canadian B train increased from being 0.5 percent to being 13 percent of the heavy vehicle traffic, although approximately one-half of this increase was due to freight previously carried on rail being diverted to B-trains. The total freight carried on the highway system was increased by 3 3/4 percent to accommodate the expected diversion of freight from rail to truck. This freight was assigned to Canadian 8 axle B-trains.

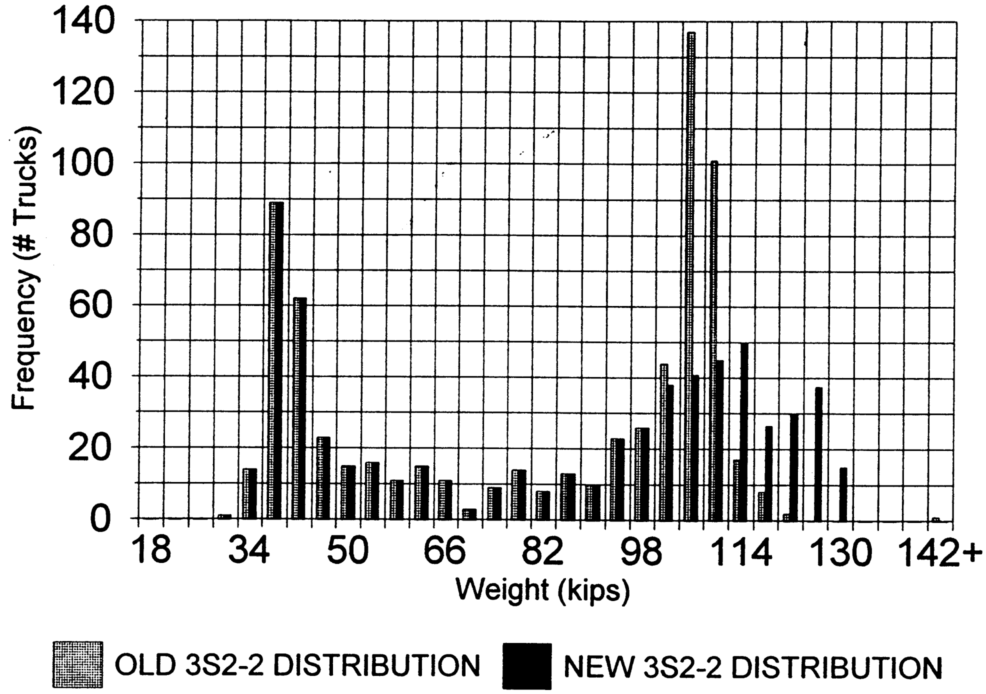


Figure 3.4.2-3 Distribution of Vehicle Weights, 3S2-2 Vehicles, Existing Limits and Canadian Interprovincial Limits, Short Term Estimate

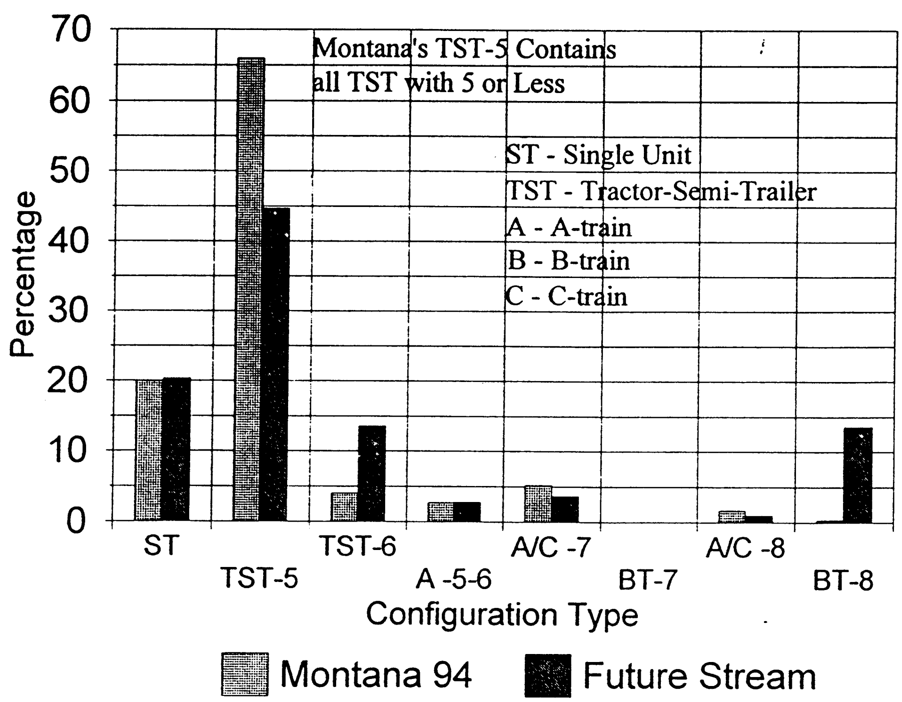


Figure 3.4.2-4 Composition of the Traffic Stream, Canadian Interprovincial Limits, Long Term Estimate

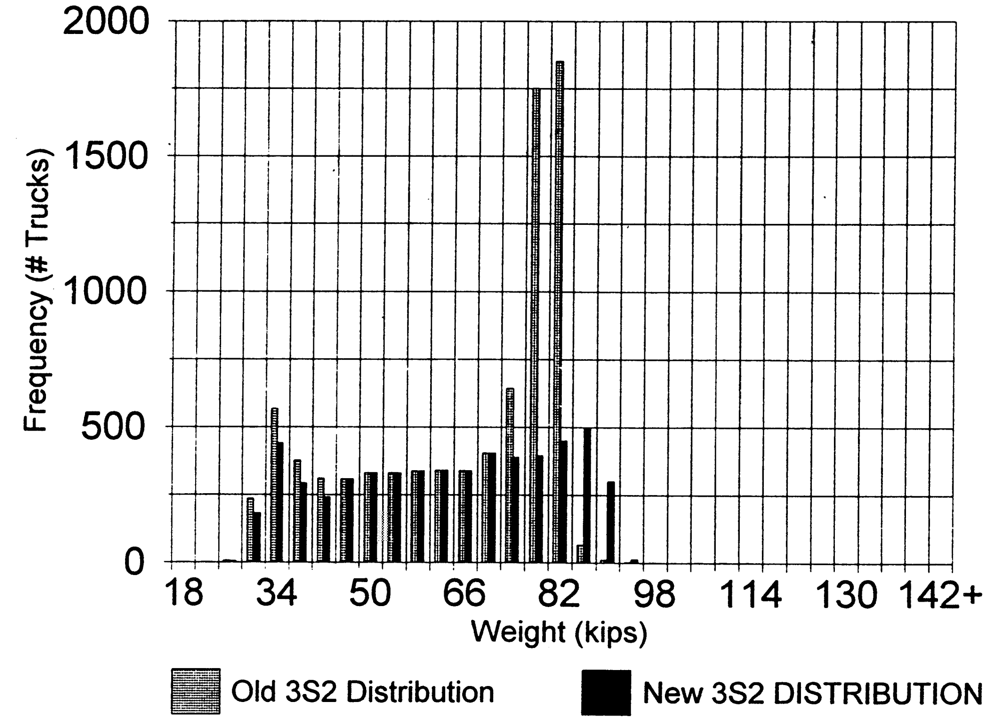


Figure 3.4.2-5 Distribution of Vehicle Weights, 3S2 Vehicles, Existing Limits and Canadian Interprovincial Limits, Long Term Estimate

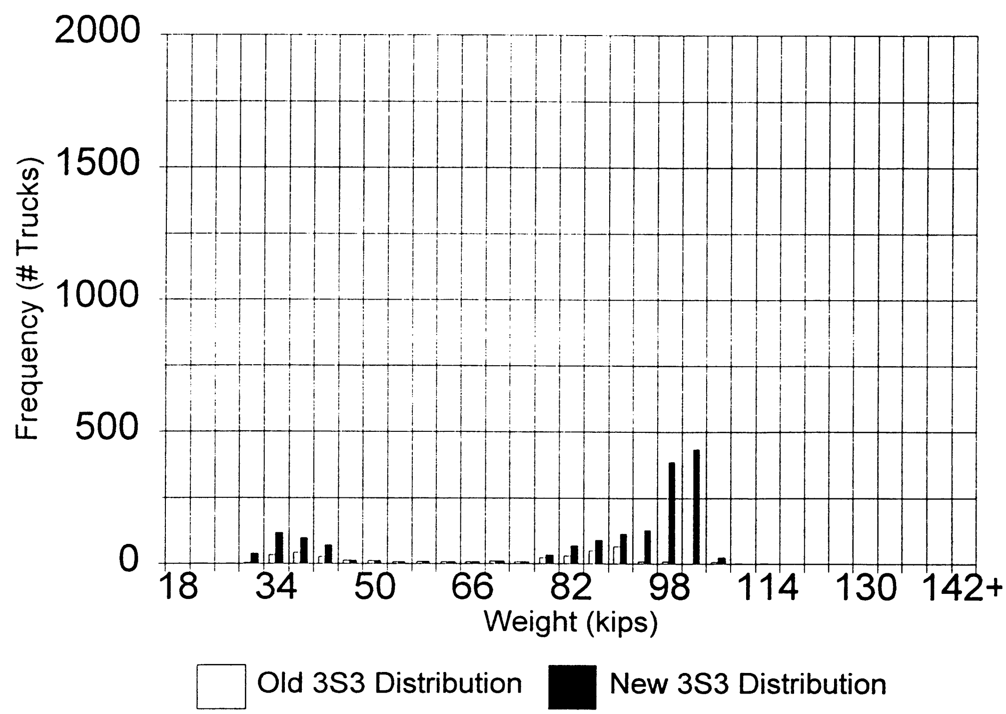


Figure 3.4.2-6 Distribution of Vehicle Weights, 3S3 Vehicles, Existing Limits and Canadian Interprovincial Limits, Long Term Estimate

3.4.3 Canamex Limits - The procedure used in estimating the future traffic stream if Canamex limits are adopted was similar to that used for Canadian Interprovincial limits. Specific diversions are summarized in Table 3.4.3-1. The "attractive" alternatives available to vehicle operators are more limited under Canamex limits compared to Canadian Interprovincial limits. Notably, gross weights of simple tractor, semi-trailer units are the same in Canamex as under present Montana Limits. Configurations that do offer weight advantages under Canamex are the large truck and trailer configurations, the 7 axle A- and C-train, and the 8 axle C-train.

In the short term Canamex scenario, freight was shifted from existing 7 and 8 axle A-trains to heavier 7 axle A-trains, 7 axle C-trains, and 8 axle C-trains. The estimated composition of the heavy vehicle fleet in the short term under the Canamex scenario is presented in Figure 3.4.3-1. The only significant change in the fleet is an increase in 8 axle A/C trains from being 2 to being 5 percent of truck traffic. The Canamex 7 and 8 axle combinations are volumetrically larger than the corresponding Canadian Interprovincial combinations. Therefore, more freight was shifted from lighter to heavier 7 and 8 axle combinations in the Canamex scenario compared to the Canadian Interprovincial scenario. Ten percent of the freight carried on 3S2 vehicles was also shifted to 8 axle C-trains. Geometrically, the vehicle lengths permitted under the Canamex scenario provide 3S2 operators with the simple option of adding a short trailer with a stabilized dolly to run an 8 axle C-train. The weight distributions for 7 axle (3S2-2) and 8 axle (3S2-3) vehicles under existing and Canamex limits are presented in Figures 3.4.3-2 and 3.4.3-3, respectively.

In the long term scenario, the same vehicle-to-vehicle freight diversions were performed as in the short term scenario, with an increase of 3 3/4 percent in the total freight carried to incorporate rail diversion effects. The diverted freight was assumed to be carried on 8 axle C-trains. This diversion further increased the proportion of 8 axle A/C-trains in the truck fleet from 5 to 12 percent.

3.4.4 Canamex Short Limits - The approach used to predict the future traffic stream under Canamex Short limits was similar in many respects to that used for Canamex limits. A summary of the diversion decisions made for the Canamex Short limits are presented in Table

Table 3.4.3-1 Summary of Vehicle Diversion Decisions, Canamex Limits

Configuration Type	Short Term Diversion		Long Term Diversion	
	% Freight Diverted[a]	Comments	% Freight Diverted[a]	Comments
SINGLE UNIT				
2SU, 3SU	0	Unaffected by new configurations	0	Unaffected by new configurations
4SU	0	Unaffected by new configurations	0	Unaffected by new configurations
TRUCK AND TRAILER				
2-2	0	Unaffected by new configurations	0	Unaffected by new configurations
3-2	0	Unaffected by new configurations	0	Unaffected by new configurations
3-4	0	Unaffected by new configurations	0	Unaffected by new configurations
4-4	0	Unaffected by new configurations	0	Unaffected by new configurations
3-5, 3-6, 2-1 3-3, 4-2, 4-3	0	Unaffected by new configurations	0	Unaffected by new configurations
TRACTOR SEMI TRAILER				
2S2	0	Unaffected by new configurations	0	Unaffected by new configurations
3S2	10	10% shift to 8 axle C-train	10	10% shift to 8 axle C-train
3S3	0	Unaffected by new configurations	0	Unaffected by new configurations
4S4	0	Unaffected by new configurations	0	Unaffected by new configurations
4S3,4S2,2S1, 2S4, 2S3,3S1	0	Unaffected by new configurations	0	Unaffected by new configurations
3 UNIT COMBINATION				
5,6 AX A T	0	No advantage over existing A- train, Assumed that they would have already switched to 7AX A-train if needed extra capacity	0	No advantage over existing A-train, Assumed that they would have already switched to 7AX A-train if needed extra capacity
7 AX A T	55	20% same configuration at heavier weight, 35% would divert to 7 AX C-train	55	xx% same configuration at heavier weight, xx% would divert to 7 AX C-train
8 AX A T	50	50% would divert to 8 AX C-train	50	50% would divert to 8 AX C-train
9 AX A T	0	Unaffected by new configurations		Unaffected by new configurations
8 AX B T		Unaffected by new configurations		Unaffected by new configurations
7,8 AX C T		Will receive freight from 7,8 AX A-train		Will receive freight from 7,8 AX A-train
5,6,7 AX B T 5,6 AX C T		No advantage over other configurations		No advantage over other configurations

[a] percentage of total freight carried by the configuration diverted to a different configuration

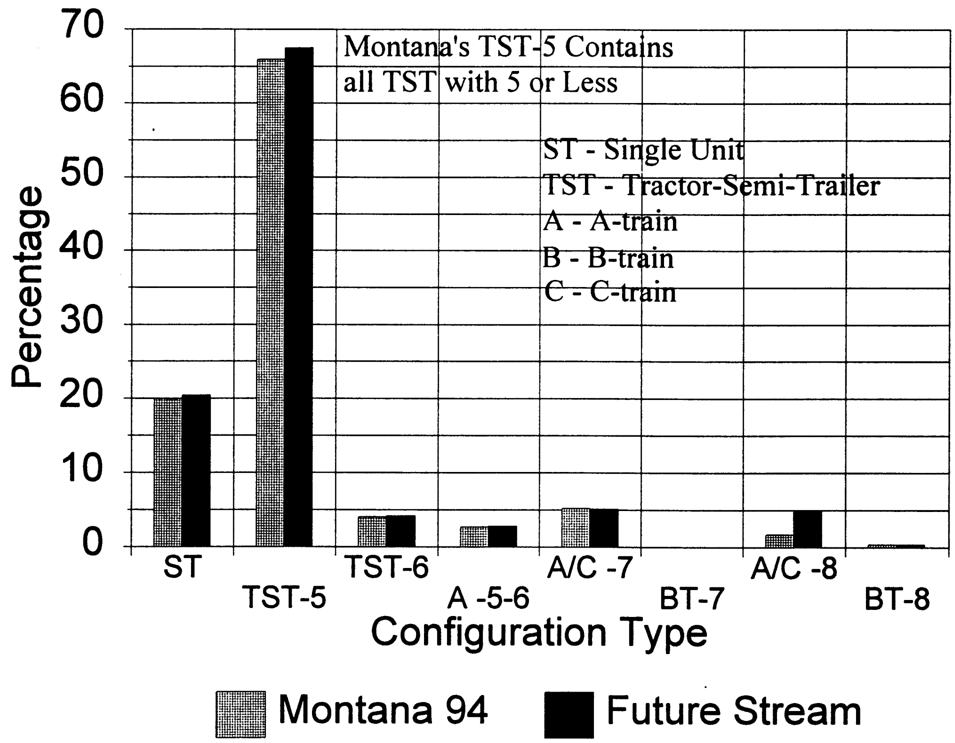


Figure 3.4.3-1 Composition of the Traffic Stream, Canamex Limits, Short Term Estimate

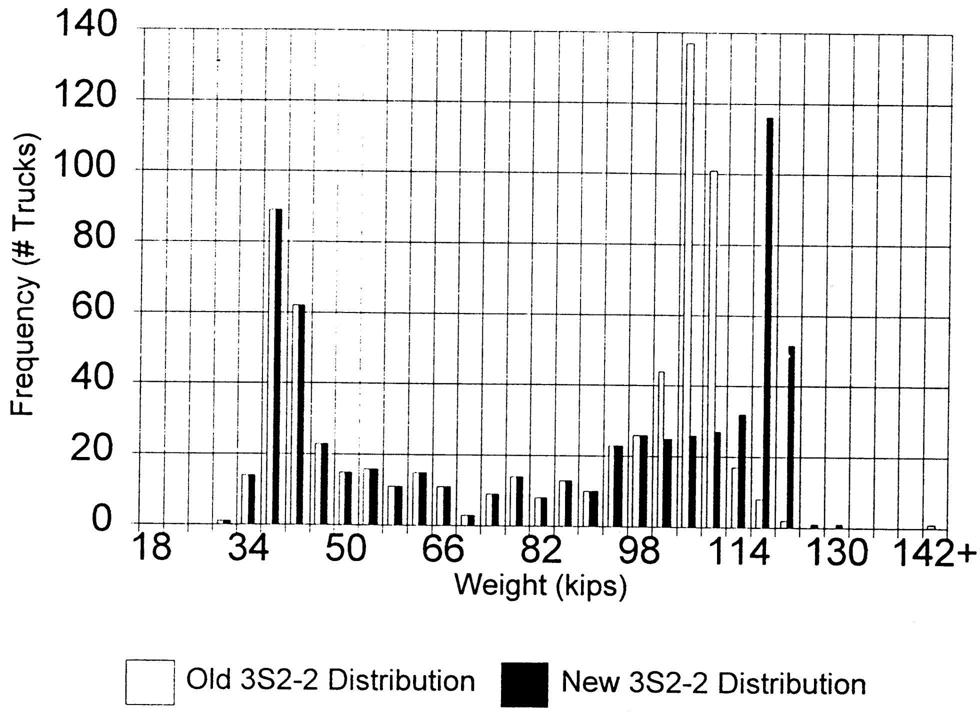


Figure 3.4.3-2 Distribution of Vehicle Weights, 3S2-2 Vehicles, Existing Limits and Canamex Limits, Short Term Estimate

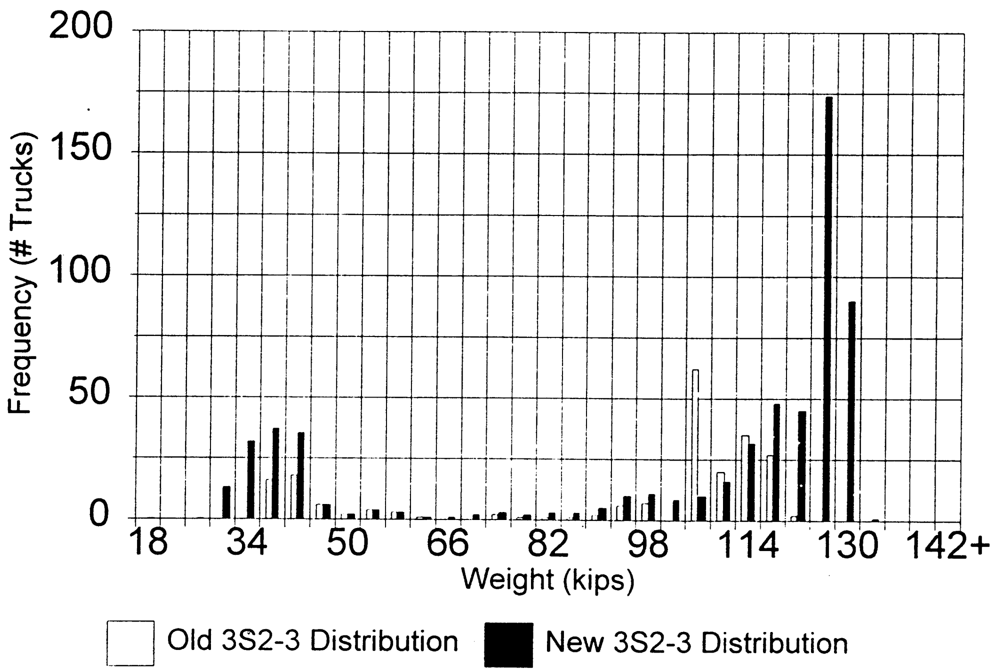


Figure 3.4.3-3 Distribution of Vehicle Weights, 3S2-3 Vehicles, Existing Limits and Canamex Limits, Short Term Estimate

3.4.4-1. The Canamex Short limits do impact more vehicle configurations than the Canamex limits. Configurations with significant increases in allowable gross weight include 3-3 and 3-4 truck and trailer units and 8 axle B-trains, in addition to the 7 and 8 axle C-trains (see Table 3.2.1-2). Under the Canamex Short limits, C-trains can operate at higher gross vehicle weights than B-trains. Based on this weight advantage, coupled with the fact that existing equipment is more compatible with C-train rather than B-train operation, the decision was made to divert freight to C-trains under the Canamex Short option.

The composition of the traffic stream predicted in the short term for the Canamex Short scenario is summarized in Figure 3.4.4-1. This composition is very similar to that for the Canamex scenario. The only major change in the fleet was again an increase in 8 axle A/C trains from being 2 to being 5 percent of the heavy vehicle traffic. Under this scenario, freight was diverted from existing 7 and 8 axle A-trains to heavier 7 and 8 axle C-trains. The weight distributions for 3S2-2 and 3S2-3 vehicles under this scenario are presented in Figures 3.4.4-2 and 3.4.4-3, respectively.

Table 3.4.4-1 Summary of Vehicle Diversion Decisions, Canamex Short Limits

Configuration Type	Short Term		Long Term	
	% Freight Diverted[a]	Comments	% Freight Diverted[a]	Comments
SINGLE UNIT				
2SU, 3SU	0	Assumed that they would have already switched to 4SU if needed extra capacity	0	Assumed that they would have already switched to 4SU if needed extra capacity
4SU	0	Canada has no 4SU to divert to	0	Canada has no 4SU to divert to
TRUCK AND TRAILER				
2-2	0	Assumed that they would have already switched to 3-2 if needed extra capacity	0	Assumed that they would have already switched to 3-2 if needed extra capacity
3-2	35	35% same configuration at higher Canadian weights	35	Would use same configuration at higher Canadian weights
3-4	40	40% same configuration at higher Canadian weights	40	Would use same configuration at higher Canadian weights
4-4	0	Canada has no 4-4 to divert to in the short term	100	Would divert to Canadian 3-4 for higher capacities and lower operating costs
3-5, 3-6, 2-1 3-3, 4-2, 4-3	mixed	Very few vehicles operating, will divert like similar configurations	mixed	Very few vehicles operating, will divert like similar configurations
TRACTOR SEMI TRAILER				
2S2	0	Assumed that they would have already switched to 3S2 if needed extra capacity	0	Assumed that they would have already switched to 3S2 if needed extra capacity
3S2	10	10% will shift to 8 AX C Train	10	10% will shift to 8 AX C Train
3S3	0	Would use same configuration, not many vehicle operating		Would use same configuration, not many vehicle operating
4S4	0	No Canadian 4S4 vehicle to divert to		No Canadian 4S4 vehicle to divert to
4S3,4S2,2S1, 2S4, 2S3,3S1	mixed	Very few vehicles operating, will divert like similar configurations	mixed	Very few vehicles operating, will divert like similar configurations
3 UNIT COMBINATION				
5,6 AX A T	0	Assumed that they would have already switched to 7AX A T if needed extra capacity	0	Assumed that they would have already switched to 7AX A T if needed extra capacity
7 AX A T	40	Would divert to 7 AX C Train	40	Would divert to 8 AX C Train
8 AX A T	45	Would divert to 8 AX C Train	45	Would divert to 8 AX C Train
9 AX A T	0	No Canadian 9 axle vehicle to divert to	35	Would divert to 8 AX C Train
8 AX B T		Current 8 AX B T will operate at Montana axle limits		Current 8 AX B T will operate at Montana axle limits
7 AX C T	-	Will receive freight from 7 AX A T	-	Will receive freight from 7, AX A T
8 AX C T	-	Will receive freight form 8 AX A T	-	Will receive freight form 8, 9AX A T
5,6,7 AX B T 5,6 AX C T		No advantage over other configurations		No advantage over other configurations

[a] percentage of total freight carried by the configuration diverted to a different configuration

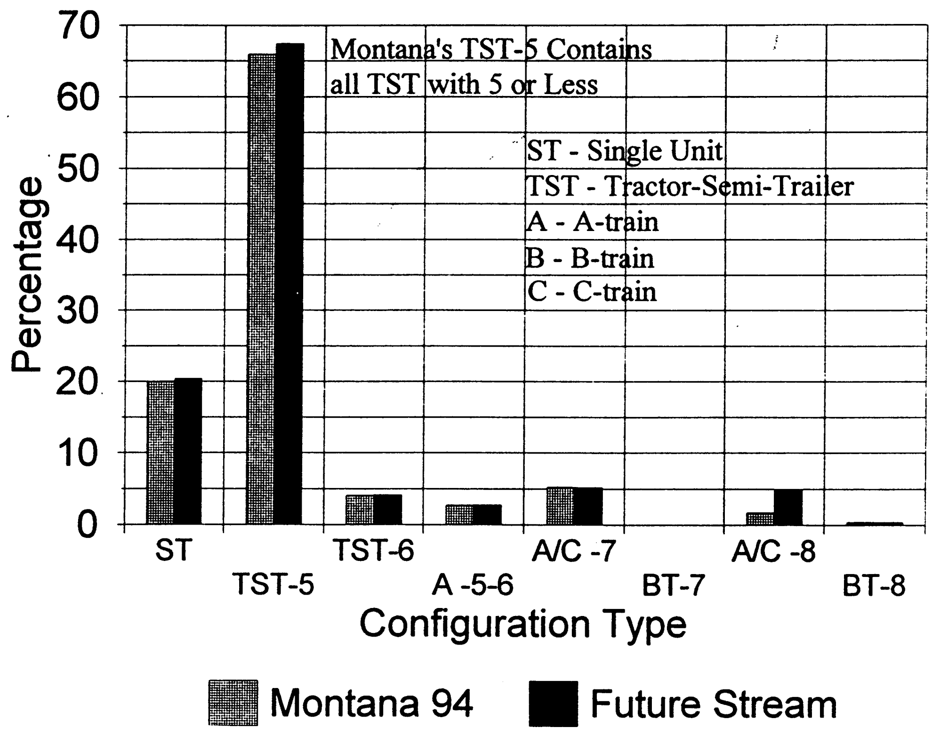


Figure 3.4.4-1 Composition of the Traffic Stream, Canamex Short, Short Term

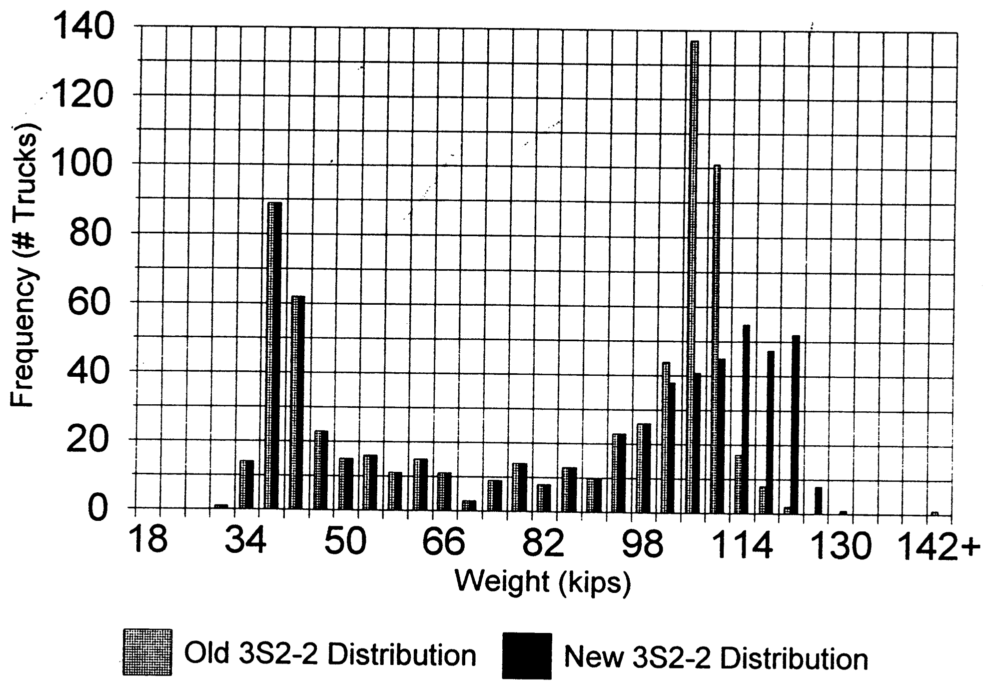


Figure 3.4.4-2 Distribution of Vehicle Weights, 3S2-2 Vehicles, Existing Limits and Canamex Short Limits, Short Term Estimate

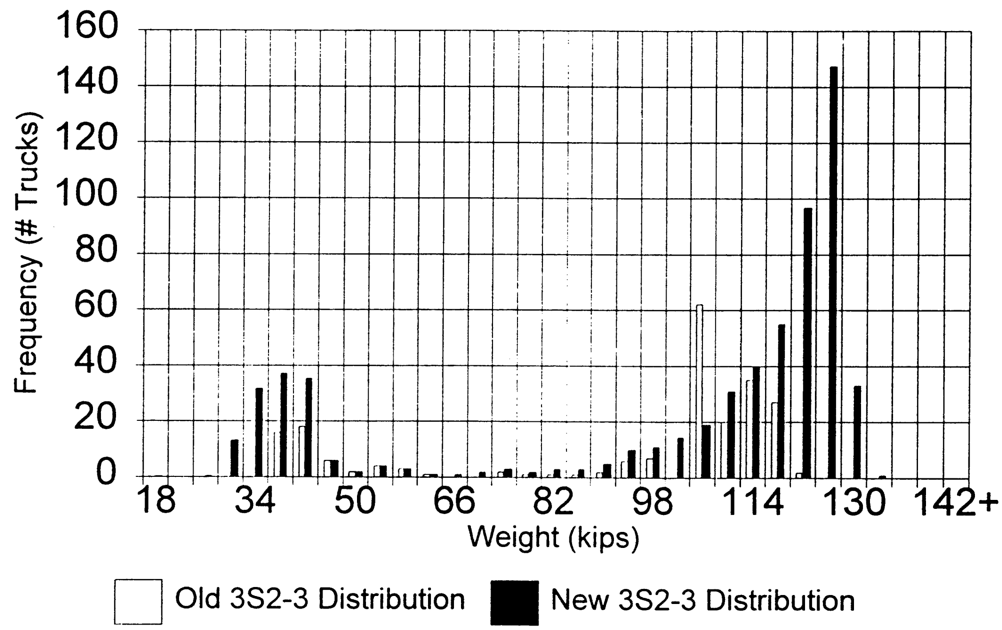


Figure 3.4.4-3 Distribution of Vehicle Weights, 3S2-3 Vehicles, Existing Limits and Canamex Short Limits, Short Term Estimate

The volume of freight shifted within the heavy combination vehicles in this scenario was less than that shifted under the Canamex scenario and a more dispersed weight distribution was assumed compared to the Canamex scenario. Volumetrically, the Canamex Short vehicles are smaller than the Canamex vehicles, and therefore it was believed that these vehicles would be less attractive to operators when evaluating possible changes in their equipment. Some freight was also shifted under this scenario to 8 axle C-trains from 3S2 vehicles, in response to the large increase in weight capacity offered by these vehicles. Freight was diverted from light to heavy 3-3 and 3-4 units, although only a nominal amount of the total freight on the system is carried by these vehicles. Shifting freight between truck and trailer units, shifting less freight on combination units, and the use of more dispersed weight distributions for 7 and 8 axle combination vehicles in the Canamex Short compared to the Canamex scenario had nominal impact on the fleet composition.

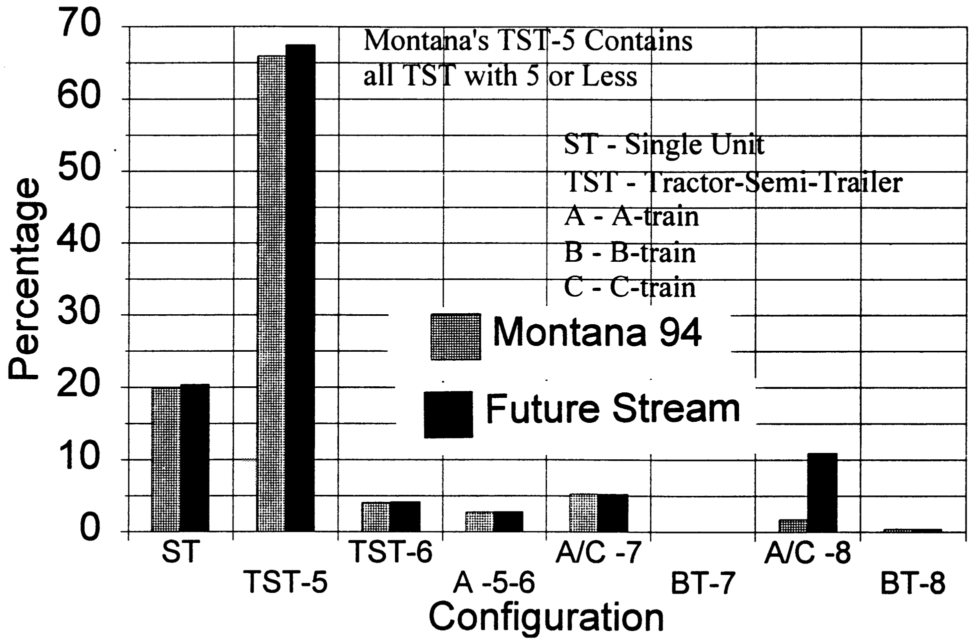


Figure 3.4.4-4 Composition of the Traffic Stream, Canamex Short, Long Term

The composition of the traffic stream predicted in the long term for the Canamex Short scenario is summarized in Figure 3.4.4-4. The same vehicle diversions were performed in the long term Canamex Short scenario as in the short term scenario, with an increase of 3 3/4 percent in the total freight carried to incorporate rail diversion effects. The diverted freight was assumed to be carried on 8 axle C-trains. As for the Canamex scenario, diversion of this freight resulted in an increase in 8 axle C trains from being 5 to being 11 percent of the truck fleet.

3.4.5 Comparison with Existing and Projected Traffic Streams in Canada - The composition of the long term Canadian Interprovincial traffic stream was compared with the vehicle fleet that has evolved and is expected to continue to evolve in Canada since the adoption of the Canadian Interprovincial Limits. Notably, the Prairie Provinces (Alberta, Saskatchewan, and Manitoba) had truck weight limits similar to those in Montana before the adoption of Canadian Interprovincial Limits. In reviewing the comparisons presented below, it is important to recognize that the composition of the Canadian traffic streams (both before and after the introduction of Canadian Interprovincial limits) are influenced by existing Montana and other U.S. limits on vehicle size in weight, in that many of these Canadian vehicles are used in cross border freight movements.

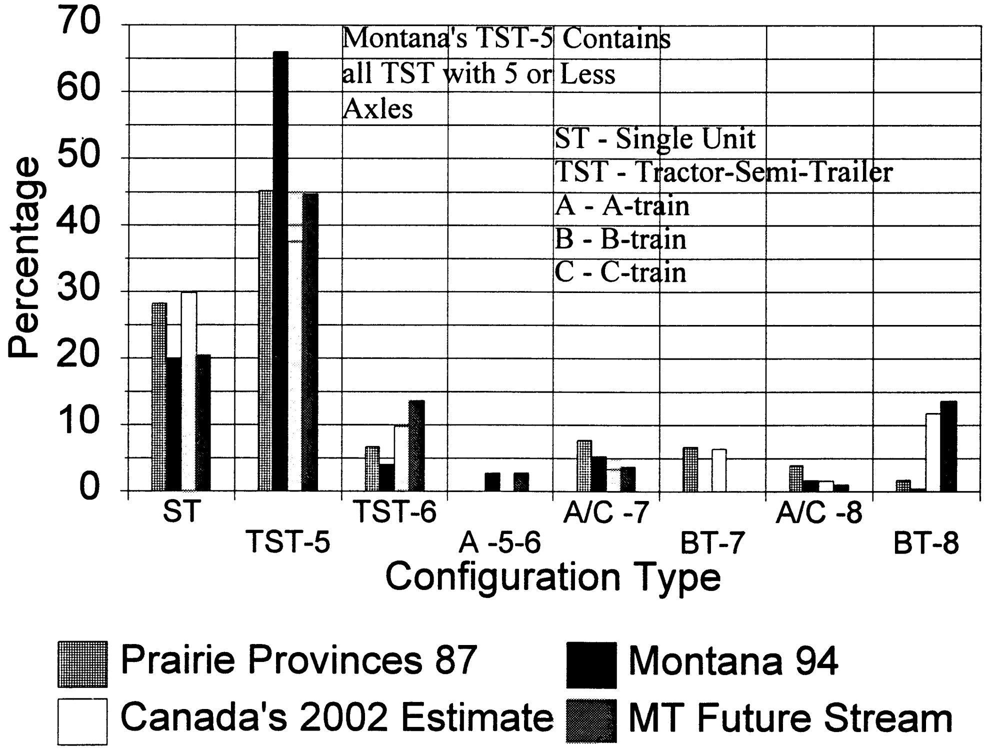


Figure 3.4.5-1 Comparison of Vehicle Fleet, Long Term Canadian Interprovincial Limits versus Present and Predicted Fleet Composition in the Prairie Provinces (Prairie Provinces projection based on data from TAC/CTRI, 1994)

The composition of the heavy vehicle fleet in the Prairie Provinces in 1987 before the adoption of Canadian Interprovincial Limits, and the projected composition of their fleet in 2002 (determined based on trends already observed in the fleet since the adoption of Canadian Interprovincial Limits (TAC/CTRI, 1994)) are presented in Figure 3.4.5-1. Shown on the same Figure is the composition of the present traffic stream in Montana and the projected composition of the traffic stream over the long term if Canadian Interprovincial Limits are adopted. The changes predicted in the Montana fleet generally mimic all the changes predicted for the Prairie Province fleet. One major difference in the two traffic compositions is in the percentages of 3S2 and 3S3 vehicles that operate. With regard to pre-adoption conditions, Montana's vehicle fleet includes almost 50 percent more 3S2 vehicles than the Prairie province fleet and approximately 100 percent fewer 3S3 configurations. This difference arises in part from differences in the underlying pre-adoption conditions in the two jurisdictions. The Prairie Provinces allowed higher tridem loads than Montana prior to the adoption of Canadian Interprovincial Limits.

Thus, in the Prairie Provinces, 3S3 vehicles already enjoyed greater popularity than they presently do in Montana.

Additional differences in the composition of the heavy vehicle traffic predicted in Montana and the Prairie Provinces results from social and economic differences in the two regions; therefore, the composition of the predicted traffic stream was also compared with the expected traffic stream in British Columbia. Some of the transportation conditions in British Columbia may be representative of those in Montana. In this case, the size and weight limits in the two regions (Montana and British Columbia) differed considerably before the implementation of Canadian Interprovincial Limits, and therefore were not considered in the comparison. The projected composition of the two traffic streams in British Columbia and Montana are compared in Figure 3.4.5-2. The two streams are very similar in composition. The difference in the volume of B-trains may be attributable to the rail diversion considered for Montana.

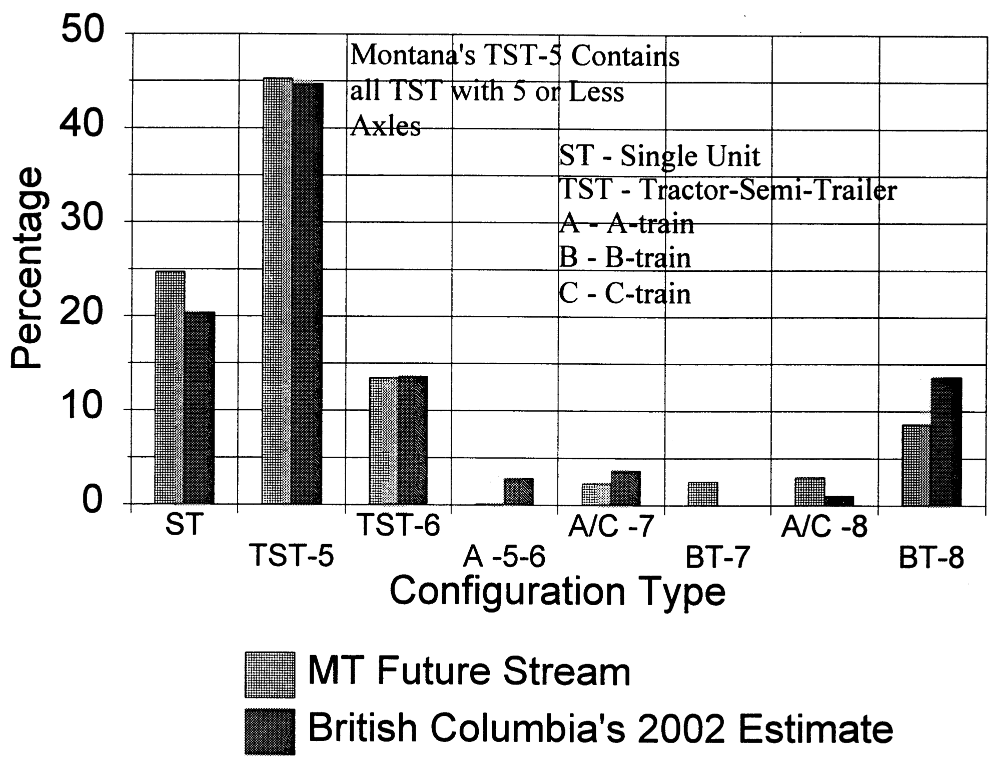


Figure 3.4.5-2 Comparison of Vehicle Fleets, Long Term Canadian Interprovincial versus Predicted Fleet Composition in British Columbia (British Columbia projection based on data from TAC/CTRI, 1994)

3.4.6 Changes in Traffic Volume - The volume of truck traffic changed as the composition of the vehicle fleet was altered and freight was carried on different vehicle configurations. The volume of truck traffic decreased under all scenarios considered by up to 3 percent, as freight was generally shifted and/or added to higher payload capacity vehicles. Nominal reductions in total truck traffic were noted even in scenarios with diversion of freight from rail to truck.

4. PHYSICAL EFFECTS ON BRIDGES

4.1 GENERAL REMARKS

Assessment of the damage expected in bridges under increased truck loads of the magnitude considered herein is a complex problem. While the loads under study exceed the legal loads in the state of Montana, they are of such a magnitude that all of the bridges on the interstate system and the majority of the bridges on the primary system will not be in imminent danger of collapse or even sustain serious damage from the occasional passage of such loads. This situation is not unexpected, in that bridges historically have been designed with a high level of conservativism with respect to strength. This level of conservatism assures a level of safety acceptable to the public. The level of safety offered by these structures may still be acceptable under the increased loads from Canadian Interprovinicial, Canamex, and Canamex short vehicles, due to the level of conservativism in the original designs.

While adequate strength and safety are the most important aspects of bridge performance, other features of bridge behavior are also important, notably serviceability and durability. Serviceability is typically evaluated in terms of the expected deflection and vibration response of a bridge. Excessive deflections and vibrations can be deemed objectionable by the motorist. Deflections will increase under the loads studied herein. With regard to durability, loads of the magnitude considered herein are not expected to cause noticeable problems in most bridges on the primary and interstate system within a few vehicle passages. Over the lifetime of a bridge, however, consisting of thousands and even millions of vehicle passages, accelerated deterioration may be become evident. The conservativism in bridge design with respect to strength may actually be responsible for the long life enjoyed by many bridges from a durability perspective.

This study considered the effects of Canadian Interprovincial, Canamex, and Canamex Short vehicles on each of the responses enumerated above (strength/safety, serviceability, and durability). Strength/safety issues were primarily evaluated by analysis, with a modest experimental effort to validate expected load paths and level of strains in typical bridges. The analyses performed generally consisted of comparing vehicle demands and bridge capacity under various conditions. By using recognized load rating methodologies and philosophies in this process, the calculated capacities for the Canadian, Canamex and Canamex Short vehicles could be related to established levels of safety, serviceability, and durability. While most of the analyses performed were done at a system-wide level using simplified procedures, a few more detailed load ratings were performed for selected bridges to obtain an indication of the relative magnitude of the load ratings generated by the various procedures available. New load rating procedures have been introduced that attempt to provide a more uniform level of safety across a broad range of conditions than was the case using older rating schemes. Limited calculations were also performed to identify possible fatigue problems, notably in steel bridges.

Serviceability issues were not analyzed in detail in this study. The magnitude of the increase in live load bridge deflections under Canadian Interprovincial and Canamex vehicles was estimated. These calculations were performed for both simple span and continuous structures. Durability issues were addressed both through the use of established load rating techniques in estimating bridge capacity and by testing some bridges under Canadian vehicle loads. Six bridges in the state were tested to determine general behavior and absolute magnitudes of stresses and strains that can be expected under Canadian Interprovincial and Canamex loads. The results of these tests were used in estimating any accelerated deterioration that might occur in response to the increased loads.

Observations were also collected, as available, on Canada's experience with their bridge system before and after the adoption of the Canadian Interprovincial limits on truck size and weight. Notably, the Province of Alberta had similar size and weight limits prior to the adoption of the Canadian Interprovincial limits as are currently used in Montana. Drawing meaningful conclusions from Alberta's experience, however, is complicated by the fact that since the middle of the 1970s they have used a higher vehicle design load than that of the United States.

Other studies have been done on the impact of adopting new vehicle size and weight limits on highway bridges, both at the state and federal level. These studies were reviewed with respect to both the methodologies employed and the results obtained.

4.2 DEMANDS OF NEW VEHICLE CONFIGURATIONS

4.2.1 General Remarks - Aspects of the vehicle configurations being considered herein that will have the greatest impact on bridge demands are a) the increased loads allowed on individual axle groups (Canadian Interprovincial Limits, only), b) the increased gross vehicle weights for large vehicles, and c) the shortened wheelbases allowed for semi-trailer and combination vehicles (Canadian Interprovincial and Canamex Short limits). Note that individual tire and axle loads are unchanged under all of the scenarios being considered. The expected effects of these loads was analytically traced through each element of a typical bridge system starting at the point of application of the load and proceeding into the ground. Stringer type bridges were selected for this purpose, as such bridges comprise 95 percent of the bridge inventory (by length). Load effects were traced from the deck, to the stringers, to the pier caps, to the footings and finally into the ground.

Demands on bridges are generally classified as either dead load or live load related, depending upon their source. Dead load demands are related to carrying the self-weight of the structure. Live load demands are related to, and caused by, use of the structure by vehicles. The dead load fraction of the demand is constant in this study; only the live load demand is being increased. In general, the relative increase in the total demand (dead load plus live load) on the structure will (a) be influenced by the ratio of dead load to live load and (b) be less than the relative increase in the live load demand alone. Fatigue is a notable exception to these observations, in that fatigue is related primarily to the live load.

4.2.2 Decks - Decks can play different roles in the structural system of a bridge depending on the nature of the design. In non-composite systems, decks simply transfer wheel loads into the stringers. In composite systems, decks transfer loads into the stringers and also act globally with the stringers in carrying loads longitudinally into the supports. Deck behaviors of interest include immediate failure under a single load event, long term failure under multiple load events, and accelerated deterioration. Decks tend to be over designed with respect to strength (Minor, White, and Busch, 1988) . Thus, while demands may increase under the scenarios postulated herein, these increases in demand may still be within the safe capacity of the deck.

With regard to transferring loads laterally into the stringers (see Figure 4.2.2-1), demands under Canadian Interprovincial, Canamex, and Canamex Short limits are expected to be similar to those placed on decks under current weight limits. In all three cases, the new limits restrict single axle loads to the same magnitudes currently allowed in Montana. Under this situation, deck demands from Canamex and Canamex Short vehicles will be equal to or lower than those from vehicles in the existing traffic stream. The situation on the relative magnitude of the demands that Canadian Interprovincial vehicles place on decks in transferring loads transversely into the stringers is less certain. The Canadian Interprovincial limits allow higher axle group loads than are presently allowed in Montana. As these loads are applied over the same outside-to-outside axle spacings as in Montana, higher localized demands are generated in the deck under the Canadian Interprovincial limits compared to existing limits. These higher demands occur because adjacent wheel loads in the axle group are applied close enough together to place overlapping demands on certain areas of the deck, as shown in Figure 4.2.2-1.

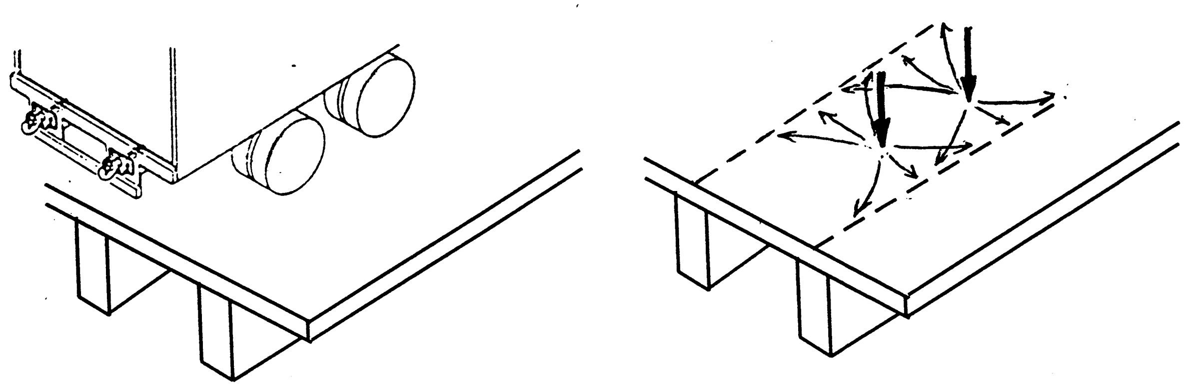


Figure 4.2.2-1 Lateral Load Transfer of a Typical Stringer/Deck System

A finite element analysis of a typical deck and stringer system indicated that demands in the transverse direction were increased by up to 17 percent under Canadian tandems and tridems compared to demands under Montana tandems and tridems. This increase in demand was observed in both positive moment at the centerline between the stringers and in negative moment over the stringers. Decks are expected to readily accommodate this increase in demand from a strength/safety perspective, due to the conservativism in their design. These higher demands and their repetitive nature could, however, result in accelerated deterioration.

Longitudinal compression stresses generated in decks under composite action will increase in magnitude under all three size and weight scenarios considered herein. The magnitude of this increase should be proportional to the increase in bending moment demand on the stinger system, as discussed below.

4.2.3 Stringers/Longitudinal Load Carrying System - Using even simple structural analyses, it is obvious that Canadian Interprovincial, Canamex, and Canamex Short vehicles will increase demands on stringer systems. The increases in axle group loads and overall gross vehicle weights on shorter wheel base vehicles will result in increased bending moment and shear force demands, higher fatigue stress ranges, and higher deflections in the stringers.

In simple span structures, maximum live load bending moment, shear force, and deflection are all a function of span length for a given vehicle. Relationships were developed between these aspects of response and span length for the Canadian, Canamex, and Canamex Short scenarios. These relationships are summarized in Figures 4.2.3-1 to 4.2.3-3. The specific vehicles used in calculating these demands (axle spacings and weights) are given in Appendix B. Referring to these Figures, the demands for each scenario have been normalized by the maximum HS20-44 design demand for the same structure. Thus, values greater than 1.0 indicate an increase in demand compared to the HS20-44 design vehicle; values less than 1.0, a decrease in demand compared to the HS20-44 vehicle.

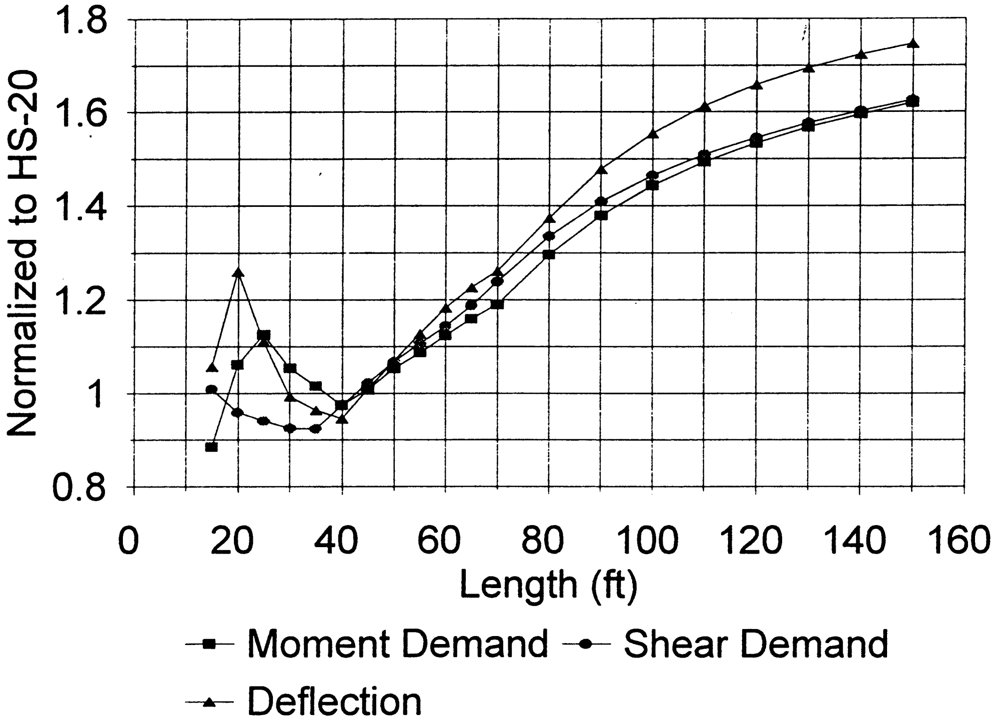


Figure 4.2.3-1 Live Load Bending Moment, Shear, and Deflection Demands, Simple Spans, Envelope for Canadian Interprovincial Limits

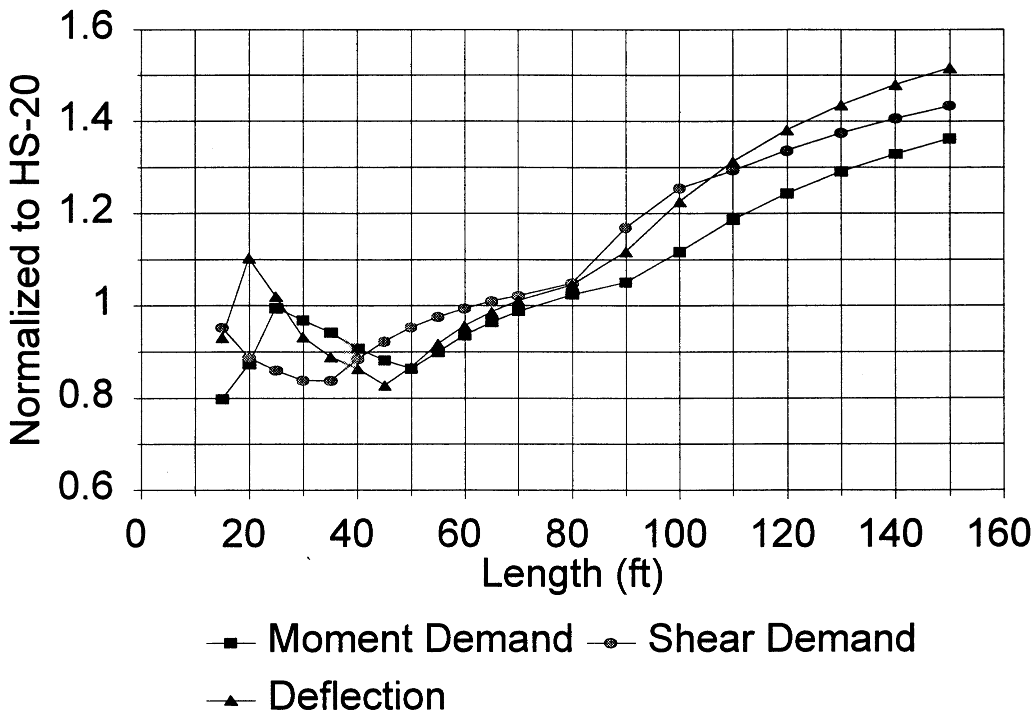


Figure 4.2.3-2 Live Load Bending Moment, Shear, and Deflection Demands, Simple Spans, Envelope for Canamex Limits

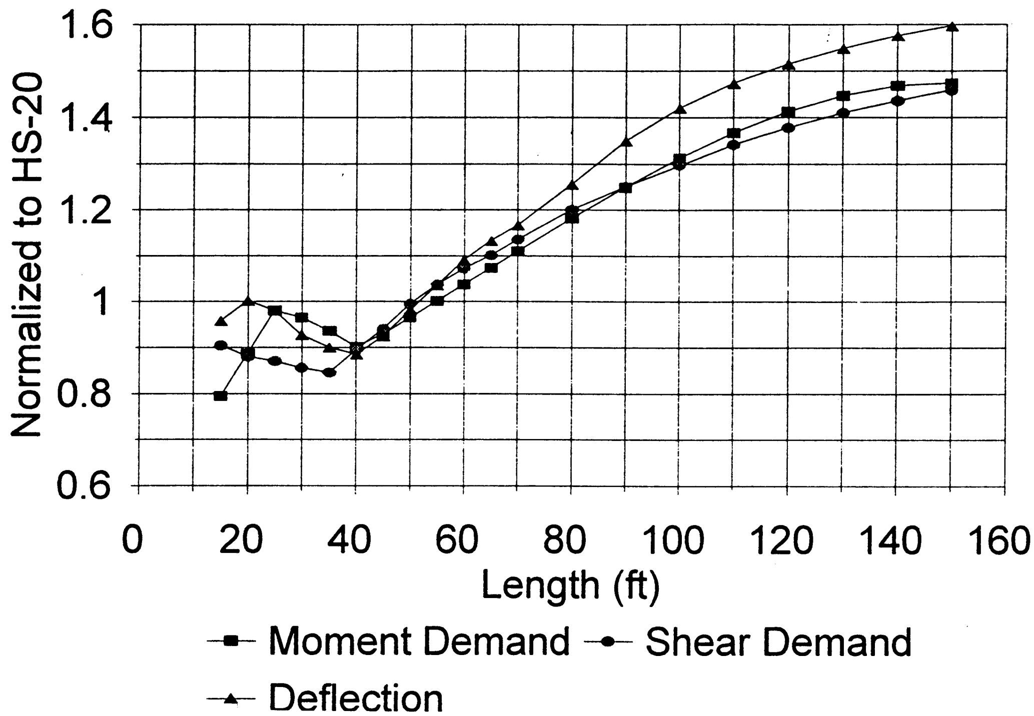
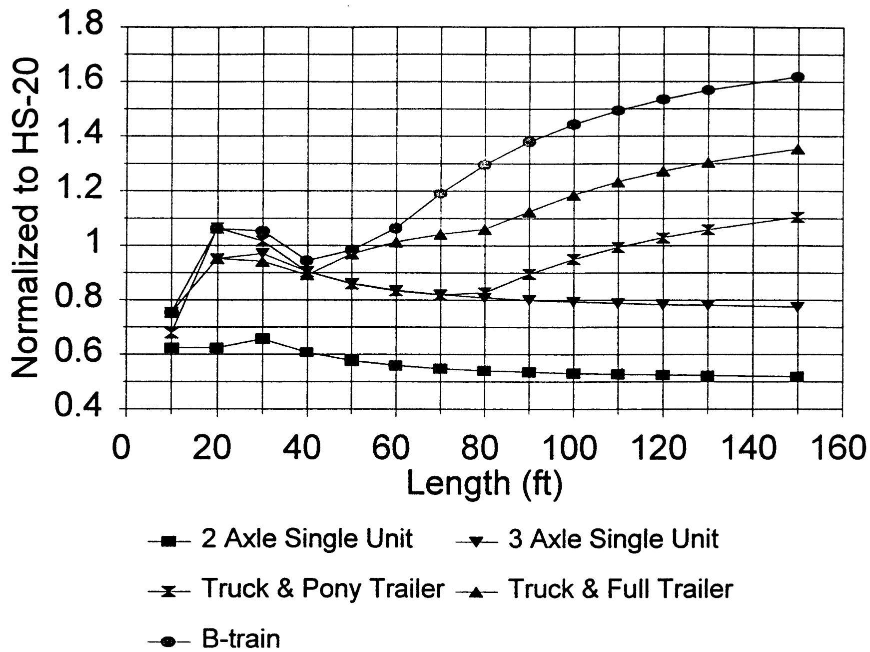


Figure 4.2.3-3 Live Load Bending Moment, Shear, and Deflection Demands, Simple Spans, Envelope for Canamex Short Limits

The relationships presented in Figures 4.2.3-1 to 4.2.3-3 were developed by successively solving for the maximum demand (bending moment, shear force, deflection) in a simply supported span as the span length was stepwise increased at 5 foot increments from 15 to 150 feet. Calculations were performed using PCBridge (Murphy, 1992), a structural analysis software package that calculates maximum demands in structures under moving loads. Calculations were performed for several vehicles within each scenario and for the HS20-44 design vehicle. Maximum bending moment results obtained for the various Canadian configurations, for example, are presented in Figure 4.2.3-4. Envelopes of the absolute highest demands for each size and weight scenario were generated by selecting the highest response at each span length generated by any vehicle within the scenario.

Referring to Figures 4.2.3-1 to 4.2.3-3, live load demands generated by the various new vehicles range from 80 to 162 percent of the design demands of the HS20 vehicles across simple span lengths of 15 to 150 feet. The demands generated by the new vehicles typically are less



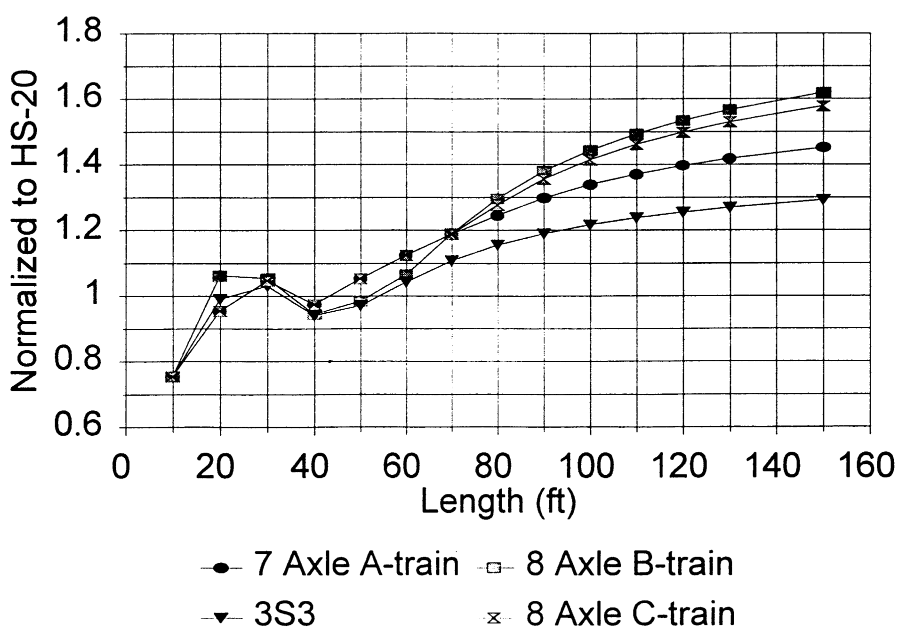


Figure 4.2.3-4 Live Load Bending Moment Demand, Simple Spans, Canadian Interprovincial Vehicles

than the HS20 demands on short span bridges. Demands in short span bridges are sensitive to single axle loads, as only one axle is close to the center of the bridge at a time. The HS20-44 design vehicle has a significantly higher single axle load (32 kips) than the load allowed on a single axle in any of size and weight scenarios under consideration (common maximum of 20 kips on a single axle). As the span length increases, maximum moment demand becomes more sensitive to the total load on each axle group, as all axles in a group can be relatively close to center of the span. Therefore, at span lengths from approximately 20 to 40 feet, the Canadian Interprovincial vehicles generate higher moment demands in the structure than the HS20-44 design load in response to the 53 kip tridem and in this system relative to the 32 k axle load of the HS20. Finally, at longer bridge lengths (starting at 40 to 60 feet), several axles are on the bridge simultaneously, generally resulting in a steady increase in moment demands under each scenario due to the higher maximum gross vehicle weights of the new vehicles (from 126 to 138 kips) compared to the HS20 design vehicle (gross weight of 72k).

The Canamex and Canamex Short demands are always less than the Canadian Interprovincial demands, as was expected. These vehicles have both lower allowable axle group loads and lower allowable gross vehicle weights than the Canadian Interprovincial vehicles. At longer span lengths, demands from the Canamex Short vehicles exceed those of the Canamex vehicles. The Canamex Short vehicles can operate at close to the same maximum load as the Canamex vehicles, but their wheelbase is considerably shorter than the Canamex vehicles. This short wheelbase results in increased moment demands on longer span bridges compared to Canamex vehicles.

Referring to Figure 4.2.3-1 the increases in relative moment and shear demands for the Canadian Interprovincial vehicles are similar in magnitude across the span lengths of interest. These demands begin to exceed HS20-44 live load demands at span lengths greater than around 45 feet. Relative increases in moment and shear demands are also similar in magnitude for the Canamex Short vehicle, and these demands begin to exceed HS20-44 demands at span lengths between 50 and 60 feet (see Figure 4.2.3-3). Demands of the Canamex vehicles begin to exceed HS20-44 demands at span lengths of 60 to 70 feet. As shown in Figure 4.2.3-2, the increase in shear demand for the Canamex vehicles (relative to HS20-44 demands) significantly exceeds the increase in the moment demand for these vehicles at span lengths greater than 40 feet (contrary to the similarity in the increases in shear and moment demands for Canadian Interprovincial and Canamex Short vehicles). While the longer wheelbase on the Canamex vehicles compared to the Canamex Short vehicles effectively reduces the maximum moment demands, the maximum shear demands are less affected.

Dead load demands on the bridges are constant under all the vehicle scenarios being considered. The dead load contribution to the total demand was calculated using an empirically derived equation that relates these contributions to the design live load demand and span length. A relationship of this type was originally proposed by Hansel and Viest (1971) for steel span structures,

$$D = 0.0132 \ L \ (1+I) \ X$$

where,

D= Dead load demand

L = Live load demand

I = Impact factor

X = Span length

Using this equation, the dead load demand steadily increases as the span length increases. This relationship was used in this study for steel, reinforced concrete, and wood bridges. The validity of the equation for these various applications was checked using actual live and dead load demands calculated for typical bridges in the Montana inventory. As might be expected, this equation underestimates dead load for typical reinforced concrete beam structures (Wilkes, 1989). This underestimation of dead load effects, however, was ultimately found to exaggerate live load effects following the analysis procedure used in this study. A second order equation was specifically developed for the dead load moments in prestressed concrete bridges in Montana based on the standard prestress bridge designs that Montana has followed for many years,

$$D = \ (5.64 \ (10)^{-5} X^2 \ + 4.63 \ (10)^{-3} X + 0.338) \ (L)$$

For wood structures on the primary and secondary systems, an allowance was made in the dead load demand calculation for the presence of asphalt overlays on the bridge decks. The magnitude of the allowance was related to the deviation in the reported Inventory rating from a basic inventory rating of 1.0 for an H15 vehicle. Most of the timber spans on the state highway system conform to a few standard configurations with respect to span length, stringer size, and stringer spacing. These standard configurations appear to have been designed for an H15 vehicle. As all these bridges are reportedly in satisfactory structural condition, any loss of capacity indicated by the inventory rating was attributed to increased dead load demands from asphalt overlays. Dead load shear and moment demand were estimated using the same expressions. This simple approach to the treatment of the two types of demand was expected to yield adequate results based on a review of the dead load demand analyses performed by Noel and his colleagues (1985).

Presented in Figure 4.2.3-5 and 4.2.3-6 are the total moment and shear demands (live load plus dead load) produced in typical simple span structures under the Canadian, Canamex, and Canamex Short loads. These results are again normalized by the HS20-44 design demand. The ratios of new vehicle demands to HS20-44 demand at longer span lengths are less than might be expected based on live load comparisons alone, due to the steady increase in dead load as span length increases. The total moment demand of Canadian Interprovincial and Canamex Short vehicles level out at approximately 122 and 116 percent of the HS20 vehicle, respectively. For Canamex vehicles, demand climbs steadily to 113 percent of the HS20 demand at a span length of 150 feet. While total moment demands for Canadian Interprovincial vehicles exceed HS20 total demands for almost all span lengths, total moment demands from Canamex and Canamex Short vehicles are less than HS20 demands out to span lengths of approximately 55 and 75 feet, respectively. Similar observations can be made for total shear demand.

Maximum demands on continuous structures are more difficult to calculate than for simply supported structures. An infinite number of unique continuous structures exist based on the number of spans and relative span lengths. Simple relationships between these parameters and vehicle demands can not be developed in the generalized sense. Therefore, every continuous span was analyzed individually to determine the maximum bending moment, shear force, and deflection generated under Canadian, Canamex, and HS20-44 design vehicle loads. Once again, these demands were normalized to the demands generated by the HS20-44 design vehicle. Both positive and negative moment demands were considered, and the highest ratio of new vehicle

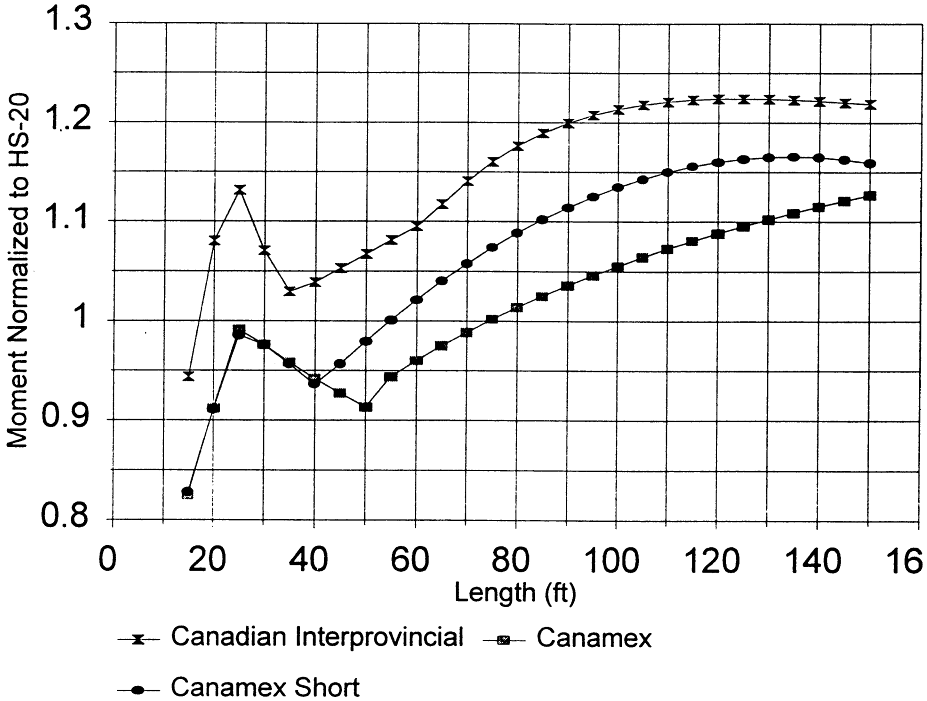


Figure 4.2.3-5 Total Bending Moment Demand, Simple Spans

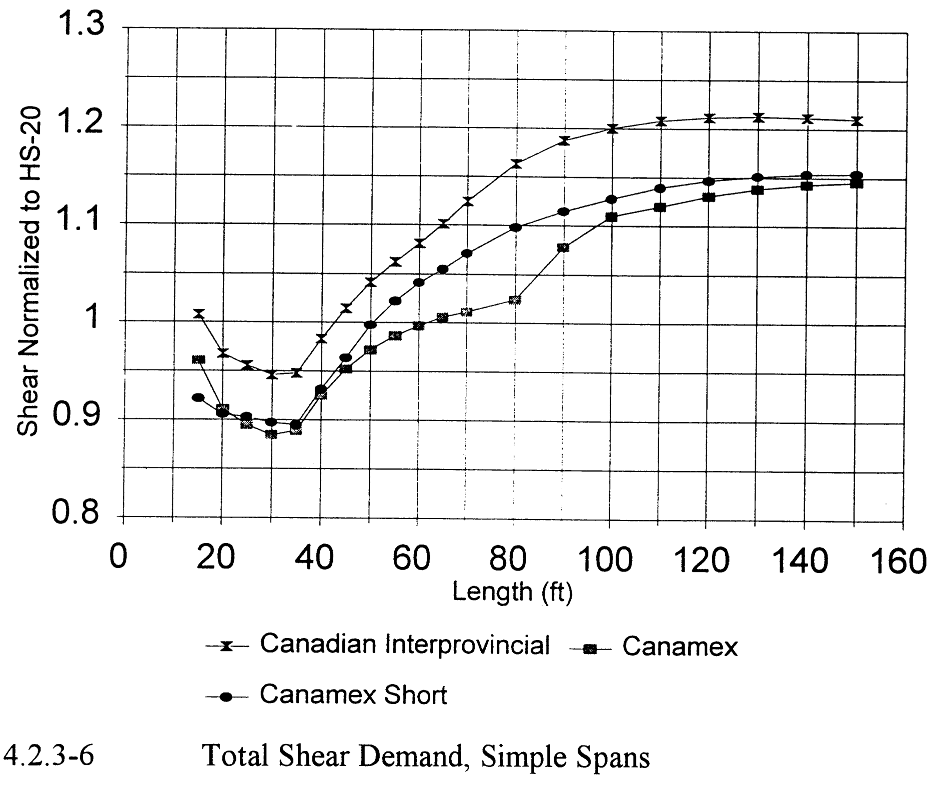


Figure 4.2.3-6 Total Shear Demand, Simple Spans

versus HS20-44 design vehicle demand was selected as critical. Typically, negative moment in the area of the supports was the controlling increase in demand. A frequency plot of the increase in negative total moment demand on continuous steel bridges for the interstate system under Canamex vehicles is presented in Figure 4.2.3-7. For a majority of these spans, the increase in bending moment demand was 20 percent or less.

The relative increases in maximum moment demands under Canadian Interprovincial and Canamex loads were expected to be nominally the same or more severe than the relative increases in maximum shear demands in continuous structures, as was the case for simply the supported structures. Moment demand is plotted as a function of shear demand for a sampling of continuous structures in Figure 4.2.3-8 (both types of demands are normalized by the HS20-44 demand). Referring to Figure 4.2.3-8, in only three of the thirty cases considered, was the increase in shear demand relative to HS20-44 demand more critical than increase in moment demand relative to HS20-44 demand. Limited analysis of the increases in moment versus shear demands for continuous structures loaded with Canamex vehicles consistently indicated that the increase in negative moment demand was more critical than the increase in shear demand.

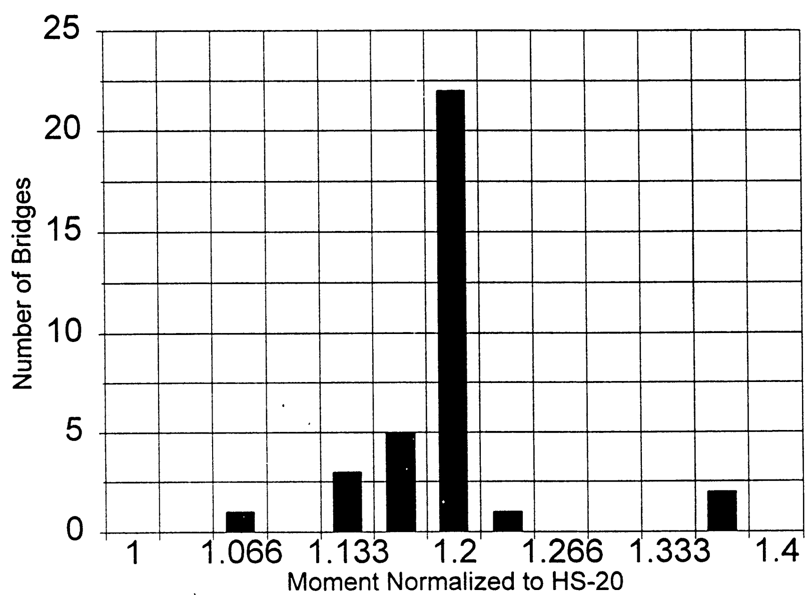


Figure 4.2.3-7 Frequency Distribution, Increase in Total Moment Demand, Continuous Steel Structures, Interstate System, Canamex Scenario

Based on the above observations of the live load and total load moment and shear demands under each scenario, the decision was made to focus these analyses on moment demand. With regard to the strength behavior of the stringers, bending moment demands were assumed to be critical in eventual comparisons of capacity versus demand for the Canadian Interprovincial and Canamex Short scenarios. Shear demand was assumed to be critical for simple span structures under the Canamex scenario; moment demand, for continuous structures. Note that moment capacity has been found to control overall bridge capacity in almost all of the analyses done by MDT for overweight vehicle permits (Murphy, 1996).

Fatigue demands on the stringer systems from individual vehicle passages will be greater under the various size and weight limits considered herein compared to present Montana load limits. Fatigue damage is related to the magnitude of the cyclic stresses experienced by a structure. In this case, the cyclic stresses are generated during the passage of each vehicle. The magnitude of these live load related stress excursions will increase in direct proportion to the

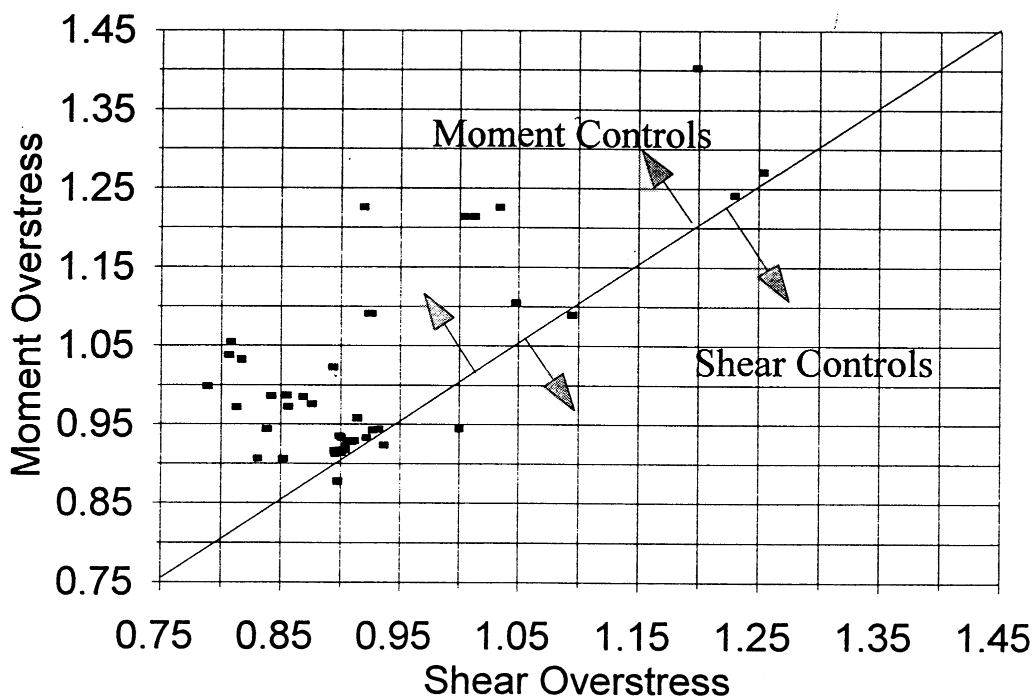


Figure 4.2.3-8 Increase in Moment Demand as a Function of Increase in Shear Demand, Sampling of Continuous Structures

increases in the live load flexural demands. This effect will be offset to some extent by the fact that fewer load excursions will be applied using the new vehicles compared to the present vehicles (fewer, larger vehicles carrying the same total freight), although diversion of freight from rail to truck counteracts this effect. Materials demonstrate different levels of sensitivity to fatigue type damage. Steel bridges are subject to fatigue damage, and, presuming they are stressed at a level higher than the fatigue limit, damage can be assumed to accumulate as a function of the third power of the tensile stress range (Schilling and Klippstein, et.al., 1978; Saklas, et.al., 1988). Thus, the fatigue demand on a bridge is proportional to (a) the number of vehicle passages over the bridge (determines number of fatigue cycles) and (b) the stress produced in the bridge by each passage (magnitude of the cycle) raised to the third power.

While the total fatigue demand over the expected life of a bridge can be determined by calculating and accumulating the damage expected from each passage of the various vehicles that use it, simplified approaches are available for evaluating this demand. One measure of demand is simply the maximum expected live load stress range and a crude approximation of the total number of load cycles to be experienced based on total traffic and percent trucks. This approach is embodied in the AASHTO Specifications (AASHTO, 1990). Notably, AASHTO provides a measure of the fatigue limit in a variety of situations by giving the allowable maximum stress range if the member is to carry over 2,000,000 million cycles of load. Stress ranges generated by loaded Canadian Interprovincial, Canamex, and Canamex Short vehicles are expected to often exceed the fatigue limits implied in the AASHTO specification. Thus, the new vehicles are expected to contribute to fatigue damage.

One approach to quantifying the relative fatigue demand of a mixed traffic stream is the equivalent fatigue truck. Moses and his colleagues (1987) suggested a basic truck configuration to be used in evaluating fatigue considerations. This suggestion has been included in an AASHTO guide on the fatigue evaluation of bridges (AASHTO, 1990). The weight of this truck, determined in its simplest form from the characteristics of the traffic stream, is indicative of the relative fatigue demand of the vehicles in a specific traffic stream upon bridges. This equivalent fatigue truck concept was simply implemented in this study to obtain an approximation of the relative fatigue damageability of the various scenarios under consideration. An equivalent fatigue truck weight was calculated for each traffic stream as,

$$W_q = \left(\frac{\Sigma\,(f_i W_i^{\,3})}{\Sigma\,(f_i)} \right)^{1/3}$$

where,

W_q = weight of equivalent fatigue truck for traffic scenario q

f_i = frequency of vehicles in category/vehicle classification I

W_i = average operating weight of vehicles in category/vehicle classification I

This expression was evaluated at randomly selected 10 mile intervals along all the interstate routes and along selected primary routes around the state using all vehicles in the traffic stream "larger" than 2 axle single units. Without exception, the equivalent fatigue vehicles for all the proposed scenarios were heavier than those for the existing traffic streams.

The relative fatigue demands for each future size and weight scenario were estimated from the equivalent fatigue vehicles determined above and the projected number of vehicles in the new traffic streams. Presuming elastic behavior of the structure at working load levels, the maximum live load stress range expected in a particular bridge under the fatigue vehicle is directly proportional to weight of the vehicle. Thus, the fatigue damage per cycle is proportional to the cube of the stress range, which is in turn proportional to the cube of the weight of the fatigue vehicle. Therefore, the relative fatigue demand associated with various traffic streams can be calculated as the product of the ratio of demand per cycle times the ratio of cycles of demand for the new stream versus the old stream,

$$\delta_{Fq} = \frac{W_q^{\,3}}{W^3} \; \frac{N_q}{N}$$

where,

δ_{Fq} = relative fatigue demand for traffic scenario q

W_q = weight of equivalent fatigue vehicle for traffic scenario q

W = weight of equivalent fatigue vehicle for existing traffic

N_q = number of vehicle passages under traffic scenario q

N = number of vehicle passages under existing traffic

These relative fatigue demands are reported in Table 4.2.3-1 for each of the six traffic streams under consideration. Fatigue demands increased for every alternate traffic stream considered. The greatest increase in fatigue demand of 34 percent was observed for Canadian Interprovincial vehicles (long term) operating on the interstate system. Fatigue demand (long term) increased by only 11 and 13 percent on the interstate system under Canamex and Canamex Short vehicles, respectively. Fatigue demand increases on the primary system were similar (although consistently and nominally lower in magnitude) to those on the interstate system. Fatigue demands were less severe for the Canamex and Canamex Short vehicles than for the Canadian Interprovincial vehicles, a direct reflection of the maximum gross vehicle weights and bending moments expected for the two types of vehicles.

Prestress concrete appears to be relatively insensitive to fatigue effects. Apparently, no fatigue failures have been experienced in prestress beams while in service (Wilkes, 1989). Beams have failed in fatigue under laboratory conditions, generally in tests in which the beams have been precracked by overloads and/or cycled at very high loads (Hanson, Hulsbos, and Van Horn, 1970; Kreger, Bachman, and Breen, 1989). Hanson and his colleagues (1970) concluded from their test results that if the stress range on the tension side of the beam remains below 6 f'c, the beam will have adequate fatigue life. Kreger and his colleagues (1989) failed a beam in shear at 900,000 cycles of load (with the first stirrup breaking at 500,000 cycles). The shear force in the beam on each cycle was approximately 70 kips, generated by applying a total of 140 kips in two point loads with a shear span length of 12 feet. Kreger and his colleagues conducted two other tests with different loads and shear spans (up to a 25 percent variation in these parameters) that survived over 3,000,000 cycles of load without failure.

From a durability perspective, however, prestress concrete may be affected by the occurrence of cracking. Cracks provide an opportunity for water and other agents to intrude into the beam, which can result in deterioration of the concrete and corrosion of the prestressing strands. The prestressing forces in a prestressed concrete member keep the section in compression, and thus they keep it uncracked. Such members possess considerable resistance to deterioration, as long they remain uncracked. The increase in bending moments discussed above will increase live load tensile stress ranges in the bottom of simply supported prestress concrete beams in direct proportion to the increase in bending moment.

Table 4.2.3-1 Relative Fatigue Demands of the Projected Traffic Streams Compared to the Existing Traffic Stream

Route[a]	Canadian Interprovincial % Change in Fatigue Demand		Canamex %Change in Fatigue Demand		Canamex Short % Change in Fatigue Demand	
	Short Term	Long Term	Short Term	Long Term	Short Term	Long Term
I-15	8.6	32.5	8.0	11.4	5.9	13.2
I-90	6.5	34.7	8.2	10.5	8.6	12.9
I-94	8.8	35.5	8.5	11.9	8.4	13.5
All Interstate	**7.7**	**34.1**	**8.2**	**11.1**	**7.7**	**13.1**
P-1	8.4	31.0	9.3	10.4	10.9	15.1
P-2	8.1	22.6	4.8	8.2	4.0	9.1
P-4	9.7	30.6	8.7	11.8	6.1	11.5
P-5	5.2	30.7	7.0	10.5	6.1	11.5
P-7	8.4	32.1	5.7	15.6	5.9	11.7
P-10	8.7	28.7	6.6	8.1	6.1	11.4
P-14	7.5	21.2	5.2	8.5	4.2	9.1
P-16	9.3	33.9	9.0	12.2	8.3	9.3
P-22	9.1	27.9	6.1	9.6	5.4	10.9
P-23	9.0	33.4	7.7	11.4	7.2	12.9
P-24	9.3	30.9	7.2	10.4	6.4	11.3
P-29	9.0	33.4	7.4	10.2	6.7	12.3
P-32	8.4	30.1	6.5	10.0	5.6	11.7
P-37	9.3	35.7	8.7	12.0	8.3	13.4
P-42	6.9	28.7	5.7	10.7	4.6	12.9
P-44	9.2	33.8	8.5	11.9	7.5	13.1
P-45	9.7	32.1	7.1	10.7	6.4	12.0
P-57	9.6	30.4	7.5	10.8	6.8	11.2
P-59	7.6	31.0	7.6	9.6	5.5	11.6
P-61	9.3	34.9	7.4	11.1	7.0	12.9
P-66	8.8	27.6	6.2	10.3	4.9	11.4
All Primaries	**8.5**	**30.1**	**7.4**	**10.5**	**7.2**	**12.2**
Interstate and Primaries	**8.2**	**31.3**	**7.5**	**10.6**	**7.3**	**12.5**

[a] route locations are shown on Figure 2.3.2-3

Fatigue stress limits are given in the AASHTO specification for highway bridges for conventional reinforcing steel. Typically, however, fatigue is not a problem in reinforced concrete bridges, in part due to the high dead load demand to total load demand of this type of structure (Wilkes, 1989). The magnitude of the live load stress range implicitly is limited by the magnitude of the capacity consumed in carrying the dead load

A review of literature available on the behavior of wood indicated that wood is neither sensitive to fatigue or cracking in the traditional sense. Wood is sensitive to duration of load, where this duration is measured as the cumulative time experienced at a particular level of stress. Residence time at higher stress levels will increase if Canadian Interprovincial, Canamex, or Canamex Short loads are allowed to routinely operate on the highway system.

While serviceability issues were not analyzed in detail in this study, the relative increase in stringer deflections under Canadian Interprovincial, Canamex, and Canamex Short vehicles were calculated. These calculations were performed for both simple span and continuous structures using PCBridge (Murphy, 1992). The increase in live load deflection increased with span length, going from 10 percent to 60 percent as the span length increased from 60 to 150 feet (see Figures 4.2.3-1 to 4.2.3-3). Deflection limits are a device (a) to control objectionable vibrations and deflection effects experienced and observed by bridge users and (b) to reduce impact effects on the structure, itself. In rural environments larger deflections are tolerated than in urban environments. For composite construction, typical of many stringer bridges on the interstate system, deflection rarely controls stringer design (Xanthakos, 1994). Even on the primary system, with a broad mix of structure types, only the timber structures are expected to be sufficiently flexible for deflection to be a problem. Thus, the decision was made not to pursue deflections further. Certainly, on the types of bridges where deflections may be critical (urban environment, wood or steel structure), further investigation of deflection problems under these vehicle loads should be considered.

<u>4.2.4 Columns/Substructure</u> - The increases in the relative demands on the girders, piers, and footings of a bridge under Canadian Interprovincial and Canamex loads are expected to be less severe than the increase in the relative demands on the stringer system. As load flows from the stringers into the girders, piers, and footings, the live load demand remains constant while the

dead load demand steadily increases. Thus, the live load demand becomes a smaller proportion of the total demand on each successive member in the bridge. Therefore, when the live load is increased, the relative increase in total demand on each subsequent element in the load path decreases.

4.3 CAPACITY OF EXISTING BRIDGES

4.3.1 General Remarks - Bridge capacity is generally determined using load rating calculations. Load ratings can be obtained using several criteria and approaches. Recognized load rating procedures include:

1) AASHTO Allowable Stress Approach (AASHTO, 1994)

2) AASHTO Load Factor Approach (AASHTO, 1994)

3) AASHTO Load and Resistance Factor Approach (Guide, AASHTO, 1989)

These approaches each utilize a slightly different philosophy in establishing member capacity, and the results obtained can vary significantly (100 percent) between approaches. The results of these ratings are typically expressed as the fraction of the maximum gross vehicle weight for a particular configuration that can safely cross the structure. Load ratings arrived at using standard procedures embody acceptable levels of safety, serviceability, and durability. Thus, if load ratings greater than 1.0 are obtained for Canadian and Canamex vehicles, such vehicles can operate on the system without compromising acceptable levels of safety, serviceability, and durability.

Two levels of load are considered in many rating procedures, namely, Inventory and Operating. The Inventory rating is defined by AASHTO as a load that can be applied to the structure an infinite number of times without any appreciable deterioration of the structure (AASHTO, 1994). A load at the Operating rating will not cause permanent distress to a bridge, but, if unlimited repetitions are allowed, it will result in a reduction in the service life (AASHTO, 1994). Operating ratings are often used with permitted truck traffic. Nominal guidance is available regarding the number of load events that are permissible at the Operating rating.

Within these choices of load rating methodologies and levels, three ratings were considered in this study to represent bridge capacity in system wide analyses:

1) Inventory ratings calculated using the AASHTO Allowable Stress approach
2) Operating ratings calculated using the AASHTO Allowable Stress approach, and
3) Inventory ratings calculated using the AASHTO Load Factor approach.

Following the AASHTO Allowable Stress approach, allowable loads are established such that an allowable stress in the material is not exceeded upon application of the dead load and live load. The allowable stress level is typically set as some fraction of the elastic limit of the material. The Load Factor approach was developed in response to a movement in structural engineering toward probabilistically based design/analysis techniques that compare "ultimate" member capacities to member demands under an overload condition. This load rating approach has been fully developed for steel, reinforced concrete, and prestressed concrete bridges. Member capacities are generally calculated as the maximum resistance (or some fraction of the maximum resistance) of the member at failure. Design overloads are calculated as the expected service loads multiplied by load factors. Overload factors of 1.3 on the dead load and 2.16 on the live load are used to obtain Inventory ratings. This approach is believed to be more rational than the allowable stress approach, in that the greater uncertainty in dead load versus life load demands is reflected in the load factors used to calculate demand. Following the allowable stress approach, live load and dead load demands are indistinguishable from each other, and the factor of safety established by the stress reduction factor is applied equally to both types of loads.

In considering Allowable Stress based Operating ratings as an acceptable level of capacity when evaluating Canadian Interprovincial, Canamex, and Canamex Short vehicles, a judgement has to be made if safety and durability are unreasonably compromised by allowing "unlimited" traffic at these load levels to use bridges. By definition, unlimited application of stresses approaching the Operating rating of a bridge are supposed to result in a reduction in the life of the bridge. Many states, however, apparently have adopted a liberal interpretation of this load rating. That is, in many states, a bridge will not be load posted until vehicles operating at maximum unpermitted legal weight limits exceed the Operating rating for the bridge (TRB, 1990a). This philosophy is followed in Montana (Murphy, 1995). Therefore, the decision was made in this study to consider Operating ratings as a measure of capacity against which Canadian Interprovincial and Canamex vehicle demands might be measured. Note, however, that in Montana, if legal vehicles exceed the Operating rating of a bridge, it is load posted back at its Inventory level. Further note that useable load ratings similar in magnitude to Allowable Stress based Operating ratings can be obtained for structures that are in good structural condition that experience light traffic using the proposed Load and Resistance Factor approach to bridge load rating (Moses and Verma, 1987).

Use of the AASHTO Load Factor based Inventory ratings as an acceptable level of capacity when evaluating Canadian Interprovincial and Canamex limits simply requires acceptance of this procedure as a legitimate load rating procedure. While MDT presently uses the allowable stress approach, they accept and are moving toward using Load Factor based ratings (Murphy, 1995).

Inventory ratings for all bridges on the state system were obtained directly from the state bridge inventory (MDT, 1994). These ratings were considered to be compatible with Allowable Stress based obtained ratings. The Allowable Stress based Operating ratings and Load Factor based Inventory ratings were calculated from information in the bridge inventory. Note that Allowable Stress based Operating ratings are reported in the bridge inventory. These values are only gross estimates of operating capacity, and they are generally not used by MDT in evaluating the capacity of specific bridges under particular demands. A simple consistency check of these ratings revealed large disparities in the ratings for similar bridges, and the decision was made not to use these values in this study. The bridge inventory contains over 90 items of information on each bridge, from which it is possible to estimate a load rating for each bridge. The inventory information is insufficient, however, to perform detailed structural analyses on individual bridges. Information used from the inventory for each bridge included the type of structural system, material, number of spans, length of the maximum span, total length of the over-all structure, and the reported Inventory rating.

Comprehensive load rating calculations include analyses of each element of the bridge system, with the minimum load rating for any given element and aspect of the response controlling the overall rating for the bridge. In this case, based on the discussion of demands presented above, attention was focused on the bridge stringer systems. Normally, under an increase in live load, the greatest proportional increase in total demand on a member would be on the first element in the load path, which in this case is the decks. Decks, however, are generally overdesigned with respect to strength. Thus, the second element in the load path, in this case the stringers, becomes the critical element with respect to increase in total demand.

4.3.2 AASHTO Allowable Stress Based Inventory Ratings - The majority of the Inventory ratings given in the state bridge inventory are simply the HS vehicle used for the original design; load rating calculations were not performed to obtain these values (Murphy, 1995). These load ratings were assumed to be a reasonable representation of the load ratings that would be obtained for the bridges using an allowable stress based analysis approach. Allowable stress based ratings were desired for this study, in that such ratings allow for simple calculation of ratios of total demand to total capacity (i.e., the proportion of the total capacity used in supporting the dead load and live load from a new vehicle). Almost all of the steel bridges on the state system, and all of the timber bridges on the state system, were designed using an allowable stress based approach, which is consistent with this interpretation of the given load ratings. Reinforced concrete bridges built prior to the late 1960's were also designed using an allowable stress based approach. Reinforced concrete bridges built after the late 1960's and all of the prestress concrete bridges on the state system were designed using a strength approach. The strength approach to the design of concrete was initially developed to produce designs similar to those obtained by the allowable stress approach for common types of structures. Therefore, the assumption was made that the allowable stress based inventory ratings for these structures would be similar to the load factor rating. In prestress concrete design, both allowable stress and strength criteria have to be met. Note that use of the design vehicle as a bridge's rated capacity does not recognize any possible increase in as-built capacity due to conservative selection of members to satisfy design demands.

4.3.3 AASHTO Allowable Stress Based Operating Ratings - Operating load ratings (AASHTO Allowable Stress based) were calculated for every span on the state highway system from the Inventory ratings given in the state bridge inventory. AASHTO defines the rating factor for a bridge as,

$$RF = \frac{C - A_1 D}{A_2\ L\ (1+I)}$$

where,

$RF =$ Rating factor

$C =$ Capacity

$A_1 =$ Dead load factor

$A_2 =$ Live load factor

$I =$ Impact factor

$D =$ Dead load demand

$L =$ Live load demand

For the Allowable Stress approach, A_1 and A_2 are taken as 1.0, and the capacity of the member is determined by the material under consideration and the type of rating being considered (Inventory or Operating). In general, flexural capacity is determined using an allowable stress that is equal to some fraction of the elastic limit of the material. Thus, the rating factor can be rewritten as,

$$RF = \frac{S\ (C_E)-D}{L\ (1+I)}$$

where,

$S =$ fraction of maximum stress at elastic limit of the material

$C_E =$ capacity of member at elastic limit of the material

Specifically, for the inventory rating,

$$RF_I = \frac{S_I C_E - D}{L(I+1)}$$

This equation can be solved for the capacity at the elastic limit of the material,

$$C_E = \frac{RF_I (L)(I+1)+D}{S_I}$$

In a similar fashion, the equation for the Operating rating factor can be written,

$$RF_O = \frac{S_O * C_E - D}{L(I+1)}$$

The expression obtained above for the capacity can be substituted into the Operating rating factor expression to obtain,

$$RF_O = \frac{S_O}{S_I}\frac{[RF_I(L)(I+1)+D]}{(L)(I+1)} - \frac{D}{L(I+1)}$$

Typical allowable stress factors for moment related stresses are given in Table 4.3.3-1 for various materials at both Inventory and Operating levels. The ratio of these factors (S_O/S_I) used in the calculation of the Operating rating is also reported in the table. In the case of composite concrete deck/steel stringer bridges, and for reinforced concrete bridges, a conservative approach was taken, with the minimum ratio for the various material possibilities involved selected for use.

Table 4.3.3-1 Allowable Stress Factors Used in Calculating Operating Ratings from Inventory Ratings (Compiled from AASHTO, 1994)

Material	Stress factor			Ratio Operating/ Inventory	Ratio used in calculating Operating rating
	Inventory	Operating	Index Stress		
Structural Steel	0.55	0.75	f_y	1.36	1.36
Reinforced Concrete Concrete Reinforcing Steel	0.4	0.6 1.39-1.50	f'c f_y	1.39	1.39
Prestressed Concrete	0.4-0.5[a]	-[b]	$(f'c)^{1/2}$	-[c]	-[c]
Wood	1.0	1.33	F_b	1.33	1.33

[a] derived from information in Bridge Design Specification (AASHTO, 1990)
[b] calculated in terms of ultimate strength of member rather than some fraction of elastic capacity
[c] undefined due to difference in models used to calculate Inventory and Operating capacities

For prestress concrete, the Operating level is defined by AASHTO (1994) in terms of the ultimate member capacity (75 percent of the ultimate capacity, M_N) rather than an allowable stress level. This measure of capacity is calculated using a different member response model than is used to obtain the allowable stress based Inventory rating. Therefore, the concept of using the stress factor ratio, S_O/S_I, in calculating the Operating rating from the Inventory rating for prestressed beams is inappropriate. In response to this situation, the decision was made to use the same S_O/S_I ratio for prestressed concrete as that used for steel. This decision implies that the capacities of prestress concrete beams at the Operating and Inventory levels have been selected in a fashion consistent with the manner in which the relative capacities of steel beams are established at the Operating and Inventory levels. This approach was believed to be adequate for these analyses. A simple comparison of the expression used above to calculate operating ratings from inventory ratings with the generic load factor based expressions for calculating operating and inventory ratings indicated that low operating ratings would be obtained by this approach for span lengths below approximately 65 feet, while increasingly high operating ratings would be obtained for span lengths above 65 feet (approximately 10 percent high for a 100 foot span). Note that average prestress span length on the state highway system is 59 feet.

A capacity ratio of 1.33 was used for wood, as given in the AASHTO manual. This capacity ratio is dependent to some extent on the accumulated duration of the applied load. If Canadian Interprovincial and/or Canamex vehicles are allowed to routinely operate on the highway, some adjustment will occur in both the maximum stress level and the time accumulated at that stress level. By theory, as the load duration increases, the allowable stress decreases. Use of 1.33 for the capacity ratio was judged acceptable in this study for the changes in load duration and stress level expected herein.

The ratios reported in Table 4.3.3-1, while derived using allowable bending stresses, were also used to represent the ratio of Operating to Inventory stresses in shear. While it can easily be shown that the allowable bending and shear stress ratios (Operating to Inventory) for wood and steel are similar in magnitude using allowable stress values recommended by AASHTO (1994), the relationship between these ratios for concrete is less obvious.

Dead load demand D was calculated as a fraction of the live load demand using the empirically derived equations for calculating dead load demands from live load demand, impact factor, and span length introduced above.

<u>4.3.4 AASHTO Load Factor Approach</u> - The Load Factor method compares the forces in a member under an overload with the strength of the member at "failure". This approach has been developed for load rating steel, reinforced concrete, and prestressed concrete bridges. Following this approach, the dead load factor, A1, is taken as 1.3, and the live load factor, A2, is taken as 2.17. Thus, the AASHTO rating factor equation for the Load Factor based Inventory level becomes,

$$RF_{LFI} = \frac{C_{LF} - 1.3\ D}{2.17\ L\ (1+I)}$$

where,

RF_{LFI} = rating factor for Load Factor based Inventory level

C_{LF} = ultimate capacity of member used in load factor procedure

Defining a new factor, K, that relates load factor based capacity to allowable stress based Inventory capacity,

$$K = \frac{C_{LF}}{S_I C_E}$$

This equation can be manipulated to solve for the load factor capacity and the results back substituted into the rating factor equation to obtain:

$$RF_{LFI} = \frac{K\ S_I\ C_E - 1.3\ D}{2.17\ L\ (1+I)}$$

From previous work, the product of $S_I C_E$ can be expressed as,

$$S_I\ C_E = RF_I\ (L)(I+1)+D$$

Making this substitution into the above equation,

$$RF_{LFI} = \frac{K\ [RF_I\ (L)(I+1)+D] - 1.3\ D}{2.17\ L\ (1+I)}$$

Thus, if the ratio of ultimate to inventory capacity, K, can be established, the Load Factor based Inventory rating can be calculated from the Allowable Stress based Inventory rating.

The capacity ratio K for steel was estimated as,

$$K = \frac{M_N}{M_{\text{allow stress}}}$$

where,

$$M_N = \text{plastic capacity} = Z\ f_Y$$

and,

$$M_{\text{allow stress}} = S\ f_S = S\ 0.55 f_Y$$

where,

Z = plastic section modulus

S = elastic section modulus

f_Y = yield stress of steel

Thus,

$$K = \frac{Z \; f_Y}{S \; 0.55 \; f_Y}$$

The ratio of plastic to elastic section modulus for steel shapes is referred to as the shape factor, and it has an average value of 1.12 for wide flanged sections (Salmon and Johnson, 1996). Making these substitutions into the equation above yields a K value for steel stingers of 2.04. Note that a shape factor of 1.12 was used in all calculations, independent of the presence or absence of composite behavior. The 1.12 value, however, is consistent with non-composite action; the relationship between Allowable Stress based and Load Factor based capacity for composite sections is complex. The assumption was simply made that despite this complexity, approximately the same capacity ratio would exist for composite sections as for non-composite sections.

A K value for reinforced concrete was estimated in a similar procedure to that of steel. The basic flexural capacity ratio for reinforced concrete can be expressed as,

$$K = \varphi \frac{M_N}{M_{\text{allow stress}}}$$

where,

φ = capacity reduction factor = 0.9

$M_N = A_S \, f_Y \, (d\text{-}a/2)$

$M_{\text{Allow stress}} = A_S f_S jd = A_S \; 0.5 \; f_Y \, jd$

A_S = area of reinforcing steel

d = effective depth

a = depth of stress block

j = moment arm factor

Use of this equation presumes that following the Allowable Stress approach that the stress state in the steel controls the capacity of the section. A useful approximation of both the quantities (d-a/2) and jd is 0.9d (Wang and Salmon, 1992). Making these substitutions into the equation above,

$$K = \frac{0.9\, A_S\, f_Y\, (0.9\, d)}{A_S\;\; 0.5\, f_Y\, (0.9\, d)}$$

Simplification of this expressions produces a K value of 1.8 for reinforced concrete.

Derivation of a simple expression for K for prestress concrete is difficult. The equations for ultimate and allowable stress capacity are involved and of different formats. As previously mentioned, the prestress concrete bridges on the state highway system were all designed using a strength approach. Therefore, the design vehicle based rating given in the bridge inventory was used to represent the load factor based inventory capacity. Following this approach, the load factor and allowable stress based load ratings for prestress concrete beams were the same.

4.4 CAPACITY VERSUS DEMAND

4.4.1 General Remarks - The impact of Canadian Interprovincial, Canamex, and Canamex Short weight limits on the bridge system was evaluated by comparing the bridge load ratings determined above with the previously estimated bridge demands for Canadian Interprovincial, Canamex, and Canamex Short vehicles. Comparisons of this type were performed for each span on the highway system for each load rating approach, and the number of deficient bridges tabulated. These analyses were performed in terms of the ratio of the total demand to the capacity of the span. In this study, this ratio is referred to as the level of "overstress". Somewhat consistent with standard structural engineering practice, and due to the nature and number of assumptions made in the various calculations, overstress levels less than 1.05 were judged to be safe when considering Inventory ratings. Overstress levels less than 1.00 were judged to be safe when considering Operating ratings.

Summaries of the bridges determined to be deficient under Canadian Interprovincial, Canamex, and Canamex Short vehicles on a system-wide basis using each load rating technique are presented in Table 4.4.1-1. Consistently fewer bridges were found to be deficient under the Canamex and Canamex Short scenarios compared to the Canadian Interprovincial scenario, which is a direct reflection of the lower allowable axle and gross vehicle loads for these vehicles

Table 4.4.1-1 Deficient Bridges, Total System by Load Rating Procedure, Canadian Interprovincial, Canamex, and Canamex Short Limits

Span Type	No. of Spans	% Deficient Canadian Interprovincial Limits			% Deficient Canamex Limits			% Deficient Canamex Short Limits		
		Allowable Stress Inv.	Allowable Stress Opr.	Load Factor Inv.	Allowable Stress Inv.	Allowable Stress Opr.	Load Factor Inv.	Allowable Stress Inv.	Allowable Stress Opr.	Load Factor Inv.
Reinf. Concrete	516	81	42	89	51	29	78	52	31	80
Cont. Concrete	602	98	10	100	64	2	82	81	3	94
Steel	656	79	21	31	55	17	29	70	16	30
Cont. Steel	886	92	25	55	89	20	34	90	20	33
Prestressed	3005	80	0	80	17	0	6	30	0	22
Cont Prestressed	3	100	0	100	33	0	33	100	0	100
Timber	2152	98	90	98[a]	98	33	98[a]	98	32	98[a]
Total	**7820**	**88**	**33**	**80[a]**	**57**	**15**	**50[a]**	**64**	**15**	**54[a]**

[a] rated timber with allowable stress inventory

compared to the Canadian Interprovincial vehicles. Nominally fewer bridges were found to be deficient under Canamex vehicles compared to Canamex Short vehicles, as would be expected due to the longer wheelbase of the Canamex vehicles. As might be expected based on the demands discussed earlier, a majority of the spans were found to be deficient under all scenarios using the Allowable Stress based Inventory capacity of the bridges. The lowest estimate of the proportion of deficient bridges system wide was 57 percent for the Canamex scenario. Eighty-eight percent of the bridges on the system were found to be deficient under Canadian Interprovincial loads; sixty-four percent, under Canamex Short loads.

Some patterns are evident in the percent of deficient bridges with respect to span type. These patterns are more pronounced in the Canamex and Canamex Short results relative to the Canadian Interprovincial results. The fewest deficiencies were observed for all three scenarios for simply supported prestressed concrete spans. Notably, under the Canamex and Canamex Short scenarios deficiencies of only 17 and 30 percent, respectively, were determined. For these spans, demands only nominally exceed HS20 demands at common span lengths. A high percent of deficiencies (98 percent) was calculated for all scenarios for timber spans. Most of these spans were designed to carry H15 rather than HS20-44 loads. With the exception of timber spans, deficiencies were generally higher in all scenarios for continuous rather than simply supported structures. Continuous structures were generally found to be inadequate to carry the negative moments generated by the new vehicles at the interior supports. Deficiencies for continuous steel structures were generally similar across all three scenarios, indicating that the negative moment demands on these structures are similar in all three scenarios.

The results obtained using Load Factor based Inventory ratings were similar to those obtained using Allowable Stress based Inventory ratings. The only notable difference in using the two rating approaches was for the percent of deficient steel bridges, which dropped significantly using the Load Factor based Inventory ratings. For the Canadian Interprovincial scenario, 45 percent of the steel bridges were found to be deficient under Load Factor based Inventory ratings, compared to 88 percent under Allowable Stress based Inventory rating. The proportion of deficient steel bridges under the Canamex and Canamex Short scenarios was found to be 32 percent using Load Factor based Inventory ratings compared to values of 75 and 81 percent, respectively, obtained using Allowable Stress based Inventory ratings.

As might be expected, the lowest percentages of deficient bridges under all three scenarios were calculated using the Allowable Stress based Operating ratings for the bridges. Only thirty-three percent of the bridges on the system were found to be deficient under Canadian Interprovincial loads; 15 percent, under Canamex and Canamex Short loads. The majority of simple span timber bridges (90 percent) were found to be deficient under Canadian Interprovincial vehicles even at Operating rating levels. The demands from Canamex and Canamex Short vehicles were sufficiently lower than the demands of Canadian Interprovincial vehicles that the proportion of deficient timber spans at Operating rating levels dropped to around 32 percent for these scenarios.

Several bridges on the state highway system (specifically on the primary, secondary, and urban systems) were found to be inadequate at their Allowable Stress based Inventory rating to analytically carry the standard rating vehicle in Montana, which is the HS20-44 design vehicle. In evaluating the effect of implementing Canadian Interprovincial, Canamex, and Canamex Short limits, it is important to appreciate the number of bridges deficient under present limits. The percentage of bridges deficient under the HS20-44 design vehicle using each of the rating approaches is indicated in Table 4.4.1-2. The lowest deficiency level is on the interstate system, where all bridges are rated as capable of carrying the HS20-44 design vehicle using Allowable Stress based Inventory ratings. Considerably higher deficiency levels are found on the primary, secondary, and urban systems, than on the interstate system, as these systems include many older bridges designed to lower standards.

4.4.2 Capacity vs. Demand, Allowable Stress Based Operating Ratings - Based on the results presented in Table 4.4.1-1, the decision was made to review in more detail the comparison of the vehicle demands under the various size and weight scenarios with the Allowable Stress based Operating ratings for the bridges on the highway system. The percent of deficient bridges and average overstress ratios calculated for each size and weight scenario using Allowable Stress based Operating ratings are presented separately for the interstate, primary, secondary, and urban

Table 4.4.1-2 Deficient Bridges, Total System by Load Rating Procedure, HS20-44 Design Vehicle

Span Type	No. of Spans	% Deficient HS20-44		
		Allowable Stress Inv.	Allowable Stress Opr.	Load Factor Inv.
Reinf. Concrete	516	49	28	60
Cont. Concrete	602	5	2	19
Steel	656	31	11	25
Cont. Steel	886	24	10	15
Prestressed	3005	7	0	6
Cont. Prestressed	3	0	0	0
Timber	2152	98	33	98[a]
Total	**7820**	**38**	**13**	**39[a]**

[a] rated timber with allowable stress inventory

systems in Tables 4.4.2-1 through 4.4.2-4. Less than 1 percent of the bridges on the Interstate system were found be deficient under Canadian Interprovincial vehicles using Allowable Stress based Operating ratings. No deficient bridges were found on the Interstate system under Canamex and Canamex Short limits. The average overstress ratio for all span types on the interstate system under Canadian Interprovincial loads was 0.823. Thus, on the average, Canadian Interprovincial vehicles exercise interstate bridge spans to 82 percent of their Allowable Stress based Operating capacity. The overstress ratios for interstate spans under Canamex and Canamex Short vehicles were even lower than for the Canadian Interprovincial Limits, averaging 0.761 and 0.775, respectively. Note that under HS-20 vehicles, the average overstress ratio on the interstate system was 0.731. Thus, the average demands of Canamex and Canamex Short vehicles only exceeded HS-20 design demands on the interstate system by 5 and 6 percent, respectively. Average Canadian Interprovincial demands were 12 percent higher than HS-20 demands on the interstate system.

The average overstress ratios determined for the bridges on the highway system under each scenario were dependent on the element of the system under consideration. The lowest

Table 4.4.2-1 Deficient Bridges and Average Overstress Levels for Allowable Stress Based Operating Ratings, Canadian Interprovincial Limits (based on number of spans)

Span type	Interstate system		Primary system		Secondary system		Urban system	
	% deficient of this type	Avg. Stress Ratio	% deficient of this type	Avg. Stress Ratio	% deficient of this type	Avg. Stress Ratio	% deficient of this type	Avg. Stress Ratio
Reinf. Concrete	0	0.754	47	0.974	57	0.937	74	2.470
Cont. Concrete	0	0.892	17	0.933	32	0.920	0	0.836
Steel	0	0.838	25	0.844	55	1.142	16	0.879
Cont. Steel	2	0.918	34	1.104	55	1.121	0	0.919
Prestressed	0	0.796	0	0.823	9	0.889	0	0.813
Cont. Prestress	0	0.905	0	0	0	0	0	0
Timber	0	0.831	91	1.044	93	1.076	3	0.988
Total	**0**	**0.823**	**54**	**0.982**	**56**	**1.015**	**12**	**1.055**
Percent Deficient under HS-20[a]	0	-	22	-	18	-	11	-

[a] see Table 4.4.2-4

Table 4.4.2-2 Deficient Bridges and Average Overstress Levels for Allowable Stress Based on Operating Ratings, Canamex Limits (based on number of spans)

Span Type	Interstate system		Primary system		Secondary system		Urban system	
	% deficient of this type	Avg. Stress Ratio	% deficient of this type	Avg. Stress Ratio	% deficient of this type	Avg Stress Ratio	% deficient of this type	Avg.Stress Ratio
Reinf. Concrete	0	0.724	38	0.920	20	0.930	66	2.134
Cont Concrete	0	0.784	6	0.795	0	0.795	0	0.758
Steel	0	0.777	18	0.847	47	1.079	16	0.779
Cont.Steel	0	0.862	32	1.028	34	1.029	0	0.856
Prestressed	0	0.739	0	0.753	1	0.809	0	0.741
Cont.Prestress	0	0.894	0	0	0	0	0	0
Timber	0	0.753	35	0.996	27	1.010	0	0.891
Total	**0**	**0.761**	**25**	**0.925**	**20**	**0.941**	**11**	**0.916**
Percent Deficient under HS-20[a]	0	-	22	-	18	-	11	

[a] see Table 4.4.2-4

Table 4.4.2-3 Deficient Bridges and Average Overstress Levels for Allowable Stress Based Operating Ratings, Canamex Short Limits (based on number of spans)

Span type	Interstate system		Primary system		Secondary system		Urban system	
	% deficient of this type	Avg. Stress Ratio	% deficient of this type	Avg. Stress Ratio	% deficient of this type	Avg. Stress Ratio	% deficient of this type	Avg. Stress Ratio
Reinf. Concrete	0	0.721	39	0.924	23	0.937	66	2.265
Cont. Concrete	0	0.837	6	0.835	6	0.859	0	0.741
Steel	0	0.796	16	0.855	47	1.086	16	0.821
Cont. Steel	0	0.852	32	1.022	32	1.024	0	0.878
Prestressed	0	0.750	0	0.770	1	0.829	0	0.764
Cont. Prestress	0	0.864	0	0	0	0	0	0
Timber	0	0.753	35	0.996	27	1.010	0	0.891
Total	**0**	**0.775**	**25**	**0.930**	**21**	**0.975**	**11**	**0.980**
Percent Deficient under HS-20[a]	0	-	22	-	18	-	11	-

[a] see Table 4.4.2-4

Table 4.4.2-4 Deficient Bridges and Average Overstress Levels for Allowable Stress Based Operating Ratings, HS-20 Limits (based on number of spans)

Span type	Interstate system		Primary system		Secondary system		Urban system	
	% deficient of this type	Avg. Stress Ratio	% deficient of this type	Avg. Stress Ratio	% deficient of this type	Avg. Stress Ratio	% deficient of this type	Avg. Stress Ratio
Reinf. Concrete	0	0.720	36	0.913	18	0.925	66	1.815
Cont. Concrete	0	0.722	5	0.752	0	0.770	0	0.719
Steel	0	0.731	6	0.813	44	1.023	16	0.763
Cont. Steel	2	0.733	14	0.838	21	0.868	0	0.733
Prestress	0	0.733	0	0.737	1	0.793	0	0.738
Cont. Prestress	0	0.719	0	0	0	0	0	0
Timber	0	0.753	35	0.996	27	1.010	0	0.891
Total	**0**	**0.731**	**22**	**0.894**	**18**	**0.914**	**11**	**0.894**

overstress ratios were consistently calculated for the interstate system, with values steadily climbing for the bridges on the primary, secondary, and urban systems. Under Canadian Interprovincial limits, for example, the average overstress ratios for the interstate, primary, secondary, and urban bridges were 0.823, 0.982, 1.015, and 1.055, respectively. These results were expected, in that the interstate bridges in Montana are generally newer and built to higher design standards than many of the bridges on the primary, secondary, and urban systems. Significant variation in overstress ratios were also observed between bridge types. Continuous steel and continuous prestress bridges consistently had the highest overstress ratios compared to other bridge types. Under the Canamex scenario, for example, the overstress ratios of continuous steel and continuous prestress bridges on the interstate system were 0.862 and 0.894, respectively. These values are 14 and 18 percent higher, respectively, than the overall average overstress ratio of 0.761 for this scenario. Overstress ratios for timber spans were generally high across all scenarios due to the lower loads used in the original designs for many of these spans. Reinforced concrete and prestressed concrete spans consistently had the lowest overstress ratios. Under the Canamex scenario, for example, the average overstress ratios for reinforced concrete and prestressed concrete spans on the interstate system were 0.724 and 0.739, respectively, compared to an average overall overstress ratio of 0.761. These overstress ratios closely approach the overstress ratios calculated for these span types under the HS20 vehicle (0.720 and 0.733, respectively).

The number of spans determined to be deficient under each scenario on each system generally reflected the relative levels of overstress discussed above. Less than 1 percent of all spans on the interstate system were deficient under all scenarios. Fifty-four percent of the bridges on primary system were found to be deficient under Canadian Interprovincial vehicles using the Allowable Stress based Operating rating; 25 percent, under both Canamex and Canamex Short vehicles. The largest difference between the Canadian Interprovincial, and the Canamex and Canamex Short vehicles was observed for timber bridges, for which 91 percent of the spans failed under Canadian Interprovincial limits (primary system), while only 35 percent failed under Canamex and Canamex Short limits. The overstress in these spans under Canadian Interprovincial limits of 1.04 was just over the acceptable level of 1.00. Most of these timber spans are 20 to 30 feet in length. At these lengths, the Canadian Interprovincial vehicles, with a heavy allowable tridem load compared to Canamex and Canamex Short vehicles, place high demands on spans, as previously shown in Figure 4.2.3-1. Relatively high levels of deficiencies (greater than 20 percent) were observed on the primary system for reinforced concrete and continuous steel spans under all scenarios.

The percentage of deficient spans on the secondary system is similar to that on the primary system. The average overstress level, however, is approximately 4 percent higher on the secondary system compared to the primary system. A similar situation exists for the average overstress levels on the urban system compared to the primary system. In the case of the secondary system, this situation is created by the increased number of bridges on the secondary system designed using H15 and lighter vehicles relative to the primary system. The percent of simple span steel bridges found to be deficient on the secondary system is over twice that found on the primary system.

As was previously observed at Inventory ratings, numerous bridges on the state highway system are also inadequate under the HS20-44 design vehicle. A summary of the bridges found to be deficient for HS20-44 by span type and system is presented in Table 4.4.2-4. All the bridges on the interstate system were adequate at Operating levels to carry the HS-20 design vehicle, as would be expected based on the Inventory ratings for these spans. Thirty-five and thirty-six percent of the timber and reinforced concrete spans on the primary system were found to be deficient to carry the HS-20 design vehicle.

4.5 LONG TERM EFFECTS - FATIGUE AND DURABILITY

4.5.1 General Remarks - If the demands of Canadian Interprovincial and Canamex vehicles were less than the Inventory ratings for all bridges on the state highway system, it could be concluded that such vehicles can safely operate on the bridge system, and that the system will not experience accelerated deterioration under such loads. Inventory ratings obtained using an accepted analysis procedure should, by definition, embody levels of safety and durability consistent with accepted practice. The consequences of using Operating ratings as a basis for setting bridge capacity are less well known. Arguments can be made that any deviation from the original definitions of load rating capacities, which have served adequately for many decades, will jeopardize at least the long term durability and possibly the safety of the system (Sorensen and Manzo-Robledo, 1992). Counter arguments can be made that if only the heaviest of vehicles in the traffic stream approach the operating rating, a bridge will only experience a finite number of these vehicles. Thus, while an unlimited number of vehicles are allowed at such load levels, in reality, only a limited number of passages will occur.

In light of the uncertain effects of using Allowable Stress based Operating ratings as an acceptable bridge capacity with respect to any form of routine vehicle operation, an effort was made to assess the long term effects of load applications that exceed the Inventory rating of a bridge but that are below the Operating rating. The demands of the Canadian Interprovincial and Canamex vehicles fall into this category, and their effect on long term integrity were considered. Attention focused on possible accelerated deterioration in concrete decks, increased fatigue damage in steel stringers, and the occurrence of cracking (that could lead to accelerated corrosion damage) in prestressed concrete. Investigation of these behaviors was accomplished both analytically and by field testing selected bridges under Canadian Interprovincial vehicles.

4.5.2 Decks - The local demand placed on decks in transmitting wheel loads from their point of application into the stringers are not expected to increase significantly under Canadian Interprovincial, Canamex, or Canamex Short loads, as previously stated. It was previously mentioned that localized demands related to transferring the wheel loads into the stringers could possibly increase up to 17 percent under Canadian Interprovincial versus existing weight limits. No increase in demand is anticipated under Canamex and Canamex Short vehicles. The increase under the Canadian Interprovincial scenario would result from the higher loads allowed on adjacent axles in an axle group under this scenario compared to present limits. The decks were judged to have adequate capacity to carry such loads from a strength perspective, in that decks apparently are generally over designed for strength (Beal, 1982; Batchelor, Hewitt, and Csagoly, 1978; Minor, White, and Busch, 1988).

Some concerns still existed, however, regarding accelerated deterioration of decks. Since the cost of repair and rehabilitation of bridge decks can be very high (Callahan, Seiss, and Kesler, 1970), it was decided that the possibility that deck deterioration would accelerate under Canadian Interprovincial and Canamex loads merited further investigation. Therefore, an extensive literature review was conducted and both analytical modeling and field testing were done to address deck deterioration concerns. A review was also done of historic bridge deck performance on Montana's highways to determine if load and traffic effects play a major role in deterioration rate.

Deck behavior and deterioration under vehicle loads has been extensively studied (Carrier and Cady, 1973; Newlon, Davis, and North, 1973; James, Zimmerman, and McCreary, 1987; Kostem, 1978; Callahan, Siess, and Kesler, 1970; Hilsdorf and Lott, 1970; Sanders and Zhang, 1994), and it has been concluded in several of these studies that vehicle and traffic effects are secondary to other causes of deck deterioration. Factors known to affect bridge deck deterioration include clear cover on the reinforcing steel, use of deicers, concrete strength, concrete air content, construction practices (finishing and curing practice), traffic volume, load intensity, bridge type, and span length. Many deck studies have further concluded that bridge deck deterioration is not limited to one cause or type of distress. Thus, assessing the specific effect of Canadian Interprovincial vehicles on deck deterioration is a difficult task.

Most deck deterioration initiates as cracking, and many studies have commented on the cause of cracking in bridges (Callahan, Siess, and Kesler, 1970; Newlon, Davis, and North, 1973; Hilsdorf and Lott, 1970; Kostem, 1978) . Cracking can occur due to consolidation of the concrete when it is in the plastic state, volumetric changes in the concrete when it is in the hardened state, structural displacements of the deck unrelated to live load applications (differential settlement of the supports, thermal movements in the supports, etc.), and structural displacements associated with vehicle loads. Callahan and his colleagues (1970) report that an analytical model used by Rejali (1966) found that maximum live load demands, if amplified in magnitude, would be expected to produce longitudinal cracks in the decks. Finite element calculations of deck response performed as part of this investigation (and described below) also indicated that load related distress in the deck would first be manifested in longitudinal cracks.

Newlon (1973) found, however, that the most prevalent type of cracks in decks are transverse cracks. His observation supports assigning responsibility for these cracks to shrinkage and plastic flow. Newlon did observe, however, that the number of cracks increased with span length and traffic volume.

Once cracks initiate, spalling and scaling can occur. Spalling and scaling problems appear to be significantly influenced by freeze-thaw action and use of de-icing agents (Callahan, Siess, and Kesler, 1970; Cady and Weyers, 1977). The factor that affects spalling the most appears to be insufficient clear cover on the reinforcing steel (Cady and Weyers, 1977; Carrier and Cady, 1973). If the clear cover is inadequate, deicing salts can penetrate to the reinforcing steel. Subsequent formation of corrosion products in the reinforcing steel creates tensile stresses in the concrete that leads to localized spalling over the bars. Scaling has been observed to increase with deck age and traffic volume. The underlying mechanisms associated with this scaling, however, may still simply be freeze thaw (may be related to the age) and use of deicers (may be related to the volume of traffic) (Newlon, Davis, and North, 1973).

In this investigation, cracking of bridge decks under Canadian Interprovincial loads was studied using finite element models of typical prestressed concrete deck stringer systems (Scoles, 1996). These models were generated in the ANSYS finite element program and consisted of up to 4,000 elements and 60,000 degrees of freedom representing a coupled deck and stringer system. Eight-noded orthotropic five layer plate/shell elements were used to represent the deck. All the elements were modeled as linear elastic materials. Wegmuller (1977) previously demonstrated that both linear and nonlinear analyses could successfully be used to study bridges under overloads.

Performance of the finite element models was verified using test data collected from two bridges on Interstate 15 in northern Montana. These bridges, with very different span lengths and stringer spacings, are typical of many bridges in the state inventory. Span lengths of the bridges were 35 and 65 feet, with stringer spacings of 8 and 5 feet, respectively. Strain data was collected from each deck in the lateral and transverse directions at the centerline between stringers, under a loaded Canadian B-train (at Canadian weights) and other vehicles traveling across the bridge at quasi-static and normal highway speeds. A typical strain history collected during the passage of B train is presented in Figure 4.5.2-1. Broad peaks in the data correspond to the passage of axle groups over the gaged locations; the sharp peaks superimposed on the broad peaks correspond to the individual axles in the group crossing the gage location. A comparison of the measured strains and the strains calculated in the finite element model are shown in Figure 4.5.2-2. The measured and calculated strains are in close agreement. Much of the observed difference in the measured and calculated response was attributed to nominal differences between the location of the transducers on the real decks and the points at which output was available in the finite element model. Based on these types of comparisons, the finite element model was judged to adequately represent the performance of real decks, and subsequent analyses focused on using the models to consider various loadings and bridge geometries.

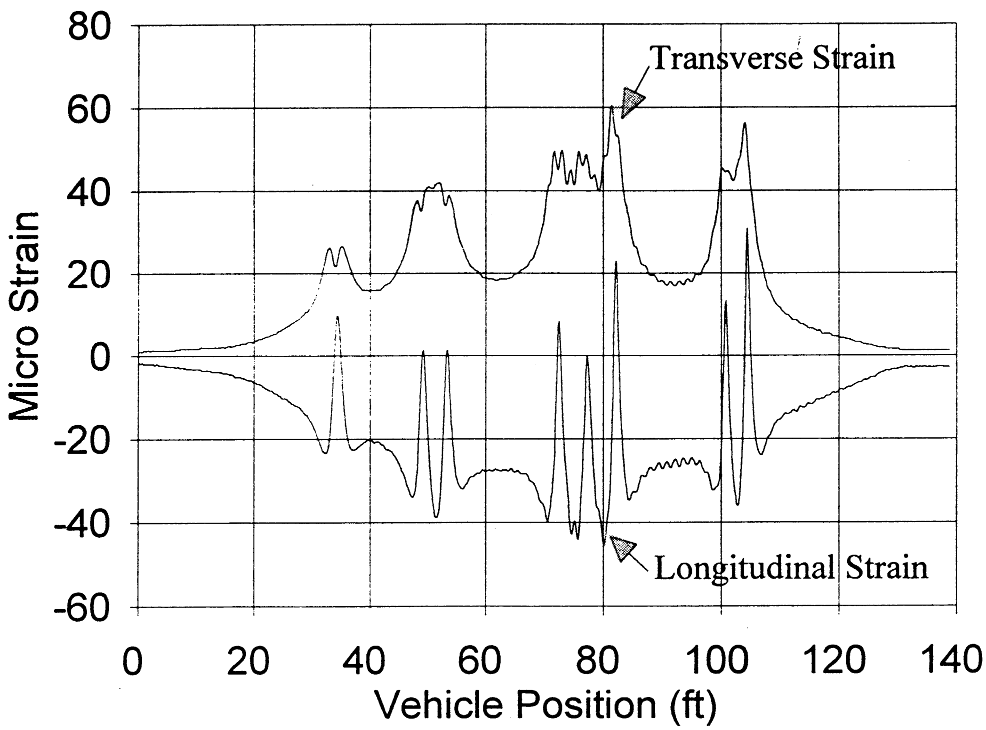


Figure 4.5.2-1 Strain History on the Bottom Surface of a Typical Concrete Deck During the Passage of a Canadian B-train

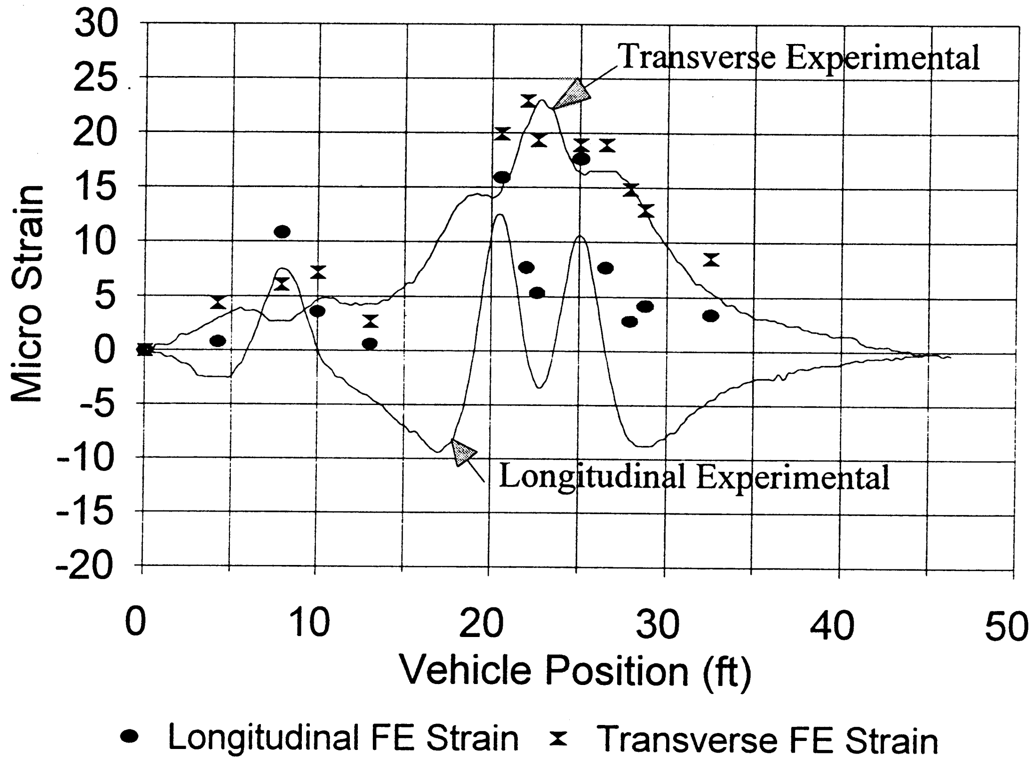


Figure 4.5.2-2 Measured and Calculated Strains on the Bottom Surface of a Typical Concrete Deck During the Passage of a 3 Axle Single Unit

The results obtained from typical finite element runs using full Montana and Canadian Interprovincial weights for single, tandem, and tridem axles on large combination vehicles are presented in Table 4.5.2-1. These results are for the stresses expected transverse to the stingers on the top surface of the deck at the centerline between stringers and over the top of the stringers. The reported values are for a wheel line centered between the stingers. Prestress concrete stringers spaced at 5 and 8 feet on center carrying a composite 7 inch thick deck were used in this calculation. Referring to Table 4.5.2-1, demands under Canadian Interprovincial limits exceed demands under current limits by up to 14 and 17 percent in tension and compression, respectively. As previously commented, this increase in demand should be readily accommodated by the decks from a strength perspective. The maximum tensile stress in the concrete of 69 psi is significantly below the expected cracking stress of the concrete, which was estimated using 7.5 f'c (Wang and Salmon, 1992) to be 474 psi (assuming a compression strength of 4000 psi). The compression stresses are significantly below the crushing stress of the concrete which was assumed to be 4000 psi. Thus, cracking and crushing was not expected to occur in the top surface of the deck under Canadian Interprovincial loads.

Table 4.5.2-1 Estimated Stress Levels at the Top Surface of Typical Bridge Decks Under Existing and Canadian Interprovincial Load Limits

Axle Group	Calculated Stresses in the Transverse Direction, Top Surface of Deck (psi)[a]					
	Centerline Between Stringers			Over Top of Stringer		
	Montana	Canadian	Canadian/ Montana	Montana	Canadian	Canadian/ Montana
Steering	269 C	269 C	1.00	40 T	40 T	1.00
Single	344 C	344 C	1.00	43 T	43 T	1.00
Tandem	331 C	388 C	1.17	50 T	56 T	1.12
Tridem	337 C	383 C	1.14	62 T	69 T	1.14

[a] T, tension; C, compression

A study of historical deck performance in Montana found that deck condition and age and traffic loading are only poorly correlated. This study considered 50 decks on prestress concrete stringer bridges located on the interstate or primary system. A majority of the decks were over 30 years old. Regression analyses were performed using linear, exponential, and polynomial models to relate deck condition rating to age and cumulative traffic (measured as AADT). The goodness of the fit was similar for all models. The simple linear regression model had a correlation coefficient (r^2) of 0.39. The actual and predicted deck condition values for this model are presented in Figure 4.5.2-3. Based on these various results, deterioration of the decks must be primarily dependent on other factors than age and traffic.

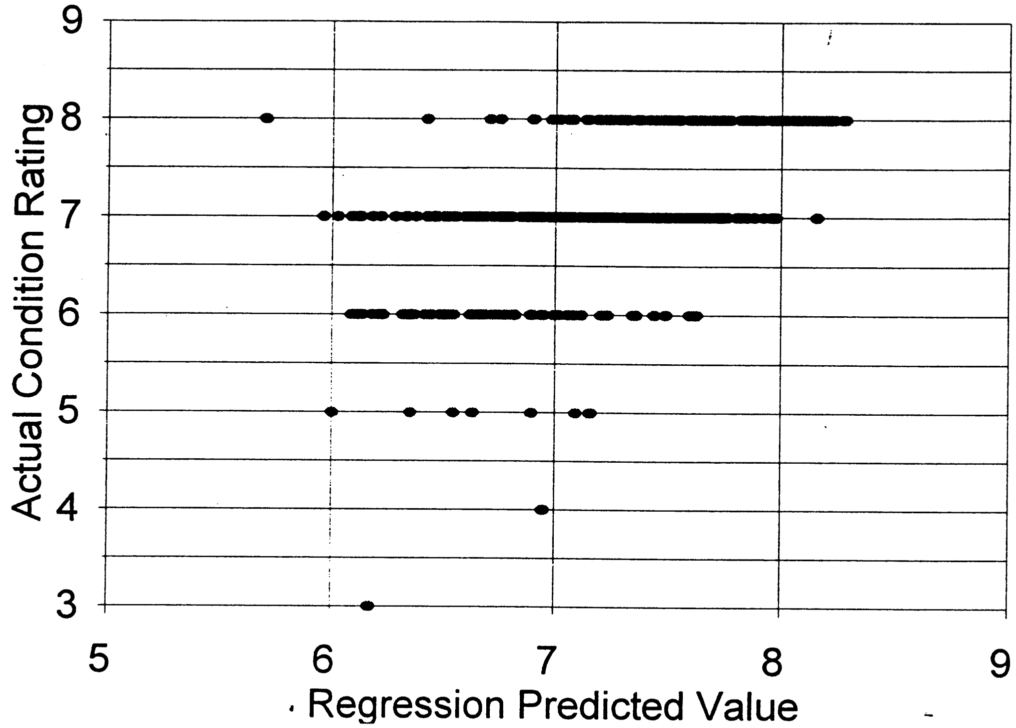


Figure 4.5.2-3 Deck Condition as a Function of Age and Traffic, Sample of 50 Bridge Decks

4.5.3 Fatigue - Steel Stringers - While the expected reduction in fatigue life of steel stringer bridges under Canadian Interprovincial and Canamex vehicles can be estimated from the weight of their equivalent fatigue vehicles and the expected number of vehicle passages, the absolute fatigue life of each bridge under the existing and new traffic streams are more tedious to calculate. While the reductions in fatigue life presented in Table 4.2.3-2 are significant, the remaining fatigue life can still be long (e.g., over seventy-five years), even after these reductions are taken. The live load stress range could be low in magnitude, if fatigue did not control the design of the original structure, and the expected fatigue life under the existing traffic stream could still be adequate. Therefore, a method was developed to estimate if the expected fatigue lives of the steel stringer bridges under Canadian Interprovincial and Canamex loads would be judged acceptable by AASHTO's fatigue criteria.

The AASHTO design code addresses fatigue in steel structures by restricting the live load stress range based on the expected cycles of load. Maximum live load stress ranges for steel stringer bridges under Canadian Interprovincial and Canamex vehicles were estimated using the allowable stress based Inventory ratings for each bridge. These stress ranges were compared to the allowable stress ranges presented in AASHTO appropriate for the cycle load regime for Montana's highways. Traffic volumes in Montana are consistent with using 100,000 stress cycles for bridges on the primary system and 500,000 cycles for the bridges on the interstate. In following this approach, the assumption was made that under the Inventory rating vehicle, the maximum allowable inventory stress is generated in the member. The stress range calculations began with the identification of the stress level expected in the stringers under the dead load demand plus the maximum live load demand. This value was estimated as,

$$f_T = C\ S_I$$

where,

f_T = total stress under dead load and maximum live load (with impact)

C = demand ratio, total demand for scenario divided by total demand for HS20

S_I = allowable stress at inventory level

From the estimate of dead load as function of live load demand presented earlier in this report, this total stress can be proportioned into dead load and live load fractions. The live load fraction was taken as the fatigue stress range. Numerical values for this stress range were calculated by assuming a steel with a yield stress of 36 ksi.

The live load stress ranges calculated above were compared to the appropriate allowable stress ranges presented in AASHTO for detail type E on a redundant load path member. Many cover plate configurations are Type E details, and these configurations are common on many steel stringer bridges in Montana. The results of this comparison are presented in Table 4.5.3-1. Seventeen and eleven percent of the steel spans system-wide were found to be deficient following this approach under Canadian Interprovincial and under Canamex and Canamex Short vehicles, respectively. On the interstate and primary system, fatigue deficiencies were higher for continuous steel spans compared to simple steel spans. The percent of deficient spans was generally highest on the secondary system.

Table 4.5.3-1 Deficient Steel Stringer Bridges, Fatigue Considerations

Scenario/ System	Simply Supported		Continuous		All Steel	
	No.	% Deficient of type	No.	% Deficient of type	No.	% Deficient of type
Canadian Interprovincial Limits						
Interstate	229	11	336	12	565	11
Primary	262	4	386	22	648	14
Secondary	116	36	155	21	271	27
Urban	49	41	9	0	58	34
Total	**656**	**15**	**886**	**18**	**1542**	**17**
Canamex Limits						
Interstate	229	0	336	8	565	5
Primary	262	2	386	17	648	11
Secondary	116	24	155	16	271	20
Urban	49	29	9	0	58	24
Total	**656**	**7**	**886**	**13**	**1542**	**11**
Canamex Short Limits						
Interstate	229	0	336	7	565	4
Primary	262	2	386	18	648	11
Secondary	116	24	155	19	271	21
Urban	49	29	9	0	58	24
Total	**656**	**7**	**886**	**14**	**1542**	**11**

The live load stress ranges calculated above may indeed be conservative compared to actual bridge performance. A simply supported steel stringer bridge typical of many bridges in the 50 to 60 foot span length on the primary and interstate systems was tested under Canadian Interprovincial vehicle loads to determine the actual live load stress ranges that can be expected (Stephens, et.al., 1996). The stringers on the bridge were instrumented at critical locations with strain transducers, and the response was measured under different vehicle loads. The live load (with impact) stress range in the stringers estimated from these tests at critical locations under two fully loaded Canadian B-trains simultaneously on the bridge was 7 ksi. The live load stress range predicted using the analytical approach described above was 12.3 ksi. At the 7 ksi level, the stringers meet the criteria for up to 2,000,000 cycles of load. Recall that the design requirement was only 500,000 cycles. The maximum live load stress range for a transverse transfer girder in the same bridge under two B-trains side-by-side on the bridge was estimated to be 8 ksi. The AASHTO allowable stress range at 500,000 cycles for this girder and detail was 10 ksi and at 2,000,000 cycles, 6 ksi. (Note that this location on the interstate (I-15 north of Great Falls), the average daily truck traffic was estimated to be less than 500 vehicles per day).

4.5.4 Cracking/Durability Prestressed Concrete - Adequate durability was expected from the prestress concrete structures if the concrete at the bottom of the stringers remained in compression at service load levels. Under these conditions, cracks would not develop that allow moisture and other agents of deterioration access to the inside of the concrete and to the prestressing strands. Possible cracking of the beams under full Canadian Interprovincial vehicles (which generate the highest demands of the scenarios considered) was checked by comparing the estimated live load strains under the maximum demand with the theoretical live load strains at which the bottom fibers of the stringers would go into tension. This check was specifically performed for three bridges on the interstate system judged to be representative of many of the prestress bridges on the state highway system. These bridges are on a segment of Interstate 15 in northern Montana upon which Canadian vehicles are already allowed to operate at full Canadian weights. The characteristics of these bridges are summarized in Table 4.5.4-1.

For each bridge under investigation, the live load strain at which tension would occur in the bottom fibers of the stringers was estimated using conventional analysis procedures. These strain values are reported in Table 4.5.4-1. Use of this level of response for evaluation of acceptable performance is more restrictive than that required by AASHTO (1990), which actually allows the stringer to be exercised up to the theoretical capacity of the concrete in tension. This approach presumed that the beam may have been previously cracked, and that any tension stress will result in the crack opening. The actual live load strains expected under Canadian Interprovincial vehicles were then estimated from data collected during field tests of the bridges (Stephens, et.al., 1996). The stringers in each bridge were instrumented with strain transducers at several locations along their length, and the strain response was measured under various vehicle loads, including a loaded Canadian B-Train. The results of these tests were used to estimate the expected live load strains under two Canadian B-trains side-by-side on the bridges. The resulting strain values are shown in Table 4.5.4-1. In all cases, the actual live load strains are a maximum of 33 percent of the calculated live load strain at which the bottom fibers of the stringers will go into tension. At these strain levels, the extreme fiber stresses on the tension edges of the beams are obviously less than 6 (f'c) $^{1/2}$ in tension. Therefore, based on the work of Hanson, Hulsbos, and Van Horn (1970), no impact on fatigue life would be expected.

Table 4.5.4-1 Estimated Service Load Strain Levels in Typical Prestressed Concrete Stringer Bridges

Bridge Characteristics		Maximum tensile strain at bottom of stringer, extrapolated from test data, 10^{-6}	Estimated live load strain for cracks to open, 10^{-6}	Ratio, maximum expected strain to live load strain to open crack
Span Length (feet)	Geometry			
36	Straight	50	213	0.23
65	Straight	107	324	0.33
65	Skew	100	324	0.31

The prestressed concrete bridges used in the test effort described above are on a section of interstate highway upon which Canadian Interprovincial vehicles are already allowed to operate at full Canadian weights. These vehicles have been allowed on this section of highway, which is located immediately south of the Canadian Border on Interstate 15, since 1991 (Galt, 1996). The bridges on this segment of highway remain in good condition (Murphy, 1995). Performance of the decks, stringers, and stringer supports has been consistent with that on similar bridges around the state. While the number of Canadian vehicles operating at full Interprovincial limits on this section of the highway has not been rigorously monitored, it is on the order of magnitude of less than 50 vehicles per day.

4.6 DETAILED LOAD RATING CALCULATIONS (BENDING)

4.6.1 General Remarks - Detailed load rating calculations were performed for the primary flexural systems on six bridges on the highway system to observe how these ratings compared with those determined above using the simple system wide analysis procedures. Five simple span structures were analyzed in bending: three prestress concrete stringer bridges, one steel stringer bridge, and one wood stringer bridge (Stephens, et.al., 1996). Note that some aspects of the response of the three prestressed concrete and the steel stringer bridge have already been discussed. One continuous structure was tested, namely, a 3 span reinforced concrete slab structure. The prestress concrete and steel stringer bridges were selected for analysis as being representative of the majority of bridges in the state inventory. The wood and concrete structures were selected for study when the system-wide analysis indicated that they were more sensitive to the demands of Canadian Interprovincial loads than the other types of bridges (further refinement of the system-wide analysis resulted in a later increase in the capacity of the concrete slab bridge). Additional rating analyses are in progress on some "typical" continuous steel structures to determine if trends observed in analyzing the capacity of simple span stringer bridges can appropriately be extended to continuous structures.

In addition to the Allowable Stress based Inventory and Operating rating, and the Load Factor based Inventory rating methodologies used in the system-wide analysis described in previous sections of this report, Load and Resistance Factor load ratings were calculated (for bending response). The intent of the Load and Resistance Factor rating approach, presented by AASHTO as a guide rather than a manual for bridge load rating (AASHTO, 1989), is to further improve the consistency in the level of safety provided by the load rating process. The approach is similar to that adopted by the structural engineering community over the past several years for most aspects of building design, and involves applying load and resistance factors that through their adjustment to site specific conditions result in designs that provide a consistent level of safety across diverse circumstances. For example, in a situation with numerous heavy vehicles, poor weight enforcement, and deteriorated structural conditions, bridge failure is more likely than in a situation with only a few heavy vehicles, good enforcement, and good structural conditions. Load factors would be applied in this case related to the number of heavy vehicles and the level of enforcement. Resistance factors would be applied related to the poor structural conditions. In areas with low traffic, good enforcement, and good structural conditions, which is the situation for most bridges in Montana, a higher rating may be obtained using this approach than is obtained using other rating procedures. This observation may be particularly appropriate for older bridges designed using earlier philosophies that applied constant factors of safety across all situations. Under low traffic, good enforcement, and good structural conditions, the Load and Resistance Factor approach has generically been shown to produce load ratings equal to or greater than Allowable Stress based Operating ratings (Moses and Verma, 1987).

4.6.2 Rating Calculations - The rating factors obtained for each bridge using the various methodologies listed above are presented in Table 4.6.2-1 (Johnson, 1995). These factors are based on the bending capacity of the primary flexural systems of each bridge under a fully loaded Canadian B-train. Presented in Table 4.6.2-1, as appropriate, are the rating factors calculated using the simple system-wide procedures outlined above. With the exception of the steel stringer and timber bridge, the Allowable stress based Inventory rating factors obtained by detailed bending analysis for all the bridges were greater than 1.0. The rating factors obtained by these analyses exceeded the factors obtained using the simple system-wide analysis procedures by 7 to 194 percent. Use of Load and Resistance Factor rating procedures resulted in higher load ratings for all structures compared to those obtained using Allowable Stress based approaches, as might be expected for conditions in Montana. Rating factors obtained by the Load and Resistance Factor approaches were all greater than 1.0, indicating that these bridges are adequate to carry full Canadian Interprovincial loads. Thus, these bridges should also be able to carry Canamex and Canamex Short vehicles, as the demands under these scenarios are less than those under Canadian Interprovincial vehicles.

The lowest rating for the prestress concrete bridges was obtained for the bridge constructed in 1977. This situation may result, in part, from the continuing evolution of bridge design codes. Codes are perpetually being revised to better represent actual conditions and to more explicitly account for observed behaviors than in previous codes. The load factors used in prestress concrete design were reduced in 1971 as part of this code refinement process. Thus, prestress stringer bridges built after 1977 may possess less reserve capacity than those built prior to 1977.

Table 4.6.2-1 Typical Results of Detailed Load Rating Calculations (Full Canadian B-Train)

Bridge Information				Rating Factor from Simple Network Analysis			Rating Factor from Detailed Analysis			
Type	Span Length (feet)	Date Built	Design Vehicle	Allowable Stress, Inventory	Allowable Stress, Operating	Load Factor, Inventory	Allowable Stress, Inventory	Allowable Stress, Operating	Load Factor, Inventory	Load and Resistance Factor
Prestressed, Simply Supported	36	1964	HS-20	0.96	1.51	0.93	1.32	1.82	1.13	1.97
	65	1961	HS-20	0.82	1.39	0.84	1.73	2.54	1.57	2.38
	65 (w/skew)	1977	HS-20	0.82	1.39	0.84	1.07	1.79	1.11	1.90
Steel, Simply Supported	56	1961	HS-20	0.89	1.47	1.03	0.95	1.65	1.35	1.88
Timber, Simply Supported	25	1957	H-15	0.61	0.88	-[a]	0.65	0.99	-[a]	-[a]
Continuous, Reinforced Concrete	23-30-23	1971	HS-20	0.74	1.16	0.68	1.35	2.64	2.11	3.19

[a] no procedure for load rating

Results of the field tests conducted on these 6 bridges were also used in the load rating process. Diagnostic testing was performed on each bridge to determine load paths and estimate absolute levels of response under service loads (Stephens et.al., 1996; Johnson, 1995). These results were used to adjust the load distribution factors in the rating calculations. Allowable Stress based Inventory load ratings obtained using distribution factors based on the field test results are presented in Table 4.6.2-2. The load ratings increased by up to 30 percent compared to those obtained strictly by analysis, with an average increase in capacity of 15 percent. The Allowable Stress based Inventory rating for the timber bridge, however, was still below 1.0.

4.7 CANADIAN EXPERIENCE

Alberta's experience with the adoption of Canadian Interprovincial Limits may be informative with regard to the impact such a step may have on Montana's bridges, in that size and weight limits in Alberta prior to the introduction of the Canadian Interprovincial limits were similar to those currently in force in Montana. Bridges in Alberta have been, and continue to be, designed according to principles and procedures consistent with those used in Montana. Prior to around 1975, these calculations were performed using a vehicle load similar to the HS20 load. Since 1975, bridges in Alberta have been designed using a load equivalent to an HS25 design load, which is 20 percent higher than the HS20 design load. Thus, bridges in the Alberta constructed after 1975 have a higher design live load capacity than bridges in Montana.

Alberta performed a detailed analysis of the bridges in the province prior to the adoption of the Canadian Interprovincial limits to determine their ability to carry the new vehicles. The bridges designed to HS25 were generally found to be adequate to carry Canadian Interprovincial vehicles. Some deficiencies were found on bridges designed using HS20, dependent on the specific span length, structural system, and material. Problems were encountered with, among other things, shear in reinforced concrete and steel bridges (notably, violations of width to thickness ratios for elements of steel members), and stability of the compression flanges of steel sections in negative moment regions of continuous spans (Moroz, 1996). Approximately 60 out of 600 bridges on the Alberta primary system (the highest level system in Alberta) were found to be deficient and in need of some remedial action based on these analyses (Zutatas, 1994).

Table 4.6.2-2 Load Rating Results, Allowable Stress Inventory vs. Allowable Stress Inventory Using Experimental Distribution Factors

Bridge Type	Span Length (feet)	Date Built	Design Vehicle	Rating Factor from Detailed Analysis		
				Allowable Stress, Inventory	Allowable Stress, Inventory with Experimental Distribution Factors	% Increase in Rating Using Experimental Distribution Factors
Prestressed, Simply Supported	36	1964	HS-20	1.32	1.51	14
	65	1961	HS-20	1.73	1.92	11
	65 (w/skew)	1977	HS-20	1.07	1.17	9
Steel, Simply Supported	56	1961	HS-20	0.95	1.13	19
Timber, Simply Supported	25	1957	H-15	0.74	0.83	12
Continuous, Reinforced Concrete	23-30-23	1971	HS-20	1.35	1.75	30

The experience in Alberta represents to some extent a combination of the various scenarios considered in this study. The situation for bridges in Alberta built before 1975 is similar to the situation faced by bridges in Montana subjected to full Canadian Interprovincial loads. The situation for bridges in Alberta built after 1975 is similar to the situation faced by bridges in Montana subjected to Canamex Short vehicles. That is, the level of overstress in an HS25 structure under full Canadian Interprovincial vehicles is similar to that of an HS20 bridge under Canamex Short vehicles. Thus, based on the Canadian experience, less than 10 percent of the bridges on the interstate system might be found deficient under Canamex Short vehicles, while more than 10 percent might be found deficient under Canadian Interprovincial vehicles.

4.8 RESULTS OF OTHER STUDIES

Several studies have been conducted on the impact of changes in truck size and weight limits on bridge performance. Generally, only indirect comparisons can be made between the results of these studies and the results obtained herein due to differences in the specific regulatory situations under investigation. One of the most pertinent studies conducted to-date was the 1990 TRB truck size and weight study (TRB, 1990a), in which the impact of adopting Canadian Interprovincial limits across the United States was addressed. The TRB study considered a Canamex version of Canadian Interprovincial limits in which U.S. axle load limits were maintained but with a 51 kip tridem compared to the 42.5 kip tridem used in this study. By using the 51 kip tridem load, however, the TRB scenario may be closer in make-up to the Canadian Interprovincial Limits than to the Canamex scenarios considered in this study. The TRB scenario also made it attractive to operate 4 axle single units consisting of a single steering axle and tridem, which could operate at 71 kips under their scenario (compared to 62.5 kips under all the scenarios considered in this study).

The TRB study found that 21 percent of the bridges on the primary and interstate system, above and beyond the bridges deficient to carry current vehicle loads, were inadequate under the Canadian Interprovincial scenario they considered. The TRB study used Operating ratings plus a 5 percent tolerance (on the rating factor) to represent capacity. In this study, using Allowable Stress based Operating ratings plus zero tolerance (on the total stress), 17 percent of the bridges on the primary and interstate system were found deficient, above and beyond the bridges found to be deficient to carry current vehicle loads. The results obtained herein, however, indicate that all these bridges are on the primary system.

A second study performed by TRB (TRB, 1990b) on the effect of Turner trucks on the highway infrastructure may also be indicative of the effect of Canadian Interprovincial and Canamex vehicles on Montana's highways, in that some of the proposed Turner trucks have wheelbases and total gross vehicle weights (and thus structural demands) similar to those of the vehicles considered in this study. This study also used Operating ratings plus a 5 percent tolerance (on the rating factor) to represent bridge capacity. The study found that the incremental deficiencies (above and beyond deficiencies under current vehicle size and weight limits) associated with adopting Turner trucks nationwide (specifically for a scenario with heavy Canadian Interprovincial type C-train vehicles) amounted to 20 percent of all bridges on the interstate and primary systems compared to 17 percent found in his study. Once again, all the bridges determined to be deficient in this study were on the primary system. The TRB study results also imply that the incremental deficiencies from adopting Turner trucks with a C-train similar to that of the Canamex scenario considered herein amounted to 8 percent of all bridges in the inventory. The study comments that the majority of these deficiencies would be concentrated in half a dozen states.

Weissman and Harrison (1991) performed a study on the impact of adopting Turnpike Doubles and Triple 28s on the Rural Interstate Bridge Network. Their study found that of the 845 bridges on the Montana interstate system, 106 bridges were deficient under these vehicles at the Inventory ratings reported in the National Bridge Inventory (plus a 5 percent tolerance on total demand). The vehicles considered in their study were a double trailer combination unit at a length of 108 feet and a gross weight of 134,000 pounds, and a triple trailer combination unit at a length of 95 feet and a gross vehicle weight of 115,000 pounds. The weights of these vehicles are of the order of magnitude of the Canadian Interprovincial and Canamex vehicles considered in this study, but the wheelbases are considerably longer, resulting in lower demands than those considered in this study. Thus, a useful comparison of their results with the results of this investigation is difficult to formulate. This study found that on the interstate system, 91, 32, and 47 percent of the bridges were deficient at their Allowable Stress based inventory capacity to carry Canadian Interprovincial, Canamex, and Canamex Short loads, respectively.

Mohammadi and his colleagues (1991) performed a study of the effect of increased truck weights upon Illinois highway bridges. The study focused on fatigue effects in steel bridges from increasing the allowable gross vehicle weight from 72,000 to 80,000 pounds on bridges with limited design load carrying capacity (54,000 pounds or less). Of the 15 bridges studied, 6 were expected to have at least a 75 year life under either vehicle weight. Seven of the bridges were expected to have reduced lives under the 80,000 pound load compared to the 72,000 pound load, with an average reduction in life of 11 percent. These results illustrate the variability in the fatigue response of bridges based on their specific configuration.

4.9 DISCUSSION AND CONCLUDING REMARKS

The impact of adopting Canadian Interprovincial, Canamex, or Canamex Short limits on the bridge system is dependent on the criteria and procedure judged to be acceptable in establishing bridge capacity. While many bridges on the state highway system are obviously inadequate to carry Canadian Interprovincial, Canamex, and Canamex Short vehicle loads, these bridges generally are on the secondary and primary systems. Typically these bridges were designed for lower vehicle loads than are used under present design standards, and their inability to carry the increased demands is to be expected. The adequacy of bridges that were designed using modern design vehicles (i.e., all of the bridges on the interstate system and many of the bridges on the primary system) to carry Canadian Interprovincial, Canamex, and Canamex Short vehicles without compromising acceptable levels of safety, serviceability, and long term durability is more difficult to determine. While these vehicles will place higher demands on these bridges than they were apparently designed to carry, many designs may have been sufficiently conservative that the bridges can reasonably accommodate these increases in load.

A summary of the predicted deficiencies in the bridge system based on simple system-wide analyses of the strength capacity of the bridges (Allow Stress based ratings) with respect to the Canadian Interprovincial, Canamex, and Canamex Short loads is presented in Table 4.9-1. These results are presented in terms of both the total percentage of bridges deficient (Table 4.9-1a) and the incremental deficiencies above and beyond the bridges already deficient under HS20-44 demands (Table 4.9-1b). The most conservative results with respect to bridge

Table 4.9-1a Summary of Deficient Bridges by System and Scenario, Total Deficiencies

System	Percent Total Deficient by Rating Level								
	Allowable Stress Based Inventory Rating			Allowable Stress Based Operating Rating			87 Percent of Maximum Total Stress at Allowable Stress Based Operating Rating		
	Canadian Inter-provincial	Canamex	Canamex Short	Canadian Inter-provincial	Canamex	Canamex Short	Canadian Inter-provincial	Canamex	Canamex Short
Interstate	91	32	47	0	0	0	32	2	6
Primary	90	75	81	53	28	28	70	66	67
Secondary	94	78	84	56	21	21	75	66	70
Urban	94	47	56	21	18	18	44	30	36
Total	**91**	**61**	**71**	**36**	**17**	**17**	**58**	**44**	**45**

Table 4.9-1b Summary of Deficient Bridges by System and Scenario, Incremental Deficiencies (above and beyond bridges already deficient to carry the HS20-44 design vehicle)

System	Percent Incremental Deficient by Rating Level								
	Allowable Stress Based Inventory Rating			Allowable Stress Based Operating Rating			87 Percent of Maximum Total Stress at Allowable Stress Based Operating Rating		
	Canadian Inter-provincial	Canamex	Canamex Short	Canadian Inter-provincial	Canamex	Canamex Short	Canadian Inter-provincial	Canamex	Canamex Short
Interstate	91	32	47	0	0	0	32	2	6
Primary	26	11	17	28	3	3	5	1	2
Secondary	25	9	15	36	1	1	16	7	11
Urban	62	15	24	3	0	0	14	0	6
Total	**47**	**17**	**27**	**20**	**1**	**1**	**16**	**2**	**3**

deficiencies are obtained using the Allowable Stress based Inventory ratings, as might be expected. The majority of the bridges on all systems are found to be deficient under all three scenarios, with the highest deficiencies under Canadian Interprovincial Limits (91 percent) and significantly lower deficiencies under Canamex and Canamex Short limits (61 and 71 percent, respectively). Incremental deficiencies are also high in this case for the interstate system, as all bridges on this system have an HS20 rating. The incremental deficiencies are low for the primary, secondary, and urban systems primarily because these systems already have so many bridges deficient for the HS20 vehicle.

If Allowable Stress based Operating ratings are used to measure capacity, the number of bridges found to be deficient declines significantly compared to that calculated using the Allowable Stress based Inventory ratings. Thirty-six and seventeen percent of the bridges system-wide were found to be deficient under Canadian Interprovinicial, and Canamex and Canamex Short loads, respectively. Only a few bridges on the interstate system were found to be inadequate at this rating level under any of the size and weight scenarios. Incremental deficiencies are therefore also low on the interstate system, as all bridges on the interstate system have at least an HS20 rating. For the primary system, 53 and 28 percent of the bridges were still found to be deficient under Canadian Interprovincial and the two Canamex scenarios, respectively, even using full Allowable Stress based Operating ratings. Incremental deficiencies on the primary system for Canamex and Canamex Short vehicles were only 3 percent, compared to 28 percent for Canadian Interprovincial vehicles.

Based on the results of limited load rating calculations and bridge testing performed herein , many bridges on the highway system may have the capacity to carry demands in excess of the HS20-44 design demands. Four simply supported steel and concrete spans, and one continuous reinforced concrete span, all believed to be deficient to carry Canadian Interprovincial vehicles at inventory levels based on their design capacity, were found to be adequate at inventory levels when evaluated using their as-built properties and new load rating procedures. Field testing further revealed an average increase in capacity of 12 percent based on the actual load transfer behavior determined for the bridges compared to the behavior assumed in the load rating models.

Short span timber structures were found to be overstressed under Canadian Interprovincial loads even at Allowable Stress based Operating ratings. Detailed analysis and diagnostic bridge testing resulted in a nominal increase in the capacity of these structures, but they were still found to be inadequate at Allowable Stress based Operating ratings. These bridges represent the vast majority of the bridges found to be deficient on the primary system under all scenarios. Many of these bridges were designed for an H15 load rather than the HS20-44 load. Proof testing of typical structures within this category may be necessary and justified, before replacement is decided, to definitively establish capacity.

Use of Load and Resistance Factor load rating procedures produced a substantial increase in load rating compared to other approaches. These load ratings were similar to the Allowable Stress based Operating ratings, as might be expected under conditions in Montana (low traffic and good structural conditions).

Based on these various considerations, it may be reasonable to expect the numbers of bridges found to be deficient under Canadian Interprovincial, Canamex, and Canamex Short loads to fall between the predictions based on full Allowable Stress based Operating ratings and some fraction of these ratings to represent bridge capacity. Full Allowable Stress based Operating ratings may represent an upper bound on the useable capacity that would be determined for most structures following Load and Resistance Factor rating procedures. The lower bound on useable capacity was estimated to be 87 percent of these full Allowable Stress based Operating ratings (assuming that the Operating ratings have been estimated from the original design demand rather than the as-built capacity). This lower bound on capacity is consistent with the level of Inventory ratings obtained in this study by using as-built and as-performing data in the Inventory rating process, rather than simply setting the Inventory capacity equal to the design demand. This lower bound on capacity represents an increase of approximately 18 percent in the as-built and as-performing capacity of the specific bridges on the Montana state highway system relative to their total original design demand. Note that while these levels of increase in the as-built and as-performing capacities compared to design demands were observed for simple span structures, they are believed to extend to continuous bridges. Additional analyses are underway to validate this assumption.

At the intermediate level of capacity of 87 percent of full Allowable Stress based Operating ratings, 32 , 2, and 6 percent of the bridges on the interstate system were found to be deficient under Canadian Interprovincial , Canamex, and Canamex Short loads, respectively. On the primary system, 70, 66, and 67 percent of the bridges were found to deficient under Canadian Interprovincial, Canamex, and Canadian Short loads. At the 87 percent level, incremental deficiency rates system-wide under Canadian Interprovincial, Canamex, and Canamex short vehicles were 16, 2, and 3 percent, respectively.

Comprehensive load rating analyses of the bridges on the system will be necessary to definitely establish bridge deficiencies. Some of the deficiencies discovered in Alberta when they performed these types of analyses for their bridge system, for example, will not be revealed by a simple network analysis. Some of the deficiencies they discovered, such as stability problems with elements in steel cross-sections, however, were simple to remedy.

Using any of the measures of bridge capacity discussed above, Canadian Interprovincial vehicles place more severe demands on bridges than Canamex and Canamex Short vehicles. The resulting numbers of deficient bridges are generally disproportionate to the difference in flexural demands between the scenarios, implying full Canadian Interprovincial vehicles are closer to a critical threshold of demand than Canamex and Canamex Short vehicles. While the flexural demands of Canadian Interprovincial vehicles are only 10 to 15 percent greater than those of Canamex Short vehicles, the number of deficient bridges under Canadian Interprovincial vehicles is generally 30 to 50 percent greater than the number under Canamex and Canamex Short vehicles. The proportion of incrementally deficient bridges under Canadian Interprovincial limits is from 75 to over 500 percent higher than for Canamex and Canamex Short limits. The proportion of deficient bridges for all span types and systems generally decreases under Canamex and Canamex Short limits compared to Canadian Interprovinical Limits, except for continuous steel structures. The nature of the demands and the geometries of these structures are such that they experience similar demands under all three scenarios.

Fatigue demands will increase under Canadian Interprovincial, Canamex, and Canamex Short vehicles. This increase in demand was investigated particularly with respect to steel bridges. The impact of this increase in demand (31, 11, and 13 percent greater for Canadian

Interprovincial, Canamex, and Canamex Short vehicles over the long term compared to existing vehicles) is difficult to predict due to the sensitivity of fatigue response to the specific structural configuration under investigation. Based on simple calculations, and the general low volume of traffic on bridges in Montana, fatigue is not expected to be an issue under the size and weight scenarios considered in this study. Bridges known to possess fatigue sensitive details will have to be carefully evaluated.

The durability of concrete bridge decks and prestressed concrete beams should be unaffected by the adoption of Canadian Interprovincial, Canamex, or Canamex Short limits. Limited analytical and experimental investigations of the behavior of these elements were undertaken, as these are the two most common elements in bridges on the state highway system. These analyses indicated that stress and strain levels under Canadian Interprovincial and Canamex limits are below the values expected to result in permanent and cumulative damage to the structures.

5. PHYSICAL EFFECTS ON PAVEMENTS

5.1 GENERAL REMARKS

As previously commented, maximum axle weights under the Canadian Interprovincial limits on truck size and weight exceed those currently allowed in Montana. The increase in weights ranges from 10 percent on a tandem to 25 percent on a tridem. While these loads are not expected to cause severe damage to the pavement in a single passage, the cumulative effect of these loads from multiple vehicle passages is of concern. This concern is heightened by the fact that the fatigue damage caused by the passage of an axle group is believed to increase by as much as the fourth power of the weight of the axle group.

Pavement wear concerns also exist if the Canamex or Canamex Short limits on truck size and weight are adopted, even though the axle weight limits under this system are unchanged from their present values. Different vehicle configurations place different demands on pavements per unit weight of freight hauled. Some configurations believed to be attractive to weight limited operators under the Canamex and Canamex Short system may be more damaging to the pavement than the existing configurations they replace. Furthermore, axle weights on large combination vehicles (7 and 8 axle double trailer units) are presently limited to less than their current maximum allowable values by bridge formula constraints. Under the Canamex and Canamex Short size and weight scenarios, some of these axles may be loaded to higher weights than have been commonly used under existing weight limits.

The impacts of the adoption of Canadian Interprovincial, Canamex, and Canamex Short limits on pavement were determined by a) estimating the demand expected to be placed on the pavement under the existing and alternate scenarios proposed herein, b) determining the remaining life of existing pavements under these demands, and c) calculating the required overlay thickness to meet these demands in the future to extend the life of the pavement an additional 20 years. These calculations were performed for a sampling of pavement segments from the entire interstate system and from typical primary routes around the state. These results were then extrapolated to cover each route in its entirety, and then further extended to represent the situation across the entire primary and interstate systems. All calculations were performed for flexible pavements. Less than 5 percent of the pavement on the state highway system is rigid, and the decision was made that a reasonable representation of total system performance would be realized by considering just flexible pavement.

Data on the secondary and urban systems is sparse with respect to the volume of traffic and the physical characteristics of the roadway. Therefore, pavements on the secondary and urban systems were not analyzed as part of this study. Pavements on the secondary system are generally believed to be less well constructed and in poorer condition than pavements on the primary and interstate systems. Thus, despite the relatively low traffic on the secondary system, operation of Canadian Interprovincial, Canamex, and Canamex Short vehicles on these roads could have a significant impact on their condition.

Vehicles operating at full Canadian Interprovincial Limits have been allowed to travel for the past 5 years on a 36 mile stretch of Interstate route 15 in northern Montana. The performance of the pavements on this segment of highway was reviewed for any anomalies that might be related to the operation of these vehicles.

5.2 TRAFFIC DEMANDS AND PAVEMENT DAMAGE

5.2.1 General Remarks - The damage sustained by a given pavement by the passage of a vehicle is affected by several factors related to both the vehicle and the pavement. Important characteristics of the vehicle include individual axle loads, axle configuration, tire configuration, tire size and pressure. Pavement related parameters of interest include pavement type, thickness, subgrade conditions, temperature, and present condition. Gillespie and his colleagues (1993) compiled an excellent summary of the relationship between these various parameters and pavement damage. Pavement demands and damage are generally viewed with respect to two mechanisms, (a) immediate structural failure of the pavement under a few applications (or even under the single application) of a severe demand, and (b) progressive fatigue and/or rutting failure of the pavement under high cycles of moderate demand. Demands under Canadian Interprovincial, Canamex, and Canamex Short vehicles are not so severe as to cause immediate structural failure of most pavements. Maximum local wheel load demands under all the scenarios considered in this study should be similar in intensity to those under existing Montana limits. The cyclic demands on pavements under Canadian Interprovincial, Canamex, and Canamex Short vehicles, however, are expected to change compared with the cyclic demands placed on the pavements under the present traffic stream.

The effects of changes in cyclic pavement demand can be investigated using pavement performance models. Considerable research has been done developing mechanistic models to relate the various vehicle and pavement characteristics listed above to pavement behavior and performance, and some of these mechanistic relationships are beginning to be used in practice. The relationship between engineering material response (considered, for example, in cycles of strain to cracking) and gross highway performance (measured, for example, in terms of ride quality) can be difficult to establish when using mechanistic approaches. In light of the apparent complexity of the problem from a mechanistic perspective, empirical relationships have traditionally been used to predict pavement performance as a function of a variety of parameters known to influence pavement damage. A well-known empirical approach used to quantify and design for fatigue type damage in pavements is the AASHTO ESAL approach (AASHTO, 1993). While this design process and the entire ESAL concept are not universally accepted, this is the design process currently used by MDT. Therefore, this approach was used in this investigation.

5.2.2 AASHTO ESAL Approach - Following the AASHTO approach to pavement design, vehicle demands on pavements are quantified in terms of equivalent single axle loads or ESALs (AASHTO, 1993). An ESAL represents the relative amount of damage inflicted by a particular type of axle (e.g. single axle, tandem, or tridem) under a specific load in terms of the number of passages of a single axle loaded at 18,000 pounds required to inflict an equivalent level of damage. Relationships between ESALs and axle loads were determined from the results of the AASHO road test (HRB, 1962). In part of this test, sections of road were loaded with repeated cycles of the same axle load until a predetermined level of deterioration was reached. Tests were performed for a limited range of axle loads and axle types. Deterioration was measured in terms of the present serviceability index (PSI), a parameter specifically developed to provide a general indication of a pavement's ability to serve traffic. The index ranges from 1 to 5, with a value of 5 corresponding to pavement in excellent condition. Pavements on major roads with a PSI of 2.5 are considered in need of repair.

The results of the AASHO road test indicated that the relationship between ESAL and axle weight was dependent on the axle configuration (single or tandem), the axle group weight, the type of pavement (flexible vs. rigid), the relative strength of the pavement, and the terminal level of PSI selected to correspond to failure. Within these parameters, axle load and axle configuration have the most effect on pavement damage. The relationships between ESAL and axle load for single and tandem axles on the same pavement are shown in Figure 5.2.2-1 While the AASHTO equations derived to relate ESALs-to-axle load indicate a fourth order dependence (ESALs increase as a fourth order of the load) (AASHTO, 1972), some investigators believe a lower order relationship (third order) maybe more appropriate (Small, et.al., 1989). Following AASHTO's approach, a Canadian tandem at a load of 37.5 kips does approximately 45 percent more damage in a single passage then the same axle loaded at the Montana limit of 34 kips.

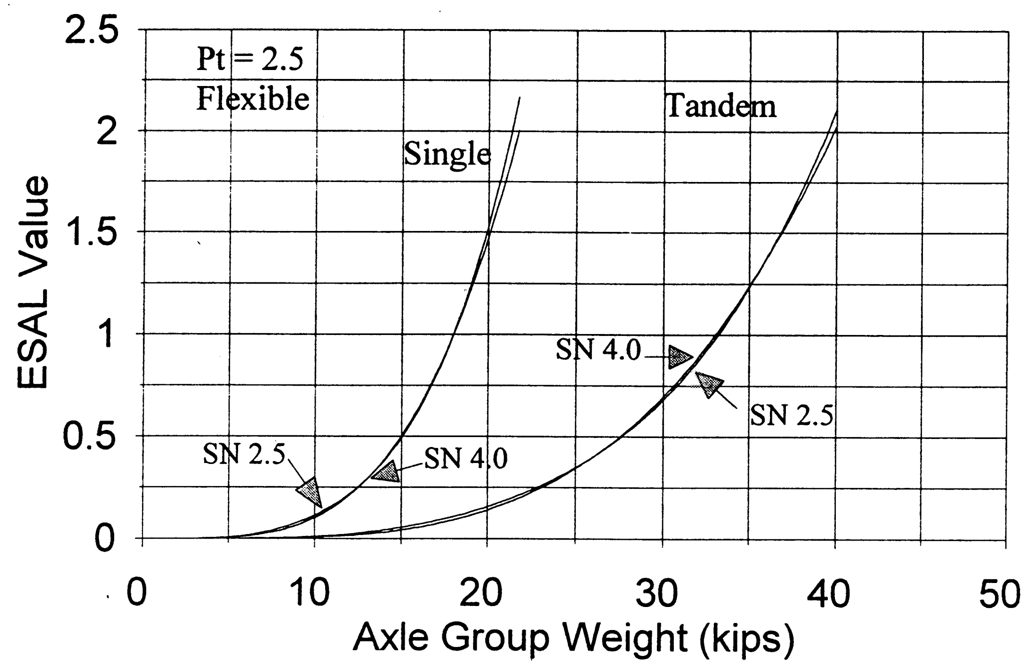


Figure 5.2.2-1 Typical Axle Load to ESAL Relationships, SN=2.5 and SN=4.0 (Based on AASHTO (1993))

The sensitivity of the ESAL-to-axle load relationship to axle group configuration (i.e., single versus tandem axle) is clearly evident in Figure 5.2.2-1. Pavement damage decreases when the applied load is carried on closely spaced axles compared to widely spaced axles. This effect has been attributed to favorable interference in the stress patterns generated by the individual axles in the group. ESAL to axle load relationships for tridems were analytically developed from the single and tandem axle expressions, as tridems were not part of the AASHO road test matrix. Thus, the validity of the tridem relationship is less certain. Efforts have been made to validate tridem, quadrum, and even quintum damage to axle group load relationships using computer models calibrated against the AASHO road tests (Southgate and Deen, 1986; Kilareski, 1989). While such calculations have supported at least the general concept that use of more axles in a group results in less pavement damage, concerns still exist that the advantage may be overstated for tridems, if the AASHTO approach is simply followed.

Tire configuration also influences the demand placed on pavements by vehicle axles. Parameters of concern with respect to tires include number (e.g., single or dual), size (e.g., width), and inflation pressure. Use of single, narrow, over inflated tires compared with conventional tires can result in a significant increase in pavement demand. A 25 psi increase in inflation pressure even for conventional dual tires, for example, can reportedly increase the damage caused in flexible pavements by a factor of two (Gillespie, et.al., 1993). The assumption was made in this study that tire and inflation pressures would be similar across all vehicles being studied, and therefore tire and inflation pressure were eliminated as variables for consideration. Axle group loads will increase under Canadian Interprovincial loads compared with current Montana limits, but no information was found that indicated inflation pressures would also increase.

The amount of damage sustained by a pavement under the passage of a particular axle load is directly dependent on the type of pavement, its thickness, and the subgrade conditions. In the case of flexible pavements, various combinations of materials, thicknesses, and subgrade conditions can be collectively evaluated using the structural number (SN) (AASHTO, 1993). Values for SN range between 1 and 6, with a value of 6 corresponding to the strongest/best flexible pavement. The influence of SN on the ESAL-to-axle load relationship is shown in

Figure 5.2.2-1. As might be expected, strong pavements are less affected by the passage of a given axle load than weak pavements, as evidenced by the lower ESAL values for pavements with higher SN values. ESAL values at the load ranges of interest, however, are relatively insensitive to SN value.

The ESAL approach provides a tool for calculating the demand placed on a pavement by a traffic stream of mixed vehicles operating at various weights. ESALs can be calculated for each axle of a vehicle based on the individual characteristics of the axles and then summed to obtain the ESALs for the vehicle. These values can be further summed across all vehicles to obtain the ESALs for the entire traffic stream. Expected total ESALs of demand at a given location can be used in the pavement design process following an approach published by AASHTO that relates pavement thickness to, among other things, strength of the base, the selected terminal condition at failure, and total ESALs of demand across its expected lifetime.

5.2.2 - ESAL Calculations - Calculations of ESAL demands were done for the existing and projected traffic streams along the interstate and selected primary routes around the state. The routes considered in this analysis are shown in Figure 2.3.2-3. Pavement segments on both systems were sampled at ten mile intervals along the length of the routes analyzed.

The total ESAL demands at each location under each scenario were calculated from the composition of the traffic stream at that location using average operating ESAL values for each vehicle type. The number of vehicles of each configuration at each location was multiplied by the average ESAL value for that configuration (and scenario) and summed to obtain the total ESAL demand. Average ESAL values for each configuration were calculated from the weight/frequency distributions previously generated for each scenario according to the procedures described in Section 3. A structural number (SN) of 3.5 was used for all pavements in performing these calculations. While this SN value was judged to be appropriate for the pavements in the state, it was also observed that ESAL magnitudes were relatively insensitive to this parameter across the range of realizations believed to be appropriate for this problem. A terminal PSI value of 2.5 was used in all ESAL calculations, which is consistent with MDT practice.

Typical ESAL values obtained from these calculations for some of the major vehicles in the various traffic streams projected for the interstate system are presented in Table 5.2.2-1.

Different ESAL values were used on the primary and interstate systems for only a few vehicle configurations. Operating weights and thus ESAL factors were found to be similar on both the interstate and primary systems for most vehicle configurations. Typical ESALs per 100,000 pounds of freight carried are also presented in Table 5.2.2-1. These values represent the relative efficiency with respect to pavement damage of various vehicles in transporting freight. These values clearly show that of the large combination vehicles, the 7 axle C-train is particularly damaging to pavement, and perhaps its use at high loads should be discouraged.

Table 5.2.2-1 Typical ESAL Values, Long Term Scenario, Interstate System

Vehicle Configuration	Average Operating ESALs	ESALs per 100,000 lbs of freight carried	ESALs per 100,000 lbs freight carried, normalized to existing 3S2
Canadian Interprovincial			
3S2	1.32	4.63	0.93
3S3	2.08	4.57	0.92
7 Ax A-train	1.64	4.64	0.93
8 Ax A-train	1.73	3.77	0.76
8 Ax B-train	2.91	4.54	0.91
Canamex			
3S2	1.41	4.91	0.99
3S3	1.43	4.26	0.86
7 Ax A-Train	1.77	5.13	1.03
7 Ax C-Train	3.34	6.03	1.21
8 Ax C-Train	3.07	4.31	0.87
Canamex Short			
3S2	1.41	4.91	0.99
3S3	1.43	4.26	0.86
7 Ax A-train	1.67	4.67	0.94
7 Ax C-train	3.38	5.86	1.18
8 Ax C-train	2.75	4.47	0.90
Existing			
3S2	1.46	4.97	1.00
3S3	1.43	4.26	0.87
7 Ax A-train	1.90	4.96	1.00
8 Ax A-train	1.79	3.61	0.73

The change in ESALs of demand predicted along the interstate and selected primary routes under each scenario considered herein is summarized in Table 5.2.2-2. Pavement demands increase on the interstate and primary systems under all scenarios considered, with a maximum average increase of 4.8 percent for Canadian Interprovincial limits over the short term and a minimum average increase of 1.3 percent for Canamex and Canamex Short limits over the short term. ESAL changes from the long term scenarios may better represent the overall changes in demands than those from the short term scenarios, as short term conditions are only expected to persist for a few years. Increases in demand for these scenarios were 3.3, 4.0, and 4.3 percent, respectively, for Canadian Interprovincial, Canamex, and Canamex Short vehicles. The relative change in ESALs between the short and long term scenarios are in opposite directions for Canadian Interprovincial, and Canamex and Canamex Short limits. That is, for Canadian Interprovincial limits, the short term ESAL demand is significantly higher than the long term ESAL demand, even though the long term scenario includes extra freight diverted from rail. Under the long term scenario, operators are expected to shift freight onto 3S3 and Canadian B-trains, which are relatively ESAL friendly compared to the vehicles assumed to operate in the short term scenario (notably heavy 3S2 vehicles) and relatively ESAL neutral compared to vehicles in the present traffic stream. Thus, most of the ESAL increase for the long term Canadian Interprovincial scenario is related to freight diverted from rail.

The opposite trend occurs for Canamex and Canamex Short limits compared to Canadian Interprovincial limits, that is, the short term ESAL demand is significantly less than the long term ESAL demand. In this case, all freight is diverted directly to heavier A- and C-trains, which are less ESAL friendly than many existing configurations, with extra freight being assigned to these same vehicles from rail diversion for the long term scenario. Rail diversion is responsible for approximately a 3 percent increase in total ESALs of demand beyond the basic 1.5 percent increase generated by vehicle-to-vehicle diversions. Thus, rail diversion accounts for approximately twice the amount of increased damage expected under Canamex and Canamex Short Limits compared to the increased damage simply due to vehicle-to-vehicle diversions.

As might be expected, vehicle demands will increase more on the interstate system (maximum increase of 7.0 percent) than on the primary system (maximum increase of only 4.2 percent). The composition of the traffic stream is different on the two systems, with the volume

Table 5.2.2-2 Predicted Changes in ESAL Demands of the Projected Traffic Streams Compared to the Existing Traffic Streams

Route	Canadian Interprovincial % Change in ESAL's		Canamex % Change in ESAL's		Canamex Short % Change in ESAL's	
	Short Term	Long Term	Short Term	Long Term	Short Term	Long Term
I-15	7.35	5.39	1.88	5.14	1.89	5.44
I-90	6.55	4.62	1.58	5.15	1.34	4.35
I-94	7.66	5.69	1.25	4.60	1.47	4.84
All Interstate	**7.05**	**5.10**	**1.61**	**5.06**	**1.54**	**4.81**
P-1	3.82	2.60	1.14	3.73	1.30	3.94
P-2	1.90	2.59	1.68	4.51	1.37	4.45
P-4	6.09	5.17	2.11	3.20	2.15	5.17
P-5	3.70	2.65	1.47	4.36	1.54	4.63
P-7	1.66	1.15	0.35	2.86	1.02	3.09
P-10	3.8	2.80	2.04	4.71	1.88	4.72
P-14	2.86	3.47	1.84	4.51	1.70	4.57
P-16	6.40	4.53	1.60	4.54	1.51	4.67
P-22	3.67	2.81	0.88	3.75	0.72	3.85
P-23	5.50	3.50	0.12	3.26	0.26	3.57
P-24	3.61	3.04	1.63	4.65	1.74	4.96
P-29	3.73	2.47	0.24	3.12	0.34	3.35
P-32	2.32	1.69	0.76	3.09	0.40	2.90
P-37	5.83	4.28	0.50	3.74	0.79	4.28
P-42	2.59	2.97	0.74	3.33	0.74	3.70
P-44	5.13	3.75	1.14	3.83	1.53	4.23
P-45	3.55	2.64	1.12	3.86	1.22	4.16
P-57	5.29	3.21	2.26	5.28	2.15	5.36
P-59	1.61	1.47	0.34	3.06	0.45	3.36
P-61	3.54	2.32	0.12	3.17	0.24	3.41
P-66	2.72	2.96	1.73	4.20	1.48	4.44
All Primaries	**3.84**	**2.89**	**1.24**	**4.00**	**1.23**	**4.23**
Total (Interstate and Primaries)	**4.82**	**3.34**	**1.31**	**4.19**	**1.26**	**4.33**

[a] route locations are shown in Figure 2.3.2-3

of heavy vehicle traffic (notably 3S2s) being significantly greater on the interstate system relative to the primary system.

Note that the uncertainty on the damageability of tridems is less critical in these various damage calculations for the Canamex scenarios relative to Canadian Interprovincial scenarios. The tridem is less attractive and less prevalent under Canamex limits compared to Canadian Interprovincial limits.

REMAINING LIFE/FUTURE OVERLAY PREDICTION METHODOLOGY

5.3.1 General Remarks - The reduction in the service life of existing pavements was estimated using a damage model based on the AASHTO design approach, which uses the ESAL concept (AASHTO, 1993). Traditionally, pavements have been designed to resist some total number of repetitions of load expressed in ESALs and some level of absolute maximum wheel load. The maximum wheel loads allowed under the size and weight scenarios considered herein remain unchanged, thus failure of the pavement in a single load event is no more or less likely than under current conditions. ESALs of demand will increase, however, as freight is loaded on heavier axles (Canadian Interprovincial limits) and existing axles can be loaded to heavier weights rather than being indirectly restricted in weight by the bridge formula (Canamex and Canamex Short limits). Thus, the total ESALs for which the pavement was designed will be reached sooner chronologically under the new scenarios compared to current conditions. Assuming acceptable operating conditions are restored by overlay, the overlay thickness required to provide a 20 year life will also be greater for the Canadian, Canamex, and Canamex Short limits then under existing conditions. These two features of the future pavement situation, that is, remaining life of the present pavement and overlay thickness to be used when it fails, were used to measure relative demands under Canadian Interprovincial, Canamex, and Canamex Short limits versus existing size and weight limits.

Remaining life and overlay calculations were performed independently for all six future traffic scenarios considered herein, even though three of the streams were developed to represent short term conditions. Consideration was given to combining the short and long term scenarios under each regulatory situation to develop a single composite picture of the future demands under each situation. The decision was made instead to consider each scenario in the long term sense to possibly bracket the actual solution and to obtain an indication of the sensitivity of the calculations to changes in ESALs of demand.

5.3.2 Pavement Performance Model - In predicting remaining pavement life, the basic AASHTO design equations were used in a fashion similar to that used by Deacon (1988) for the TRB study of truck size and weight (TRB, 1990a). The basic equation used by AASHTO for flexible pavement design is,

$$\log_{10}(W_{18}) = Z_R * S_o + 9.36 * \log_{10}(SN + 1)$$

$$- 0.20 + \frac{\log_{10}\left(\frac{\Delta\ P.S.I.}{4.2 - 1.5}\right)}{0.40 + \frac{1094}{(SN + 1)^{5.19}}}$$

$$+ 2.32 * \log_{10}(M_R) - 8.07$$

where,

W_{18} = total number of equivalent 18,000 pound single axle loads applied

Z_R = standard normal deviate (taken as -1.64 for this study)

S_o = combined standard error of traffic and performance prediction (taken as 0.45)

SN = structural number for the pavement section

Δ P.S.I. = change in present serviceability index over the design life of the pavement

M_R = resilient modulus

The above equation was subsequently modified following the work of Deacon (1988), to include environment effects. The assumption was made that the equation above specifically addressed pavement deterioration as function of traffic loading. Pavement life, however, is also consumed by non-load related phenomena which are often collectively labeled environmental effects. Environmental deterioration was incorporated in this model by assigning some of the change in serviceability in the above equation to environmental effects,

$$\Delta\ P.S.I = 4.5 - p_t - S_E$$

where

S_E = environmentally related loss in serviceability over life of the pavement

p_t = terminal serviceability

and an initial serviceability index of 4.5 has been assumed. The environmental loss in serviceability was expressed as a function of time, using a format proposed by Deacon,

$$S_E = k * (1 - e^{-0.07t})$$

where,

k = environmental constant adjusted to conditions in area of interest

t = elapsed time in years

The total life of each pavement section was estimated from the above equation using an iterative solution technique. For each successive year into the future, the ESALs accumulated to-date and the environmental PSI loss were calculated and substituted into the equation and a check made for equality. In calculating ESALs in any given year, past ESALS were accumulated based on data on the existing traffic stream; future ESALs, based on the estimates of the traffic stream determined above for the scenario under consideration. In performing these calculations, structural numbers were determined from actual roadway profile information provided by MDT. Effective resilient modulus values were also provided by MDT for most segments of highway.

Missing values were extrapolated from information available from adjacent segments. Layer coefficients were selected consistent with MDT design practice. Once total life was estimated for each segment, remaining life was simply calculated as,

$$t_R = T_W + t_T - T_A$$

where,

t_R = remaining life in years

T_W = year last worked on

t_T = total life in years

T_A = current year

The function of this performance model was cursorily checked along a few routes by comparing the present condition of the pavement as calculated by the model with the actual condition as determined in the pavement inspection program. The environmental deterioration model was adjusted based on these comparisons to bring model performance into better conformance with actual observed performance. Initially, the coefficient k on this expression was varied as a linear function of ESALs of demand, in an effort to incorporate some direct interaction between level of traffic and rate of accumulation of environmental damage. Eventually, a constant value of k of 1.95 was used. This environmental deterioration model is plotted in Figure 5.3.2-1. A typical relationship between the actual and the calculated PSI ratings obtained using this environmental model is presented in Figure 5.3.2-2. Referring to Figure 5.3.2-2, the model is doing an adequate, but not outstanding job predicting performance ($R^2 = 0.6$). A linear regression fit through the points in Figure 5.3.2-2 found a slope of 1.08 and an intercept of 0.08 (ideal values would be 1.00 and 0.00 for these parameters, respectively). While the model performance is only adequate, little bias toward either under or overpredicting performance is evident, and it was decided that reasonable results would be obtained in a system-wide analysis using this a model.

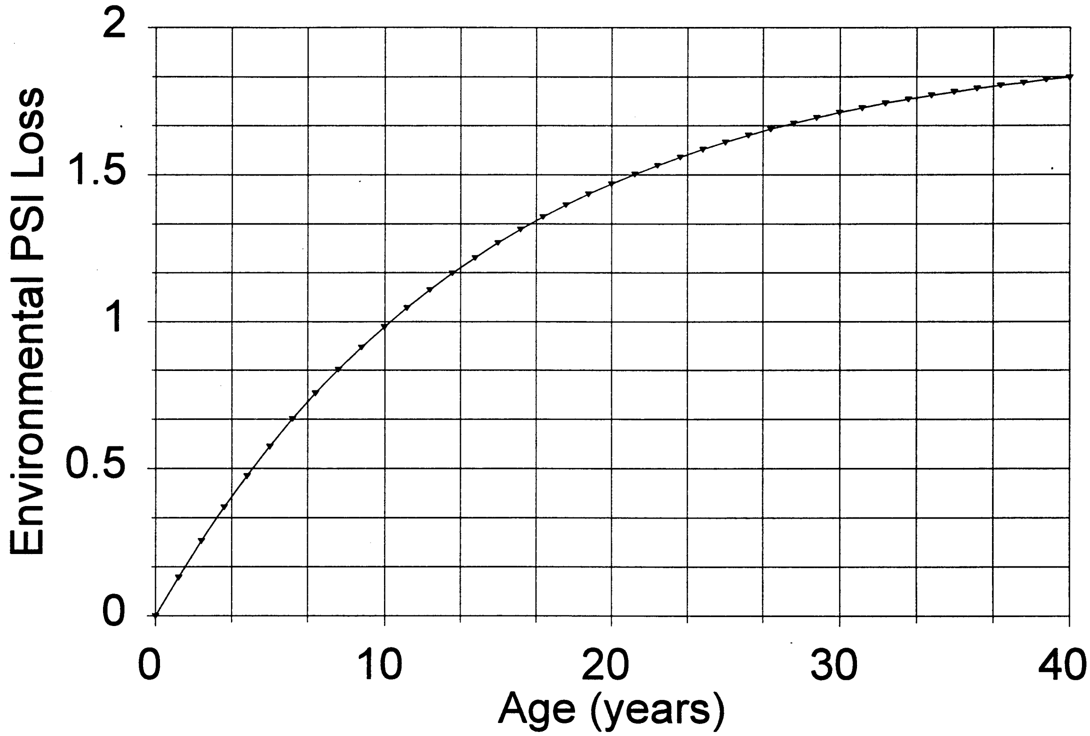


Figure 5.3.2-1 Environmental Deterioration Model

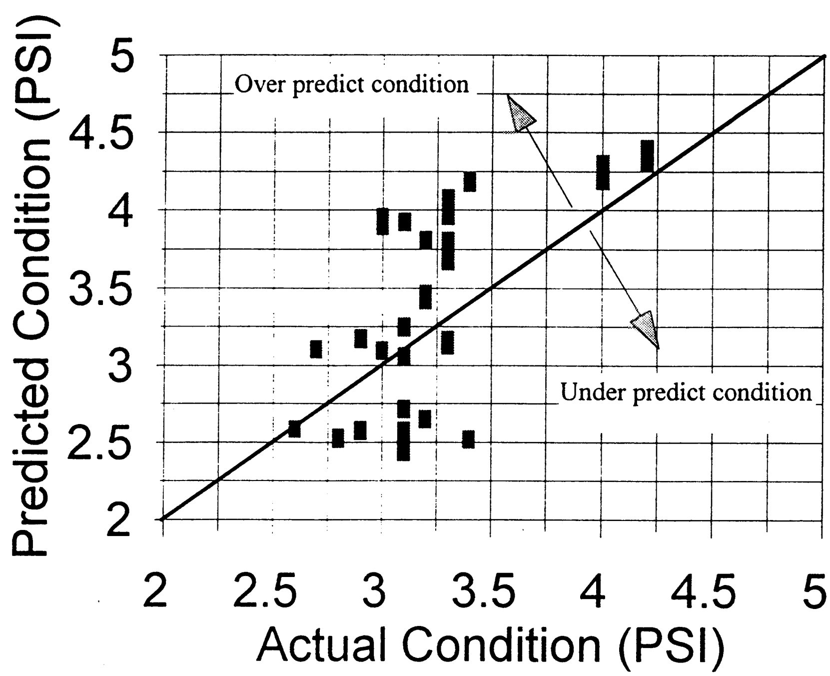


Figure 5.3.2-2 Actual and Predicted PSI Ratings

The change in remaining life under the future scenarios considered herein compared to under the current situation was typically less than 1 year for all scenarios considered. The thickness of the overlay required at the end of the remaining life of each pavement section required to provide 20 more years of service was calculated using the same performance model described above. For this case, the ESALs and environmental demands over the 20 year design life were known, the structural number required was calculated, and a pavement thickness to produce this structural number was determined. The average increase in overlay thickness only ranged from 0.3 to 1.5 percent (note that a minimum overlay thickness of 0.25 feet was enforced). Required overlay thicknesses were consistently larger for the new traffic streams relative to the existing traffic stream, as was expected based on the increased ESALs for these streams. Furthermore, as again was expected based on the relative ESALs of demand, overlay thicknesses under Canadian Interprovincial Limits were higher than those for and Canamex Short limits.

5.4 OBSERVED PAVEMENT PERFORMANCE

Vehicles running at full Canadian Interprovincial limits have been allowed to operate on a 36 mile section of Interstate in northern Montana for the past 5 years. The order of magnitude of the average number of vehicles operating on this section of roadway at full Canadian weights is less than 50 per day. Precise data on the volume of Canadian Interprovincial vehicles that have traversed this section of roadway, however, is unavailable. Accelerated deterioration beginning at the introduction of Canadian Interprovincial limits was only noted for one pavement section on that portion of the highway traveled by vehicles operating at full Canadian weights. This segment of roadway, however, has historically experienced accelerated deterioration due to poor subgrade conditions. Additional investigation of pavement performance in this area is underway.

5.5 EXPERIENCE IN CANADA

The effect of the adoption of Canadian Interprovincial limits in Canada on pavement deterioration is uncertain. TAC/CTRI (1994) comment in their study of the effects of the adoption of Canadian Interprovincial limits that the change is expected to have a neutral effect on pavements. In a survey conducted by TAC/CTRI (1994), no unusual impacts attributable directly to the adoption of Canadian Interprovincial limits were commented on by the various provinces. Some provinces did indicate in the survey that aging highways and increased volumes of truck traffic were resulting in pavement problems, but that these problems may have occurred independent of the adoption of Canadian Interprovincial limits. Alberta has experienced accelerated deterioration of its pavements since 1988, but it attributes this deterioration more to increased truck traffic rather than directly to Canadian Interprovincial vehicles (Zutatas, 1996). Alberta's structural design procedures for pavements were not changed based on the introduction of Canadian Interprovincial limits (TAC/CTRI, 1994). The increase in ESALs of demand in Alberta predicted prior to the adoption of Canadian Interprovincial limits was 5 to 10 percent (Khalil, 1996).

5.6 RESULTS OF OTHER STUDIES

Several studies have been conducted on the impact of changes in truck size and weight limits on pavement performance. As with bridges, one of the most pertinent studies conducted to-date was the 1990 TRB truck size and weight study (TRB, 1990a), in which the impact of adopting Canadian Interprovincial limits across the United States was addressed. As previously stated, the TRB study considered a Canamex Short type version of Canadian Interprovincial limits with a 51 kip tridem load compared to the 42.5 kip tridem load used in this study. By using the 51 kip tridem load, the TRB scenario in some respects is closer in make-up to the Canadian Interprovincial limits than to the Canamex scenario considered herein. The TRB scenario also made it attractive to operate 4 axle single units consisting of a single steering axle and tridem, which could operate at 71 kips under their scenario (compared to 62.5 kips under all the scenarios considered herein).

The TRB study predicted a 15.2 percent increase in ESAL-miles under their Canadian Interprovincial scenario. To some extent, the reference against which this increase is judged is different from that found in Montana, in that the TRB increase is with respect to conditions around the country under existing size and weight limits, which typically do not include use of the Uncapped Formula B to control allowable gross vehicle weights. This prediction can be crudely adjusted for the situation in Montana by subtracting off the predicted impact of adopting Uncapped Formula B nationwide, which was estimated in the TRB study as a 1.2 percent increase in ESAL miles. The subsequent predicted increase in ESAL-miles under Canadian Interprovincial Limits is 14 percent.

The change in pavement demands predicted in this study under the long term Canadian Interprovincial, Canamex, and Canamex Short limits were 3.3, 4.2, and 4.3 percent, respectively. These predictions are significantly lower than the TRB prediction. The TRB figure is a national average, as previously stated, and certainly some of this difference may be related to the specific transportation conditions in Montana compared to the rest of the country (notably with regard to level of commercial vehicle activity and likelihood of rail diversion). Additionally, the difference in tridem axle and 4 axle single unit load limits may be responsible for some of the observed differences in the results.

When first asked about the Canamex option, the immediate reaction of MDT pavement design engineers was that only a nominal change in pavement demand would occur, since axle loads remain unchanged under this scenario (presuming no rail diversion) (Galt, 1995). This reaction is consistent with the results obtained herein. The change in demand under Canamex is primarily related to (and is sensitive to) the amount of freight diverted off rail.

Further comparison of the results of this investigation with other studies is more difficult, in that each study begins and ends with different regulatory scenarios then those considered herein. A TRB truck size and weight study done in 1989 (TRB, 1989) investigated the effects on the highway system of changing configurations of certain vehicles within combination trucks. Note that the pavement impact methodology used in this study is the same as that used in the later TRB study referenced in the paragraph above. As part of this study, the change in ESALs of demand was calculated for switching 10 percent of the freight carried on all combination vehicles from configurations with an estimated average ESAL/ 100,000 pounds of freight carried of 2.4 (the value used in establishing the base case) to configurations with an average ESAL/100,000 pounds of freight carried of 6.5. The result was a change in ESALs of 10 percent. In this study, more freight is being shifted between vehicles (up to 38 percent on some vehicles) and some freight has been diverted from rail to truck (an additional 3.75 percent of the truck freight), but the configurations receiving the freight are only nominally less ESAL friendly then the vehicles losing it. Thus, upon closer inspection, the results obtained herein and those in this TRB study are consistent, based on the freight diversions being performed.

6. OTHER CONSIDERATIONS

6.1 GENERAL REMARKS

While the focus of this study was on the direct impact that the adoption of Canadian Interprovincial, Canamex, or Canamex Short limits on truck size and weight may have on pavements and bridges, other bridge and pavement related activities and highway features may also be affected by such a step. With regard to geometric features of the highway system, consideration needs to be given to lane width, curve geometry, intersection geometry, grades, length of merging lanes and passing zones, etc. With regard to bridges, inspection and detailed load rating analyses may need to be performed for some structures prior to the adoption of the new truck size and weight limits. After adoption of new limits, the inspection interval on some of the more vulnerable bridges may need to be decreased. With regard to pavements, consideration needs to be given to any increased maintenance requirements that may arise under Canadian Interprovincial, Canamex, and Canamex Short vehicles.

6.2 GEOMETRIC CONSIDERATIONS

<u>6.2.1 Lane Width and Intersection Geometry</u> - Adoption of Canadian Interprovincial, Canamex, or Canamex Short limits on vehicle size and weight will not require any special considerations with respect to lane width and intersection geometry beyond those considerations already made in the design process to accommodate existing vehicles. The basic width of vehicles allowed under both Canadian Interprovincial and existing Montana limits is 8.5 feet. Thus, adoption of Canadian Interprovincial limits will require no change in roadway width based simply on vehicle width. Required roadway widths at intersections and curves, however, is influenced by the handling characteristics of a vehicle in addition to its basic width. The ability of a vehicle to negotiate intersections and curves without encroaching on adjacent lanes is related to its offtracking characteristics. Offtracking is defined as the lateral deviation of the path of the steering axle compared to the path of the rearmost axle as a vehicle negotiates a turn (TRB, 1989). In low speed turns (speeds of approximately 40 mph and lower), the rearmost axle tends

to travel along a path inside the path of the steering axle. Low speed offtracking thus is a problem in intersections and on slow speed roads with curves. In high speed turns, the rearmost axle tends to travel along a path outside the path of the steering axle. Therefore, high speed offtracking is a problem on high speed roads with curves. Low speed offtracking is insensitive to vehicle weight; high speed offtracking increases with gross vehicle weight.

Canadian Interprovincial vehicle configurations were developed with target offtracking limits at both low and high speeds (RTAC, 1987). Low speed offtracking was restricted to 19.7 feet on a 90 degree turn at an outside radius of 36.1 feet. High speed offtracking was limited to 1.5 feet at a speed of 62 miles per hour on a curve with a radius of 1289 feet. Both of these requirements appear to be similar to the expected performance of a conventional 48 foot tractor, semi-trailer (based on descriptions of this performance given by TRB, 1989). The Canamex Short vehicles have the same geometry as Canadian Interprovincial vehicles and operate at a lower gross weight. The Canamex vehicles are similar in geometry to the Rocky Mountain doubles that already are allowed to operate in Montana under permit. The Canamex vehicles are 4 percent longer and 10 percent heavier than Rocky Mountain doubles.

6.2.2 Roadway Features Related to Vehicle Power - It is anticipated that if Canadian Interprovincial, Canamex, or Canamex Short limits are adopted, existing vehicles will be used to transport increased weight without modification of the power units. Therefore, vehicle speeds on grades and general acceleration rates will decrease. Slow moving Canadian, Canamex, and Canamex Short vehicles on grades could increase the demand for passing lanes on two-lane roadways. The reduction in acceleration could affect the adequacy of existing merging lanes as the new vehicles attempt to enter high speed facilities. The reduction in acceleration could also necessitate increased sight distances at intersections, curves, and changes in grade, as vehicles approaching these features encounter slow moving vehicles. Design sight distances at intersections in Alberta were increased by 30 percent following adoption of Canadian Interprovincial limits both to accommodate long combination vehicles and 82 foot B-trains (TAC/CTRI, 1994).

Existing vehicle regulations in Montana do not directly address the problems described above. It is anticipated that these various problems will be less severe for Canamex and Canamex Short vehicles compared to Canadian Interprovincial vehicles, as might be expected based on their relative allowable gross vehicle weights. The heavy combination vehicles allowed under Canadian Interprovincial and Canamex Short limits are shorter than the long combination vehicles already allowed in Montana, which should mitigate some of these power related effects. The Canamex vehicles, however, will be able to carry up to 10 percent more weight than is presently allowed on existing vehicles with similar geometry, and power related effects will merit further consideration. This situation can possibly be addressed by legislating minimum weight to power ratios. Pending further investigation of these issues, no impact to the roadway was assessed based on vehicle power.

6.3 BRIDGE ANALYSIS AND INSPECTION

Many bridges will have to be analyzed and possibly inspected prior to the adoption of either Canadian Interprovincial or Canamex limits. These analyses should be performed at the discretion of the MDT bridge engineers. The number of bridges requiring detailed load rating analysis was estimated in this study based on expected overstress level. All bridges stressed at and above 87 percent of their Allowable Stress Based Operating Rating were considered as obvious candidates for analysis. This level of demand is approximately midway between Allowable Stress based Inventory and Operating ratings. Montana has employed standard designs for both timber and prestressed concrete structures for many years, which may expedite these analyses. Field inspection and possibly field testing of critically stressed bridges may be prudent prior to formulating a final decision on their disposition. A significant increase (15 percent average, 9 percent minimum) was found in the load ratings of the bridges tested as part of this study.

6.4 PAVEMENT MAINTENANCE

The assumption was made in this investigation that the impact of Canadian Interprovincial, Canamex, and Canamex Short vehicles on pavements would be considered through the reduction in remaining pavement life rather than the increase in maintenance activities necessary to obtain the same life under the new traffic streams. The increases in traffic demand predicted in Section 5 of this report were judged to be not so severe as to expect a dramatic acceleration in damage associated with the interaction between environmental and traffic effects (such an acceleration in total damage with increase in traffic level has been qualitatively observed at several locations (e.g., Hudson and Flanagan,1987)).

7. COST IMPACT

7.1 GENERAL REMARKS

The cost impacts on the highway system of adopting Canadian Interprovincial, Canamex, and Canamex Short limits on vehicle size and weight were determined by assessing the costs of the various physical impacts identified above. These costs were calculated and expressed as equivalent uniform annual costs (EUAC) for each scenario. A dollar value was assigned to the changes necessitated by the adoption of the new limits based on the present cost of similar work. These estimates were adjusted for inflation and projected return on investment, with due consideration for (a) the remaining life of existing facilities under present and proposed truck weight limits and (b) the design requirements and design life of new facilities. Both total and incremental pavement and bridge costs associated with adopting Canadian Interprovincial, Canamex Short, and Canamex limits were calculated. A gross estimate of changes in user cost responsibilities associated with the new size and weight limits was determined by assigning the incremental costs associated with each scenario to the new vehicles in the traffic stream.

7.2 BRIDGE COSTS

7.2.1 General Remarks - The cost of the engineering impacts on the bridge system if Canadian Interprovincial or Canamex limits are adopted include the costs of:

1) detailed load rating analyses of selected bridges required before adoption of the new limits, to definitively establish those bridges that require replacement,
2) immediate bridge replacement required due to inadequate strength, as identified above,
3) increased frequency of inspections of selected bridges after adoption of the new limits, and
4) long term fatigue damage (steel bridges).

For this preliminary analysis, the deficient bridges that require immediate replacement were assumed to be those bridges identified in the simple network wide analysis as deficient at full

Allowable Stress based Operating ratings (lower bound on costs) and those identified as deficient at 87 percent of these ratings (upper bound on costs). Bridge replacement costs were generally the largest part of the total bridge costs associated with each size and weight scenario.

7.2.2 Cost of Detailed Load Rating Analyses - The expected cost of load rating analyses recommended to be performed prior to the adoption of Canadian Interprovincial, Canamex, or Canamex Short limits is summarized in Table 7.2.2-1. The decision was made that all bridges stressed under Canadian Interprovincial, Canamex, Canamex Short or the HS20 design vehicle to 87 percent (or more) of their capacity at their Allowable Stress based Operating rating should undergo a detailed load rating analysis. These load rating analyses should serve as the decision mechanism by which the need for actual bridge replacement is verified. While MDT is in the process of streamlining and semi-automating its load rating procedures, it was estimated that following current procedures an engineer could, on the average, spend 4 to 8 hours load rating a bridge (Murphy, 1996). Eight hours per load rating was selected for this study. The load rating costs presented in Table 7.2.2-1 reflect this estimate. While this cost was applied against each bridge, Montana has historically employed standard designs in timber and prestress spans, which may result in some cost savings on these types of structures. Any such cost savings will be offset by increased expenditures on analyzing continuous bridges, which constitute a major portion of the bridges to be reviewed.

Table 7.2.2-1 Bridge Load Rating Costs

System	Load Rating Cost, Millions of 1996 Dollars		
	Canadian Interprovincial	Canamex	Canamex Short
Interstate	0.11	0.01	0.02
Primary	0.32	0.31	0.32
Secondary	0.17	0.15	0.16
Urban	0.01	0.01	0.01
Total	**0.61**	**0.48**	**0.51**

7.2.3 Bridge Replacement - A summary of the cost to immediately replace the bridges found to be deficient based on strength under each scenario (as simply estimated for this study from the network-wide analysis performed in Section 4) is presented in Table 7.2.3-1. Based on the earlier discussion of the engineering impacts on the bridge system of adopting Canadian Interprovincial, Canamex, and Canamex Short limits, the decision was made to present costs for two levels of assumed capacity, full Allowable Stress based Operating ratings and 87 percent of these ratings. Actual costs to replace all currently deficient bridges (under HS20) and to upgrade those bridges which become deficient under the new scenarios were expected to fall between those predicted using full Allowable Stress based Operating ratings and those determined using 87 percent of those ratings. In calculating the costs presented in Table 7.2.3-1, it was assumed that strengthening existing bridges was not an option. Increasing the load carrying capacity of structures can be accomplished by reducing the dead load (e.g., by using light weight decks) and/or strengthening members. Such options can be both awkward and expensive to implement, depending on the structural system and material (Murphy, 1996). Therefore, the conservative assumption of complete replacement was selected over strengthening. Note that this approach is probably overly conservative for the situation herein, in that a high percentage of the deficient bridges are of steel construction, the one type of structure that can possibly be upgraded to carry increased demands. Many of the steel bridges in Alberta found to be deficient when Canadian Interprovincial limits were adopted, for example, were strengthened (at nominal cost) rather than replaced (Moroz, 1996). The further assumption was made that all spans of a structure would be replaced at a width of 40 feet. In simple span structures it may be feasible to only replace the deficient spans (presuming the remaining spans meet current geometric standards).

Bridge replacement costs were simply calculated using a unit cost of $109 per square foot for the Canadian Interprovincial, Canamex, and Canamex Short limits, and $105 per square foot for the HS20 design vehicle. The average unit cost used by MDT to estimate bridge replacement costs is $105 per square foot. This cost includes both external contract and internal MDT costs. This cost was increased by 3.5 percent for Canadian Interprovincial, Canamex, and Canamex Short limits to accommodate an increase in design standard for new bridges from HS20-44 to HS25-44. The relative magnitude of this cost increase was estimated from work done by Moses

Table 7.2.3-1 Total Bridge Replacement Costs

System	Immediate Replacement Cost, Millions of 1996 Dollars			
	Canadian Interprovincial	Canamex	Canamex Short	HS-20
Allow Stress based Operating Ratings				
Interstate	3.7	0.0	0.0	0.0
Primary	246.8	144.4	146.4	101.1
Secondary	135.5	49.9	48.8	43.0
Urban	11.8	11.3	11.3	11.3
Total	**397.8**	**205.6**	**206.5**	**155.4**
87 Percent of Allow Stress based Operating Ratings				
Interstate	310.2	29.8	54.5	0.0
Primary	498.1	286.1	305.2	234.4
Secondary	228.1	154.1	171.4	101.7
Urban	58.7	18.4	39.3	18.4
Total	**1095.1**	**488.4**	**570.4**	**354.5**

for the Ohio Department of Transportation (Moses, 1992) and others (Weissman, Reed, and Feroze, 1994). The new design level of HS25 was selected as representative of the increase in moment demand placed on bridges across the system by Canadian Interprovincial vehicles. Canadian Interprovincial vehicles produce a 25 percent increase in live load demand relative to the HS20 demands on simply supported structures at a span length of 75 feet. The increase in negative moment in continuous structures under the new vehicles was also found to be less than or equal to 20 to 25 percent for a majority of structures under all scenarios.

Suggesting an increase in design demands for new bridges while judging the majority of existing bridges (which were designed under lower demands) as adequate, may seem inconsistent. Most existing bridges in Montana, however, were designed using relatively simple procedures that applied global factors of safety to account for a variety of behaviors and load situations not explicitly considered in the analysis. Those design procedures have been refined and new design procedures have been introduced that explicitly consider many of these behaviors and load situations in the design process. Therefore, the "reserve" capacity that some structures designed using older procedures may possess in any given situation is explicitly taken into account following new design procedures, and such procedures should be applied using the actual expected demands.

Referring to Table 7.2.3-1, the lowest cost impacts are for Canamex and Canamex Short limits using full Allow Stress based Operating ratings. The total cost estimate for these scenarios and level of assumed capacity was approximately 205 million dollars, with no cost impact projected for the interstate system. The cost to immediately replace all the bridges deficient to carry Canadian Interprovincial limits using full Allowable Stress based Operating ratings was estimated at 498 million dollars, with only a 3.7 million dollar impact projected on the interstate system. Costs climb dramatically for the primary system under all vehicle scenarios considered in these analyses. This increase in costs is a reflection of both the higher number of bridges and the higher percentage of reduced capacity bridges on the primary system compared to the interstate system. The costs to upgrade the primary system for Canadian Interprovincial, Canamex, and Canamex Short vehicles were estimated at 247, 143, and 146 million dollars, respectively, using full Allowable Stress based Operating ratings. Estimated costs to upgrade the secondary and urban systems are lower in magnitude than those for the primary system in approximate proportion to the number of bridges on each system.

Estimated cost impacts increased significantly when bridge capacity was assumed at 87 percent of Allowable Stress based Operating ratings rather than at full Allowable Stress based Operating levels. The cost to immediately upgrade the interstate system to carry Canadian Interprovincial vehicles was estimated at 310 million dollars, compared to just 3.7 million dollars based on full Allowable Stress based Operating ratings. Corresponding costs for the Canamex and Canamex Short limits were approximately 80 to 90 percent less than those for the Canadian Interprovincial limits (29.8 and 54.5 million dollars, respectively). Costs to upgrade the primary system at the 87 percent capacity level increased approximately 100 percent compared to those obtained using full Allowable Stress based Operating ratings, to 498, 286, and 305 million dollars for Canadian Interprovincial, Canamex, and Canamex Short vehicles, respectively.

These cost results again reflect the sensitivity of the underlying engineering analyses to both the level of imposed demand and level of assumed capacity. While Canadian Interprovincial vehicles impose only 10 to 15 percent greater total demand (dead load plus live load) on bridges than Canamex and Canamex Short vehicles, their cost impact is approximately 100 percent greater than that of Canamex and Canamex Short vehicles. Similarly, assuming a 13 percent lower capacity for bridges (87 percent of Operating rating) generated a 140 to 175 percent increase in total cost impact for all three scenarios.

The costs discussed above are total costs to upgrade all deficient bridges to carry Canadian Interprovincial, Canamex, and Canamex Short vehicles, which includes bridges that are already deficient with respect to the HS20-design vehicle. Incremental bridge costs associated with the adoption of Canadian Interprovincial, Canamex, and Canamex Short loads were estimated simply by subtracting the HS20-44 costs from the total replacement costs for each scenario. These incremental costs are reported in Table 7.2.3-2. Broad trends in these incremental costs are consistent with trends in total costs. That is, significantly lower costs are associated with Canamex and Canamex Short vehicles compared to Canadian Interprovincial vehicles (from one-third to one-fifth lower) and significantly lower costs are associated with using full Allowable Stress based Operating ratings compared to 87 percent of these ratings (from one-third to one-quarter lower). Costs for Canamex vehicles are nominally 0 to 50 percent less than costs anticipated for Canamex Short vehicles.

Zero incremental costs were calculated for Canamex and Canamex short vehicles operating on the interstate system using full Allowable Stress based Operating ratings. Incremental costs were generally low across all systems for Canamex and Canamex Short vehicles under full operating ratings, with a total estimated incremental cost impact of approximately 50 million dollars for both scenarios. The corresponding total incremental cost impact for Canadian Interprovincial vehicles was 242 million dollars. At 87 percent load ratings, incremental costs increased dramatically (by factors of 3 to 4), as was observed for total replacement costs, and the variation in costs between size and weight scenarios widened.

<u>7.2.4 Cost of Increased Bridge Inspections</u> - Consideration was given to possibly increasing the frequency of bridge inspections due to the accelerated demands of Canadian Interprovincial,

Table 7.2.3-2 Incremental Bridge Replacement Costs, Above and Beyond the Costs to Replace Bridges Deficient Under HS20-44

System	Incremental Replacement Costs, Millions of 1996 Dollars		
	Canadian Interprovincial	Canamex	Canamex Short
Allow Stress based Operating Ratings			
Interstate	3.7	0.0	0.0
Primary	145.7	43.3	45.3
Secondary	92.5	6.9	5.8
Urban	0.5	0.0	0.0
Total	**242.4**	**50.2**	**51.1**
87 Percent of Allow Stress based Operating Ratings			
Interstate	310.2	29.8	54.5
Primary	263.7	51.7	70.8
Secondary	126.4	52.4	69.7
Urban	40.3	0.0	20.9
Total	**740.6**	**133.9**	**215.9**

Canamex, and Canamex Short vehicles on bridges. After a discussion with MDT bridge personnel (Murphy, 1996), it was concluded that the present inspection interval was adequate in light of the low traffic volumes of Montana's highways. MDT is considering reviewing and revising the bridge inspection schedule around the state to replace arbitrary inspection intervals with a rational inspection schedule based on site specific conditions such as type of bridge, physical condition, level of traffic, etc.

7.2.5 Fatigue Costs - Fatigue costs are difficult to quantify due to the underlying uncertainty in the amount of expected fatigue damage. The calculations performed herein certainly indicate that traffic streams containing Canadian Interprovincial, Canamex, and Canamex Short vehicles will place increased fatigue demands on the bridge system. The calculated maximum average increase in the fatigue demands on the interstate and primary systems (combined) ranged from 7.3 to 31.3 percent. If the controlling demand on the bridge design was not fatigue, it may have considerable fatigue capacity and a long fatigue life despite these increases in demand. The decision was made not to assign any difference in future costs against the various scenarios based on fatigue demand. In the 1990 TRB study of truck size and weight limits (TRB, 1990a), fatigue accounted for only 3 percent of all bridge related costs attributable to the adoption of Canadian Interprovincial limits.

7.3 PAVEMENT COSTS

7.3.1 Cost Calculation - The impact on pavement costs of the adoption of Canadian Interprovincial, Canamex, and Canamex Short limits is related to the reduction in service life of the existing pavement and the cost of the subsequent overlays required to provide 20 years of additional service. As was done in assessing pavement impacts, pavement costs were assessed for all six scenarios (short and long term) in the long term sense to obtain a range of cost impacts across a variety of demand levels. These costs were calculated for the entire interstate system and typical routes on the primary system for each traffic scenario using the remaining life and overlay thickness information generated in Section 5 of this report. The results of these analyses were then extrapolated as necessary across the entire system. In each case, the cost of the required overlay was calculated in terms of 1996 dollars and then adjusted to the actual cash flow. The results obtained for each scenario were then expressed in terms of an equivalent uniform annual cost (EUAC).

Overlay costs in 1996 dollars were calculated using generic cost information provided by MDT (see Table 7.3.1-1) multiplied by a factor of 1.5 to cover internal MDT costs incurred on each project. Overlay cost was expressed as an EUAC using the equation,

$$EUAC = P_{OL} \frac{i\,(1 + i)^{-n_{RL}}}{(1 + i)^{n_{OL}} - 1} \; \frac{(1 + i)^{n}}{(1 + i)^{n} - 1}$$

where,

P_{OL} = present cost of overlay

n_{RL} = remaining life of present pavement

n_{OL} = design life of overlay (assumed = 20 years)

n = total number of periods being considered (assumed = 75 years)

i = discount rate (assumed = 7 percent)

Table 7.3.1-1 Basic Cost Data for Overlays (adapted from information from Wissinger, 1995)

Item	Unit Cost (dollars/ton)
Bituminous Asphalt	240.00
Aggregate	10.00
Placement Cost	10.25
Sealer	215.00
Cover Aggregate	20.00
Mobilization	6 % of total cost
Traffic control	4 % of total cost
Miscellaneous	14 % of total cost

The EUAC values calculated for each route are summarized in Table 7.3.1-2. The costs presented in Table 7.3.1-2 are total EUAC for each scenario considered. The costs for all scenarios are similar in magnitude and are around 100 million dollars. Incremental costs

Table 7.3.1-2 Total EUAC of Overlays

Route[a]	Canadian Interprovincial Limits		Canamex Limits		Canamex Short Limits		Existing Limits
	Short term	Long Term	Short Term	Long Term	Short Term	Long Term	
I-15	14.00	13.94	13.84	13.94	13.83	13.94	13.78
I-90	19.70	19.61	19.30	19.65	19.29	19.61	19.24
I-94	10.02	10.00	9.85	9.96	9.85	9.95	9.84
Interstate	**43.72**	**43.55**	**42.99**	**43.55**	**42.97**	**43.50**	**42.86**
P-1	6.40	6.37	6.31	6.40	6.30	6.41	6.27
P-2	0.66	0.67	0.67	0.67	0.66	0.67	0.66
P-4	0.66	0.66	0.66	0.66	0.66	0.66	0.64
P-5	2.32	2.32	2.31	2.33	2.31	2.33	2.30
P-7	1.67	1.66	1.66	1.67	1.66	1.67	1.65
P-10	1.88	1.87	1.87	1.88	1.87	1.88	1.86
P-14	1.97	1.98	1.99	1.99	1.97	1.99	1.96
P-16	0.87	0.86	0.85	0.86	0.85	0.86	0.84
P-22	0.47	0.47	0.47	0.47	0.47	0.47	0.47
P-23	1.88	1.88	1.85	1.85	1.85	1.87	1.85
P-24	1.11	1.11	1.09	1.12	1.09	1.12	1.09
P-29	1.58	1.57	1.57	1.58	1.57	1.58	1.56
P-32	0.32	0.32	0.32	0.32	0.32	0.33	0.32
P-37	1.52	1.52	1.49	1.51	1.49	1.52	1.49
P-42	0.32	0.32	0.32	0.32	0.32	0.32	0.32
P-44	0.24	0.24	0.24	0.24	0.24	0.24	0.24
P-45	0.16	0.16	0.16	0.16	0.16	0.16	0.16
P-57	3.78	3.76	3.74	3.78	3.74	3.78	3.70
P-59	0.23	0.23	0.23	0.23	0.23	0.23	0.22
P-61	1.50	1.49	1.47	1.49	1.47	1.49	1.48
P-66	0.15	0.15	0.15	0.15	0.15	0.15	0.15
Primaries	**55.81**	**55.66**	**55.25**	**55.84**	**55.24**	**55.91**	**54.97**
Total	**99.53**	**99.21**	**98.24**	**99.39**	**98.21**	**99.41**	**97.83**
Normalized by Existing	1.017	1.014	1.004	1.016	1.004	1.016	1.000

[a] locations of routes shown on Figure 2.3.2-3

associated with the adoption of Canadian Interprovincial, Canamex, and Canamex Short limits, presented in Table 7.3.1-3, were simply calculated by subtracting cost predictions for the existing traffic stream from cost predictions for the other scenarios.

Referring to Table 7.3.1-3, future pavement costs increased for all scenarios with respect to the predicted conditions under the existing traffic stream (as was expected based on the increase in ESALs of demand for all scenarios). These predicted cost increases, however, are all less than 3 percent. The maximum EUAC increase of 1.7 million dollars or 1.7 percent is for the short term scenario under Canadian Interprovincial limits for which a 4.8 percent increase in ESALs of demand was predicted. The minimum cost increase of 0.38 million dollars (corresponding to less than 0.4 percent increase) is for the short term scenario under Canamex limits. The long term scenarios may provide a better indication of true costs, as traffic is expected to approach long term conditions in just a few years. The increase in EUACs are 1.38, 1.56, and 1.58 million dollars, respectively for the long term Canadian Interprovincial, Canamex, and Canamex Short scenarios. These values represent increases of 1.4, 1.6, and 1.6 percent, respectively, over comparable costs under the existing traffic stream.

Table 7.3.1-3 Incremental Overlay Costs

System	Increase in Overlay Costs by Scenario, Millions of 1996 dollars											
	Canadian Interprovincial				New Canamex				Canamex			
	Short Term		Long Term		Short Term		Long Term		Short Term		Long Term	
	$	%	$	%	$	%	$	%	$	%	$	%
Interstate	0.86	2.0	0.69	1.6	0.13	0.30	0.69	1.6	0.11	0.3	0.64	1.5
Primary	0.84	1.5	0.69	1.3	0.28	0.51	0.87	1.6	0.27	0.5	0.94	1.7
Total	1.70	1.7	1.38	1.4	0.41	0.42	1.56	1.6	0.38	0.4	1.58	1.6

The percentage change in cost is consistently less than the percentage change in ESALs of demand, as has been observed by other investigators (Khalil, 1996; Deacon, 1988). The marginal change in costs relative to increase in ESALs of demand is greater for the primary system than for the interstate system. Thus, the increase in ESALS of demand is more readily accommodated on the stronger interstate pavements compared to the primary pavements.

7.4 CHANGE IN COMBINED PAVEMENT AND BRIDGE COSTS

7.4.1 Absolute Costs - The combined bridge replacement and overlay costs associated with adopting Canadian Interprovincial, Canamex, or Canamex Short limits are presented in Tables 7.4.1-1 and 7.4.1-2. Combined cost calculations were only performed for the interstate and primary systems, as the data necessary to perform pavement calculations for the secondary and urban systems were unavailable. Combined costs were calculated as EUAC. The various lump sum bridge costs calculated above were converted to EUACs assuming a 75 year life and a discount rate of 7 percent,

$$EUAC = P_{BR} \frac{i\,(1 + i)^n}{(1 + i)^n - 1}$$

where,

P_{BR} = present cost of bridge replacement

i = discount rate (7 percent assumed)

n = number of time periods (75 years assumed)

These costs were then added to the pavement EUAC calculated above to obtain the combined cost for each scenario.

Differences in total costs observed in Table 7.4.1-1 and 7.4.1-2 generally reflect differences between bridge costs under the various scenarios considered in this study, as pavement costs were similar in all cases. Thus, higher costs were observed (a) for Canadian Interprovincial compared to Canamex and Canamex Short vehicles, and (b) for a bridge capacity of 87 percent of Allowable Stress based Operating capacity compared to full Allowable Stress based Operating capacity. Relative differences in costs between scenarios and between using different assumptions on bridge capacity decreased when combined costs were considered rather than just bridge costs. The relatively large magnitude of the pavement cost contribution to total costs, which was fairly constant across all scenarios and independent of bridge capacity assumptions, tended to force some uniformity in total costs across all scenarios. The maximum difference in total costs across all situations was 45 percent compared to a maximum difference in bridge costs across all situations of 460 percent.

Table 7.4.1-1 Total EUAC of Bridges and Overlays for Each Future Scenario, Full Allowable Stress Based Operating Ratings

Item	Total Equivalent Uniform Annual Cost, Millions of Dollars						
	Canadian Interprovincial		Canamex		CanamexShort		Existing Limits
	Short Term	Long Term	Short Term	Long Term	Short Term	Long Term	
Interstate							
Bridges	0.3	0.3	0.0	0.0	0.0	0.0	0.0
Overlays	43.7	43.6	43.0	43.5	43.0	43.5	42.9
Total	**44.0**	**43.9**	**43.0**	**43.5**	**43.0**	**43.5**	**42.9**
Primary							
Bridges	17.3	17.3	10.1	10.1	10.2	10.2	7.1
Overlays	55.8	55.7	55.3	55.8	55.2	55.9	55.0
Total	**73.1**	**73.0**	**65.4**	**65.9**	**65.4**	**66.1**	**62.1**
Total							
Bridges	17.6	17.6	10.1	10.1	10.2	10.2	7.1
Overlays	99.5	99.3	98.3	99.3	98.2	99.4	97.9
Total	**117.1**	**116.9**	**108.4**	**109.4**	**108.4**	**109.6**	**105.0**

Table 7.4.1-2 Total EUAC of Bridges and Overlays for Each Future Scenario, 87 Percent of Full Allowable Stress Based Operating Ratings

Item	Equivalent Uniform Annual Cost, Millions of 1996 dollars						
	Canadian Interprovincial		Canamex		Canamex Short		Existing Limits
	Short Term	Long Term	Short Term	Long Term	Short Term	Long Term	
Interstate							
Bridges	21.7	21.7	2.1	2.1	3.8	3.8	0.0
Overlays	43.7	43.6	43.0	43.5	43.0	43.5	42.9
Total	**65.4**	**65.3**	**45.1**	**45.6**	**46.8**	**47.3**	**42.9**
Primary							
Bridges	34.9	34.9	20.0	20.0	21.4	21.4	16.4
Overlays	55.8	55.7	55.3	55.8	55.2	55.9	55.0
Total	**90.7**	**90.6**	**75.3**	**75.8**	**76.6**	**77.3**	**71.4**
Total							
Bridges	56.6	56.6	22.1	22.1	25.2	25.2	16.4
Overlays	99.5	99.3	98.3	99.3	98.2	99.4	97.9
Total	**156.1**	**155.9**	**120.4**	**121.4**	**123.4**	**124.6**	**114.3**

The lowest EUAC for the primary and interstate systems combined was 108 million dollars for the short term Canamex and Canamex Short scenarios under full Allowable Stress based Operating ratings. The corresponding EUAC under Canadian Interprovincial Limits was 117 million dollars. Once again, results for the long term scenarios may be more indicative of actual costs, as short term conditions are expected to exist for only a few years. The total EUAC for the primary and interstate systems combined were approximately 117 and 109 million dollars for the long term Canadian Interprovincial and the long term Canamex and Canamex Short scenarios, respectively. Higher combined costs were determined using 87 percent of full Allowable Stress based Operating ratings for bridge capacity relative to using full Allowable Stress based Operating ratings for bridge capacity, due to the increase in bridge deficiencies. The total combined EUAC for the primary and interstate systems reached 156, 121, and 125 million dollars, respectively, for the Canadian Interprovincial, Canamex, and Canamex Short scenarios.

Bridge costs were only from 10 to 15 percent of total costs when full Allowable Stress based Operating ratings were used to represent bridge capacity. The contribution of bridge costs to total costs increased to 20 to 35 percent, when 87 percent of full Allowable Stress based Operating ratings are used for bridge capacity. In either case (and as might be expected), these results indicate that new proposals that significantly affect pavement costs will have greater impact on total highway costs than proposals that impact only bridge costs.

7.4.2 Incremental Costs - The increase in incremental EUAC associated with adopting Canadian Interprovincial and Canamex limits are presented in Tables 7.4.2-1 and 7.4.2-2 for the primary and interstate systems. These incremental costs were calculated simply by subtracting the total costs projected under current size and weight limits from those projected under Canadian Interprovincial, Canamex, and Canamex Short limits. Referring to Tables 7.4.2-1 and 7.4-2-2 bridge costs generally formed the majority of the incremental EUAC system wide. Only under full Operating ratings and on the interstate system do incremental pavement costs exceed bridge costs for any scenario. In most cases, incremental bridge costs represented over 75 percent of total incremental costs. With bridge costs driving total incremental costs, significant differences again emerged between the costs of the various scenarios. Highest incremental costs were consistently observed for the Canadian Interprovincial limits, with the incremental costs for the

Table 7.4.2-1 Incremental EUAC of Bridges and Overlays for Each Future Scenario, Full Allowable Stress Based Operating Ratings

Item	Incremental Equivalent Uniform Annual Cost, Millions of 1996 dollars											
	Canadian Interprovincial				Canamex				Canamex Short			
	Short Term		Long Term		Short Term		Long Term		Short Term		Long Term	
	$	%	$	%	$	%	$	%	$	%	$	%
Interstate												
Bridges	0.3	-	0.3	-	0.0	0	0.0	0	0.0	0	0.0	0.0
Overlays	0.8	2	0.7	2	0.1	0	0.6	1	0.1	0	0.6	1
Total	**1.1**	**3**	**1.0**	**2**	**0.1**	**0**	**0.6**	**1**	**0.1**	**0**	**0.6**	**1**
Primary												
Bridges	10.2	142	10.2	142	3.0	42	3.0	42	3.1	44	3.1	44
Overlays	0.8	2	0.7	1	0.3	1	0.8	2	0.2	0	0.9	2
Total	**11.0**	**18**	**10.9**	**18**	**3.3**	**5**	**3.8**	**6**	**3.3**	**5**	**4.0**	**6**
Total												
Bridges	10.5	148	10.5	148	3.0	42	3.0	42	3.1	44	3.1	44
Overlays	1.6	2	1.4	1	0.4	0.4	1.4	1	0.3	0	1.5	2
Total	**12.1**	**12**	**11.9**	**11**	**3.4**	**3.2**	**4.4**	**4**	**3.4**	**3**	**4.6**	**4**

Table 7.4.2-2 Incremental EUAC of Bridges and Overlays for Each Future Scenario, 87 Percent of Full Allowable Stress Based Operating Ratings

Item	Incremental Equivalent Uniform Annual Cost, Millions of 1996 dollars											
	Canadian Interprovincial				Canamex				Canamex Short			
	Short Term		Long Term		Short Term		Long Term		Short Term		Long Term	
	$	%	$	%	$	%	$	%	$	%	$	%
Interstate												
Bridges	21.7	-	21.7	-	2.1	-	2.1	-	3.8	-	3.8	-
Overlays	0.8	2	0.6	1	0.1	0	0.6	1	0.1	0	0.6	1
Total	**22.5**	**52**	**22.3**	**52**	**2.2**	**5**	**2.7**	**6**	**3.9**	**9**	**4.4**	**10**
Primary												
Bridges	18.5	113	18.5	113	3.6	22	3.6	22	5.0	30	5.0	30
Overlays	0.8	2	0.7	1	0.3	1	0.8	2	0.2	0	0.9	2
Total	**19.3**	**27**	**19.2**	**27**	**3.9**	**5**	**4.4**	**6**	**5.2**	**7**	**5.9**	**8**
Total												
Bridges	40.2	245	40.2	245	5.7	35	5.7	35	8.8	54	8.8	54
Overlays	1.6	2	1.3	1	0.4	0	1.4	1	0.3	0	1.5	2
Total	**41.8**	**37**	**41.5**	**36**	**6.1**	**5**	**7.1**	**6**	**9.1**	**8**	**10.3**	**9**

Canamex and Canamex Short limits being similar in magnitude to each other and significantly lower than Canadian Interprovincial costs. Long term incremental costs of 12 and 5 million dollars, respectively, were calculated for Canadian Interprovincial and for Canamex and Canamex Short limits using full Allowable Stress based Operating ratings for bridge capacity. These costs represent increases of 11 and 4, percent, respectively, compared to projected bridge and overlay costs under existing weight limits.

Long term incremental costs increased significantly when 87 percent of Allowable Stress based Operating ratings were used to represent bridge capacity. Incremental costs of 42, 7, and 10 million dollars were estimated for Canadian Interprovincial, Canamex, and Canamex Short vehicles, respectively, at this bridge capacity level. These costs correspond to increases of 36, 6 and 9 percent in future bridge and pavement expenditures above those estimated for existing weight limits. These results again reflect the sensitivity of these analyses to assumed bridge capacity and the specific demands of the scenarios under investigation. Using 87 percent of Operating capacity versus full Operating capacity (a change of only 13 percent) increased projected costs by 61 (Canamex) to 250 percent (Canadian Interprovincial). Based on the manner in which capacity has been estimated and costs have been calculated, actual costs are expected to range between the estimates for 87 percent of full Operating ratings and full Operating ratings.

7.5 USER COST ALLOCATION

Analyses were performed to allocate the increased (incremental) pavement and bridge costs identified in this study to the new vehicles in the traffic stream that occasioned them. These analyses were performed for the new vehicles projected to operate on interstate and primary routes over the long term under each size and weight scenario. The results were expressed in terms of cost responsibility per mile driven by each new configuration. Incremental bridge costs were allocated based on ton miles of travel; incremental pavement costs, by ESAL-miles of travel. While these allocators and the specific manner in which they were used may be controversial, the intent of these calculations was to simply obtain an order of magnitude estimate of cost responsibility.

The cost responsibilities determined for each scenario and at each bridge capacity are presented in Table 7.5-1. Based on the various assumptions made in these analyses in

Table 7.5-1 Incremental Vehicle Cost Responsibility if Canadian Interprovincial, Canamex, or Canamex Short Limits on Vehicle Size and Weight are Adopted

Item	Incremental Vehicle Cost Responsibility (dollars per mile driven)					
	Canadian Interprovincial		Canamex		Canamex Short	
	Full[a]	87 Percent[b]	Full[a]	87 Percent[b]	Full[a]	87 Percent[b]
Interstate						
Bridge	0.00	0.17	0.00	0.06	0.00	0.13
Overlay	0.01	0.01	0.02	0.02	0.02	0.02
Total	**0.01**	**0.18**	**0.02**	**0.08**	**0.02**	**0.15**
Primary						
Bridge	0.12	0.22	0.15	0.18	0.16	0.25
Overlay	0.01	0.01	0.04	0.04	0.05	0.05
Total	**0.13**	**0.23**	**0.19**	**0.22**	**0.21**	**0.30**

[a] calculated using bridge capacity at full Allowable Stress based Operating levels
[b] calculated using bridge capacity at 87 percent of full Allowable Stress based Operating levels

determining engineering impacts and costs, actual cost responsibilities may lie between the values reported at each bridge capacity level. Due to the similarities observed in the cost responsibilities between vehicles within each scenario, only an average cost responsibility collectively calculated for all the vehicles within each scenario is presented in Table 7.5-1. Typical cost responsibilities by vehicle type within a scenario are presented in Table 7.5-2 (Canamex scenario shown). Referring to Table 7.5-2, bridge and overlay costs were relatively constant across all configurations within the scenario. Nominally higher bridge costs (e.g., 0.07 versus 0.06 dollars per mile, interstate system) were calculated for the heaviest vehicle in the scenario, which for the Canamex scenario was the 8 axle C-train. Higher pavement costs (e.g., 0.05 versus 0.04 dollars per mile) were determined for the least ESAL friendly vehicle in the scenario, which for the Canamex scenario was the 7 axle C-train. While trends of this type were

observed across the vehicles within all scenarios, costs within a scenario generally varied by less than 20 percent between vehicles.

Table 7.5-2 Incremental Cost Responsibility, Canamex Scenario, 87 Percent of Allowable Stress Based Operating Ratings

Item	Incremental Cost Responsibility (dollars per mile driven)		
	7 Axle A-train	7 Axle C-train	8 Axle C-train
Interstate			
Bridge	0.06	0.06	0.07
Overlay	0.02	0.02	0.02
Total	**0.08**	**0.08**	**0.09**
Primary			
Bridge	0.15	0.13	0.15
Overlay	0.04	0.05	0.04
Total	**0.19**	**0.18**	**0.19**

Returning to differences in cost responsibilities between scenarios (Table 7.5-1), the lowest cost responsibility per vehicle mile driven on the interstate and primary systems was determined for Canadian Interprovincial vehicles when bridge capacity was set at full Allowable Stress based Operating levels. These cost responsibilities were 0.01 and 0.13 dollars per mile driven on the interstate and primary systems, respectively. Unit cost responsibilities in general were lower for Canadian Interprovincial vehicles than might have been expected based on the absolute total cost of their impact on the highway system. While the impact of the operation of Canadian Interprovincial vehicles had previously been found to produce highest absolute bridge and pavement costs in all situations, more operators were expected to shift to new vehicles if these limits are adopted than if Canamex or Canamex Short limits are adopted. Lowest cost responsibilities per vehicle mile driven using 87 percent of full Operating bridge capacity were determined for the Canamex scenario. These costs were 0.08 and 0.22 dollars per mile driven on the interstate and primary system.

Cost responsibilities on the interstate system were 0.01 and 0.02 dollars per mile driven, respectively, for the Canadian Interprovincial and for the Canamex and Canamex Short vehicles (full Operating bridge capacity). Pavement costs dictated total cost responsibility in this particular situation (interstate system, full operating ratings). Bridge costs dominated total cost responsibility in all other situations. On the interstate system, at 87 percent of full Allowable Stress based Operating ratings, the lowest cost responsibility of 0.08 dollar per mile driven was calculated for the Canamex limits. Corresponding cost responsibilities for the Canadian Interprovincial and Canamex Short limits were 0.18 and 0.15 dollars per mile driven, respectively.

Lower cost responsibilities were consistently calculated for vehicle operation on the interstate system relative to the primary system. Calculated cost responsibilities on the primary system were from 1.3 to 10 times greater than cost responsibilities estimated for the interstate system. Costs increases between the two systems were generally higher using full Operating stress levels for bridge capacity compared to using 87 percent of these levels. The lowest cost predicted for the primary system was 0.13 dollars per mile driven for Canadian Interprovincial vehicles using full Operating bridge capacity. The highest cost responsibility for the primary system was 0.30 dollars per mile driven for Canamex Short vehicles with 87 percent of Operating capacity as a measure of bridge adequacy.

Based on the disparity in cost responsibility for operation on the interstate and primary systems, it may be prudent to consider allowing these new vehicles to operate only on specific parts of the highway system (i.e., the interstate routes), while minimizing their operation on other parts of the system.

8. CONCLUSIONS AND IMPLEMENTATION

8.1 SUMMARY AND CONCLUSIONS

1.1.1 Summary - The adoption of Canadian Interprovincial, Canamex or Canamex Short limits on vehicle size and weight will have a definite impact on the Montana State highway system. All three of these systems of size and weight allow vehicles to operate on the highway system at gross vehicle weights that are higher than those presently permitted in Montana. Canadian Interprovincial Limits additionally allow higher axle group loads than are presently legal in Montana. Vehicles operating under these new size and weight limits will place increased demands on both the bridge and pavement systems, which will result in increased highway costs to maintain the same level of service.

If Canadian Interprovincial limits are adopted, the incremental increase in combined bridge and pavement costs on the interstate and primary systems is projected to be between 12 and 42 million dollars per year, which represent increases of 12 and 36 percent, respectively, relative to comparable costs under the present traffic stream. These cost increases are specifically associated with (a) replacing bridges on the system found to be inadequate under the heavy Canadian vehicle loads and (b) overlaying roads earlier than expected using pavements nominally thicker than would be required under the existing traffic stream. The impact of adopting Canamex limits is projected to be less than that for Canadian Interprovincial limits, which would be expected based on the relative magnitude of the allowable loads under the two systems. If Canamex limits are adopted, the incremental increase in pavement and bridge costs on the interstate and primary systems is projected to be between 4 and 7 million dollars per year, which represent increases of 4 and 6 percent, respectively, over comparable costs projected under the current traffic stream. Adoption of Canamex Short limits would result in a nominal increase in costs compared to Canamex limits. Costs on the interstate and primary systems would be 5 and 10 million dollars per year, respectively, corresponding to a 4 and 9 percent increases over projected expenditures for the existing size and weight system.

The costs presented above were determined based on the changes that are expected to occur in the composition of the traffic stream if new weight limits are adopted, as operators move to take advantage of any economic benefit offered by the new vehicles. The new limits generally offer the ability to transport greater weight than present limits, so weight limited operators are expected to migrate to the new heavy configurations. As this evolution occurs, substantial increases (300 percent) in the number of 3S3 and B-trains in the traffic stream are predicted under Canadian Interprovincial limits. These vehicles will make up 27 percent of all heavy truck traffic in the new traffic stream, compared to less than 5 percent in the existing traffic stream. Under Canamex and Canamex Short limits, 8 axle C-train use is projected to increase from 2 to 12 percent of the traffic stream. Under all three scenarios, diversion of freight from rail to truck is expected, and an allowance was made for this occurrence as the new traffic streams were developed.

The new vehicles in the Canadian Interprovincial, Canamex, and Canamex Short traffic streams will have an impact on the bridges and pavements on the highway system. The Canadian Interprovincial and Canamex Short limits allow shorter and heavier vehicles to operate than are presently legal in Montana; the Canamex limits allow vehicles similar in configuration to existing vehicles in Montana to operate at higher gross vehicle weights than are allowed in Montana.. The combination of increased weight (all scenarios) and decreased length (Canadian Interprovincial and Canamex Short scenarios, only) places higher demands on bridges than those that were used in the original designs. Bridges, however, have traditionally been conservatively designed, and many of these structures may possess adequate reserve capacity to offer an acceptable level of safety under these new demands. Limited analyses performed in this study found that the as-built, as-performing, and as load-rated capacity of many bridges is significantly higher than the design demand. Thus a lower bound on useable bridge capacity was established at an intermediate capacity between the design Allowable Stress based Inventory and Operating levels for the bridge. An upper bound on useable bridge capacity was established at the full Allowable Stress based Operating level of the bridge. Useable bridge capacities similar in magnitude to the full Allowable Stress based Operating level of a bridge can be obtained for conditions in Montana (low traffic, good structural conditions) using Load and Resistance Factor load rating procedures. These procedures were recently developed in an effort to provide uniform levels of safety in bridge load ratings across a wide variety of in-service conditions.

Using these bounds on useable bridge capacity, the analyses performed in this study found that 16 to 20 percent of all the bridges on the state highway system are deficient to carry Canadian Interprovincial vehicles (above and beyond the bridges currently deficient under HS20 design loads). Significantly fewer bridges are deficient (above and beyond those bridges already deficient to carry the HS20 design vehicle) under Canamex and Canamex Short limits compared to Canadian Interprovincial limits. Between 1 and 3 percent of all the bridges on the state highway system are deficient under Canamex and Canamex Short limits (above and beyond those already deficient under the HS20 design vehicle).

Projections of bridge deficiencies are sensitive to the level of assumed bridge capacity. Under full Allowable Stress based Operating ratings, for example, less than 0.5 percent of the bridges on the interstate were found to be deficient under all scenarios. Assuming a bridge capacity midway between Allowable Stress based Inventory and Operating ratings, however, resulted in bridge deficiency rates of 32, 2, and 6 percent, respectively, under Canadian Interprovincial, Canamex, and Canamex Short limits. Bridge deficiency rates were also dependent on the element of the highway system under consideration. The lowest percentages of deficient bridges were consistently found on the interstate system. Deficiency rates on the primary and secondary systems ranged around 70 percent, compared to corresponding rates on the interstate system of less than 32 percent. These results were expected, in that the primary and secondary systems include many older bridges designed for lower loads than are used for interstate bridges.

Bridge deficiency rates are also dependent on bridge type. The highest percentage of deficient bridges were noted for short span simply supported timber structures and continuous steel structures. The short span timber structures, predominantly on the primary and secondary systems, were generally designed for H15 load. The continuous steel structures, found on all systems, appear to be sensitive to the increase in negative bending moment at the supports under the vehicles in all scenarios. While the maximum positive moments expected in simply supported spans vary significantly between scenarios, the negative bending moments generated in continuous structures appear to be similar for all three scenarios. Work is currently underway on analyzing typical continuous structures to determine if trends on bridge capacity observed for simply supported spans extend to continuous structures. The lowest bridge deficiency rates were observed for simply supported prestressed and reinforced concrete bridges.

While strength is of primary importance in evaluating bridge performance, durability is an important consideration from a practical perspective. A limited experimental and analytical investigation of bridge behavior at Canadian Interprovincial load levels indicated that long term durability and performance should not be compromised under these loads. These investigations considered accelerated deterioration of concrete decks and prestress concrete beams, and accelerated fatigue in steel stringers. A network analysis of fatigue response in steel bridges indicated that less than 20 percent of the bridges on the system will have less than a 75 year life under the new vehicles considered herein, although long term fatigue demands are predicted to increase by approximately 35 and 10 percent under Canadian Interprovincial and under Canamex and Canamex Short loads, respectively.

Pavement demands will increase under both Canadian Interprovincial and Canamex vehicles. Long term pavement demands, as measured in ESALs, were projected to increase approximately 3 and 4 percent, respectively, for the long term Canadian Interprovincial and the Canamex and Canamex Short scenarios compared to projected demands for the current traffic stream. These demands will result in a nominal reduction in the life of existing pavements (typically less than 1 year) and a nominal increase in the thickness of future overlays (typically less than 2 percent), based on calculations performed using an AASHTO ESAL based pavement performance model.

Costs were assessed for the impacts identified above by calculating costs for equivalent work at current prices, projecting these costs into the future as necessary, and determining equivalent uniform annual costs for the resulting cash flow. Total bridge costs system-wide associated with adopting Canadian Interprovincial limits were estimated to be between approximately 400 and 1,100 million. These figures represent the immediate cost to totally replace bridges identified as deficient. The bridge costs estimated for Canamex and Canamex Short limits were approximately 50 percent lower than those for Canadian Interprovincial limits, ranging from approximately 200 to 550 million dollars. These estimates include the cost of replacing all bridges that are already deficient under the HS20-44 design vehicle. Incremental costs above and beyond the costs to replace bridges already deficient under HS20 were estimated at 240 to 740 million dollars and 50 to 220 million dollars, respectively for Canadian Interprovincial and for Canamex and Canamex Short limits. The lower costs reported in each of the above ranges was calculated using full Allowable stress Based Operating ratings for bridge capacity; the upper costs, using a rating midway between Allowable stress Based Operating and Inventory ratings. Lowest bridge replacement costs were consistently calculated for the interstate system compared to the primary and secondary systems.

Equivalent uniform annual overlay costs were calculated for each scenario based on the expected remaining life of the existing pavement and future overlay thickness required to provide 20 years of additional service. A 7 percent discount rate was used in these calculations. These calculations were only performed for the interstate and primary systems due to constraints on the data available for secondary and urban systems. Total equivalent uniform annual costs for overlays under all scenarios were closely grouped around 100 million dollars. The similarity in total costs between scenarios was expected, in that the total pavement demands varied by only a few percent between scenarios. The expected long term overlay costs under all scenarios exceeded comparable costs projected for the existing traffic stream by less than 2 percent.

Combined bridge replacement and pavement overlay costs were calculated for each scenario for the primary and interstate systems. Immediate bridge replacement costs were converted to equivalent uniform annual costs using a 75 year life and a 7 percent discount rate. Total (bridge plus overlay costs) equivalent uniform annual costs projected under the long term Canadian Interprovincial scenario range from 120 million to 160 million dollars. The lower cost in this range was calculated using full Allowable Stress based Operating ratings to represent bridge capacity; the higher cost, using a bridge capacity midway between Allowable Stress based Inventory and Operating ratings. These costs represent increases of 11 to 36 percent over comparable costs under the existing traffic stream. The corresponding equivalent uniform annual costs for Canamex limits (long term) range from 110 million dollars to 120 million dollars, which represent cost increases of 4 and 7 percent over comparable costs under the existing traffic stream. The corresponding equivalent uniform annual costs for Canamex Short limits (long term) range from 110 million dollars to 125 million dollars, which represent cost increases of 4 and 9 percent over comparable costs under the existing traffic stream. The majority of the total cost under all scenarios was for pavement overlays.

If Canadian Interprovincial limits are adopted, the incremental increase (above and beyond the costs projected under existing size and weight limits) in combined bridge and pavement costs on the interstate and primary systems is projected to be between 12 and 42 million dollars per year, as previously stated. The impacts of adopting Canamex and Canamex Short limits are projected to be significantly less than that for Canadian Interprovincial limits, which would be expected based on the relative magnitude of the allowable loads under the two systems. If Canamex limits are adopted, the incremental increase in pavement and bridge costs on the interstate and primary systems is projected to be between 4 and 7 million dollars per year. If Canamex Short Limits are adopted, the incremental increase in pavement and bridge costs on the interstate and primary systems is projected to be between 5 and 10 million dollars per year. In most cases, the majority of the incremental costs are associated with bridge impacts. In all cases, the cost impacts for the primary system significantly exceed those for the interstate system.

The increase in user cost responsibility associated with adopting Canadian Interprovincial, Canamex, or Canamex Short limits was estimated based on the increased costs for the highway system identified above and the projected use of the system by the new vehicles. The estimated cost responsibility for Canadian Interprovincial vehicles operating on the interstate system ranges from 0.01 and 0.18 dollars. Cost responsibilities ranging from 0.02 to 0.08 and from 0.02 to 0.15 dollars per mile driven are estimated for Canamex and Canamex Short vehicles, respectively, operating on the interstate system. In each instance, the first figure was calculated using full Allowable Stress based Operating ratings to represent bridge capacity; the second figure, using an intermediate bridge capacity between Allowable Stress based Inventory and Operating levels. Actual cost responsibilities are expected to fall within these ranges. The sensitivity of these estimates of cost responsibility to the assumed level of bridge capacity is obvious. The cost responsibilities for Canadian Interprovincial vehicles may be lower than would be expected based on the total cost impacts of 12 to 42 million dollars stated above. Adoption of Canadian Interprovincial limits, however, will affect the greatest number of vehicles in the traffic stream, reducing cost responsibility per vehicle mile driven.

Lower cost responsibilities were consistently calculated for vehicles operating on the interstate relative to the primary system. Calculated cost responsibilities on the primary system are from 1.3 to 10 times greater than cost responsibilities estimated for the interstate system. While cost responsibilities were not calculated for the secondary system, these costs are expected to be higher than those for the interstate and primary system. The lighter pavements and bridges on the secondary system are expected to less tolerant of the increases in load under Canadian Interprovincial, Canamex, and Canamex Short vehicles than the more substantial pavements on the primary and interstate systems.

8.1.2 Conclusions - Overall, Canadian Interprovincial limits will result in significantly higher demands on the highway system than Canamex or Canamex Short limits, as would be expected based on the difference in loads allowed under the three systems. Demands under Canamex Short limits, in turn, are nominally higher than the demands under Canamex Limits. These differentials in demand are associated primarily with the bridge system, where Canadian Interprovincial vehicles stress more structures closer to their ultimate capacity than Canamex Short and Canamex vehicles. In general, fewer bridges were found to be deficient on the interstate compared to other systems. Based on these various results, it may be practical to focus the operation of the new vehicles on designated routes within the state, notably the interstate routes. The interstate system should be able to handle either Canadian Interprovincial, Canamex, or Canamex Short vehicles without substantial modification. It will be possible, however, to open more of the system to Canamex vehicles than to either Canamex Short or, particularly, Canadian Interprovincial vehicles. Collector routes along the interstate (primary, secondary, and urban routes) may also be able to better handle Canamex vehicles than Canamex Short and Canadian Interprovincial vehicles. In almost all cases, the majority of the incremental uniform annual cost is bridge related. Thus, costs associated with specific routes could be significantly lower than the average costs presented above, if these routes contain only a few (or no) deficient bridges.

8.2 IMPLEMENTATION

A judgement needs to be made regarding the economic feasibility of adopting Canadian Interprovincial, Canamex, or Canamex Short limits on vehicle size and weight. Gross estimates of the costs associated with providing the new vehicles with highway service were determined in this study. It may be possible based on these estimates to determine if vehicle operators will be willing to pay the indicated costs to operate these vehicles. In arriving at such a decision, consideration should be given to the specific weights at which these vehicles will be allowed to operate and the possibility of their operation over limited routes.

If it appears that the new vehicles will be adopted by operators, it may be prudent to next perform comprehensive load rating calculations (using Allowable Stress, Load Factor, and Load and Resistance Factor approaches) on a broader sampling of typical bridges on the system than were considered in this investigation. Attention could be focused on bridges known to be vulnerable based on this investigation. Bridges in the sample could also be selected from specific routes upon which initial operation of the new vehicles is expected. These analyses would reduce some of the present uncertainties regarding the specific impact these new limits will have on the bridge system. These analyses might also reveal that operation of these vehicles can be further expanded to incorporate more of the highway system.

Strictly from an infrastructure perspective, consideration should then be given to allowing Canadian Interprovincial, Canamex, and/or Canamex Short vehicles to operate on a trial basis on certain routes around the state. Considerable evidence exists that the interstate system will be able to handle these vehicles without substantial modification. Some primary routes may also be found that can be upgraded at nominal cost to carry the new vehicles (notably Canamex or Canamex Short vehicles). If these vehicles are allowed to operate on a trial basis, they should be expected to pay additional fees to cover their projected increase in cost responsibility for the highway system. The performance and condition of the bridges and pavements on any routes upon which Canadian Interprovincial, Canamex, or Canamex Short vehicles are allowed to operate should be closely monitored over time. The data collected on these routes would be useful in validating existing performance models and definitively establishing, as possible, the physical effects of the new vehicles on the highway system. This information would be used 1) to determine whether these vehicles should be allowed to continue to operate on the system and 2) to establish appropriate user fees.

REFERENCES

Alberta Transportation and Utilities (1992), "RTAC in Alberta", Alberta Transportation and Utilities, Edmonton, Alberta.

Alberta Transportation and Utilities (1994), "Memorandum of Understanding Regarding the Canamex Trucking Corridor", memorandum attached to letter of transmittal of material to David Galt, Montana Department of Transportation, from Carl Procuik, Alberta Transportation and Utilities dated November 28, 1994.

American Association of State Highway and Transportation Officials (1972), AASHTO Interim Guide for Design of Pavement Structures, AASHTO, Washington, D.C.

American Association of State Highway and Transportation Officials (1989), AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges, AASHTO, Washington, D.C.

American Association of State Highway and Transportation Officials (1990), AASHTO Standard Specification off Highway Bridges, AASHTO, Washington D.C.

American Association of State Highway and Transportation Officials (1993), AASHTO Guide for Design of Pavement Structures, AASHTO, Washington, D.C.

American Association of State Highway and Transportation Officials (1994), AASHTO Guide for Condition Evaluation of Bridges, AASHTO, Washington, D.C.

Batchelor, B., et.al. (1978), "Investigation of the Ultimate Strength of Deck Slabs of Composite Steel/Concrete Bridges", TRR 664, Transportation Research Board, National Research Council, Washington, D.C.

Beal, D.B. (1982), "Load Capacity of Concrete Bridge Decks", Journal of the Structural Division, ASCE, Vol. 108, No.ST4.

Cady, P.D. and Weyers, R.E. (1984), "Deterioration Rates of Concrete Bridge Decks", Journal of Transportation Engineering, ASCE, Vol. 110, No. 1.

Callahan, J.P, Siess, C.P., and Kesler, C.E. (1970), "Effect of Stress on Freeze-Thaw Durability of Concrete Bridge Decks", NCHRP Report 101, National Research Council, Washington, D.C.

Carrier, R.E. and Cady, P.D. (1973), "Deterioration of 249 Bridge Decks", HRR 423, Transportation Research Board, National Research Council, Washington, D.C.

Cloud, W. (1995), personal communication, Planning Division, Montana Department of Transportation, Helena, Montana.

Deacon, J.A. (1988), "Pavement Rehabilitation Cost Model", TRB, National Research Council, Washington, D.C.

Dunker, K.F. and Raubat, B.G. (undated), "Performance of Highway Bridges", undated manuscript provided by Scott Walters, Elk River Prestress, Helena, Montana, 1994, first author from Department of Civil Engineering, Iowa State University, Ames, Iowa.

Federal Highway Adminstration (FHWA) (1988), "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges", U.S. Department of Transportation, Washington, D.C.

Galt, D. (1996), personal communication, Director, Motor Carrier Services Division, Montana Department of Transportation, Helena, Montana.

Gillespie, T.D, et.al. (1993), "Effects of Heavy-Vehicle Characteristics on Pavement Response and Performance, NCHRP Report 353, National Cooperative Highway Research Program, National Academy Press, Washington, D.C.

Hansell, W.C. and Viest, I.M. (1971), "Load Factor Design for Steel Highway Bridges", AISC Engineering Journal, October 1971.

Hanson, J., Hulsbos, C., and VanHorn, "Fatigue Tests of Prestress Concrete I-Beams", Journal of the Structural Division, American Society of Civil Engineers, Vol. 96, No. ST9, Nov. 1970.

Highway Research Board (1962), "The AASHO Road Test, Report 5, Pavement Research", SR 61E, Highway Research Board, National Research Council, Washington, D.C.

Hilsdorf, H.K., and Lott, J.L. (1970), "Revibration of Retarded Concrete for Continuous Bridge Decks", NCHRP Report 106, National Cooperative Highway Research Program, National Academy Press, Washington, D.C.

Hudson, W.R. and Flanagan, P.R. (1987), "An Examination of Environmental Versus Load Effects on Pavments", TRR 1121, Transportation Research Board, National Research Council, Washington, D.C.

Hult, D. (1995), personal communication, Data Collection/Analysis Section, Montana Department of Transportation, Helena, Montana.

James, R.W., Zimmerman, R.A, and McCreary,C.R. (1987), "Effects of Overloads on Deterioraton of Concrete Bridges", TRR 1118, Transportation Research Board, National Research Council, Washington, D.C.

Johnson, L.P. (1995), "Load Rating Four Bridges Subject to Loading by Canadian B-Train Double Trucks", Professional Paper submitted to the Department of Civil Engineering, Montana State University, Bozeman, Montana.

Khalil, S. (1996), personal communication, pavement engineer, Alberta Transportation and Utilities, Edmonton, Alberta, Canada.

Kilareski, W.P. (1989), "Heavy Vehicle Evaluation for Overload Permits, TRR 1227, Transportation Research Board, National Research Council, Washington, D.C.

Kostem, C. N. (1978), "Overloading of Highway Bridges--Initiation of Deck Damage", TRR 664, Transportation Research Board, National Research Council, Washington, D.C.

Kreger, M.E., Bachman, P.M., and Breen, J.E. (1989), "An Exploratory Study of Shear Fatigue Behavior of Prestressed Concrete Girders", PCI Journal, July-August 1989.

Meyer, J. (1996), personal communication, Bridge Bureau, Montana Department of Transportation, Helena, Montana.

Minor, J., White, K.R., and Busch, R.S. (1988), "Condition Surveys of Concrete Bridge Components: Users Manual", NCHRP Report 312, Transportation Research Board, National Research Council, Washington, D.C.

Mohammadi, J.; Guralnick, S.A.; and Polepeddi, R. (1991), "The Effect of Increased Truck Weights Upon Illinois Highway Bridges", FHWA/IL/RC-013, Department of Civil Engineering, Illinois Institute of Technology, Chicago, Illinois.

Montana Code Annotated (1995), Montana Legislative Council, State of Montana, Helena, Montana.

Montana Department of Transportation (1991), "State of the Interstate Report", Highway Division, Helena, Montana.

Montana Department of Transportation (1993a), "State of the Primary Pavement Report", Highway Division, Materials Bureau, Pavement Management Section, Montana Department of Transportation, Helena, Montana.

Montana Department of Transportation (1993b), "1993 Montana Federal Aid Roadlog", Secondary Roads and Statistics Bureau, Montana Department of Transportation, Helena, Montana.

Montana Department of Transportation (1994), "Montana Bridges 1994", Bridge Bureau and Transportation Planning, Montana Department of Transportation, Helena, Montana.

Moroz, A. (1996), personal communication, bridge engineer, Alberta Transportation and Utilities, Red Deer, Alberta, Canada.

Moses, F. (1992), "Truck Weight Effects on Bridge Cost", Report No. FHWA/OH-93/001, Dept. of Civil Engineering, Case Western Reserve University, Cleveland, Ohio.

Moses, F., Schilling, C.G., and Raju, K.S. (1987), "Fatigue Evaluation Procedures for Steel Bridges", NCHRP Report 299, National Cooperative Highway Research Program, National Research Council, Washington, D.C.

Moses, F. and Verma, D. (1987), """Load Capacity Evaluation of Existing Bridges, NCHRP Report 301, National Cooperative Highway Research Program, National Research Council, Washington, D.C.

Murphy, J. (1992), "PCBridge Version 2.61", Madison, Wisconsin.

Murphy, M. (1995), personal communication, Bridge Bureau, Montana Department of Transportation, Helena, Montana.

Murphy, M. (1996), personal communication, Bridge Bureau, Montana Department of Transportation, Helena, Montana.

Newlon, H.H., Jr., Davis, J., and North, M. (1973), "Bridge Deck Performance in Virginia", HRR 423, National Research Council, Washington, D.C.

Noel, J.S., James, R.W., Furr, H.L., and Bonilla, F.E. (1985), "Bridge Formula Development", FHWA/RD-85-088, Texas Transportation Institute, Texas A&M University, College Station, Texas.

Rejali, M.H.M. (1966),"Distribution of Transverse Moments on Concrete Bridge Decks", Master's Thesis, University of Illinois, Champaign, Illinois.

Ritter, M. (1990), "Timber Bridges: Design, Construction, Inspection, and Maintenance", United States Department of Agriculture, Forest Service, Washington, D.C.

Roads and Transportation Association of Canada (1987), "Recommended Regulatory Principles for Interprovincial Heavy Vehicle Weights and Dimensions", Roads and Transporation Association of Canada.

Saklas, J.G., et.al. (1988), "Bridge Fatigue Damage Cost Allocation, Executive Summary", Production Software Inc., College Park, Maryland.

Salmon, C.G. and Johnson, J.E. (1992), Steel Structures, Design and Behavior, HarperCollins College Publishers, New York, New York.

Sanders, D.H. and Zhang, Y.J. (1994) "Bridge Deterioration Models for States with Small Bridge Inventories", TRR 1442, Transportation Research Board, National Research Council, Washington, D.C.

Schillings, C.G. and Klippstein, K.H., "New Method for Fatigue Design of Bridges", Journal of the Structural Division, American Society of Civil Engineers, Vol. 104, No. ST3, March, 1978.

Scoles, J. (1996), "Impact on Montana Bridge Decks of the Adoption of Canadian Interprovincial Limits on Vehicle Size and Weight", Professional Paper submitted to the Department of Civil Engineering, Montana State University, Bozeman, Montana.

Small, K.A., Winston, C. and Evans, C.A. (1989), Road Work, A New Highway Pricing and Investment Policy, The Brookings Institution, Washington, D.C.

Sorenson, H.C. and Robledo, F.M. (1992), "Turner Truck Impact on Bridges," FHWA -WA-RD 287.1, Washington State Transportation Center, Washington State University, Pullman, Washington.

Southgate, H.F. and Deen, R.C. (1986), "Effects of Load Distributions and Axle Tire Configurations on Pavement Fatigue," Research Report UKTRP-86-6, Kentucky Transportation Research Program, College of Engineering, University of Kentucky.

Stephens, J.E., et.al., "An Efficient Method for Evaluating the Load Response Behavior of Steel and Prestressed Concrete Bridges", Proceedings, Structural Materials Technology NDE Conference, San Diego, California, February 1996.

Transportation Association of Canada (TAC), Canadian Trucking Research Institute (CTRI) (1994), "Impact of Canada's Heavy Vehicle Weights and Dimensions Research and Interprovincial Agreement", Report No. VW-IC-1-01994, TAC/CTRI, Ottawa, Canada.

Transportation Research Board (1989), Providing Access for Large Trucks, SR 223 National Research Council, Washington, D.C.

Transportation Research Board (1990a), Truck Weight Limits, Issues and Options, SR 225, National Research Council, Washington, D.C.

Transportation Research Board (1990b), New Trucks for Greater Productivity and Less Road Wear, An Evaluation of the Turner Proposal, SR 227, National Research Council, Washington, D.C.

Wang, C-K. and Salmon, C.G. (1992), Reinforced Concrete Design, HarperCollins Publishers Inc., New York, New York.

Wegmuller, A.W. (1977), "Overload Behavior of Composite Steel-Concrete Bridge, Journal of the Structural Division, ASCE, Vol. 110, No. ST9.

Weissmann, J. and Harrison, R. (1991), "Impact of Turnpike Doubles and Triple 28s on the Rural Interstate Bridge Network, TRR 1319, Transportation Research Board, National Research Council, Washington, D.C.

Weissmann, J., Reed, R.L., and Feroze, A. (1994), "Incremental Bridge Construction Costs for Highway Cost Allocation", TRR 1460, Transportation Research Board, National Research Council, Washington, D.C.

Wilkes, W.J. (1989), "Perspective: Fatigue, Concrete vs. Steel", PCI Journal, July-August 1989.

Wissinger, Mark (1995), personal communication, Montana Department of Transportation, Helena, Montana.

Xanthakos, P.P. (1994), Theory and Design of Bridges, John Wiley & Sons, Inc., New York.

Zutatas, B. (1994), private communication, Motor Transport Services Division, Alberta Transportation and Utilities, Red Deer, Alberta, Canada.

METRIC CONVERSION TABLE

1 foot = 0.3 m
1 sq ft = 0.09 m^2
1 cu ft = 0.03 m^3
1gal (U.S.) = 0.004 m^3
1 in = 25.4 mm
1 sq in = 645 mm^2
1 lbf = 4.5 N
1 lbm = 0.5 kg
1 psf = 48 Pa
1 psi = 6.9 kPa
1 mi = 1.6 km

These conversion factors are intended to allow for easy and expeditious estimates of quantities in SI units from English units. Detailed calculations should be performed using more precise conversion factors than those presented above.

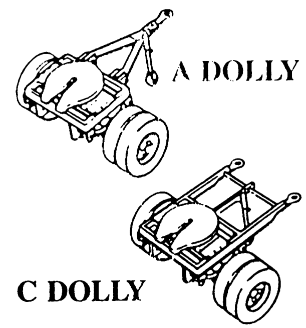
APPENDIX A

CANADIAN INTERPROVINCIAL AND CANAMEX LIMITS ON TRUCK SIZE AND WEIGHT

This appendix contains a description of the Canadian Interprovincial and Canamex limits on vehicle size and weight as implemented in Alberta.

WHAT IS RTAC?

- On May 18, 1914 the Canadian Good Roads Association was founded in Montreal. In 1970, the CGRA changed its name to the Roads and Transportation Association of Canada (RTAC), due to its increasing involvement with all transportation modes except marine. However, Roads and Road Transportation and Public Transit are the main areas of RTAC's interest. RTAC is now over 75 years old and since 1991, is known as Transportation Association of Canada (TAC).
- In February of 1988, a Memorandum of Understanding was signed to improve uniformity of weight and dimension regulations for commercial vehicles operating between provinces and territories on a nationwide highway system.
- On September 1, 1988, Alberta passed laws bringing the guidelines of the RTAC agreement into effect. All equipment manufactured after Nov. 15 1988, must comply to the regulations as outlined in this brochure.
- Then on November 1, 1992, revised regulations were put into force to incorporate a new revised Memorandum of Understanding which also included three additional vehicle configurations. These vehicle configurations have proven to be safe and economical methods of transporting goods while at the same time minimize road and bridge damage.



GENERAL INFORMATION

The information listed below is in addition to the information shown on the diagrams in this brochure.

- Loaded Height: Max. 4.15 metres (13.6 ft.)
- Track Width: 2.5 - 2.6m (8.2 - 8.5 ft.) (for trailers)
  Outside tire - Outside tire
  Min. 2.4 m prior to Nov.15, 1988
- Vehicle Combination Length - Max. 25 m (82 ft.)
- **Maximum**
  Gross Combination Weights: Must not exceed

Tractor Semitrailer	46,500 kg	(102,514 lbs)
A Train Double	53,500 kg	(117,946 lbs)
B Train Double	62,500 kg	(137,787 lbs)
C Train Double **	60,500 kg	(133,378 lbs)

  ** This weight is allowed on "C" trains that meet the required C-train dimensions (see reverse).
- Axle weight must be shared equally within 1000 kg (2200 lbs) on each axle within an axle group.
- Axle load must not exceed the lesser of maximum axle or axle group load allowance or the rated capacity of the tires, or 10 kg per mm of width or 254 kg per inch of tire width.
- Seasonal weight restrictions exist.
- Use of lift axles is prohibited in Alberta.
- C-Dolly drawbar lengths must not exceed 2m (79") in length. This drawbar length requirement is for new "C" dollies that meet the Canadian Motor Vehicle Safety Standard (C.M.V.S.S.) when it comes into effect.

INTERAXLE SPACINGS

- Single - Single
  Min. 3.0m (9.8 ft)
- Single - Tandem
  Min. 3.0m (9.8 ft)
- Tandem - Tandem
  Min. 5.0m (16.4 ft)
- Tandem - Tridem
  Min. 5.5m (18 ft)

This brochure outlines a general overview of the weights and dimensions allowable in ALBERTA under the RTAC Agreement. However, it should be noted that other jurisdictions may allow additional weights and vehicle configurations above that of the Memorandum of Understanding. For further information on the Memorandum, please contact the appropriate authority in the jurisdiction.

British Columbia	604-387-4404
Alberta	403-427-8901
Saskatchewan	306-787-4801
Manitoba	204-945-3890
Ontario	416-235-3587
Quebec	514-873-2605
New Brunswick	506-453-2802
Nova Scotia	902-424-5973
Prince Edward Island	902-368-5221
Newfoundland	709-729-6069
Yukon	403-667-5920
Northwest Territories	403-984-3341

Information updated: 92/10/1

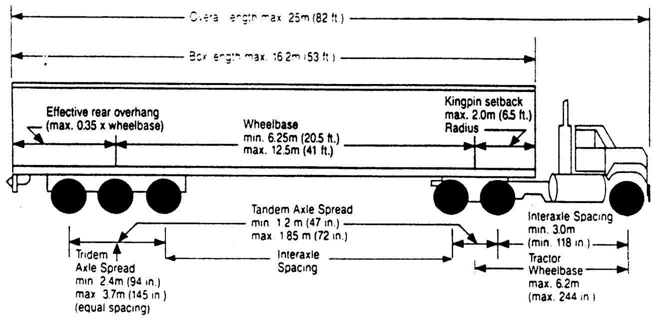
Note: This brochure is only a guide to the regulations. Please consult the actual regulation for an exact interpretation:
-Alberta Public Vehicle Weight Regulation
-Alberta Public Vehicle Dimension Regulation

Safety in Motion

Figure A-1 Canadian Interprovincial Limits (as implemented in Alberta, 1992) (page 1of 4)

TRACTOR SEMITRAILER

Dimensions: (For interaxle spacing table - see reverse)



Maximum Allowable Axle Group Weights

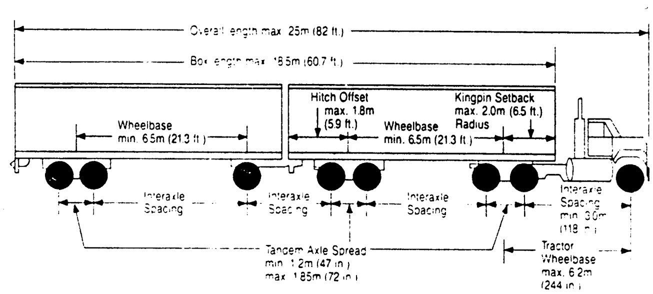
● Steering	5500 kg (12125 lbs.)
● Dual Tires	9100 kg (20061 lbs.)
●● Tandem	17000 kg (37477 lbs.)
●●● Tridem	
Min. 2.4m spread	21000 kg (46296 lbs.)
Min. 3.0m spread	23000 kg (50705 lbs.)
Min. 3.6m - Max. 3.7m	24000 kg (52910 lbs.)

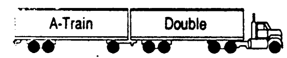
Maximum Gross Combination Weights

Axles	kg	lbs.
3	23700	52249
4	31600	69665
5	39500	87081
6	46500	102514

"A" TRAIN DOUBLE

Dimensions: (For interaxle spacing table - see reverse)





Maximum Allowable Axle Group Weights

● Steering	5500 kg (12125 lbs.)
● Dual Tires	9100 kg (20061 lbs.)
●● Tandem	17000 kg (37477 lbs.)

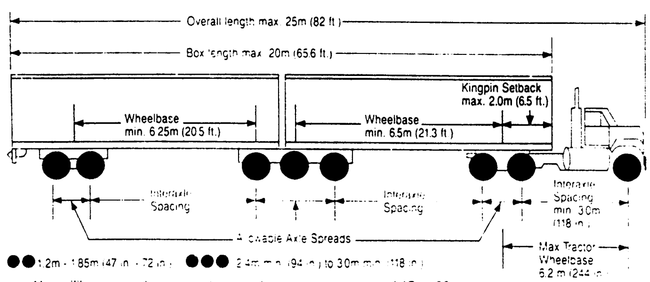
Second semitrailer axle weights are limited to a sum of 16000 kg (35273 lbs.)

Maximum Gross Combination Weights

Axles	kg	lbs.
5	39700	87523
6	47600	104939
7	53500	117946
8	53500	117946

"B" TRAIN DOUBLE

Dimensions: (For interaxle spacing table - see reverse)



●●: 2m - 1.85m (47 in. - 72 in.) ●●●: 2.4m - min. (94 in.) to 3.0m max. (118 in.)

Note: The sum of the two trailer wheelbases must not exceed 17m (55 ft.). This refers to the sum of wheelbases only, not when the trailers are hooked together.

Maximum Allowable Axle Group Weights

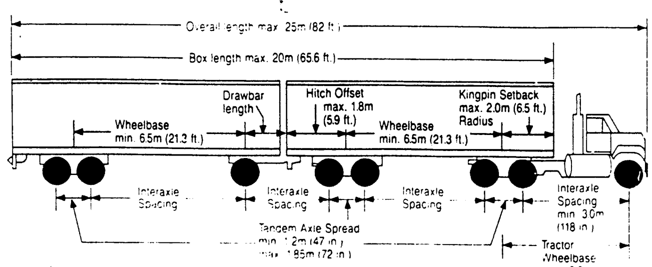
● Steering Axle	5500 kg (12125 lbs.)
● Dual Tires	9100 kg (20061 lbs.)
●● Tandem	17000 kg (37477 lbs.)
●●● Tridem	
Min. 2.4m spread	21000 kg (46296 lbs.)
Min. 3.0m	23000 kg (50705 lbs.)

Maximum Gross Combination Weights

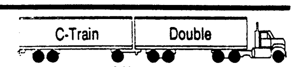
Axles	kg	lbs.
5	40700	89727
6	48600	107143
7	56500	124559
8	62500	137787

"C" TRAIN DOUBLE

Dimension: (For interaxle spacing table - see reverse)



Note: Converter dollies ... Gross Combination Weight ...



Maximum Allowable Axle Group Weights

● Steering Axle	5500 kg (12125 lbs.)
● Dual Tires	9100 kg (20061 lbs.)
●● Tandem	17000 kg (37477 lbs.)

Second semitrailer axle weight's are limited to a sum of 21000 kg (46,297 lbs.)

Maximum Gross Combination Weights

Axle	kg	lbs.
5	41900	93372
6	49800	109788
7	57700	[illegible]

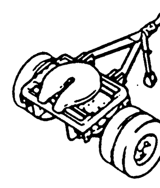
Figure A-1 Canadian Interprovincial Limits (as implemented in Alberta, 1992) (page 2of 4)

TRUCKS
TRUCK AND PONY TRAILER
TRUCK AND FULL TRAILER

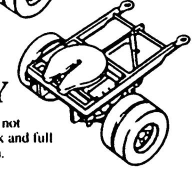
Alberta
TRANSPORTATION
AND UTILITIES

WHAT IS RTAC?

- On May 18, 1914 the Canadian Good Roads Association was founded in Montreal. In 1970, the CGRA changed its name to the Roads and Transportation Association of Canada (RTAC), due to its increasing involvement with all transportation modes except marine. However, Roads and Road Transportation and Public Transit are the main areas of RTAC's interest. RTAC is now over 75 years old and since 1991, is known as Transportation Association of Canada (TAC).
- In 1992, a revised Memorandum of Understanding was signed to improve uniformity of weight and dimension regulations for commercial vehicles operating between provinces and territories on a nationwide highway system, to include trucks and trucks with trailers.
- On November 1, 1992, Alberta passed laws bringing the guidelines of the new RTAC agreement into effect. All equipment manufactured after Sept. 1, 1993, must comply to the regulations as outlined in this brochure.
- These vehicle configurations have proven to be safe and economical methods of transporting goods while at the same time minimize road and bridge damage.



A DOLLY



C DOLLY

Note: A C-dolly is not permitted in a truck and full trailer combination.

GENERAL INFORMATION

The information listed below is in addition to the information shown on the diagrams in this brochure.

- Loaded Height: Max. 4.15 m (13.6 ft.)
- Track Width: 2.5 - 2.6 m (8.2 - 8.5 ft.)(for trailers)
  Outside tire - Outside tire
  Min. 2.4 m prior to Nov. 15, 1988
- Vehicle Combination Length - Max 23m (75.5 ft.)
- Maximum
  Gross Combination Weights: Must not exceed

Truck (3 axle)	24,300 kg	(53,571 lbs)
Truck (tandem steer)	30,600 kg	(67,460 lbs)
Truck & Pony Trailer	45,300 kg	(99,868 lbs)
Truck & Full Trailer	53,500 kg	(117,946 lbs)

- Axle weight must be shared equally within 1000 kg (2200 lbs) on each axle within an axle group.
- Axle load must not exceed the lesser of maximum axle or axle group load allowance or the rated capacity of the tires, or 10 kg per mm of width or 254 kg per inch of tire width.
- Seasonal weight restrictions exist.
- Use of lift axles is prohibited in Alberta.

INTERAXLE SPACINGS

- Single - Single
  Min. 3.0m (9.8 ft)
- Single - Tandem
  Min. 3.0m (9.8 ft)
- Tandem - Tandem
  Min. 5.0m (16.4 ft)
- Tandem - Tridem
  Min. 5.5m (18 ft)

This brochure outlines a general overview of the weights and dimensions allowable in ALBERTA under the RTAC Agreement. However, it should be noted that other jurisdictions may allow additional weights and vehicle configurations above that of the Memorandum of Understanding. For further information on the Memorandum, please contact the appropriate authority in the jurisdiction.

British Columbia	604-387-4404
Alberta	403-427-8901
Saskatchewan	306-787-4801
Manitoba	204-945-3890
Ontario	416-235-3587
Quebec	514-873-2605
New Brunswick	506-453-2802
Nova Scotia	902-424-5973
Prince Edward Island	902-368-5221
Newfoundland	709-729-6069
Yukon	403-667-5920
Northwest Territories	403-984-3341

Information updated: 92/10/1

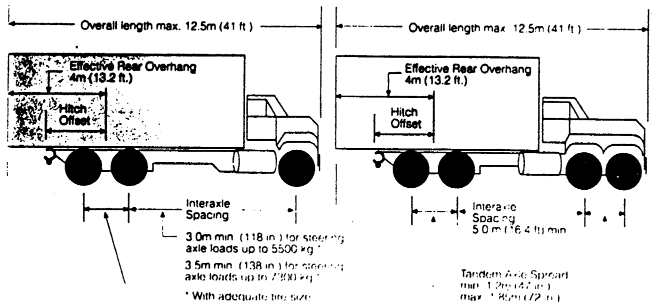
***Note:** This brochure is only a guide to the regulations. Please consult the actual regulation for an exact interpretation:*
-Alberta Public Vehicle Weight Regulation
-Alberta Public Vehicle Dimension Regulation

Alberta
MOTOR TRANSPORT SERVICES

Safety in Motion

Figure A-1 Canadian Interprovincial Limits (as implemented in Alberta, 1992) (page 3of 4)

TRUCK



Note: A tandem-steer axle truck (right) is not a RTAC vehicle but is shown here for reference. Tandem-steer trucks are used in such applications as mixers, permanently mounted cranes and service rigs, bed trucks, etc.

Maximum Allowable Axle Group Weights

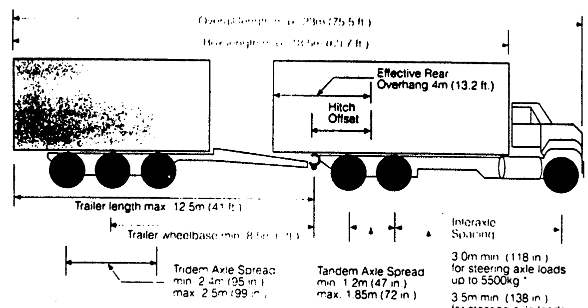
●	Steering	7300 kg (16093 lbs.)
●●	Steering	13600 kg (29982 lbs.)
●	Dual Tires	9100 kg (20061 lbs.)
●●	Tandem	17000 kg (37477 lbs.)

Note: Hitch offset maximum
- for 5th wheel hitch 1.8m (71 in.)
- other hitches 1.5m (59 in.)

Maximum Gross Vehicle Weight

Axles	kg	lbs.
2	16400	36155
3	24300	53572
4	30600	67461

TRUCK and PONY TRAILER



Note: The control dimensions for the pony trailer only effects trailers with a gross trailer weight rating exceeding 10 000 kg (22 046 lbs.)

Maximum Allowable Axle Group Weights

●	Steering	7300 kg (16093 lbs.)
●●	Steering	13600 kg (29982 lbs.)
●	Dual Tires	9100 kg (20061 lbs.)
●●	Tandem	17000 kg (37477 lbs.)
●●●	Tridem	21000kg (46296 lbs.)

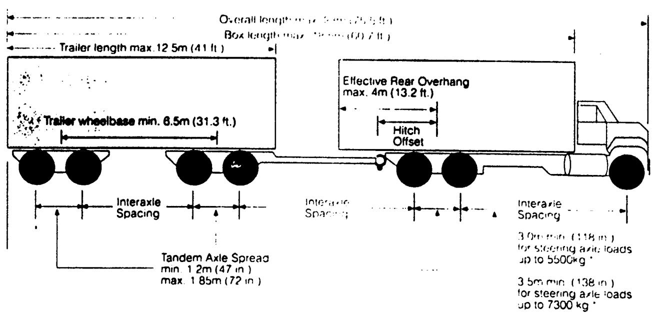
Note: Hitch offset maximum
- for 5th wheel hitch 1.8m (71 in.)
- other hitches 1.5m (59 in.)

Maximum Gross Combination Weight

Axles	kg	lbs.
3	25500	45217
4	33400	73633
5	41300	91050
6	45300	99868
7*	51600	113757

* Truck has tandem steering

TRUCK and FULL TRAILER



Note: A C-Dolly is not permitted in a truck and full trailer combination.

Maximum Allowable Axle Group Weights

●	Steering	7300 kg (16093 lbs.)
●●	Steering	13600 kg (29982 lbs.)
●	Dual Tires	9100 kg (20061 lbs.)
●●	Tandem	17000 kg (37477 lbs.)

Note: Hitch offset maximum
- for 5th wheel hitch 1.8m (71 in.)
- other hitches 1.5m (59 in.)

Maximum Gross Combination Weight

Axles	kg	lbs.
4	34600	76279
5	42500	93695
6	50400	111111
7	53500	117946
8	53500	117946

NOTES

Figure A-1 Canadian Interprovincial Limits (as implemented in Alberta, 1992) (page 4of 4)

CANAMEX
ROCKY MOUNTAIN DOUBLE
GENERAL CONDITIONS

	A TRAIN CONFIGURATION	C TRAIN CONFIGURATION
Overall Length Overall Height Overall Width	Max. 30m (98 ft. 5 in.) Max. 4.27m (14 ft.) Max. 2.6m (8 ft. 6 in.)	Max. 30m (98 ft. 5 in.) Max. 4.27m (14 ft.) Max. 2.6m (8 ft. 6 in.)
Lead Semitrailer Length (box length)	Min. 12.8m (42 ft.) Max. 16.2m (53 ft.)	Min 12.8m (42 ft.) Max. 16.2m (53 ft.)
Wheelbase	Min. N/A Max. 12.5m (41 ft.)	Min. N/A Max. 12.5m (41 ft.)
Hitch Offset Trailers ≤ 13.7 m (45 ft) Trailers > 13.7 m (45 ft)	Max. 1.8m (6 ft.) Max. 2.8m (9.2 ft.)	Max. 1.8m (6 ft.) Max. 2.8m (9.2 ft.)
Effective Rear Overhang	Max. 35% of WB	Max. 35% of WB
Converter Dolly Drawbar Length Max. No. of Axles	Not Controlled 2	Max. 2.0m*(6 ft. 6 in.) 1
Second Semitrailer or Full Trailer Wheelbase Effective Rear Overhang	Min. 6.5m (21 ft. 4 in.) Max. 35% of WB	Min. 6.5m (21 ft. 4 in.) Max. 35% of WB
WEIGHTS		
Gross Vehicle Weight	Max. 53 500kg (118,000 lb.)	Max. 58 200kg (128,000 lb.)

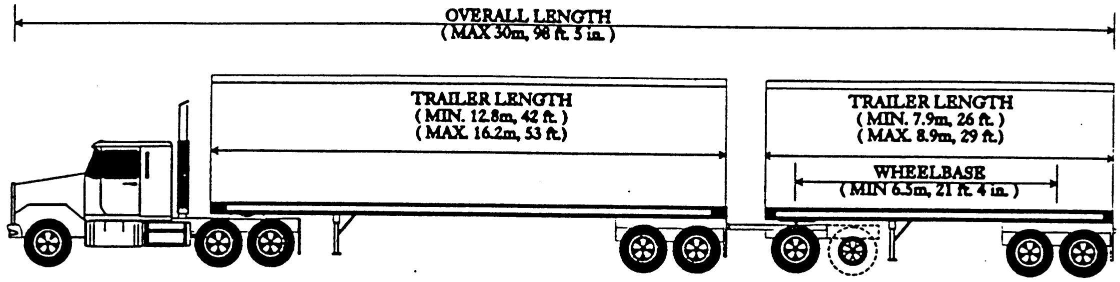
In all cases, the lead semitrailer of the configuration, must be heavier than the second semi-trailer.

*The 2.0 metre (6 ft. 6 in.) maximum drawbar length is applicable to "C" dollies manufactured in 1993 or later in accord with the compliance requirements to the CMVSS under the Motor Vehicle Safety Act, Canada.

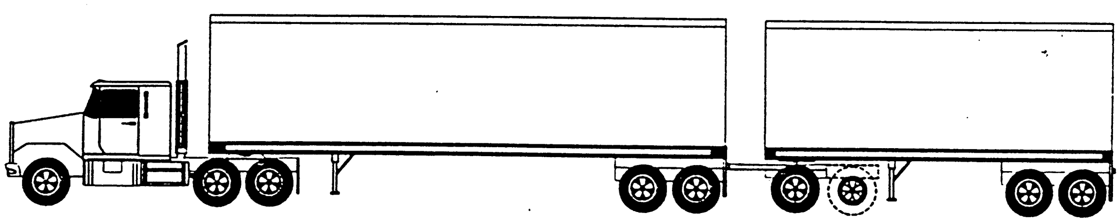
Figure A-2 Canamex Size Limits (as described in Alberta Transportation and Utilities 1994 (page 1 of 3)

CANAMEX
ROCKY MOUNTAIN DOUBLE
A train and C train

DIMENSIONS



WEIGHTS



MAXIMUM AXLE WEIGHTS	STEERING 5 500kg 12000 lb.	SINGLE : 9 100kg/ 20,000 lb. TANDEM : 15 454kg/ 34,000 lb.

COMBINED AXLE GROUP WEIGHTS	SEE TABLE 1

		A TRAIN	C TRAIN
MAXIMUM GROSS VEHICLE WEIGHT	5 AXLE	41 900kg / 92,000 lb.	41 900kg / 92,000 lb.
	6 AXLE	48 200kg / 106,000 lb.	48 200kg / 106,000 lb.
	7 AXLE	53 500kg / 118,000 lb.	54 500kg / 120,000 lb.
	8 OR MORE AXLES	53 500kg / 118,000 lb.	58 200kg / 128,000 lb.

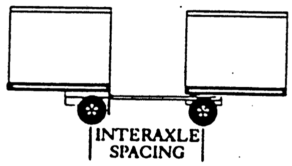
NOTE : Second or rear trailer must be lighter than the lead trailer or lead trailer must be heaviest.
Refer to jurisdictions' transport regulations governing kingpin set back and effective rear overhang.

Figure A-2 Canamex Size Limits (as described in Alberta Transportation and Utilities 1994 (page 2 of 3)

COMBINED AXLE GROUP WEIGHT ALLOWANCE

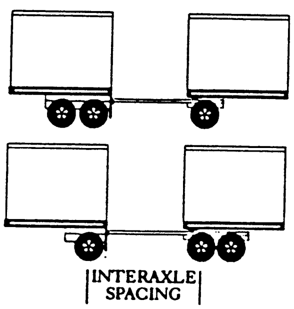
NOTE : The Axle Group Weights shown consist of the lead trailer rear axle group plus the front axle group of the second trailer.

SINGLE-SINGLE



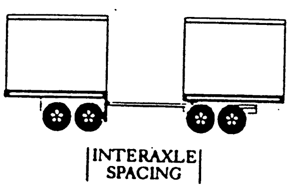
INTERAXLE SPACING	ALLOWED COMBINED WEIGHT
Equal to or greater than 3.0m (10 ft.)	18 200kg/ 40,000 lb.
Less than 3.0m (10 ft.)	15 454kg/ 34,000 lb.

TANDEM-SINGLE
SINGLE-TANDEM



INTERAXLE SPACING	ALLOWED COMBINED WEIGHT
Equal to or greater than 3.0m (10 ft.)	24 545kg/ 54,000 lb.
Less than 3.0m (10 ft.) but greater than 2.5m (8 ft. 2 in.)	23 000kg/ 50,600 lb.
Less than 2.5m (8 ft. 2 in.) but greater than 2.0m (6 ft. 6 in.)	21 000kg/ 46,000 lb.

TANDEM-TANDEM



INTERAXLE SPACING	ALLOWED COMBINED WEIGHT
Equal to or greater than 4.3m (14 ft.)	50 900kg/ 68,000 lb.
Less than 4.3m (14 ft.) but greater than 4.1m (13 ft. 6 in.)	29 900kg/ 65,600 lb.
Less than or equal to 4.1m (13 ft. 6 in.) but greater than or equal to 3.8m (12 ft. 6 in.)	28 400kg/ 62,500 lb.

Under no circumstances shall the following be exceeded :

(a) 3 650kg/ 8000 lb. per tire

(b) the capacity of the tire as determined by multiplying the cross section dimension of the tire as stamped on the tire by its manufacturer by : 10kg/ mm width of tire or 560 lb./ inch width of tire

(c) the rated capacity of the tire as stamped on the tire by its manufacturer

whichever is the lesser.

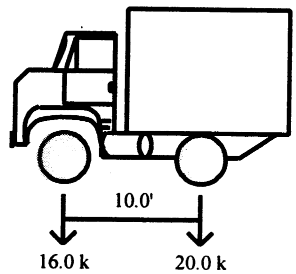
Figure A-2 Canamex Size Limits (as described in Alberta Transportation and Utilities 1994 (page 3 of 3)

APPENDIX B

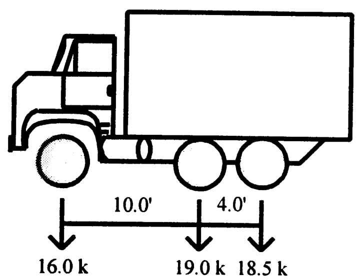
VEHICLE CONFIGURATIONS
USED FOR
SYSTEM-WIDE BRIDGE CALCULATIONS

This appendix contains weights and dimensions of the Canadian Interprovincial, Canamex, and Canamex Short vehicles used in the bridge analyses.

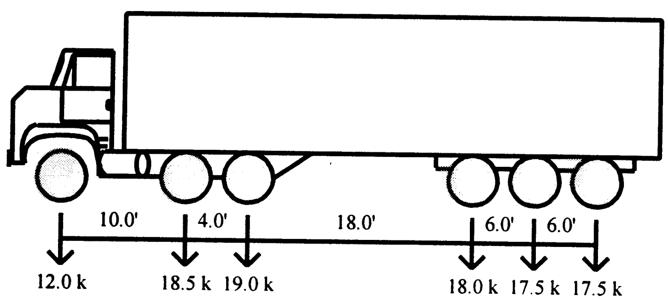
2 AXLE SINGLE UNIT



3 AXLE SINGLE UNIT



3S3



TRUCK & PONY TRAILER

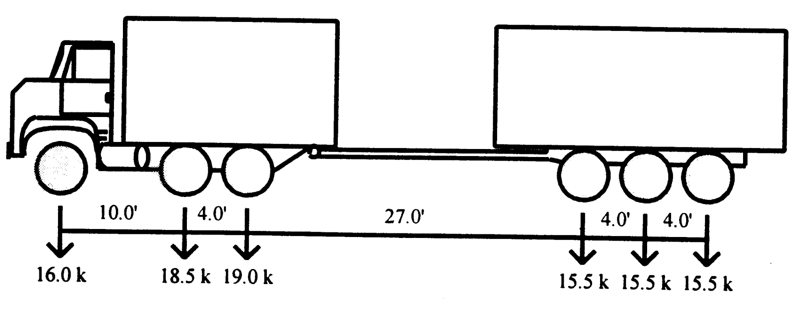
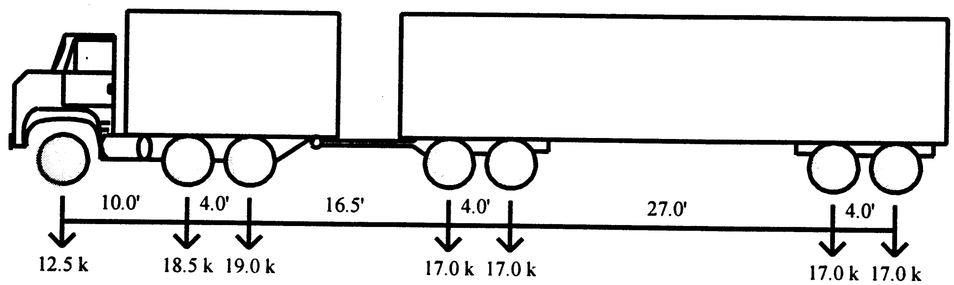
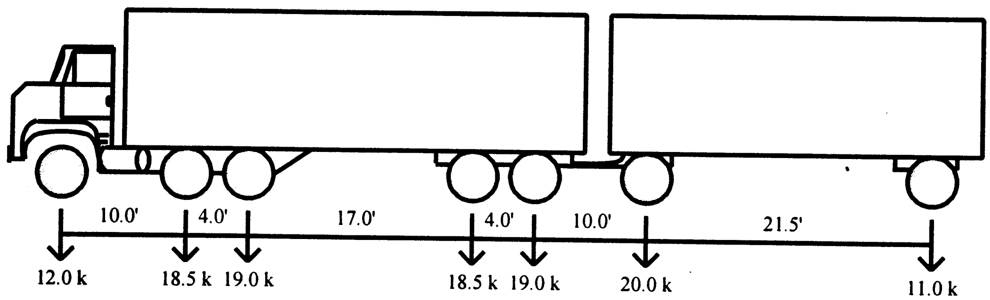


Figure B-1 Specific Weights and Dimensions Used in Calculation of Bridge Demands for Canadian Interprovincial Vehicles (page 1 of 2)

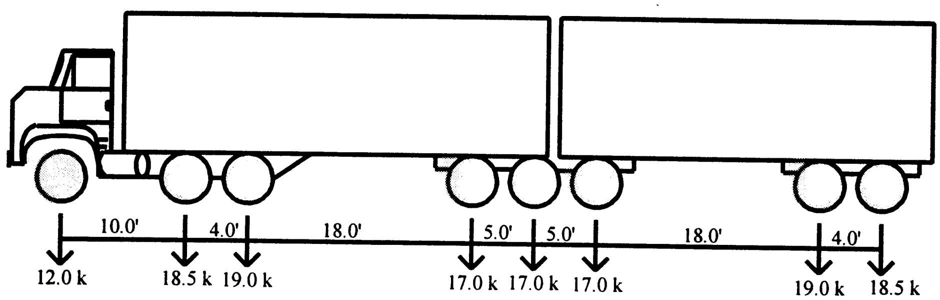
TRUCK & FULL TRAILER



7 AXLE A TRAIN



8 AXLE B TRAIN



8 AXLE C TRAIN

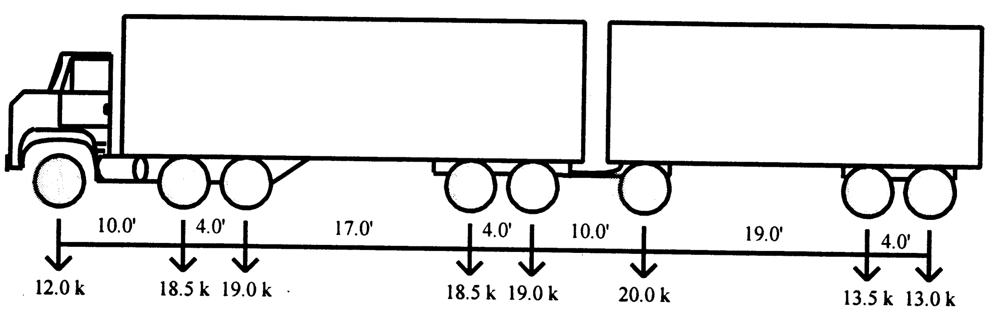
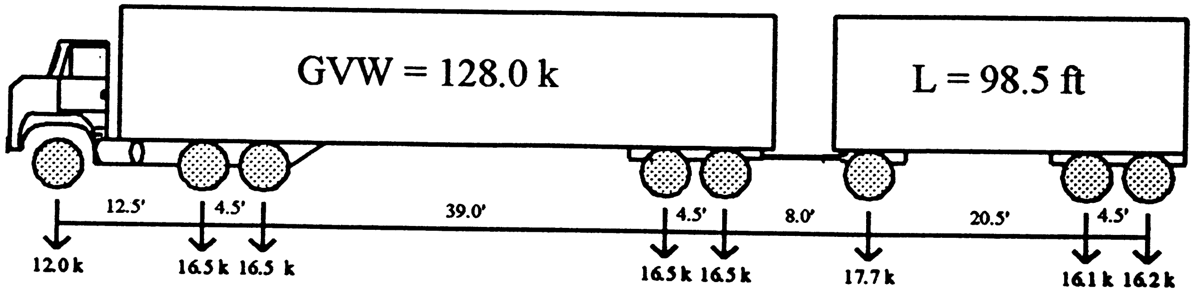


Figure B-1 Specific Weights and Dimensions Used in Calculation of Bridge Demands for Canadian Interprovincial Vehicles (page 2 of 2)

8 AXLE C TRAIN



8 AXLE C TRAIN

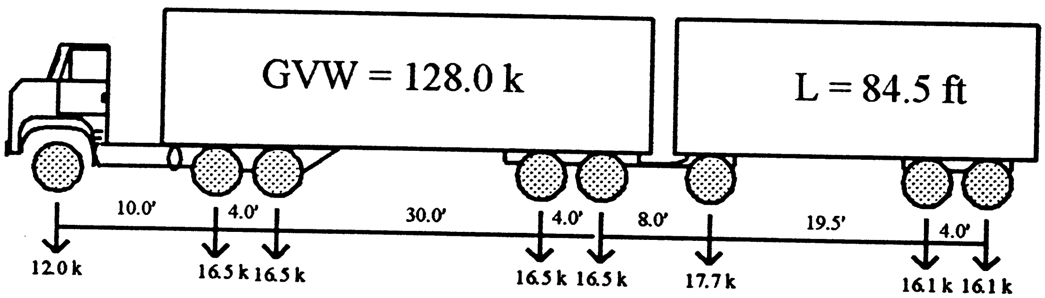
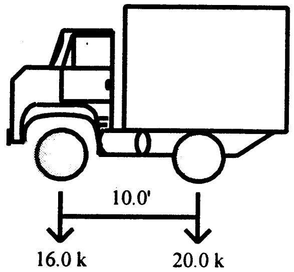
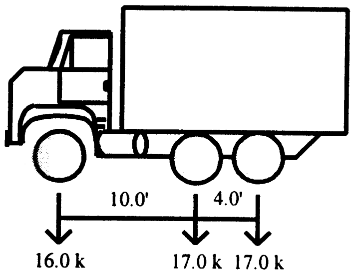


Figure B-2 Specific Weights and Dimensions Used in Calculation of Bridge Demands for the Canamex Vehicles (page 1 of 1)

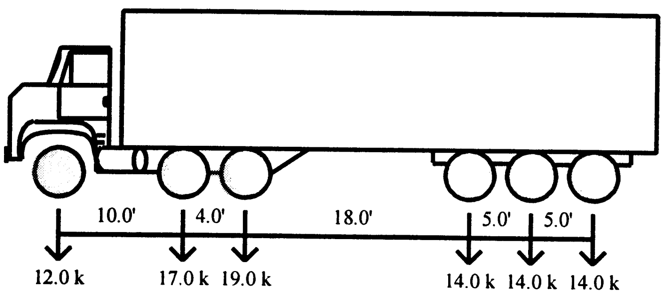
2 AXLE SINGLE UNIT



3 AXLE SINGLE UNIT



3S3



TRUCK & PONY TRAILER

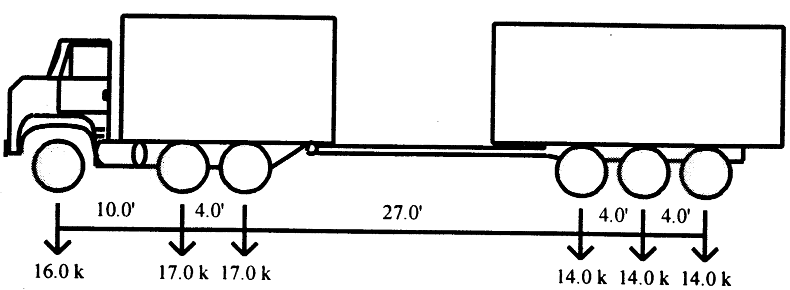
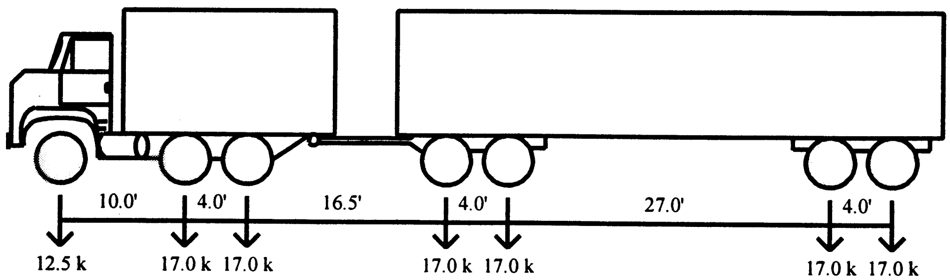
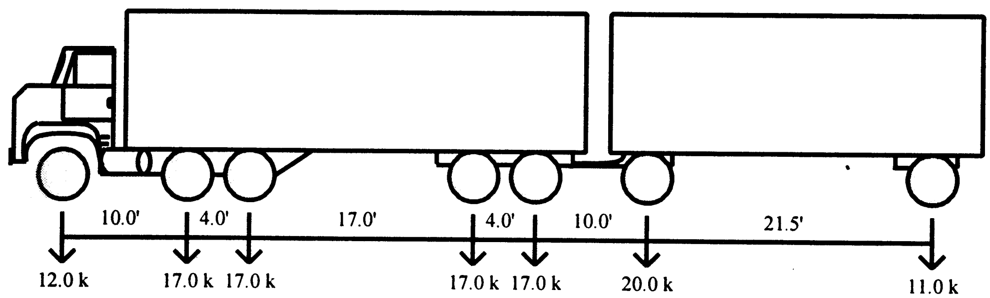


Figure B-3 Specific Weights and Dimensions Used in Calculation of Bridge Demands for Canamex Vehicles (page 1 of 2)

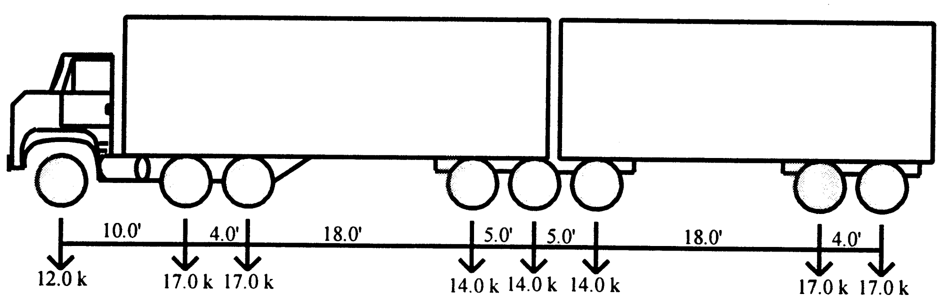
TRUCK & FULL TRAILER



7 AXLE A TRAIN



8 AXLE B TRAIN



8 AXLE C TRAIN

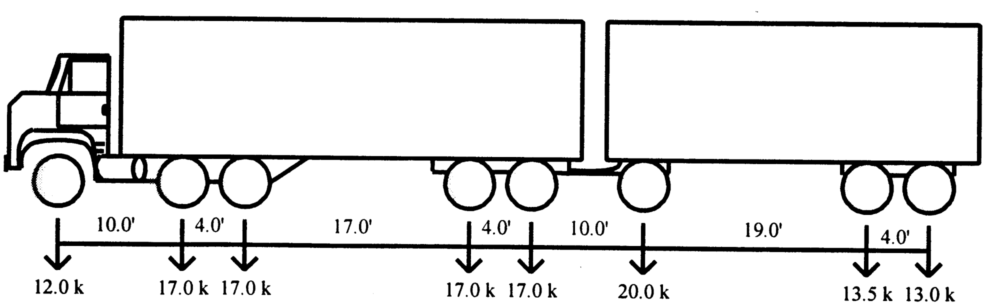


Figure B-3 Specific Weights and Dimensions Used in Calculation of Bridge Demands for Canamex Vehicles (page 2 of 2)