U.S. Department
of Transportation

**Federal Highway
Administration**

Bridge Overstress Criteria

Research and Development
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

Publication No. FHWA-RD-92-082
May 1995

FOREWORD

This report presents the results of a study conducted by the City College of New York for the Federal Highway Administration. It will be of interest primarily to planners and bridge engineers involved in bridge management, bridge rating and truck routing, and in load research.

Bridge Overstress Criteria utilizes reliability theory to approach the question of bridge capacity, and consequently provides an instructive introduction to the practical use of safety indices. This study considers the effects of a target safety index of 2.5 by applying a generalized capacity formula to a sample of bridge data taken from the National Bridge Inventory. A set of 12 existing steel, prestressed and reinforced concrete bridges was rated for the HS truck model and for the 2.5 target safety index.

Copies of this report will be distributed to FHWA division and regional offices and to State highway agencies. Additional copies for official use are available from the Federal Highway Administration, HRD-11, Turner Fairbank Highway Research Center, 6300 Georgetown Pike, McLean, Virginia 22101-2296.

Additional copies for the public may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Charles J. Nemmers

Charles J. Nemmers, P.E.
Director, Office of Engineering and Highway Operations Research and Development

NOTICE

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-RD-92-082	PB-95221040	

4. Title and Subtitle	5. Report Date
BRIDGE OVERSTRESS CRITERIA	May 1995
	6. Performing Organization Code
7. Author(s) Michael Ghosn, Charles G. Schilling, Fred Moses, and Gary Runco.	8. Performing Organization Report No.
9. Performing Organization Name and Address The City College of the City University of New York Department of Civil Engineering New York, New York 10031	10. Work Unit No. (TRAIS) NCP 3D1a 1032
	11. Contract or Grant No. DTFH61-88-C-00096
12. Sponsoring Agency Name and Address Office of Engineering and Highway Operations R&D Federal Highway Administration 6300 Georgetown Pike McLean, Virginia 22101-2296	13. Type of Report and Period Covered Final Report October 1988 - September 1992
	14. Sponsoring Agency Code

15. Supplementary Notes
Contracting Officer's Technical Representative: John O'Fallon (HNR-10)

16. Abstract
This report presents a reliability-based procedure to determine the optimal allowable loads on highway bridges considering both static and dynamic effects. A truck weight (bridge) formula was developed to provide acceptable levels of safety for bridges designed according to the 15th edition of the AASHTO specifications. Using the safety index as a measure of safety, the truck weight formula was developed to produce a safety index value of 2.5.

Twelve bridges of different material types, span lengths and configurations were analyzed for truck loads corresponding to the proposed truck weight formula. The results of the rating evaluation of these bridges showed large variations between the rating values for LFD and WSD procedures and inventory or operating stresses. Application of the higher truck weight limits to a large sample of bridges from the National Bridge Inventory indicated an increase in the number of deficient bridges if the inventory rating stress is used in the evaluation procedure. However, very few of the existing bridges would be considered deficient if the operating ratings are used.

A fatigue analysis determined the relative fatigue damage caused by various new truck types and traffic scenarios that might result from changes in truck regulations. The fatigue calculations for actual bridges suggest that many existing bridges would not be affected by the possible truck regulation changes. Even for bridges with fatigue stresses above the fatigue limit, the reduced fatigue lives with the new regulations may still be sufficient for practical requirements.

17. Key Words	18. Distribution Statement
Bridge, overstress, loads, rating	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	213	

Form DOT F 1700.7 (8-72)
(PF V2.1, 12/13/93)

Reproduction of completed page authorized

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

LIST OF FIGURES

LIST OF FIGURES (Continued)

VOLUME II

LIST OF TABLES

LIST OF TABLES (Continued)

LIST OF TABLES (Continued)

LIST OF TABLES (Continued)

LIST OF TABLES (Continued)

VOLUME II

CHAPTER ONE

INTRODUCTION

Considerable concern is being expressed by the public and government agencies over the deteriorating state of the country's infrastructure. At the same time, there is a general feeling among transportation interests that excessive regulations and weight restrictions imposed on the trucking industry are hampering efforts to strengthen the Nation's economy and plans to make it more efficient in a competitive world market. These concerns clash when it comes to determining the safe load carrying capacity of highway bridges. On the one hand, bridge owners feel that they have to control the loading of highway bridges because of the general state of deterioration of the structures; and on the other hand, the trucking industry feels that it is unfairly restricted by overly conservative bridge owners who are aware that our bridges are overdesigned and are capable of safely carrying greater loads.

The focus of this clash is the Federal legislation regulating truck weights known as the Federal-aid Highway Act. This legislation restricts the gross weights of trucks and the weights of individual axles and axle groups. The maximum vehicle weight allowed in the current law is 80,000 lb (36,300 kg). In addition, the Federal limits for single axle and tandem weights are 20,000 lb (9000 kg) and 34,000 lb (15,400 kg), respectively. The axle group weights are regulated based on what is known as the "bridge formula" (the word bridge refers to the internal vehicle bridge between axle groups and not to the structure) or the "truck weight formula." The formula which is applied to all axle groups of a truck is given by:

$$W = 500\left(\frac{BN}{N-1} + 12N + 36\right) \tag{1}$$

where: W is the overall gross weight in pounds on any group of two or more axles, B is the length in feet of the axle group, N is the number of axles in the axle group.

Equation (1) was designed to avoid overstressing HS-20 bridges by more than 5 percent and H-15 bridges by more than 30 percent.[1] The justification for these overstress ratios seems to be rather arbitrary; furthermore, parameters other than stress might be more appropriate to assure safety and serviceability. These additional parameters might include factors such as reliability or safety index, permanent deformation and accumulated fatigue damage. In this study, emphasis will be placed on the safety index to evaluate the safe load capacity of bridges. The accumulated fatigue damage will also be used to evaluate the consequences of raising the truck weight limit of steel bridges.

The overstress criteria used in the development of the original truck weight formula are based on the belief that overstressing the H-15 bridges by 30 percent is acceptable for bridges in good conditions despite the shortened life span that will be created. This was allowed because H-15 bridges were mostly built on secondary highways that are exposed to low heavy-truck volumes. HS-20 bridges however, are the current norm for Interstate highways and according to current engineering practice, overstressing them by more than 5 percent involves very high risks that should not be permitted. [2]

The bridge formula has been criticized as overly conservative and critics often cite the experience of the province of Ontario, the State of Michigan, the New England States...[3,4] These jurisdictions allow higher loads than permitted by the current Federal legislation for bridges designed based on the same AASHTO code criteria as the other States that follow the Federal truck weight limits. [5] According to these observers, highway bridges subjected to large volumes of heavy truck loads exceeding the Federal weight limits do not seem to fail nor deteriorate at a faster rate than other U.S. bridges.[2]

In Ontario, where until very recently bridges were designed according to AASHTO's code, a truck weight formula was developed based on the statistics of observed truck traffic. The formula is given as:

$$W = 10000 + 3000\ B_M - 32.50\ B_M^2 \tag{2}$$

where W in Kg is the total weight allowed for a truck or an axle group and B_M is the equivalent base length in meters. The equivalent base length is defined as the length of a uniformly distributed load which has a total weight equal to that of the truck or axle group and causes maximum moments which do not deviate unreasonably from those caused by the axle group or truck applied directly. [3]

A recent study by a group of researchers under FHWA sponsorship, developed another truck weight formula which will be referred to as the TTI formula. [1] The new formula is based on the same overstress criteria as the existing formula but is more effective in matching the specified overstress ratios. The TTI formula differs from the current formula by using only axle spacings to determine the maximum weight that can be carried by axle groups. Compared to the current regulations, TTI's formula allows more weight for short vehicles while reducing the weights for longer trucks.[6] The formula is given as:

$$\begin{aligned} W &= (34 + B)\ 1000 \qquad \text{for } B < 56 \text{ ft} \\ W &= (62 + B/2)\ 1000 \qquad \text{for } B > 56 \text{ ft} \end{aligned} \tag{3}$$

W is the gross weight in pounds for any group of axles and B is the length in feet of the axle group.

In 1987, the U.S. Congress commissioned the Transportation Research Board (TRB) to conduct a study on various issues regarding truck weight regulations.[2] At the same time, a parallel study reviewed the "Turner" proposal which would reduce the limits on axle loads while allowing increases in the gross weights. These studies developed estimates of the impacts of various proposals for changes in truck weight regulations including bridge cost impacts. A panel of experts gathered to conduct the TRB study also recommended that the bridge formula be changed to the following:

$$W = (26 + 2B)\,1000 \qquad B < 24 \text{ ft}$$
$$W = (62 + B/2)\,1000 \qquad B > 24 \text{ ft} \tag{4}$$

This TRB formula was adopted from the TTI formula by considering the stress limits on HS 20 bridges only. It would allow for significantly more weights than the TTI formula developed for both H 15 and HS 20 bridges. The TRB panel also recommended that all States should be allowed to establish permit programs for trucks with up to nine axles allowing them to carry weights over 80,000 lb (36,300 kg) when the permit vehicles satisfy the current bridge formula given in equation 1. This was based on the observation that no noticeable damage was observed under the effect of vehicles satisfying this formula. [2]

SCOPE OF THIS STUDY

The use of the arbitrary overstress ratios in the original and proposed formulas has been widely criticized. These ratios were solely based on judgment without any consideration of the increased damage due to repeated load applications or of the likelihood of overloads and simultaneous truck presence. It is widely accepted that a more rational approach should be developed based on structural reliability theory. [7] The aim is to obtain the overload capacity using statistical data on bridge safety. The steps involved in such an analysis should be based on determining acceptable safety levels using statistics on the safety margins of typical bridges including the likelihood of overloads, simultaneous truck presence and impact allowance. New safety criteria should then be used to develop a truck weight formula on the basis of limiting the number of deficient bridges with high risk of failure. This approach will thus account for the current and projected truck traffic conditions, the conditions of the existing bridge network and the funds available for rehabilitation. This report will present a reliability-based procedure to determine the optimal allowable loads on highway bridges considering both static and dynamic effects, and will review the existing truck weight formula in order to determine the impact of increasing the legal load limit based on safety and cost criteria.

The objectives of this study are: (1) To determine the optimal allowable overloads on highway bridges, considering both static and dynamic effects, and developing new appropriate criteria for safety; and (2) To review the existing truck weight formula to determine the feasibility of increasing or decreasing the legal load limit, depending upon maintenance and enforcement levels. This means that the proposed truck weight formula will be developed as a function of the existing strength of bridge members, the estimated accumulated fatigue damage and accounting for the projected fatigue safety and projected future loads including the possibility of overloads.

In this study, the objectives will be achieved using reliability theory which has been extensively used in recent years. In fact, structural code writing groups have found it advantageous to consider formal probabilistic techniques in assessing the safety of existing provisions and in introducing new code checking formats.[8] The usual application of reliability theory for bridges has been in determining the risks of structures under current loads or the calibration of new design or evaluation codes. For example, NCHRP is currently in the process of developing a program to update the AASHTO design code using formal reliability theory. [9] Also, guide specifications for the evaluation of the load capacity of existing bridges and the fatigue evaluation of steel bridges based on LRFD and reliability theory have been developed and approved by AASHTO.[10,11] The problem of developing overload criteria and consequently obtaining a truck weight formula is an inverse problem, in the sense that, we are to determine the allowable loads over a network of existing bridges with different configurations and materials. This should be based on an acceptable level of risk based on economic conditions and the level of maintenance expected in the near future.

The steps involved in applying reliability procedures to select a new truck weight formula may be summarized as follows:

(1) Choose suitable safety criteria: In this study the safety index or reliability index beta is used as the safety criterion for the evaluation of the load capacity of bridge members under bending. The effect of shear and fatigue damage are treated separately in a second stage of this study.

(2) Select an acceptable reliability level. For example, a safety index of 2.5 seems to provide a reasonable safety target based on the performance of existing bridges. A safety index of 2.5 corresponds to a probability of about 0.6 percent that the safety criteria will not be satisfied in any member of the bridge. Keep in mind that the failure of one member will not necessarily produce a complete failure of the bridge.

(3) Choose a range of typical bridges with different design criteria, span lengths, configurations, material types and capacity levels giving a representative sample of the Nation's bridges. These bridges should include simple as well as continuous spans, both steel and concrete bridges should be considered. To limit the amount of data to be handled in this study, the truck weight formula will be developed based on simple span steel bridges. The effect of the proposed formula on the other types of bridges is then evaluated in a second stage of the analysis.

(4) Use statistics on the safety margins of these typical bridges including the likelihood of overloads and simultaneous truck occurrence to find the safe loading level that will produce the target safety index.

(5) Calibrate a truck weight formula such that the effect of the truck traffic produced after the implementation of the formula will produce the required safe loading obtained in the preceding step.

(6) Check the effect of the proposed truck loads on the existing network of bridges. Also, check the additional fatigue damage and estimate the effect on the fatigue life of existing bridges.

(7) Verify that the number of bridge deficiencies under the new regulation will be acceptable in terms of the additional costs required to maintain the existing bridge network.

BRIDGE SAFETY

Bridges are initially designed with relatively high safety factors in the sense that the overall load effects that they are expected to carry are generally lower than what the design codes stipulate. Over the lifetime of the structure, however, several factors affect the performance of the bridge. Some of these factors are: (1) Errors in design and construction that produce a lower safety factor than initially intended. (2) Deterioration of the structure due to environmental effects and the lack of appropriate maintenance levels. (3) Overload distress caused by extra-heavy loads. (4) Fatigue distress caused by numerous applications of normal loads.

On the other hand, a bridge, like other structures, have additional reserve strength that is not usually accounted for in the current simplified design criteria. These reserve strengths often lead to higher margins of safety than originally assumed. The reserve strength is usually difficult to estimate, as it is related to structural details, the procedure followed during construction, site location and traffic conditions. The factors contributing to these reserve strengths include: (1) Simplified conservative load distribution factors given in the design specifications which tend to overestimate the component forces. (2) System effects; bridges are designed on a component by component basis, however the interaction of the various components produce a much higher system capacity than predicted. (3) Nominal material strengths used in design are lower bound values with a high probability of being exceeded. (4) The combination of nominal loads used in design are worst case loads that are unlikely to be actually applied to the bridge.

In the last few years, there has been an increasing awareness of the importance of correlating design and evaluation safety factors with the uncertainties inherent in estimating the strength of members, the behavior of structural systems and the actual loading of the bridges. One step in that direction was the adoption by AASHTO of Load Factor Design (LFD) which specifies different factors for different loads based on the uncertainty associated in determining these loads. [5] The Ontario Bridge Design Code went one more step in that direction by calibrating the safety factors using formal reliability theory. [12] AASHTO has already adopted as guide specifications the reliability-based formulations developed for fatigue evaluation of bridges and for load capacity evaluation. [10,11] Finally, as already noted, NCHRP is developing a new bridge design specification based on reliability principles. [9]

A reliability-based methodology consists of analyzing the safety margin defined by:

$$Z = R - S \tag{5}$$

where Z is the safety margin, R is the capacity, S is the total load effect. The total load, $S = D + L + E + \ldots$ where D is the dead load effect, L is the live load effect, E is the environmental load effect, etc. The risk is often described by a safety index (beta). Beta gives the number of standard deviations that the mean of the safety margin is on the safe side, or:

$$\beta = \frac{\bar{Z}}{\sigma_Z} \tag{6}$$

where $\bar{Z}$ is the mean safety margin and σ_Z is the standard deviation of the safety margin. The safety index β is directly related to the probability of failure if all the variables follow a normal distribution. The same safety measure, however, can still be used for non-normal distributions as a relative measure of risk. Targets for acceptable safety levels are obtained by evaluating the safety index of existing bridges. The safety index is a function of the mean and standard deviation of each term in Z. These in turn are affected by the design values and their uncertainties. For existing bridges, safety will be related to the level of deterioration (i.e. actual current strength rather than as-designed strength), serviceability and the truck loading at the site. It is these unknowns, in addition to the procedures followed in designing the existing bridge, that will affect the levels of safety and determine the level of loads that can be safely applied. Chapter Two of this report presents a more complete review of reliability theory and practice as it relates to bridge safety analysis and development of bridge safety criteria.

EVALUATION OF MEMBER CAPACITY

To quantify the effect of some of the factors mentioned above, several experimental and analytical models were developed to determine bridge member capacity for the analysis of bridge safety. For example, statistical models for beams and columns were established for buildings but some of the results are also applicable for bridge members.[13] Models were developed to analyze the member capacity of prestressed concrete bridge girders, composite steel bridge beams, and bridge slabs.[14,15,16] Several studies reviewed the performance of steel members for AISC and are currently reviewing existing bridge evaluation procedures and investigating the possibility of considering the inelastic behavior of steel beams and girders in the evaluation process.[17,18] The plastic capacity of compact beams has been thoroughly investigated and Autostress Guide Specifications based on plastic design concepts have recently been adopted by AASHTO.[19] Considerable data on member behavior under regular truck traffic has been assembled by several researchers using Weigh-In-Motion studies that provided information on the response of bridge members including stress levels, composite action, dynamic effects and stress distributions.[20,21]

Bridges are subjected to millions of truck crossings in their lifetime. Therefore, damage accumulation models are important for the analysis of the safety of bridge members. Test data for different types of bridge components and materials are available from experiments conducted at Lehigh University and the University of Maryland.[22,23] Additional experimental studies are being conducted under FHWA sponsorship at the universities of Pittsburgh and Maryland.[24] Techniques for reliability based fatigue design and life prediction have been developed for prestressed concrete and steel members.[11]

EVALUATION OF SYSTEM BEHAVIOR

Bridges are designed as a combination of single members; but analytical methods to estimate the ultimate capacity of bridges beyond the first member's failure have been developed. Recent full-scale testing of real bridges has shown that they have considerable reserve strength exceeding the analytical estimates.

The analytical techniques developed for determining the ultimate strength of bridges are divided into two categories: (1) Plastic analysis that uses methods such as the yield line theory or that studies the various collapse mechanisms to predict the ultimate load behavior of bridges. (2) Elastic-plastic methods that follow the complete behavior of bridge structures until collapse. These usually use either the finite difference or the finite element methods. For example, finite element programs to predict the overload response of reinforced concrete, prestressed concrete and composite steel beam bridges were developed in references [25,26]. The importance of structural redundancy in the safety evaluation of highway bridges was studied in reference [27]. The review concludes that very few bridges will collapse if only one of the main load carrying members fails. The lateral distribution of live loads to longitudinal members was studied and the conservatism of the current AASHTO method was noted.[28] Current techniques for the determination of lateral load distribution factors were reviewed and a number of alternative simplified methods were proposed.[29]

Experimental studies have been concerned with either model or single member testing in laboratories or field testing under regular or proof loads. Some full scale, ultimate load tests have been performed on actual bridges, but the results have been mainly used to check the validity of the analytical methods. The maximum load carrying capacity of some bridges was studied and the test results were compared to several analytical models.[30,31] Using proof load tests, experimental results were compared to analytical predictions. Modeling the support conditions was observed to be one of the major weaknesses encountered in analytical analyses.[32,33] It was also noticed that bridges often exhibit much higher strengths than predicted under even the most sophisticated analyses. A report for NCHRP attempted to explain the reasons for different load capacities as determined by test measurements when compared to analytical results.[34] The following contributing factors were observed: (1) Load distribution effects, (2) unintended composite action, (3) unintended end support restraint, (4) unintended continuity, (5) effect of floor system and secondary members, (6) dynamic effects, (7) membrane slab action and (8) actual versus assumed material properties. The same factors were cited in another study in addition to the effects of deterioration, strain hardening and skews.[35] The importance of making appropriate assumptions about the boundary conditions has also been emphasized.[36]

BRIDGE LOAD MODELS

The determination of the mean and standard deviation of the load models are of primary importance in evaluating the safety of existing or planned bridges. Dead load models are available from research performed in the development of the Ontario Highway Bridge Code.[37] Live load models for heavy truck crossings are more difficult to estimate; considerable work has been performed on that subject in references [38,39]. An extensive data base obtained from weigh-in-motion studies has been utilized in previous reliability-based analyses to determine all factors pertaining to bridge loading.[20] The available data includes distribution of load to individual members, dynamic effects, and multiple truck presence. In addition, truck weight statistics, axle weight distribution, and fatigue load spectra were obtained.[24,40] This work is currently being expanded and the model is being modified to reflect truck weight data from Wisconsin and Florida with emphasis placed on the statistics of the extreme permit and illegal weights.[41]

A current study at the University of Colorado is also acquiring weigh-in-motion truck load data. This information will be very helpful in providing additional information on truck loads and truck load effects.

For this study, the available data and models will be extrapolated to obtain various loading scenarios corresponding to different proposed load limits and the final load limits to be obtained.

EFFECTS OF CHANGING THE BRIDGE FORMULA

The actual fatigue life of a bridge depends on the actual truck traffic that passes over it rather than on the assumed design or rating vehicle. The present truck traffic can be broken down into six main truck types.[11,24] The statistics of the gross weights, axle spacings and axle weights for each truck type have been determined from nationwide weigh-in-motion studies.[20,39,40] The percentages of the different types of trucks in the traffic have also been determined from these studies. To assess the effect on fatigue life stemming from a change in the bridge formula, it is necessary to first estimate how the present truck traffic would be altered by such a change. It is possible that the change would result in the production of new truck types; or, it might merely change the percentages of the present types in the traffic. In either case, it would be necessary to modify the present truck traffic data to represent expected conditions after the bridge formula is changed. Different assumptions regarding the changed character of truck traffic should be investigated. The fatigue damage caused by the present and modified truck traffic can be accurately calculated by procedures developed in several NCHRP and FHWA projects.[11,24,42] These effects are expected to be greatest for steel bridges, but almost negligible for concrete bridges. The fatigue behavior of prestressed concrete bridges is still under investigation and little is known about the expected damage under traffic conditions. This approach, which considers expected changes in the actual truck traffic, is the only accurate way to assess the effects of changes in the bridge formula on fatigue. A report to NCHRP and TRB describes the costs of bridge upgrading and replacement due to: (a) Changes in the legal weight limits as proposed by FHWA's Turner Truck Study, (b) various proposed scenarios for truck weight changes, and (c) elimination of grandfather exemptions.[43] This study will rely on these techniques in order to study the effect of the changes in the truck weights and truck traffic if the proposed criteria are used to develop a new truck weight formula.

REPORT OUTLINE

The approach proposed for this study utilizes structural reliability theory to analyze the effect of increasing the legal load limit upon the safety of the nation's bridges. The study will consider the state of the existing bridge network and will account for the expected level of enforcement. The existing accumulated fatigue damage will be included in the analysis in addition to the actual behavior of bridges under regular loads and overloads.

Chapter one of this report summarizes the objective of this project, outlines the procedure to be followed and gives a description of the pertinent factors that need to be considered in developing the work plan.

Chapter two gives a background on the theory of structural reliability and details the live load models and their applicability to the objectives of this study. Also in chapter two, a method is developed to obtain a truck weight formula and the results are presented for the base case.

Chapter three looks at the sensitivity of the results to the assumptions made while developing the base case and investigates the consequences of the adoption of the proposed formula on the existing bridge network in terms of increased number of deficient bridges and estimated costs.

Chapter four presents the results of a detailed analysis of 12 typical existing bridges in terms of effects of implementing the proposed truck weight formula and costs of upgrading the deficient bridges.

Chapter five performs the fatigue analysis and studies the increased fatigue damage expected after the implementation of the proposed formula.

Finally, chapter six summarizes the work and the results obtained during the conduct of this study.

CHAPTER TWO

CALCULATION OF TRUCK WEIGHT FORMULA

Historically, truck regulations have maintained controls on axle and gross weights with legal load formulas based on limiting allowable stresses in certain types of bridges. These stress limitations do not usually lead to consistent or defensible safety levels and also ignore the cost impact of the weight regulation on all highway systems. This chapter illustrates how new truck weight regulations can be developed to provide acceptable safety levels. Target safety levels are derived from existing AASHTO bridge evaluation and rating procedures apllied to structures showing safe and adequate performance levels. Reliability indices are used to relate the statistics of bridge load effects based on existing or proposed truck weight regulations to the dynamic behavior and resistance variables of existing bridges.

INTRODUCTION

In the United States, the maximum vehicle weight allowed in the current law for general operation is 80,000 lb (356 kN). In addition, the Federal limits for single axle and tandem weights are 20,000 (89 kN) and 34,000 lb (151 kN), respectively. The axle group weights are regulated based on what is known as the "Bridge Formula" given in equation 1. A recent study under FHWA sponsorship developed a new bridge formula based on the same overstress criteria as the existing formula. The new formula is more effective in preventing stresses from exceeding the specified overstress ratios. It is accepted that a more rational approach based on structural reliability theory is needed.[7] The steps involved in such an analysis are based on determining acceptable safety levels using statistical analysis of the safety of typical existing bridges. This

Preceding page blank

analysis should consider the likelihood of overloads, simultaneous truck presence, impact allowance and live load distribution. New safety criteria should then be developed on the basis of limiting the number of posted bridges based on traffic and funds available for rehabilitation.

The objectives of this chapter are to present a reliability-based procedure to determine the optimal allowable overload on highway bridges considering both static and dynamic effects. The desirable safety levels will be calculated according to reliability theory that has been extensively used in recent years. The usual application of reliability theory for bridges has been in determining the risks of structures under current loads or the calibration of design or evaluation codes. In this study, the steps to follow consist of working backward starting from a desired level of safety to obtain the maximum permissible overload.

STRUCTURAL RELIABILITY THEORY

Load intensity, bridge response and structural strength parameters are not known with certainty. The aim of structural reliability theory is to account for the uncertainties in evaluating the load carrying capacity of structural systems or in the calibration of safety factors for structural design codes. Such uncertainties may be represented by random variables and their probability distributions...[8]

The value that a random variable can take is described by its probability law which is characterized by a probability distribution function. That is, a random variable may take a specific value with a certain probability and the ensemble of these values and their probabilities are described by a distribution function. The most important characteristics of a random variable are its mean value or average, and the standard deviation which gives a measure of dispersion or a measure of the uncertainty in determining the variable. The standard deviation of a random variable R with a mean $\overline{R}$ is defined as σ_R. A dimensionless measure of the uncertainty is the coefficient of variation (COV) which is the ratio of standard deviation divided by the mean value. For example the COV of the random variable R is defined as V_R such that:

$$V_R = \frac{\sigma_R}{\bar{R}} \qquad (7)$$

Typical COV's for structural applications range from 8 to 15 percent for material strength, 5 to 10 percent for dead load, and 15 to 30 percent for live load and even higher for wind and seismic effects.

Codes often specify safe or nominal values for the variables used in the design equations. These nominal values are related to the means through bias values. The bias is defined as the ratio of the mean to the nominal value used in design. For example, if R is the member resistance, the mean of R, $\bar{R}$ can be obtained from the nominal or design value R_n using a bias factor such that:

$$\bar{R} = b_r \ R_n \qquad (8)$$

where: b_r is the resistance bias and R_n is the nominal value as specified by the design code. For example, A36 steel has a nominal design yield stress of 36 ksi (248,220 kPa) but coupon tests show an actual average value close to 40 ksi (275,800 kPa). Hence the bias of the yield stress is 40/36 or 1.1.

In structural reliability, safety may be described as the situation where capacity (strength, resistance, fatigue life, etc.) exceeds demand (load, moment, stress ranges, etc.). Probability of failure, i.e., probability that capacity is less than applied load, may be formally calculated; however, its accuracy depends upon detailed data on the probability distributions of loads and resistances. Since such data is often not available, approximate models are used for calculation.

Let the reserve margin of safety of a bridge component be defined as, Z, where:

$$Z = R - S = R - (D + L) \qquad (9)$$

R is the resistance or member capacity , S is the total load effect (S=D+L), D is the dead load effect, and L is the live load effect.

Probability of failure P_f is the probability that the resistance R is less than the total applied load effect S. This is symbolized by the equation:

$$P_f = Pr [R < S] \tag{10}$$

If R and S follow independent normal distributions then:

$$P_f = \Phi [- \frac{\bar{R} - \bar{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}}]$$

$$P_f = \Phi [- \frac{\bar{Z}}{\sigma_Z}]$$

$$P_f = \Phi [- \beta] \tag{11}$$

where Φ is the normal probability function that gives the probability that the normalized random variable is below a given value. $\bar{Z}$ is the mean safety margin and σ_Z is the standard deviation of the safety margin. The safety index is defined as:

$$\beta = \frac{\bar{Z}}{\sigma_Z} \tag{12}$$

The safety index (β) is often used as a measure of structural safety. β gives the number of standard deviations that the mean margin of safety falls on the safe side.

β as defined in equation 12 provides an exact evaluation of risk (failure probability) if R and S follow normal distributions. Although β was originally developed for normal distributions, similar calculations can be made if R and S are lognormally distributed. A random variable R whose logarithm is normally distributed is said to have a lognormal distribution. If both R and S have lognormal distributions then the failure function can be defined as:

$$Z = \ln R - \ln S \tag{13}$$

and

$$\beta = \frac{\ln\left[\frac{\bar{R}}{\bar{S}}\sqrt{\frac{V_S^2+1}{V_R^2+1}}\right]}{\sqrt{\ln\left[\left(V_R^2+1\right)\left(V_S^2+1\right)\right]}} \tag{14}$$

V_S and V_R are the COV of S and R respectively. In general, β's from either normal or lognormal models are used as estimates of the reliability of a structural member even if its capacity and applied load are neither normal nor lognormal. To improve on these estimates "Level II" methods have been developed.[8] Level II methods involve an iterative calculation to obtain an estimate to the failure probability. This is accomplished by approximating the failure surface (i.e. when Z=0) by a tangent multi-dimensional plane at the point on the failure surface closest to the origin. A more detailed explanation of these principles and derivations of the equations given in this chapter can be found in reference [8].

The safety index approach has been used by many code writing groups throughout the world to express structural risk. β in the range of 2 to 4 is usually specified for different structural applications. Structural safety calculations for bridges differ somewhat from other applications because truck loads (which constitute the dominant live load) increase with time due to new truck regulations and increases in truck volume. Meanwhile member capacity is decreasing due to inadequate maintenance and environmental effects. Thus, for new bridge constructions, β is relatively high, say on the order of 3.5. But, over a bridge's life span, a typical β may fall to about 2.5. A β of 3.5 implies about a 0.000233 risk or a probability of failure of about 0.0233 percent while a beta of 2.5 corresponds to a probability of failure on the order of 0.621 percent. These values usually correspond to the failure of a single component. If there is adequate redundancy, overall system safety indices (β) will be higher.

β is not calculated solely for making statistical risk statements but rather for recommending the proper load and strength safety factors for design or evaluation specifications. One commonly used approach is that each type of structure should have uniform or consistent reliability levels over the full range of applications; e.g. similar β values should be obtained for bridges of different span lengths, number of lanes, simple or continuous spans, roadway categories, etc. Thus, a single target beta must be achieved for all applications. Some engineers and researchers on the other hand are suggesting that higher values of β should be used for longer spans or for bridges that carry more traffic. This latter approach has not been accepted yet and no practical mechanism has been developed to determine the distribution of β with span length. For this reason, this study will use a single β value as the target safety index for all bridge spans.

Appropriate target β are obtained based on existing designs. That is, if the safety performance of bridges designed according to current criteria has generally been found satisfactory, then the safety index obtained from current designs is used as the target that any new design should satisfy. This calibration with past performance also helps to minimize any inadequacies in the data base as has been previously reported and as will be seen later in this report. [39]

The calibration effort is usually executed by code groups as follows:

- Safety indices are calculated for current code design and performance of existing structures based on statistical information about the randomness of the strength of members and the statistics of applied loads. For medium to short span bridges, the load S in equation 9 is divided into two parts: Dead load and live load. R on the other hand is determined by looking at the statistics of the resistance of typical bridge members. This is usually done for a range of applications such as different span lengths, beam spacings, materials and traffic conditions.

- In general, there will be considerable scatter in such computed safety indices. If the existing code is believed to provide an average satisfactory performance, a target β can then be directly extracted. This is done by examining the performance and experience of selected bridge examples and averaging the β values.

- For the development of new design codes, safety factors and design loads and strengths for a new format are selected by trial and error to satisfy the target β as closely as possible for the whole range of applications. Similarly, for the development of new truck weight regulations, maximum permissible live load moment envelopes will be determined by trial and error to satisfy the target safety index for all bridge types considered. Then the truck weight formula that will produce the permissible live load envelope is determined.

This chapter will illustrate how these live load envelopes and a truck weight formula can be developed based on rational reliability concepts. The next section discusses the basic statistical data base required to execute the calculations; and section following that describes the step-by-step procedure developed for this study.

DATA BASE AND LIVE LOAD MODELS

To execute the safety index calculations, one needs to obtain the statistical data of all the random variables that affect the safety margin Z of equation 9. These are the member resistances, the dead load effect and the live load effect. Experimental and simulation studies have developed statistical estimates of member resistances for different types of bridges. Data on the live load statistics, however, are less common; in fact, besides the limited data from the weigh-in-motion studies, little information is available on bridge-related truck load statistics in the United States.[20,21] The statistical data used for new designs has to be averaged from several sites because no specific information is available on the volume of trucks or the loading and response of a bridge before it is built and opened to traffic. Similarly, average statistical values for truck volumes, truck types and bridge responses are used herein for the safety index calculations and the development of new truck weight regulations. This section presents a summary of the statistical data used by the author and his colleagues in this study and in several other studies on bridge reliability.

Dead Load

Dead load effects are obtained from the self-weight of the structure including the weight of the wearing surface and other non-structural elements. The dead load effect for steel members was found to be related to the design live load and the span length by the formula:[44]

$$D_n = 0.0132 \, (L_n + I_n) \, SL \tag{15}$$

where L_n is AASHTO's design live load effect on a member. I_n is the nominal impact load effect on the member. SL is the span length in feet. The mean dead load value $\bar{D}$ was also obtained from equation 15 i.e. the dead load bias is estimated at 1.0. The dead load coefficient of variation used is 9 percent based on the typical values given in reference 10. A similar relationship between the dead load and span length was used for prestressed concrete bridge members based on the data provided in reference 1:

$$D_n = 0.014 \, (L_n + I_n) \, SL \tag{16}$$

A similar formula has been recommended for concrete T beams such that: [45]

$$D_n = (L_n + I_n) \, (0.6967 - 0.00762 \, SL + 0.0002554 \, SL^2) \tag{17}$$

Resistance Data

Statistical data including biases and COV for different categories of steel members and prestressed concrete members were established in reference 10 based on earlier research work. [17,13] For example, steel members in new condition were assigned a bias of 1.1 relative to the nominal capacity as specified by AASHTO procedures and a COV of 12 percent. Partially corroded steel members with some slight loss of section were associated with a bias of 1.05 and a COV of 16 percent. Severely corroded sections with noticeable loss of section have a bias of 1.0 and a COV of 20 percent. For prestressed concrete members in good condition a bias of 1.15 and a COV of 8 percent were used. A bias of 1.1 and a COV of 12 percent were recommended for concrete T beams. These biases and coefficients of variation account for the uncertainties in the material properties, fabrication and scatter in prediction theory.

Live Load Modeling

Bridges are designed to safely withstand the maximum load expected over the service lifetime of the structure. In short to medium span bridges, maximum live load is usually due to the occurrence of several heavy trucks simultaneously on the bridge. Each occurrence of one or more vehicles on the bridge (herein called a loading event) is characterized by the number of trucks in the event, their gross weights, axle spacings, axle weight distribution and the relative position of these trucks with respect to each other. All these factors are random variables which should be accounted for in a model to calculate the maximum loading on a bridge.

Simulation programs have been developed to study the truck loading problem.[46] In these programs, the bridge surface is divided into rectangular slots and a truck loading event occurs when there is at least one truck on any one of the assumed slots. The first truck that arrives on the bridge as part of a loading event is considered the "main" truck. The probability of having the main truck in a given lane can be obtained from the truck traffic statistics for a site. For example, on a two-lane section of Interstate I-90 in Ohio, it was found that 83 percent of the trucks travel in the right lane.[20,21]

The possible combinations of vehicles in all the slots can be obtained and each truck combination is associated with a probability of occurrence. This probability will be referred to as the headway combination probability and can be calculated based on field data. In reference 46, the headway combination probability was calculated based on the truck arrival data gathered from the weigh-in-motion field measurements.[20,21] The weigh-in-motion data includes the conditional lane occurrence probability and the probability of slot occupancy. These conditional probabilities might be site and traffic dependent. Field headway data is only available for two-lane highways, thus the results obtained herein are valid for two-lane bridges only.

The conditional lane occurrence probability gives the probability that a vehicle occupies a certain lane if the lane occupied by another vehicle that arrived on the bridge ahead of it is known. For example, given a main truck in the right lane, the probability that the second truck occurrence is also in the right lane for the I-90 site is measured as 83.5

percent. Given a main truck in the left lane, the probability that the following truck is in the right lane was measured to be 85.4 percent.

The probability of slot occupancy is obtained from the probability which gives the location of the second truck relative to the previous truck. For example, given that the main truck is in the right lane, and given that the next truck is in the left lane, the probability that the two trucks occupy adjacent slots (side-by-side case) is given as 5.8 percent. The probability that the two trucks occupy consecutive slots is 5.2 percent.

The final headway probability for each simultaneous occurrence of trucks (loading event) is the product of the probability of the lane occupied by the main truck, times the conditional lane occurrence probabilities of the following trucks, times the probabilities of slot occupancy.

In reference 46, each truck involved in the loading event was assumed to be either of a single unit type or a semi-trailer type. Each truck in the event will also have a different gross weight. Depending on the type, each truck involved in the event will be associated with a gross weight and a corresponding probability obtained from gross weight histograms for the different truck types considered.

Given the truck positions and given the gross weights of all the trucks in the event, the maximum moment response associated with the event can be easily calculated from the influence line of the bridge. The response of the bridge due to the event is also associated with a headway probability and probabilities of the gross weights of the trucks. The corresponding moment response is then associated with a probability equal to the product of the headway probability and the gross weight probabilities. This assumes independence between the headway and the gross weights and between the gross weights of the different trucks in the event. Thus far, weigh-in-motion data have not shown any correlation between headways and weights. This observation however can be modified pending additional field data. Of particular importance to most bridges is the correlation of side-by-side heavy trucks.

The maximum moment calculation is executed for all possible truck weights and all possible truck combinations. A histogram giving each calculated bending moment and its associated probabilities can be assembled. This histogram gives the cumulative probabilities for the occurrence of one loading event: $F_X(x)$. The number of events in a one day period can be estimated from field data. For example, an average interstate site will have about 2000 such events/day. To calculate the maximum response over the lifetime of a bridge herein assumed to be 50 years, the number of events N is 36.5 x 10^6 and the probability distribution of the maximum lifetime response is:[46]

$$G_m(T) = [\,F_X(x)]^N \tag{18}$$

where $G_m(T)$ is the cumulative maximum probability associated with the maximum moment response for a projection period T (T=50 years) which corresponds to the number of events, N over this period.

Reference 46 demonstrates that a good representation of the tail of the weight histogram at a given site can be obtained from the gross weight value corresponding to the upper 5 percent fractile of all the gross weights collected at that site. This characteristic gross weight will be denoted here as $W_{.95}$. Also, a good representation of the maximum lifetime response was found to be the median of the maximum moment distribution (i.e. 50 percent fractile of the maximum moment). The headway factor H is defined as the ratio of the median of the 50 year maximum moment and the maximum moment due to one standard truck with a gross weight equal to $W_{.95}$. H was calculated for different sites and span lengths. This H ratio was found to be consistent for each span within an estimated standard error less than 7 percent for the sites investigated. The H ratio is then a multiplicative factor that relates the maximum moment due to a truck with a weight equal to $W_{.95}$ to the median of maximum moment over a period of 50 years. While $W_{.95}$ is site dependent, H was found to be consistent from site to site assuming similar truck volumes and traffic conditions.

So far in this analysis, the axle spacings and axle weight distribution for each truck type have been assumed to be constant at given values corresponding to the standard simulation trucks. To account for this limitation in the analysis, a correction factor m is introduced; m represents the variation of a random truck effect on a bridge compared with the effect of the standard simulation truck. This m factor is calculated as the ratio of the maximum response of a random truck to the maximum response of a standard semi-trailer truck or a single unit truck of the same gross weight. To be exact, m should be a function of the location of each vehicle in the loading event when the maximum moment is calculated. Thus, the values of m calculated as explained herein should be applied as a correction only to the response of the trucks located at the critical point of the bridge. This fact is neglected and m is used as a correction factor on the total response.

Based on the results of the discussion of the previous paragraph, the median of the total response of the maximum load in 50 years for a general truck traffic at a given site is approximated by the load formula [46]:

$$M = a\, m\, W_{.95}\, H \tag{19}$$

where a is a deterministic value dependent on the standard truck configuration used in the simulation, the span length and the response variable (midspan moment, end shear...). m is a random variable reflecting the type of truck traffic configuration present at the site e.g. single unit trucks, semi-trialers, etc. It is also a function of span length. H is a random variable and gives the overload factor due to the presence of closely spaced vehicles, side-by-side and following vehicles. H also reflects the probability that vehicle weights exceed the 95th percentile in combination with closely spaced events. It was found from the simulation model as discussed in the previous paragraph and in reference 46. H is a function of the truck volume and depends on the span length. $W_{.95}$ is a 95th percentile characteristic value of the truck gross weights and is assumed to be a random variable to reflect possible errors in the estimation of the variable and to reflect the difference values from one site to another.

Equation 19 is used in estimating the maximum live load applied on a bridge structure in its lifetime (usually taken as 50 years). This equation gives the total static load on a bridge. To obtain the load effect on a member under highway traffic, two additional factors are required and these are the impact factor (or dynamic amplification factor), i, and the girder distribution factor, g. The total load effect on a bridge member L is then the product of the maximum lifetime static load effect (equation 19) the girder distribution factor and the dynamic amplification factor:

$$L = a\ m\ W_{.95}\ H\ g\ i \tag{20}$$

For a 50-year design, possible growth in the weights of heavy trucks traveling over the highways should be included in the reliability analysis. The approach adopted here is to include load growth explicitly as one of the variables denoted as Gr. A mean Gr factor of 1.15 along with a C.O.V. of 10 percent were assumed for the evaluation of the safety indices. The live load formula used in the safety index calculations for new designs becomes:

$$L = a\ m\ W_{.95}\ H\ g\ i\ Gr \tag{21}$$

Except for the factor a, all the variables of equation 21 are random variables with statistics based on examination of a number of sites. Table 1 gives the values of a, m and H obtained for a 50-year projection of the maximum load effect based on weigh-in-motion data collected at several sites.[46]

Table 1. Input data for reliability analysis.

Span (ft)	a (kip-ft)	m mean	m C.O.V.	H mean	H C.O.V.
30	6.07	0.92	15 %	2.63	10 %
40	8.57	0.93	12 %	2.69	10 %
60	13.57	0.94	6 %	2.75	10 %
80	13.40	0.93	9 %	2.78	7 %
100	18.40	0.95	7 %	2.80	7 %
125	24.40	0.96	6 %	2.86	7 %
150	30.90	0.96	5 %	2.87	7 %
175	36.90	0.97	4 %	2.98	7 %
200	43.40	0.97	4 %	3.05	7 %

1 ft = 0.3048 m
1 kip-ft = 1.356 kn-m

Statistical data based on field measurements and theoretical analysis were collected on the load distribution factor g.[10] They showed that for steel bridges a bias of 0.90 with a COV of 13 percent exists between the AASHTO recommended load distribution factor and the values obtained by researchers. A bias of 0.90 with a COV of 8 percent was however, used herein based on the field data and the calculation given in references 20, 21 and 46. The decrease in the COV to 8 percent is partially due to the 50-year projection of the maximum expected load distribution factor. For concrete T-beams, the bias obtained was 1.01 with a COV of 5 percent. Prestressed concrete bridges were associated with a bias of 0.96 and a COV of 8 percent.[10,45]

Similarly, the impact factor was found to be a function of surface roughness.[10] Three different values were recommended for the mean dynamic impact. These are 1.1, 1.2 and 1.3 for smooth, medium and rough surfaces respectively and these were all associated with a COV of 10 percent. A value of 1.2 and a COV of 8 percent were used in these calculations for an average site for steel and prestressed bridges based on the field data and the 50-year projections of the data as performed in reference [46]. Concrete T beam bridges are associated with a mean impact factor of 1.15 and a COV of 10 percent.

Two different $W_{.95}$ values are used depending on whether the span length is less than or more than 60 ft. For spans less than 60 ft (18.29 m), the single unit trucks produce the critical loads; for spans longer than 60 ft (18.29 m), semi-trailers control the loading. These $W_{.95}$ values are 47 kips (209 kN) and 75 kips (333.6 kN) with 15 percent and 10 percent COV's respectively.[46]

DETERMINATION OF TARGET SAFETY INDEX

The development of new truck weight regulations requires first the determination of a target safety index. Since the performance of bridges designed by the current standards are generally satisfactory, the target safety index will be determined based on current design procedures.

The resistance R of a member designed using the current code has a nominal value R_n that can be calculated from two different design formulae:

1) AASHTO's WSD:

$$R_n = \frac{1}{.55} (D_n + L_n + I_n) = 1.82 (D_n + L_n + I_n) \tag{22}$$

2) AASHTO's LFD:

$$R_n = 1.3 D_n + 2.17 (L_n + I_n) \tag{23}$$

In both cases, the nominal load, L_n, is the static moment effect for one member using the AASHTO design vehicle and girder distribution factor and I_n is the dynamic effect obtained from L_n using AASHTO's impact formula.[5] D_n is the nominal dead load effect.

The object of this study is to develop a formula to regulate the truck weights over the existing bridge network. This network is composed of more than 600,000 bridges of different materials, span lengths, geometries etc. The ensemble of these bridges should maintain an acceptable level of safety. In order to adequately manage the large number of parameters associated with the safety evaluation of the complete network, a truck weight formula will herein be developed to provide acceptable levels of safety for steel bridges. These steel bridges are assumed to be simple sapns, designed according to AASHTO's Working Stress Design method (WSD) with HS-20 loading. The effect of the adoption of the proposed truck weight formula on the complete network composed of bridges of different configurations, material types and different rating levels is investigated in chapter 3 in terms of the safety impact and economic costs.

AASHTO's WSD method calculates the required safe minimum member capacity of a bridge utilizing a live load envelope representing AASHTO's truck and lane loads. An approximate analysis procedure selects a load distribution factor based on the type of bridge, and a dynamic factor is determined based on the span length. These design or nominal values for loads and load effects determine the required nominal resistance R_n as shown in equation 22 which can also be represented by:

$$R_n = \frac{1}{0.55} (D_n + L_n + I_n) \tag{24}$$

where D_n is the nominal dead load of the member, L_n is the static live load moment effect on one member as specified by AASHTO's HS 20 vehicle and lane load, I_n is AASHTO's dynamic load. The total load applied on the bridge is divided to each member using AASHTO's load distribution factor g_n. I_n is calculated as a fraction of L_n using AASHTO's impact factor. $L_n + I_n$ can also be presented as $L_n i_n$ where the impact factor i_n varies between 1.0 to 1.3. The limiting allowable stress ratio for steel beams is 0.55; this corresponds to a safety factor equal to 1.82. Based on several experimental studies, it was found that the mean resistance $\bar{R}$ of a bridge member is higher than R_n. $\bar{R}$ can be obtained from the nominal resistance using a bias b_r of 1.1. The coefficient of variation associated with the resistance is 12 percent.

D_n is usually obtained from the self-weight of the structure including the weight of the wearing surface and other non-structural members. For these calculations, the dead load value D_n is obtained from equation 15 and the dead load bias is estimated at 1.0. The dead load coefficient of variation used in these calculations is 9 percent.

AASHTO uses an HS-20 design vehicle whose effect on bridges should bracket the effect of the actual truck loads. Several studies, however, showed that the HS-20 vehicle does not provide a consistent envelope for all spans under current truck traffic conditions. For this reason, the nominal live load effect L_n is calculated using AASHTO's HS-20 truck but the mean live load effect including impact and its coefficient of variation will be calculated using equation 21. The load model estimates the actual live load moment on a member in the lifetime of a bridge structure due to the passage of truck loads and is given by:

$$L = a\, m\, W_{.95}\, H\, g\, i\, Gr \tag{21}$$

The factor a calculates the maximum moment effect of a typical standard truck assuming a one unit gross weight. The factor, a, is deterministic and its value for different span lengths is given in table 1. For spans of 60 ft (18.29 m) and less, a is obtained from a typical single unit truck. For spans longer than 60 ft (18.29 m) a typical semi-trailer truck is used as the basis for calculating the factor a.[46]

The factor m is a random factor that accounts for the variability in the axle configuration of random trucks compared to the standard trucks used for the calculation of a. Table 1 shows the different mean values of m and their coefficients of variation.

$W_{.95}$ is the value corresponding to the upper 5 percent of the gross weight histogram. From a study for FHWA, gross weight histograms were collected at several sites throughout the U.S.[40] The average $W_{.95}$ obtained for semi-trailer trucks was 75 kips (333.6 kN) and the C.O.V. is 10 percent. For single unit trucks, the average is 47 kips (209 kN) and the C.O.V. is 15 percent.

H is a factor that relates the moment of one truck with the characteristic gross weight $W_{.95}$ to the estimated maximum lifetime load on two-lane bridges. H was obtained from the results of a simulation program with different truck histograms and is dependent on the span length as shown in table 1.

The dynamic impact factor is i. In this analysis the mean dynamic to static load effect ratio is assumed to be 1.2 with a coefficient of variation (C.O.V.) of 8 percent.

The factor g is the lateral distribution factor. For slabs on steel-beam bridges, field measurements yielded a bias between the measured and the AASHTO load distribution factors equal to 0.9 and is associated with a C.O.V. of 8 percent.

Gr is a lifetime growth factor. For a 50-year lifetime, one needs to account for possible growth in the weights and volume of heavy trucks traveling over the bridge network. In this study, a growth factor Gr with a mean of 1.15 and a C.O.V. of 10 percent is assumed.

Equation 21 accounts for the current truck load effect by considering the statistics of current truck gross weights, multiple occurrence on the bridge and truck configuration. Also indirectly considered in the H factor is the truck traffic volume. In these calculations, an average truck volume of 2500 trucks/day is assumed.

Using this data base, the safety index calculations are executed for simply supported steel beam bridges of different span lengths designed using WSD criteria for an HS-20 loading. The safety index inherent under current situation is given in table 2 for different span lengths. The safety index thus obtained varies from about 2.5 to 4.2. The results indicate that the shorter spans exhibit a higher risk of failure than the longer spans. This is due to the basic procedure in WSD which applies one safety (or load) factor for all loads regardless of whether they can be accurately estimated as with dead loads or can be estimated less accurately as with live loads. Shorter spans, having relatively lower dead loads, will then have lower safety indices.

Table 2. Safety index versus span length for current designs.

Span (ft)	Safety Index
30	2.47
40	3.00
60	3.66
80	3.76
100	3.69
125	3.67
150	3.77
175	4.01
200	4.21

1 ft= 0.3048 m

Table 2 shows that the lowest safety index value accounting for the growth in traffic is about 2.5. This value corresponds to the shortest simple span considered, i.e. 30 ft (9.14 m). The largest safety index is 4.21 corresponding to the 200-ft (61 m) span. The average safety index value from table 2 is 3.58. These calculations include a growth factor which would account for possible changes in the truck weights and truck volume. Since the target safety index used to develop a new code is often calibrated based on the performance of existing structures, changes in the safety index inherent in current criteria and design procedures is justified only when there is a consensus among knowledgeable engineers that current criteria do not provide adequate safety. Based on calculations similar to the ones presented herein, OHBDC used a safety index target of 3.5 corresponding to the average safety index value for short to medium span bridges.[12] Similarly, a safety index value of 3.5 is proposed in reference [9]. A target value of 3.5 was used for these new design models; but it is generally accepted that a lower target safety index value can be used for the evaluation of existing bridges. For example, Moses used a safety index target of 2.3 for the calibration of safety factors for the load capacity evaluation of existing bridges when no growth factor is considered. His calculations are also based on operating stresses (i.e. using a safety factor of 1/0.75 instead of the 1/0.55 of equation 24). Operating stresses are regularly used by a number of States for rating existing bridges.[10]

In this study, it is decided to use a target safety index value of 2.5 for the calculation of the required truck weight formula. It will be noted in chapter 3 below that this value of 2.5 is approximately equivalent to a value of 3.0 if no growth rate factor is considered. The 2.5 target index used herein would then be more conservative than Moses' approach but less conservative than the value used for new designs. This is justified based on the fact that Moses' approach assumes a detailed inspection of bridges every 2 years as mandated by Federal regulations. Also, it is usually much more practical to be conservative with new designs than for the evaluation of existing bridges.

Another justification for using a target of 2.5 is the fact that the bridge engineering community is generally satisfied with the current performance of simple span highway bridges including short-span bridges.[2] Since bridges with 30-ft spans (9.14 m) designed by current specifications provide adequate safety, and since this safety level is represented by a safety index β equal to 2.5, the use of the 2.5 value to calibrate the new bridge formula is justified. The target safety index measures the level of safety against failure (yielding) of the most critical bridge member. Due to structural redundancy existing in most bridges, failure of the most critical member will not necessarily lead to the collapse of the complete structure. Also, the 30-ft span (9.14 m) controls the loading on short wheeled vehicles. With these vehicles using the Nation's bridges without any observed or documented risk to bridge safety, one cannot then justify increasing the required safety level which would mandate a reduction in the short wheeled vehicles' legal weights.

CALCULATION OF SAFE LOAD ENVELOPE

The steps involved in applying reliability procedures to obtain a bridge formula may be summarized as follows:

(1) Choose suitable safety measures. The safety index widely used in structural reliability theory as a measure of structural safety is used in this study as the basis for the determination of the safety of bridge members.

(2) Select an acceptable reliability level. A safety index of 2.5 for redundant bridges seems to provide a reasonable safety target based on the performance of existing bridge members. This safety index target of 2.5 corresponds to a probability of about 0.6 percent that the safety criteria will be satisfied in any member of the bridge.

(3) Choose a range of typical bridges with different design loads, span lengths, and configurations giving a representative sample of the Nation's bridges. These bridges should include both steel and concrete bridges having simple as well as continuous spans. In this study, simple-span steel bridges are used to obtain a truck weight formula. The implications in terms of safety and cost of other types of bridges are studied separately in chapter 3 as part of the cost and safety analysis.

(4) Use statistics on the safety margins of these typical bridges including the likelihood of overloads and simultaneous truck occurrences to obtain the live load envelope that will produce the target safety index. It will be assumed that the uncertainties (C.O.V.) of the live load random variables will remain the same as currently observed. The live load envelope as defined herein is the maximum mean total bridge live load effect that will achieve the target safety index for each span length.

(5) Calibrate a truck weight formula that will produce the load envelope obtained in step (4).

(6) Verify that the proposed formula will lead to an acceptably small number of bridges requiring upgrading to support the projected additional load.

(7) Review the implications of adopting the proposed formula in terms of safety of typical steel and concrete bridges of simple and continuous spans. This should include both strength and fatigue requirements.

(8) Study the costs required to maintain the bridge infrastructure if the proposed formula is implemented.

Changes in the legal truck weight regulations will produce changes in the characteristics of the regular truck traffic over highway bridges. Specifically the weights and the axle configurations of the trucks will be affected. According to our basic truck load model (equation 21), it is $W_{.95}$ that will be affected the most. The effect of the changes in the truck weight regulations on H will be secondary. If trucks are allowed to carry more loads, the total number of truck trips required to carry the total loads in a jurisdiction might decrease if there is no shift in the modes of transportation. The number of truck trips on the highway system will affect the number of simultaneous occurrences on a given bridge site, this will be reflected in a change in the value of the variable H. H is sensitive to only very large changes in truck volumes, and can be assumed to remain constant for the purposes of this study. Also, H represents the shape of the weight histogram above the $W_{.95}$ value, thus we are also assuming that the percentage of overloads will remain the same after the implementation of the new weight regulations.

In this study, we are assuming that changes in the legal weight limits will produce a shift in $W_{.95}$ such that the ratio of $W_{.95}$ to the legal limit remains constant. At the same time, m is assumed to be equal to 1.0 relative to standard vehicles that will exactly satisfy the new truck weight formula after the implementation of the new regulation. These are purely assumptions since no data is available to consider future changes in these parameters.

The next step in this analysis consists of determining the live load envelope required to produce an acceptable safety level. This is done as follows: Assuming that current bridges are designed according to AASHTO's WSD method with HS 20 loading (equation 24), a program is developed to determine the mean live load L required such that the target safety index of 2.5 is matched exactly for each span length considered. Table 3 gives these values for steel bridges designed according to equation 24. The values shown give the maximum total static moment effect M_t permitted on two lane simple span steel bridges such that their safety index does not fall below 2.5.

Table 3. Maximum live load moment envelope for HS 20 steel bridges.

Span (ft)	Maximum Moment (kip-ft)
30	717
40	1231
60	2438
80	3902
100	5434
125	7500
150	9986
175	13820
200	18380

1 ft = 0.3048 m
1 kip-ft = 1.356 kN-m

From the load envelope developed above, a truck weight formula is obtained using the procedure outlined in reference 1. The truck weight formula is designed to give a relationship between the weight of a truck and its length. The steps involved are summarized as follows:

(1) A truck satisfying the bridge formula is assumed to have a total weight W and total truck length B.

(2) Assume that the truck weight W is uniformly distributed over the truck length B.

(3) Several values of B are used such that B varies between 1 ft (0.305m) and 120 ft (36.58 m).

(4) Given a truck length B, find the moment effect M_r of a unit load uniformly distributed over B. This is done for each span length.

(5) For every span length, find the load envelope M_t required to obtain the target safety index 2.5 (see table 3).

(6) For each span length, find $W_{.95\ t}$ (target 95 percentile weight) such that $mM_r\ W_{.95\ t}\ H$ is equal to M_t. a is not included herein since it is implicitly considered in the unit load influence term, M_r.

(7) Repeat the previous steps for every truck length B. For each truck length, several $W_{.95\ t}$ values are obtained corresponding to the load effect of every span considered.

(8) For each truck length B, find the lowest $W_{.95\ t}$ which is labeled $W_{.95\ t\ min}$.

(9) A characteristic weight factor gives the ratio of the legal load W to $W_{.95\ t\ min}$. A factor of 1.07 is used for spans greater than 60 ft (18.29 m) based on current truck weight statistics for semi-trailer trucks. The ratio for shorter spans governed by single unit trucks is 1.09.

(10) Plot W versus B. This curve provides an envelope that the distributed load W should satisfy in order to ensure that all span lengths will produce a safety index of at least 2.5.

(11) Find an algebraic expression that will fit the W versus B curve as closely as possible. This will be the truck weight formula to be utilized.

In this approach we assume that distributing truck weights uniformly over the truck length provides a safe envelope for all typical truck configurations and axle spacings.[1] Figure 1 shows the W versus B curve obtained using the calculations performed in this analysis. The curve is compared to the curve given by equation 3 and the one proposed by the TRB Truck Weight Study (equation 4).[1,2] Figure 1 compares the proposed truck weight formula to the TTI formula for an overstress criteria equal to 1.05 x HS20 moments and 1.33 x H15. Also shown is the plot of the formula developed by TRB for the "Truck Weight Study" project. It is interesting to note that the maximum difference between the TRB formula and the formula developed here using the safety index criteria differ at most by 12 percent. The TRB formula was obtained by adopting the formula given in equation 3 for trucks under 80 kips (355.84 kN) and matching the current Bridge Formula for trucks over 80 kips (355.84 kN).[2]

The safety index formulation yields a truck weight formula almost identical to the HS20 formula developed in reference 1 for lengths less than 35 ft (10.67 m). The TRB formula for trucks over 40 ft (12.19 m) long was obtained based on field observations

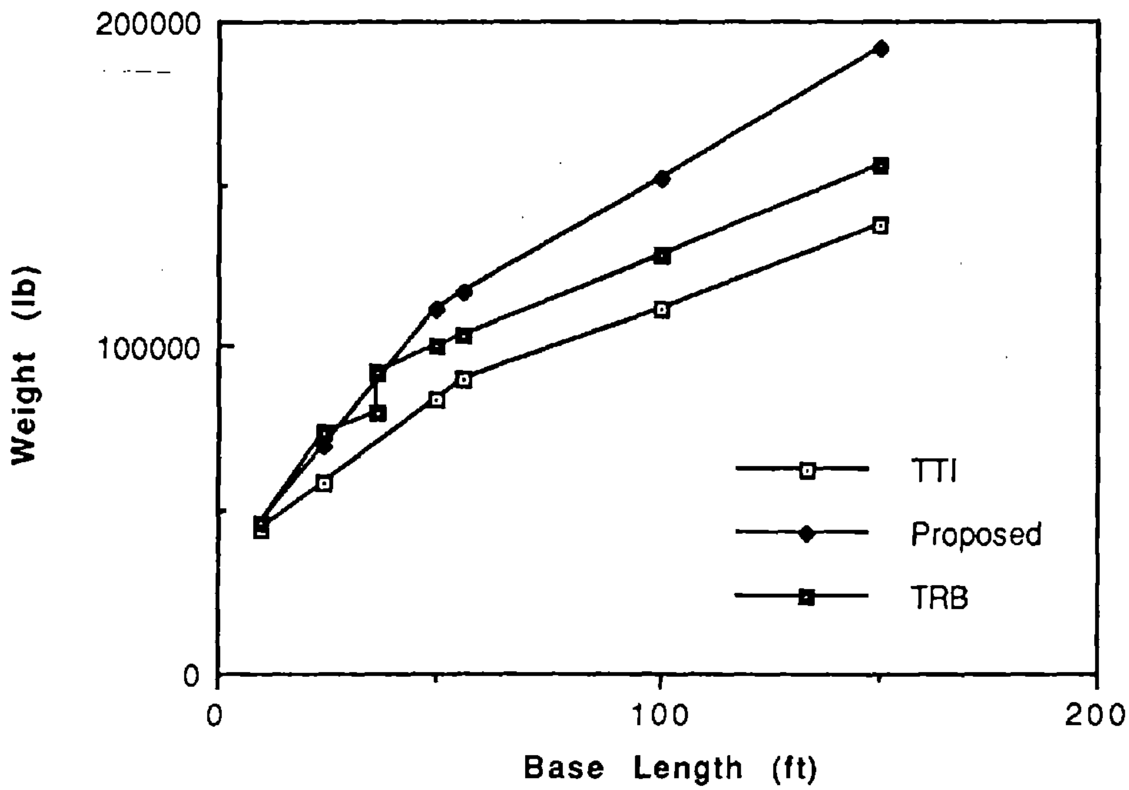


Figure 1. Comparison of truck weight formulas.

from bridge engineers from several States associated with the TRB study indicating that existing bridges currently subjected to loads corresponding to the current bridge formula do not exhibit any increased damage or any additional risk. Interestingly enough these observations are corroborated by the reliability analysis undertaken herein.

The proposed formula, as discussed earlier, was developed such that a simply-supported steel bridge designed to satisfy AASHTO's WSD criteria for HS-20 loading will have a safety index beta of 2.5 when subjected to the loads expected over the next 50 years if the new truck weight formula is implemented. The projections of the loads is based on the assumption that a lateral shift in the gross weight histograms accompanies any shift in the legal limit. Also, to account for future increases in truck weights and in truck traffic and the more frequent number of heavy multiple truck occurrences caused by that, a traffic growth random variable with a mean of 1.15 and a COV of 10 percent is included in the maximum load model. The proposed formula obtained by fitting the W versus B results is as follows:

$$\begin{aligned} W &= (1.64\ B + 30)\ 1000 \qquad \text{for } B < 50 \text{ ft} \\ W &= (0.80\ B + 72)\ 1000 \qquad \text{for } B > 50 \text{ ft} \end{aligned} \tag{25}$$

Typical vehicles satisfying the proposed truck weight formula are given in figure 2. The axle spacings of these vehicles is based on the axle spacings of the typical vehicles proposed in the TRB "Truck Weight Study."[2]

CONCLUSION

A new truck weight formula that regulates the weight of heavy trucks and axle groups is developed based on rational safety criteria. The procedure used to obtain the proposed formula utilizes a reliability analysis such that the projected truck load effect will produce a uniform safety index for existing bridges designed according to current AASHTO criteria. The proposed formula is based on the reliability analysis of simple span steel bridges satisfying AASHTO's WSD specifications. The effect on the national bridge network resulting from the implementation of the proposed formula will be studied in the following chapters.

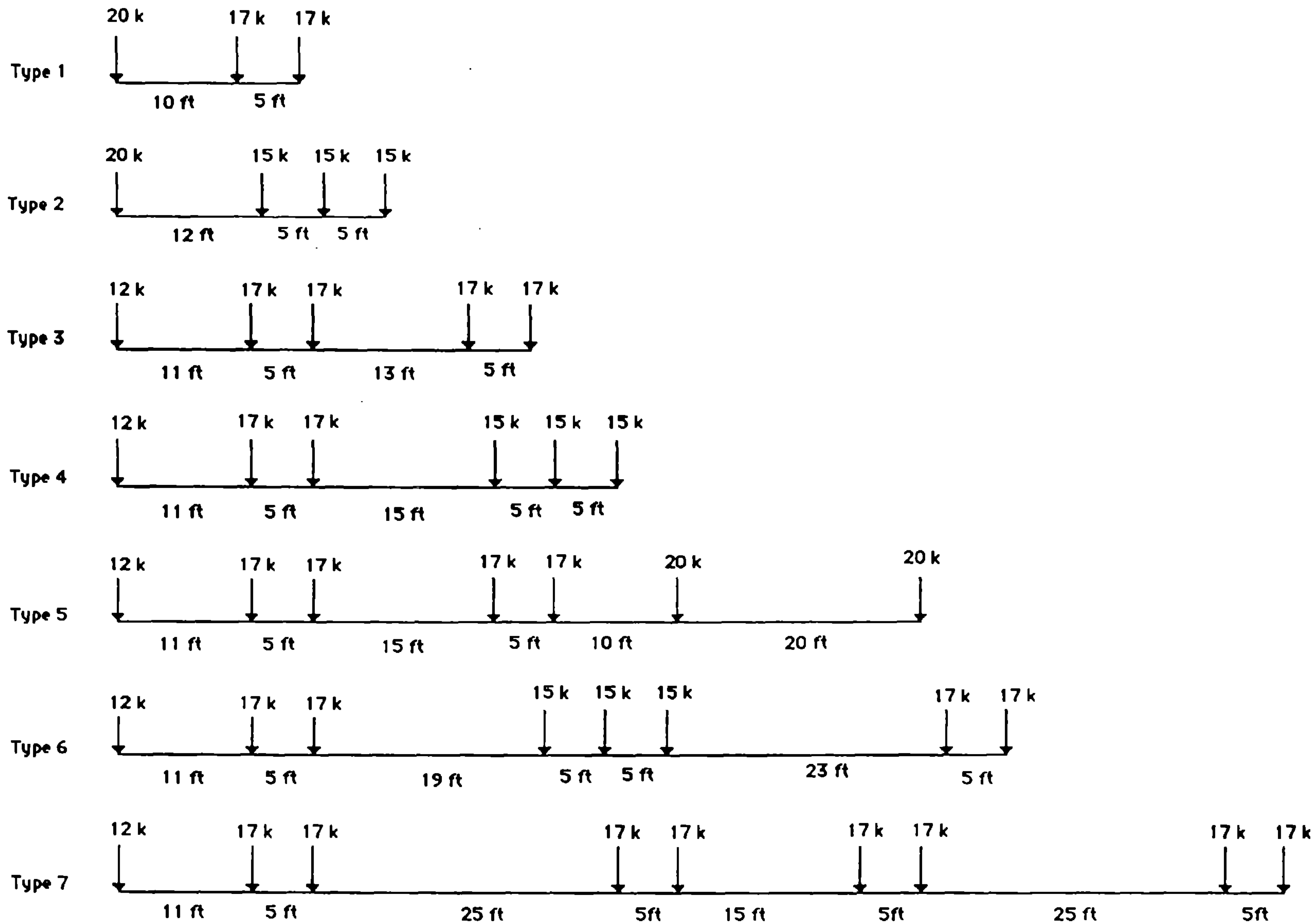


Figure 2. Typical truck configurations under proposed formula.

CHAPTER THREE

SENSITIVITY AND COST ANALYSIS

Chapter two presented a new truck weight formula developed using structural reliability techniques. In this chapter, a sensitivity analysis is performed to study the effect of the assumptions made on the new truck weight formula. Also, the consequences of the implementation of the formula on the existing bridge network composed of simple and continuous steel and concrete bridges is investigated.

SENSITIVITY TO GROWTH FACTOR

As explained in chapter two, the truck weight (bridge) formula proposed in equation 25, was based on a safety index target of 2.5. This target value was obtained from current design criteria such that it matches the safety index inherent under current traffic conditions for short-span steel bridges designed according to WSD procedures with HS-20 loading. The reliability formulation accounts for the possibility of multiple occurrences of random trucks on two-lane simply supported steel bridges. An allowance for possible growth in the truck traffic volume or in the number of overloads due to changes in the legal limits or the level of weight enforcement over the next 50 years was considered in the reliability calculations through a growth factor Gr. This factor Gr is included in the live load formulation to account for:

(a) Possibility of increases in the truck volume in the future which will increase the number of simultaneous truck loadings over the spans,

(b) changes in legal limits within a jurisdiction or the increases in permit or illegal overloads in the future, and

(c) uncertainties associated with the proposed modeling techniques which may become more significant with future traffic and truck configurations.

In the original calculations, a Gr value of 1.15 and a corresponding coefficient of variation equal to 10 percent were used. To study the effect of this factor on the proposed truck weight formula, a sensitivity analysis is performed in this section whereby a new target beta is extracted without considering the growth factor Gr.

A comparison between the safety indices for WSD HS-20 steel bridges under current traffic conditions obtained assuming a Gr factor and without a Gr factor are shown in table 4 below.

Table 4. Effect of Gr factor on safety index.

Span (ft)	Safety index	
	(a) with Gr	(b) without Gr
30	2.47	3.06
40	3.00	3.65
60	3.66	4.36
80	3.76	4.48
100	3.69	4.37
125	3.67	4.30
150	3.77	4.34
175	4.01	4.51
200	4.21	4.65

1 ft = 0.305 m

Column (a) in table 4 shows that the lowest safety index value accounting for growth in traffic is about 2.5. This value corresponds to a 30-ft span (9.14 m), the shortest considered. As already explained in chapter two, current practice in structural reliability is for the target safety index utilized in developing a new code to be calibrated based on the performance of structures built to satisfy the current code. Changes to the safety index inherent in current design procedures are justified when there is a consensus among expert engineers that current procedures do not provide adequate safety. Under WSD, a 30 ft (9.14 m) simple span has a very high live to dead load stress ratio, but is still considered to provide adequate safety under current loads. Since this safety level is represented by a safety index beta equal to 2.5 when accounting for growth in traffic, then there is no justification for utilizing a higher safety index when calibrating the new load limits. Keep in mind that the target safety index as defined herein, measures the level of safety against yielding of the most critical bridge member. If yielding of that member does occur, then the other members may provide additional safety against bridge failure by providing additional reserve strength.

Table 4 shows the safety indices for two cases: (a) accounting for growth in truck loading through the Gr factor, and (b) no truck loading growth is considered. Using the same logic developed above, if we do not consider the Gr factor in the calculations, the target safety index should be on the order of 3.0. Table 5 shows the maximum permitted moment effect required to maintain the target safety indices for the two cases considered: (a) the required load envelope for a target safety index of 2.5 accounting for the growth factor; and (b) the required load envelope for a target safety index of 3.0 with no growth factor. The results for the two cases considered are very similar as would have been expected based on several previous studies on the effect of errors in parameter estimation during reliability-based code calibration.[46]

Table 5. Limiting moment envelopes for HS-20 steel bridges.

Span (ft)	Moment envelope (kip-ft) (a) with Gr	(b) No Gr
30	717	739
40	1231	1273
60	2438	2536
80	3902	4095
100	5434	5687
125	7500	7798
150	9986	10316
175	13820	14189
200	18380	18756

1 ft = 0.305 m
1 kip-ft = 1.36 Kn-m

Table 6 shows a comparison between the maximum allowable (legal) truck weights in function of the truck lengths for the two cases considered. The maximum difference observed for the 60-ft (18.29 m) truck is on the order of 4.5 percent. Most of this difference is due to round off errors. For example, target indices of 2.5 and 3.0 were used rather than the exact values of 2.47 and 3.06. The similarity between the two columns shown in table 6 confirms that the reliability approach proposed in this study is capable of smoothing out errors in modeling future truck traffic growth.

Table 6. Effect of Gr factor on proposed truck weight formula.

Truck length (ft)	Legal truck weight (kip)	
	(a) with Gr	(b) No Gr
10	48	49
20	60	61
30	79	82
40	97	100
50	109	115
60	118	123
70	125	130
80	132	137
90	140	146
100	149	154
110	157	162
120	165	171

1 ft = 0.305 m
1 kip = 4.48 kN

SENSITIVITY TO BRIDGE TYPE

A comparison between the proposed truck weight formula (equation 25) that was based on simple span steel bridges satisfying the WSD criteria and formulas that could be developed using reinforced concrete (R/C) T-beam or prestressed concrete (Ps/C) bridges with either WSD or LFD criteria is undertaken in this section. The objective is to check whether equation 25 provides a safe envelope for all types of bridges. Table 7 compares the maximum live load moment envelopes for HS-20 WSD steel bridges to those for HS-20 R/C T-beam bridges and HS-20 prestressed concrete bridges designed by both LFD and WSD criteria. The calculations are executed using a target safety index of 2.5 considering the growth factor Gr.

Table 7. Maximum load envelopes for different bridge types.

Span (ft)	Maximum moment envelope (kip-ft)					
	Steel LFD	Steel WSD	R/C LFD	R/C WSD	Ps/C LFD	Ps/C WSD
30	767	718	800	1047	738	695
40	1275	1232	1330	1789	1237	1205
60	2378	2438	2531	3717	2352	2432
80	3600	3903	3988	6589	3643	3973
100	4762	5435	5548	10433	4912	5619
125	6184	7501	-	-	6528	7880
150	7774	9986	-	-	8399	10641
175	10187	13820	-	-	11263	14912
200	12869	18384	-	-	14546	20047

1 ft = 0.305 m
1 kip-ft = 1.36 Kn-m

Table 8. Legal truck weights for different truck lengths and bridge types.

Span (ft)	Truck weights (kips)					
	Steel LFD	Steel WSD	R/C LFD	R/C WSD	Ps/C LFD	Ps/C WSD
10	51	48	53	69	49	46
20	64	60	66	87	61	58
30	83	79	86	116	80	77
40	88	97	100	145	93	96
50	93	109	112	168	98	110
60	97	118	121	196	103	122
70	101	125	130	225	109	131
80	105	132	141	254	114	139
90	110	140	154	285	119	147
100	116	149	170	317	125	157
110	122	157	187	349	132	167
120	127	165	204	380	139	176

1 ft = 0.305 m
1 kip = 4.448 kN

Table 8 gives the legal truck weights for different truck lengths for the different bridge types considered. The results given in tables 7 and 8 illustrate the following points:

• The maximum moment envelope obtained using steel bridges provide a safe envelope when compared to concrete T-beam bridges designed by either WSD or LFD criteria. Notice that concrete bridges are assumed to have a maximum span length of 100 ft (30.48 m).

• The maximum moment envelope obtained using steel bridges falls on the unsafe side of the prestressed concrete bridge envelope for WSD designed bridges less than 60 ft (18.29 m) in length. The difference is however small on the order of only 3 percent which can be ignored.

• WSD criteria provide an unsafe maximum moment envelope for LFD bridges with spans greater than 40 ft (12.19 m). This is expected since LFD criteria were developed in order to reduce the margin of safety of bridges above 40 ft (12.19 m). This observation is also reflected in table 8 where LFD criteria would produce legal truck weight limits for trucks over 40 ft (12.19 m) in length lower than the legal weights obtained if WSD criteria were used. Most existing bridges however were built to WSD criteria and therefore this observation should not affect the safety of most bridges in the existing network.

SENSITIVITY TO RATING CAPACITY

Equation 25 was based on simple span steel bridges satisfying AASHTO's WSD design with HS-20 loading. The existing network consists not only of bridges designed by this criteria, but also bridges designed to different loading criteria or that have deteriorated to lower live load capacities. The live load capacities of existing bridges are often expressed in terms of HS ratings. For example, an HS-15 bridge is a bridge capable of supporting its dead weight plus 75 percent (15/20) of the HS-20 live load. Table 9 gives the safety index values for HS-15, HS-20 and HS-25 WSD steel bridges under

Table 9. Safety indices for steel bridges with different HS-capacities.

Span (ft)	Safety index		
	HS-15	HS-20	HS-25
30	1.70	2.47	3.10
40	2.24	3.00	3.63
60	2.93	3.66	4.28
80	3.04	3.76	4.38
100	3.03	3.69	4.27
125	3.07	3.67	4.21
150	3.23	3.77	4.27
175	3.51	4.01	4.48
200	3.75	4.21	4.65

1 ft = 0.305 m

Table 10. Maximum moment envelopes for steel bridges with different HS-capacities. (target safety index = 2.5)

Span (ft)	Maximum moment envelope (Kip-ft)		
	HS-15	HS-20	HS-25
30	564	717	872
40	979	1231	1485
60	1972	2438	2900
80	3200	3902	4598
100	4512	5434	6348
125	6311	7500	8677
150	8495	9986	11455
175	11878	13820	15734
200	15940	18380	20789

1 ft = 0.305 m
1 kip-ft = 1.36 Kn-m

current truck traffic conditions. Table 10 gives the maximum moment envelopes required to provide a safety index of 2.5 for WSD bridges with HS-15, HS-20 and HS-25 load capacities. Table 11 gives the maximum legal truck weights permitted to maintain the target safety index of 2.5 for the three cases considered.

Table 11. Legal truck weights for different truck lengths and HS-capacities.

Length (ft)	Truck weight (kips)		
	HS-15	HS-20	HS-25
10	37	48	58
20	47	60	72
30	62	79	96
40	78	97	115
50	89	109	129
60	99	118	137
70	105	125	144
80	111	132	153
90	118	140	162
100	126	149	171
110	133	157	180
120	141	165	190

1 ft = 0.305 m
1 kip= 4.448 kN

The results show that if in the interest of conservatism, the truck weight formula is calibrated so that HS-15 bridges provide the target safety index of 2.5 rather than the HS-20 bridges, the weight versus truck length equation should be lowered on the order of 23 to 14 percent with the higher percentage corresponding to the shorter vehicles. Since the truck weight formula obtained using HS-20 bridges gives the same legal limits as the current formula for short wheeled vehicles, then using the HS-15 criteria will produce about a 23% lower legal limits for short wheeled vehicles under current regulations. This conservatism, however, seems unjustified since expert engineers do not observe any excessive damage to existing bridges under currently legal short wheeled vehicles.

Similar calculations were executed for several HS criteria ranging from HS-30 to HS-10. Table 12 below gives the different truck weight formulas developed for all the cases considered. The results show that an increase of 5 units on the HS scale will produce a change in the legal limits varying between 35% to 13% with the higher percentage corresponding to the short wheeled trucks. This indicates that the shorter vehicles are more sensitive to the HS criteria used than the longer vehicles especially when lower HS ratings are used.

Table 12. Truck weight formulas for different HS criteria.

Live load capacity	Truck Weight Equation	
HS-10	$W = 1.04\ B + 15$	$B<60$
	$W = 0.64\ B + 39$	$B>60$
HS-15	$W = 1.35\ B + 22$	$B<50$
	$W = 0.73\ B + 53$	$B>50$
HS-20	$W = 1.64\ B + 30$	$B<50$
	$W = 0.80\ B + 72$	$B>50$
HS-25	$W = 1.87\ B + 38$	$B<46$
	$W = 0.87\ B + 84$	$B>46$
HS-30	$W = 2.21\ B + 43$	$B<44$
	$W = 0.96\ B + 98$	$B>44$

W = is legal weight in kips
B = Truck base length in ft

SENSITIVITY TO RATING PROCEDURES

Equation 25 was developed based on steel bridges that satisfy equation 22. The safety factor 1/.55 (corresponding to inventory rating stresses) is widely used when designing new bridges; for the rating of existing bridges however, most States allow the use of a less conservative safety factor associated with what is known as operating rating stresses.[10] Table 13 compares the safety indices obtained for bridges with an HS-20 inventory rating to the safety indices obtained for bridges with an HS-20 operating rating (i.e. equation 22 with a safety factor of 1/.75 rather than 1/.55). Both types of bridges are assumed to carry the same truck traffic. In both cases, the projection of maximum load is done for a 50-year period rather than the 2-year period used by Moses.[10] In addition, the calculations executed in this section assume a traffic growth factor Gr which was not considered by Moses. The average safety index value obtained using the operating rating stresses is 1.85. This is compared to an average of 3.58 using inventory rating stresses. The 1.85 average value indicates that existing bridges could be rated using safety index criteria such that a safety index of 1.85 provides an acceptable safety level (accounting for growth rate) if the engineering community is satisfied with current operating stress criteria. Operating ratings are normally used on highways with limited heavy truck traffic and bridges with excellent maintenance. Adopting operating rating stresses as criteria implies a reduction in the acceptable safety index on the order of 48 percent compared to inventory rating.

Table 14 compares the loading envelopes required to achieve the target safety index of 2.5 for bridges with inventory and operating stress ratings . Table 15 gives the legal truck weights required for different truck lengths. The results of table 15 show that if bridges that satisfy the HS-20 rating using operating stress criteria should have a safety index value of 2.5 as a minimum, the legal truck weights should be reduced by as much as 48 percent when compared to the proposed legal limits of equation 25.

Table 13. Safety indices for HS-20 steel bridges using different rating criteria

span (ft)	Safety Index	
	Inventory	Operating
30	2.47	1.25
40	3.00	1.67
60	3.66	2.13
80	3.76	2.02
100	3.69	1.86
125	3.67	1.78
150	3.77	1.81
175	4.01	1.99
200	4.21	2.15

1 ft = 0.305 m

Table 14. Maximum live load moment envelopes for different rating criteria using a target safety index of 2.5

span (ft)	Moment envelope (kip-ft)	
	Inventory	operating
30	717	488
40	1231	818
60	2438	1546
80	3902	2353
100	5434	3130
125	7500	4091
150	9986	5169
175	13820	6811
200	18380	8634

1 ft = 0.305 m
1 kip-ft = 1.36 Kn-m

Table 15. Legal truck weights for different axle base lengths and different rating criteria. (target safety index = 2.5)

Length (ft)	Weight (kips)	
	Inventory	Operating
10	48	32
20	60	40
30	79	54
40	97	58
50	109	61
60	118	64
70	125	67
80	132	70
90	140	73
100	149	77
110	157	81
120	165	85

1 ft = 0.305 m
1 kip= 4.448 kN

SENSITIVITY TO TARGET SAFETY INDEX

The target safety index chosen for the development of the truck weight formula was 2.5. This section looks at the sensitivity of the results to changes in the target index. Table 16 gives a comparison between the maximum moment envelopes if different safety index targets were chosen for the development of the truck weight formula. These calculations are based on simple-span steel bridges with WSD HS-20 inventory rating criteria but using different target safety indices ranging from 2.0 to 3.5. The results show that a change in the target index on the order of 0.5 will change the required load envelope on the order of 15 to 17 percent.

Table 16. Maximum moment envelopes for different target safety indices.

Span (ft)	Moment (kip-ft)			
Target Index:	2.0	2.5	3.0	3.5
30	839	718	614	525
40	1433	1232	1058	909
60	2821	2438	2104	1815
80	4485	3903	3390	2940
100	6254	5435	4712	4075
125	8662	7501	6474	5565
150	11571	9986	8581	7332
175	16066	13820	11823	10041
200	21451	18384	15654	13211

1 ft = 0.305 m
1 kip-ft = 1.36 Kn-m

Table 17. Legal truck weights for different axle base lengths for different target safety indices

Length (ft)	Weight (kip)			
Target Index:	2.0	2.5	3.0	3.5
10	56	48	41	35
20	70	60	51	44
30	93	79	68	58
40	112	97	83	72
50	126	109	95	82
60	136	118	102	88
70	144	125	108	93
80	153	132	114	98
90	162	140	121	104
100	173	149	128	109
110	182	157	135	115
120	192	165	142	121

1 ft = 0.305 m
1 kip= 4.448 kN

Table 17 gives the weight versus axle base length tables for the different safety index criteria. From the results it is observed that a 0.5 change in the safety index target will produce a change in the truck weight on the order of 15 to 17 percent. This change is relatively uniform for all the truck base lengths and is also similar to the change in the maximum moment envelopes given in table 16.

SENSITIVITY TO ERRORS IN THE DATA BASE

As a check on the effect of possible errors in the data base, small variations in the statistical data were assumed. The data used in reference [10] for the calibration of a bridge evaluation code is used in this section for the development of the maximum live load moment envelope. The differences between the original data used in the previous sections and the altered data used in this section are as follows: The altered data uses a bias on AASHTO's girder distribution factor equal to 0.90 with a COV of 13 percent. Also, in the altered data, the impact factor is assumed to have a mean of 1.20 with a COV of 10 percent corresponding to a bridge deck surface of medium roughness. The altered data used in this section is due to a larger number of field tests compared to the number of tests that produced the original data. Also, the altered data uses a maximum projection period for bridge lifetime of 2 years compared to the 50-year period assumed in the original data. The lower projection period produces a lower mean value but a higher C.O.V.

The results obtained using the altered data are shown in tables 18 and 19 and compared to the results obtained using the original data. Column (a) of table 18 shows the safety index values obtained under current traffic conditions using the altered data. The average safety index drops from 3.58 (with the original data) to 3.16 with a minimum value of 2.13 and a maximum of 3.83. If the logic used in the previous chapter is followed here to determine the target safety index using the altered data, a value of 2.13 (rounded up to 2.15) would be chosen as the target safety index. This value corresponds to the lowest acceptable safety level associated with the altered data.

Column (b) of table 18 gives the maximum moment envelope obtained if a safety index target of 2.5 is used with the altered data. Column (c) gives the maximum moment envelope if a target beta of 2.15 is used with the altered data. Column d shows the maximum moment envelope obtained with the original data and a target safety index of 2.5.

Table 18. Maximum moment envelopes obtained using altered and original data.

Span (ft)	Beta with altered data		Moment envelope (kip-ft)		
	(a)		(b) Altered data	(c) Altered data	(d) Original data
		Target index:	2.5	2.15	2.5
30	2.13		640	719	717
40	2.61		1098	1228	1231
60	3.20		2164	2418	2438
80	3.27		3453	3842	3902
100	3.22		4810	5355	5434
125	3.22		6642	7413	7500
150	3.35		8843	9898	9986
175	3.60		12259	13734	13820
200	3.83		16317	18326	18380

1 ft = 0.305 m
1 kip-ft = 1.36 Kn-m

The results of table 18 show that under current loading conditions and using the altered data, the maximum moment envelope obtained using the target safety index of 2.15 (column (c) does not change significantly from the maximum moment envelope obtained using the original data with the original target index of 2.5.

Table 19 gives the legal weight versus truck base length for the cases considered. It is noted that if a truck weight formula is to be calculated to achieve a 2.5 safety index target with the altered data, then a shift in the weight versus base length relationship on the order of 11 percent is observed for all truck lengths. If a target safety index of 2.15 rather than 2.5 is used for the development of the truck weight formula with the altered data, then, the truck weight versus base length relationship would remain practically unchanged when compared with the results obtained with the original data. This observation confirms the fact that changes in the data base will not affect the final results if the target safety index is changed accordingly.

Table 19. Variability of legal truck weights versus base length with data base.

Span (ft)	Truck weight (kip)		
	(a) Altered data	(b) Altered data	(c) Original data
Target index:	2.5	2.15	2.5
10	42	48	48
20	53	60	60
30	71	79	79
40	86	97	96
50	97	108	109
60	105	117	118
70	110	123	125
80	117	131	132
90	124	139	140
100	132	148	149
110	139	155	157
120	147	164	165

1 ft = 0.305 m
1 kip= 4.448 kN

EFFECT OF NEW FORMULA ON SAFETY OF BRIDGES

To study the implication of the adoption of the proposed truck weight formula on the safety of existing bridges, the maximum live load moment envelope given in table 3 is applied to bridges with different WSD inventory ratings ranging from HS-10 to HS-30. Table 20 gives the safety indices obtained for the range of assumed ratings. Table 21 gives the same results but using the moment effects of the typical vehicles given in figure 2 of chapter 2. Because the truck weight formula is applied to all axle groups, in some instances, weight limitations on a subgroup of axles will mean that the maximum legal weight that the full truck length could achieve may not be reached. This means that the moment effects of the typical vehicles give a more conservative moment envelope than the envelope of table 3. Also, in the derivation of the truck weight formula i.e. equation 25 it was assumed that the vehicle weights are uniformly distributed over the truck length. This assumption is conservative and when applied to actual trucks, it will produce moment envelopes more conservative than the maximum moment envelopes derived in table 3. These observations mean that using the proposed formula and the typical vehicles of figure 2 will lead to higher safety than originally intended and will produce safety indices higher than the target value of 2.5 used in the derivation of table 3. This is observed in the results of table 21 for the HS-20 case as will be further explained below.

Table 20 gives the safety indices obtained if simple-span steel bridges are loaded by vehicles producing moments equal to the maximum moments given in table 3. The calculations are executed for bridges designed according to WSD criteria with inventory ratings varying between HS-10 to HS-30. For example, for HS-20 bridges, the safety index beta of 2.5 was achieved for all span lengths when the moments of table 3 are used. Bridges with lower than HS-20 ratings will produce safety indices lower than the target 2.5. Bridges with higher than HS-20 rating produce safety indices higher than the target 2.5. The longer the span lengths the closer the safety index is to the 2.5 value since for long spans the dead loads become more dominant and the effect of the live loads becomes less important.

Table 21 gives the results for the same calculations executed using the moment effects of the typical vehicles of figure 2. The results show that if the moment effects of the typical vehicles of figure 2 are used, then the range of the calculated safety index will vary from 2.91 to 3.04 for the bridges with HS-20 inventory ratings. The difference ranges between 0.41 to 0.54. The same difference is observed for all bridge ratings. This confirms that the proposed vehicles provide a high degree of conservativeness i.e. they produce a lower moment envelope compared to the maximum moment envelope of table 3.

In a previous section while analyzing the results of table 13 it was found that if on the average, engineering experts are satisfied with the safety level implied by the operating stress ratings, then a safety index of 1.85 should provide an acceptable safety level. Based on this observation one can assume that bridges that produce a safety index beta of 1.85 or higher are safe. Using this assumption and the résults shown in table 21 then all the bridges with inventory ratings equal to HS-15 or higher will be considered safe if the vehicles proposed in figure 2 are allowed to operate freely on the existing bridge network. In addition, most bridges that have an HS rating of 12.5 will still be safe under the proposed vehicles.

Table 20. Safety indices for bridges with different inventory ratings using maximum moment envelope.

HS rating:	10	12.5	15	17.5	20	22.5	25	27.5	30
span (ft)									
30	0.74	1.28	1.73	2.14	2.50	2.83	3.13	3.41	3.66
40	0.80	1.31	1.75	2.14	2.50	2.83	3.12	3.40	3.66
60	0.93	1.38	1.79	2.16	2.50	2.81	3.10	3.37	3.63
80	1.00	1.42	1.81	2.17	2.50	2.81	3.10	3.37	3.63
100	1.13	1.51	1.86	2.19	2.50	2.79	3.06	3.32	3.57
125	1.28	1.61	1.93	2.22	2.50	2.76	3.01	3.25	3.48
150	1.40	1.69	1.98	2.25	2.50	2.74	2.97	3.20	3.41
175	1.50	1.77	2.02	2.27	2.50	2.72	2.94	3.14	3.34
200	1.59	1.83	2.06	2.29	2.50	2.71	2.91	3.10	3.28

1 ft = 0.305 m

Table 21. Safety indices for bridges with different inventory ratings using moment of typical vehicles.

HS rating: span (ft)	10	12.5	15	17.5	20	22.5	25	27.5	30
30	1.14	1.67	2.14	2.54	2.91	3.24	3.54	3.81	4.07
40	1.36	1.86	2.31	2.71	3.07	3.40	3.70	3.97	4.23
60	1.43	1.89	2.30	2.68	3.02	3.38	3.63	3.90	4.16
80	1.31	1.73	2.12	2.49	2.82	3.13	3.42	3.70	3.96
100	1.30	1.69	2.04	2.37	2.68	2.97	3.25	3.51	3.75
125	1.38	1.71	2.03	2.32	2.60	2.87	3.12	3.36	3.59
150	1.53	1.83	2.11	2.38	2.64	2.88	3.11	3.34	3.55
175	1.81	2.08	2.33	2.58	2.82	3.04	3.26	3.47	3.67
200	2.11	2.35	2.59	2.92	3.04	3.25	3.45	3.65	3.84

1 ft = 0.305 m

Table 22. Safety indices for bridges with different operating ratings using maximum moment envelope.

HS rating: span (ft)	10	12.5	15	17.5	20	22.5	25	27.5	30
30	-0.44	0.13	0.61	1.04	1.43	1.77	2.09	2.37	2.64
40	-0.49	0.05	0.52	0.94	1.32	1.67	1.98	2.27	2.54
60	-0.54	-0.05	0.39	0.78	1.15	1.48	1.79	2.08	2.35
80	-0.64	-0.19	0.22	0.61	0.96	1.29	1.60	1.89	2.16
100	-0.60	-0.19	0.19	0.54	0.87	1.18	1.47	1.74	2.01
125	-0.51	-0.15	0.18	0.50	0.79	1.08	1.34	1.60	1.84
150	-0.44	-0.12	0.18	0.46	0.73	0.99	1.24	1.48	1.70
175	-0.38	-0.09	0.18	0.44	0.69	0.93	1.16	1.37	1.59
200	-0.31	-0.05	0.19	0.43	0.66	0.88	1.09	1.29	1.49

1 ft = 0.305 m

Table 23. Safety indices for bridges with different operating ratings using moments of typical vehicles.

HS rating: span (ft)	10	12.5	15	17.5	20	22.5	25	27.5	30
30	-0.05	0.52	1.00	1.44	1.83	2.17	2.49	2.78	3.04
40	0.04	0.59	1.07	1.50	1.88	2.23	2.55	2.84	3.11
60	-0.06	0.44	0.88	1.29	1.65	1.99	2.31	2.60	2.87
80	-0.35	0.10	0.52	0.91	1.27	1.60	1.91	2.21	2.48
100	-0.43	-0.03	0.35	0.71	1.04	1.35	1.64	1.92	2.18
125	-0.42	-0.06	0.28	0.59	0.89	1.17	1.44	1.70	1.94
150	-0.31	0.00	0.30	0.59	0.86	1.12	1.37	1.61	1.84
175	-0.09	0.20	0.47	0.73	0.99	1.23	1.46	1.68	1.89
200	0.17	0.43	0.68	0.93	1.16	1.38	1.60	1.81	2.08

1 ft = 0.305 m

Tables 22 and 23 give the same results as tables 20 and 21 using HS ratings assuming operating stress criteria. Since in general a 5 percent tolerance is allowed while rating a bridge before deciding on closings or posting, this same tolerance is assumed here. Again, using a safety index of 1.85 as criterion for acceptance of the risk inherent in a bridge, table 23 indicates that all bridges with HS-30 and higher ratings using operating stresses will be acceptable if the trucks given in figure 2 are adopted. Bridges with HS-25 ratings and spans less than 100 ft (30.48 m) will be acceptable, so will bridges with HS-27.5 ratings if their spans are less than 125 ft (38.10 m).

EFFECT OF CHANGING TRUCK WEIGHT REGULATION ON EXISTING BRIDGES

A major portion of the total cost impacts for new truck weight regulations is the effect on the existing bridge population. Some 130,000 bridges are now rated structurally deficient with an estimated $53 billion replacement or upgrading cost. If a new truck weight regulation introduced higher legal loads, a larger number of structural deficiencies will result; this will increase the estimated replacement and upgrading costs. The objective of this section is to develop an estimate of the replacement and upgrading costs that will result from implementing the proposed truck weight formula. The procedure outlined here for the cost analysis, follows the method developed for TRB's Truck Weight Study.[43]

To provide a base case for the cost analysis, bridges under current truck regulations are rated using vehicles corresponding to the current Federal Bridge Formula. This means that they should provide adequate capacity under the AASHTO legal vehicles.[47] A bridge is considered deficient under this base case scenario if these AASHTO vehicles cause stresses that exceed the operating stress level plus 5 percent tolerance. The operating stress level is obtained based on equation 22 with a safety factor 1/0.75 rather than 1/0.55 and a load L_n obtained from the AASHTO rating vehicles not the HS-loading. These proposed criteria are similar to the rating methods used by many State agencies.[43]

The cost allocation study considered a large sample of bridges that represent the highway classifications and regions in the U.S. Bridges of different spans, geometries, material and age were analyzed. The sample was obtained from the Federal Highway National Bridge Inventory System (NBI). The predictions using the Base Case model produced a total number of deficiencies close to the 130,000 estimated deficiencies which appears in the Secretary of Transportation annual report. [43]

In reference [43], it was assumed that as a result of changing the weight regulations, new designs would follow the same checking format of AASHTO given in equation 22. Thus, after the implementation of the proposed formula, new bridges will be designed for live loads L_n obtained using an HS-type vehicle with a gross weight that will provide an upper bound to the new legal loads. It was found that typically, an increase in the HS level causes only a small increase in the cost of a new bridge. For example, a 25 percent increase in design HS level increases the cost of a new bridge construction by only about 5 percent. Based on this estimate, the effect of changes in truck weight regulations on the construction costs of new bridges is minimal and can be neglected compared to the costs to the existing bridge network. Thus, the latter case will be the subject of the rest of this section.

The current US bridge network consists of 600,000 bridges. A number of these bridges would need upgrading if a new legislation allowing higher truck weights is implemented. In this analysis, it is assumed that any bridge that is found to be deficient under a new legislation will have to be replaced i.e. the possibilities of upgrading, posting or closing an existing bridge are not considered. The possibility of posting or closing bridges has been ignored because these options will entail economic and productivity costs to the shipping industry exceeding the cost of replacing the affected structures. Upgrading options are seldom used in practice because of Federal rules requiring that upgraded bridges should satisfy all regulations on geometry, lane widths, side barriers...[43]

As previously mentioned, additional bridge deficiencies resulting from adopting a new truck weight formula are found by rating all bridges using operating stress levels plus 5 percent load tolerance. The rating calculations use live load values due to typical vehicles satisfying the truck weight formula under consideration. Replacement costs of deficient bridges do not reflect the existing condition and age of the structures. Table 24 shows the estimates of current number of deficient simple span bridges in comparison to the expected number under the new regulations proposed in equation 25. This is presented for two different highway categories: primary systems and secondary systems. These estimates are also given separately for steel, reinforced concrete and prestressed concrete bridges. The cost implications of such number of deficiencies is given in terms of the total length of deficient bridges. Construction costs are usually given for unit area and since bridges have standard widths, the total length of deficient bridges is directly related to the total cost. [43]

The results of table 24 show that a large increase in the total length of deficient steel bridges on primary highway systems will accompany the implementation of equation 25. The change in the length of deficiencies is about 5.7 times the current length (or the total length of deficient bridges will be 6.7 times the current length). The additional length of deficiencies for the other types of bridges and highway classifications is in general less than 2.9 the current levels. One reason for this large number of deficiencies is that many existing bridges do not satisfy the WSD HS-20 inventory stress criteria used to determine the target reliability level. Still, the total number of deficiencies under the assumed trucks for the primary highways is 16,813 which roughly constitutes 18 percent of the total number of simple span bridges. Knowing that the proposed increases in the truck weights will result in a large increase in economic productivity, the increase in the number of deficiencies would be justified (see for example TRB's Truck Weight Study for a complete comparison between levels of productivity and economic costs for bridges and highways). The problem appears to be with the secondary bridge system. There, the total number of deficiencies under the existing legal limits is on the order of 38 percent; and the proposed change in truck weight regulation will produce a total number on the order of 50 percent of these bridges.

Table 25 shows the same results for continuous spans. Here, the number of deficiencies seems to increase drastically. The reason for this seemingly large increase in the number of deficiencies is that the NBI data does not specify the location of the deficiencies or whether the rating calculations were for the positive or negative bending moments. For this report, the number of deficiencies was calculated assuming the worst ratio of loads for positive and negative moments. Thus the number of deficiencies shown is an upper bound. Also, the number of current deficiencies was calculated assuming that one of AASHTO's legal vehicles is in one lane of the bridge as currently done by most rating engineers. [43] In the calculations performed for this report, it is assumed that it is possible to have two of the proposed typical vehicles following each other in one lane. It is estimated that these assumptions increased the estimated number of deficiencies by at least a factor of two.

The calculations were executed based on the moment effect of the seven representative vehicles chosen in figure 2 of chapter 2. Keep in mind that the number of deficiencies per material type does not necessarily reflect the quality of concrete versus steel bridges as the maintenance and age conditions of the bridges are not reflected in this tabulation.

Table 24. Consequences of implementation of proposed truck weight formula for simple span bridges.

	Total # of Bridges	Current Deficiencies		Expected Deficiencies	
Primary Highways		Number	Length (1000 ft)	Number	Length (1000 ft)
Steel	41140	1738	119.3	9051	798.8
R/C	25510	1174	43.7	3532	168.1
Ps/C	26474	1264	95.6	4230	345.3
Secondary Highways					
Steel	154401	80243	3759.1	99141	4993.6
R/C	81671	21510	652.8	33134	1173.8
Ps/C	48568	5503	247.5	11802	626.4

1 ft = 0.305 m

Table 25. Consequences of implementation of proposed truck weight formula for continuous span bridges.

	Total # of Bridges	Current Deficiencies	Expected Deficiencies
Primary Highways		Number	Number
Steel	32348	3182	18276
R/C	23973	2626	9393
Secondary Highways			
Steel	24250	6794	17187
R/C	26376	7226	14073

The moment effect of the typical vehicles given in figure 2 represent a conservative lower bound of the maximum moment envelope produced in table 3. This means that the results given in table 24 are a lower estimate of the number of deficiencies that is expected if the moment envelope generated in table 3 is directly used to determine the live load effect. Table 26 shows the expected number of deficiencies for this latter case. Compared to table 24, table 26 shows a total increase in the expected number of deficiencies of 8921 for primary highways and 25897 for secondary highways. These changes represent less than 10 percent of the total number of simple-span bridges in the network.

Table 26. Consequences of implementation of proposed truck weight formula for simple span bridges using maximum moment envelope.

	Total # of Bridges	Current Deficiencies	Expected Deficiencies
Primary Highways		Number	Number
Steel	41140	1738	13271
R/C	25510	1174	6109
Ps/C	26474	1264	6354
Secondary Highways			
Steel	154401	80243	109461
R/C	81671	21510	41458
Ps/C	48568	5503	19055

As proposed in reference [43], the calculation of the number of deficiencies has been performed from NBI files using three span length categories. These are: spans less than 60 ft (18.29 m) in length, spans between 60 and 120 ft (18.29-36.58 m) in length and spans greater than 120 ft (36.58 m). The existing network of simple span bridges counts roughly 380,000 bridges, 73 percent of which are less than 60 ft (18.29 m), 23 percent are between 60 ft (18.29 m) and 120 ft (36.58 m), while 4 percent are greater than 120 ft (36.58 m). The proposed truck weight formula (equation 25) was based on providing uniform safety levels for simple span bridges of all span lengths. The proposed formula produced much higher weight limits for the longer vehicles; these control the loading of the longer spans. The details of the calculations performed in this section indicated that the number of deficient bridges in each span length category remains roughly unchanged. In fact, out of the roughly 195,000 bridges found deficient in table 26, 73 percent had spans less than 60 ft (18.29 m), 23 percent were between 60 and 120 ft (18.29 - 36.58 m) long and 4 percent were longer than 120 ft (36.58 m). The fact that these percentages are the same as the percentages of bridges in each span length category, indicates that the proposed method dealt with all span categories uniformly. This uniformity occurs because longer bridges are in general better maintained and have higher ratings than the short span bridges; therefore, a large increase in the loading of the longer bridges does not necessarily imply a large increase in the number of deficiencies. Since the loading of the longer bridges is controlled by long vehicles, then heavier longer vehicles can be permitted without changing the percentage of deficiencies in each span length category.

Effect of changes in the target safety index and HS ratings on the number of expected bridge deficiencies

The truck weight formula and the maximum moment envelope used to develop table 26 are based on using a safety index target value of 2.5. If different safety index values are used then the maximum envelopes obtained will be as shown in table 16. These maximum moment envelopes can be used to calculate the number of expected bridge deficiencies assuming different target safety index values. The results are given in table 27. This table shows what would happen to the existing bridge network if the truck weight formula was developed based on HS-20 WSD steel bridges using safety index targets ranging between 2.0 and 3.5.

If the target safety index is increased, this means that a lower truck load effect is required producing a more conservative truck weight formula and a smaller number of projected deficiencies. The results given in table 27 indicate that if a safety index target equal to 3.5 is used, this will still produce a larger number of bridge deficiencies than currently observed. This is because a large number of existing bridges do not produce a safety index of 3.5 under current conditions. Remember that the safety index value of 3.5 corresponds to the average safety index obtained from the current WSD AASHTO code under current truck traffic conditions. Table 27 also shows that a change in the safety index target of 0.5 will produce a relative change in the number of deficient bridges on the order of 10 to 13 percent of the total number of simple-span bridges. That is, if one decides to reduce the expected number of deficiencies obtained in table 26 by 10 to 13 percent then the safety index value used to develop the truck weight formula should be increased from 2.5 to 3.0. If a decrease in the number of deficiencies on the order of 20 to 26 percent is required then, the truck weight formula should be developed based on a target safety index value equal to 3.5.

Table 27. Consequences of implementation of proposed truck weight formula for simple span bridges assuming different target safety index values.

		Number of Deficiencies				
		Current	Expected			
Target safety index:			2.0	2.5	3.0	3.5
	# of bridges					
Primary Highways						
Steel	41140	1738	20705	13271	8580	4636
R/C	25510	1174	9846	6109	3172	1312
Ps/C	26474	1264	12732	6354	4194	2090
Secondary Highways						
Steel	154401	80243	121718	109461	96949	83147
R/C	81671	21510	52912	41458	29732	16064
Ps/C	48568	5503	28222	19055	12902	7162

Table 26 was developed based on the maximum moment envelope (table 3) obtained assuming WSD steel bridges satisfying an inventory rating equal to HS-20 and using a target safety index value of 2.5. If bridges with different HS ratings are used to calculate the truck weight formula, then the maximum moment envelopes obtained will be as shown in table 10. Using these moment envelopes to calculate the number of expected bridge deficiencies is illustrated in table 28.

The results of table 28 indicate that in order to achieve the same or lower number of deficiencies than currently observed, a safety index target of 2.5 should be used on WSD simple-span steel bridges with inventory stress ratings equal to HS-10. It is also observed that a change in the HS-rating of 5 tons produces a change in the total number of deficiencies on the order of 11 to 17 percent. That is, if one decides to reduce the expected number of deficiencies obtained in table 26 by 11 t o17 percent then the truck weight formula should be developed to satisfy a target safety index value of 2.5 with bridges having inventory stress ratings of HS-15 rather than HS-20 as originally used. If a decrease in the number of deficiencies on the order of 22 to 34 percent is required then, the truck weight formula should be developed based on bridges with ratings equal to HS-10.

Table 28. Consequences of implementation of proposed truck weight formula for simple span bridges assuming different moment envelopes with different HS ratings.

	Current	Expected				
		Number of Deficiencies				
HS ratings:		HS-30	HS-25	HS-20	HS-15	HS-10
Primary Highways						
Steel	1738	27849	22033	13271	6838	1677
R/C	1174	16344	10947	6109	1988	296
Ps/C	1264	17388	13736	6354	3082	1213
Secondary Highways						
Steel	80243	135314	124633	109469	89389	64016
R/C	21510	65787	55177	41459	21584	5476
Ps/C	5503	36963	30151	19061	9340	3097

CONCLUSIONS

A sensitivity analysis has been performed to study the effect of errors in the data base and in the consequences of implementing the proposed truck weight formula on the safety and the ratings of existing bridges. The results of this sensitivity analysis demonstrate that the proposed formula is not sensitive to the assumed data base if the safety index criteria are changed accordingly.

If the proposed formula is adopted, it will increase the number of bridge deficiencies. These are calculated based on current rating methods which are not necessarily compatible with the reliability approach used in the determination of the proposed truck weight formula.

The 2.5 uniform safety index criteria used in this study is slightly more conservative than criteria used by Moses to develop load factors for the evaluation of the capacity of existing bridges, but is less conservative than the criteria proposed by Kulicki in the calibration of a new AASHTO bridge design code.[9,10] The results however provide an acceptable level of safety consistent with the observation of many State engineers.[2]

The proposed truck weight formula provides a uniform and rational approach to truck weight regulation with a uniform level of safety over all span lengths. It is also observed that the increase in the number of bridge deficiencies is uniformly spread over all span lengths.

The sensitivity analysis performed in this chapter also analyzed the change in the expected number of bridge deficiencies that would be obtained if different criteria than those of chapter two were used. These criteria include different target safety indices and bridges that have different HS rating levels.

Chapter 4 will confirm some of the observations made in this chapter by performing a detailed analysis of 12 typical U.S. bridges including concrete and steel, two-girder and multi-beam, simple and continuous bridges.

CHAPTER FOUR

ANALYSIS OF EXAMPLE BRIDGES

The analysis performed in chapter two for the determination of the allowable safe loads and the proposed truck weight formula applied a generalized load model (equation 21) to simplified bridge models i.e. simple span bridges with moment capacities satisfying AASHTO's WSD criteria for HS-20 loadings. For this chapter, a detailed analysis of specific bridges is performed. Twelve bridges of different material types, span lengths and configurations are analyzed for truck loads (see figure 2) corresponding to the proposed truck weight formula (equation 25) as determined in chapter two. The analysis is performed to check the ratings of these bridges if the proposed formula is adopted. The calculations compare the ratings assuming either one or two legal vehicles in each lane of the bridge. The two-vehicle case is used in this study in lieu of AASHTO's lane loading for long spans and continuous bridges. Many States use only one vehicle when rating their bridges. The analysis performed in this chapter will compare the results for these two cases. Different rating criteria are also compared including WSD and LFD ratings using inventory and operating stresses. Also, the ratings obtained for the typical vehicles are compared to the ratings obtained under AASHTO's HS-20 loading. In addition to the ratings, the safety index of each bridge is calculated assuming current loading conditions and compared to the safety index calculated assuming that the proposed truck weight formula (equation 25) is adopted. The cost of rehabilitating the bridges that did not meet the inventory rating criteria is estimated to project the costs incurred if the proposed new truck weight formula is adopted. The analysis also provides stress ranges at critical bridge locations that are used in chapter five for the fatigue analysis.

The 12 bridges analyzed were selected to give a rough representation of typical U.S. bridges. These include two-girder and stringer, simple and continuous span bridges of steel, reinforced concrete and prestressed concrete members. This chapter gives a summary of the observations made for each of the 12 structures. The rating calculations performed for bridge no. 1 are given in detail in the following section and summarized in table 29. However, only a summary of the results of the rating calculations for the other 11 bridges is given along with the tables presenting the final results.

In addition to the conventional rating, two of the steel bridges were analyzed using a 3-D finite element program. This is done to compare the results of the conventional rating to the more exact finite element results.

COMPOSITE STEEL MULTI-BEAM BRIDGES

1. Old Riverdale Drive

This bridge on Old Riverdale Drive in Danville, Pittsylvania County is owned by Virginia Department of Highways and Transportation (VDOHT). The bridge is a two-span, continuous-composite multibeam structure which is assumed to be noncomposite for negative moment regions. It has unequal spans of 95 ft and 70 ft (28.96 and 21.34 m). The W36x245 and W36x135 beams are spaced at 8 ft (2.44 m) center to center. This two-lane right bridge provides an example of typical rolled beam structures. The layout of the bridge is given in figure 3.

WSD rating

WSD ratings for all sections of the bridge were calculated for the seven typical vehicles shown in figure 2. The results show that the most critical section is the 0.4 point of span 1 when loaded with vehicle 5. The dead load moments at that section are 869 kip-ft (1178 kN-m) for the regular dead load including diaphragms and haunches, and 189 kip-ft (256 kN-m) for the superimposed dead loads such as sidewalks and parapets. The live load plus impact moment for vehicle 5 is 1233 kip-ft (1672 kN-m). The

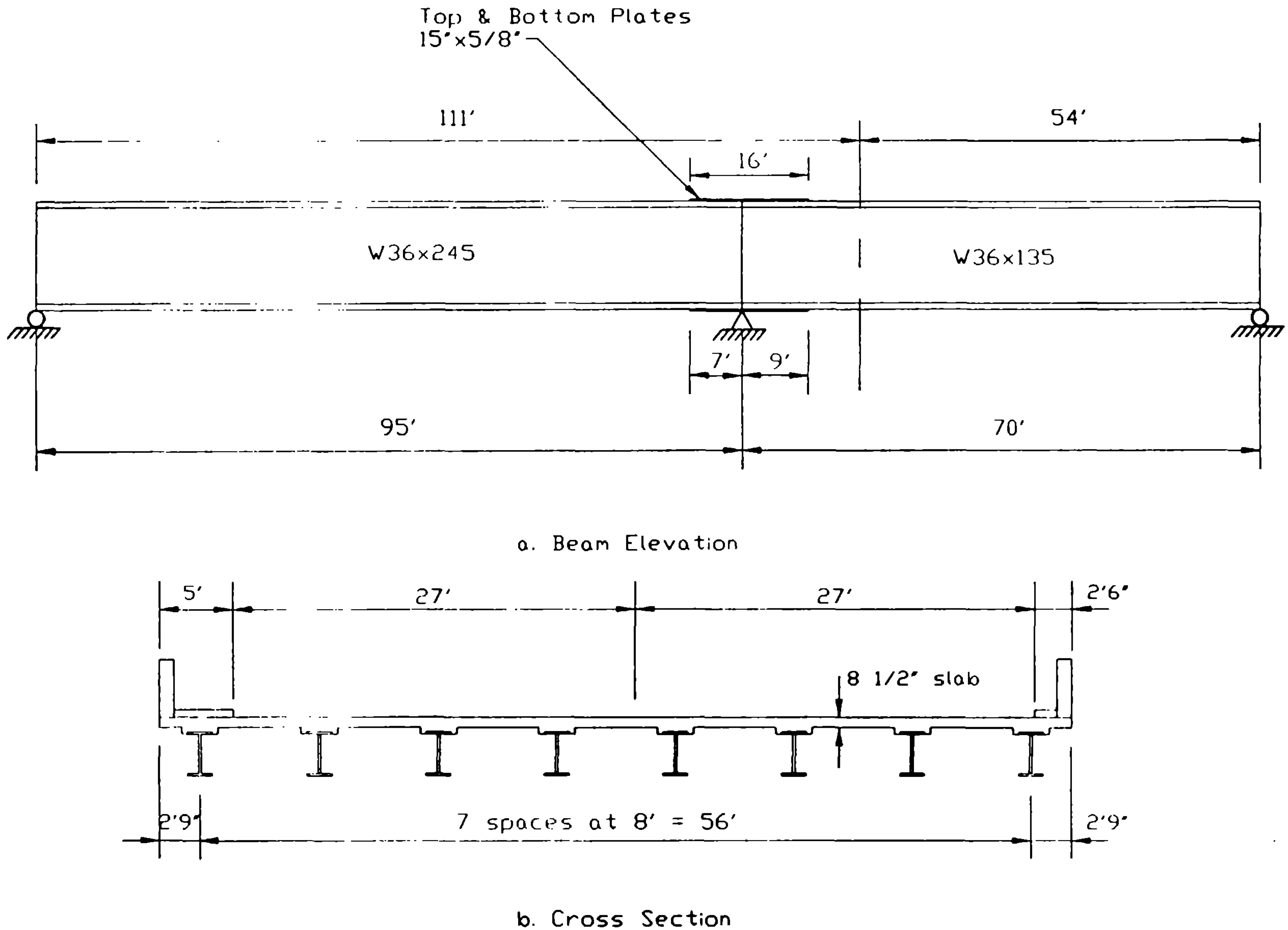
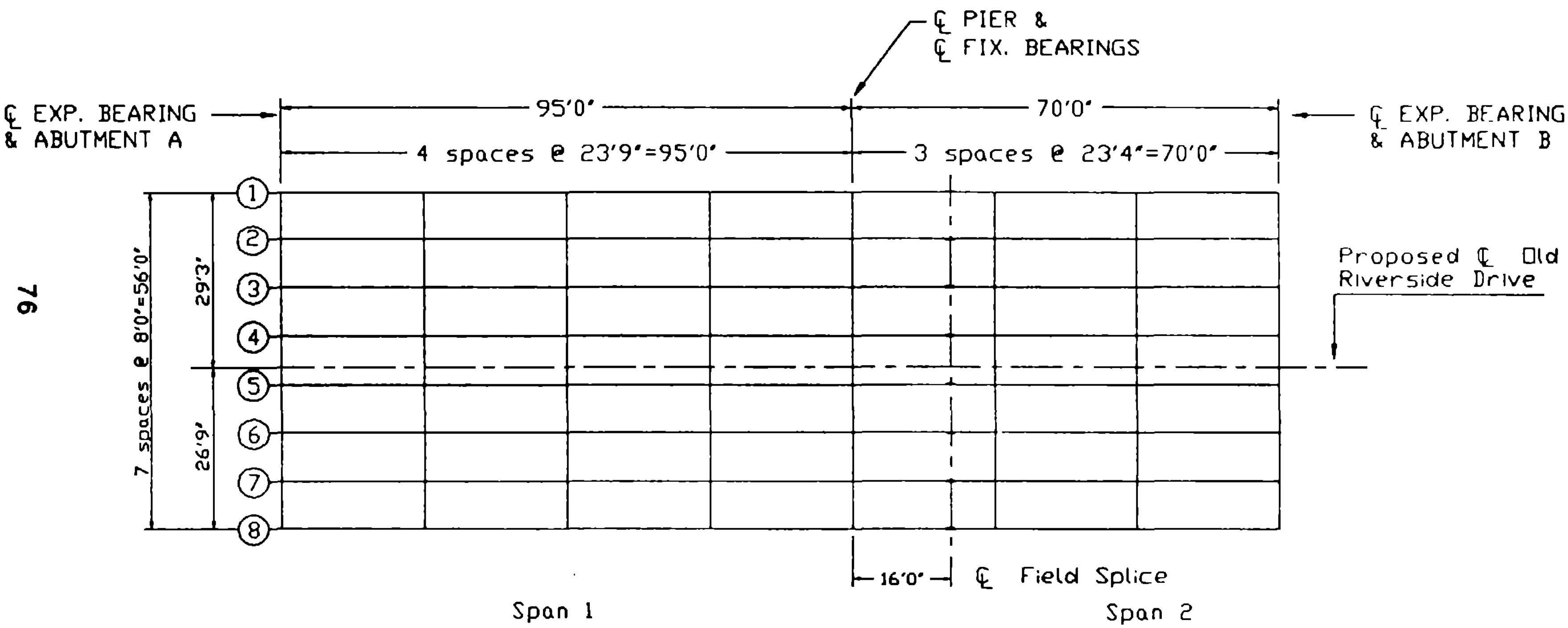


Figure 3. Layout of bridge #1.



c. Framing Plan

Figure 3. Layout of bridge #1 (continued).

W36x245 beam is compact and the positive moment capacity was calculated to be 7007 kip-ft (9501 kN-m). The composite moment of inertia is 41,100 in^4 (17106×10^6 mm^4) producing a section modulus S=1280 in^3 (20.98×10^6 mm^3). Rating factors (RF) equal to 1.18 (inventory) and 2.03 (operating) are obtained for vehicle 5. The HS-20 truck produced a maximum live load plus impact moment of 1018 kip-ft (1380 kN-m). The corresponding ratings are HS-29.4 for inventory stresses and HS-50.6 for operating stresses for the positive moment region (RF=1.47 and 2.53 respectively).

Since no lane loading is provided with the typical vehicles given in figure 2, rating calculations were also performed for two trucks following each other in the same lane. In order to minimize the calculations, it was always assumed that the two vehicles following each other are of the same type. The headway distance between the two vehicles was varied from 30 ft (9.14 m) up to a maximum of 80 ft (24.38 m). In this particular example, the critical point was also the 0.4 point of the first span under two type 5 vehicles. The rating factors obtained were unchanged, i.e., 1.18 for the inventory level and 1.23 for operating.

LFD rating

Using LFD criteria for rating, the critical point is over the pier under the effect of one vehicle of type 7. The negative moment due to regular dead load at the pier is 1235.7 kip-ft (1676 kN-m) and for the superimposed dead load is 260.4 kip-ft (353 kN-m). The negative live load plus impact moment is 964.20 kip-ft (1307 kN-m). The section over the pier was found to be braced but noncompact for negative moment. Ultimate negative moment capacity assuming noncomposite action is 5000 kip-ft (6780 kN-m). The inventory rating factor was 1.46 and the operating factor was 2.44. For HS-20 loading the maximum negative live load moment at the pier is 845.6 kip-ft (1147 kN-m) producing rating factors of 1.66 or HS-33.2 (inventory) and 2.78 or HS-55.6 (operating). When two vehicles are assumed to follow each other in one lane, the maximum negative moment obtained becomes 1365.8 kip-ft (1852 kN-m) for two type 5 vehicles. This reduces the inventory rating to HS-20.6 (RF=1.03) and the operating rating to HS-34.4 (RF=1.72). A summary of the input data and the results of the rating calculations are given in table 29.

Table 29. Rating summary for bridge # 1.

Rating method	WSD	WSD	LFD	LFD
No. of vehicles considered	1	2	1	2
critical vehicle type	5	5	7	5
critical section:	0.4 L_1	0.4 L_1	over pier	over pier
description of section:	W36x245	W36x245	W36x245 w/ Pl. 5/8x15	W36x245 w/ Pl. 5/8x15
dead load moments (kip-ft)	869 + 189	869 + 189	1236 + 260	1236 + 260
special vehicle (LL+I) (kip-ft)	1233	1233	964	1366
HS-20 moment (kip-ft)	1018	1018	846	846
section type	composite	composite	noncomposite	noncomposite
section modulus (in^3)	1280	1280	-	-
ultimate moment capacity (kip-ft)	-	-	5000	5000
special vehicle inventory rating	1.18	1.18	1.46	1.03
special vehicle operating rating	2.03	2.03	2.44	1.72
HS inventory rating	(HS-29.4) 1.47	(HS-29.4) 1.47	(HS-27.0) 1.66	(HS-27.0) 1.66
HS operating rating	(HS-50.6) 2.53	(HS-50.6) 2.53	(HS-55.6) 2.78	(HS-55.6) 2.78

Calculation of safety index

The above rating evaluation (except for the new truck types used) is a traditional deterministic method to check whether an existing bridge is capable of supporting the proposed vehicles. A reliability method can also be used to determine the risk of bridge failure under the proposed new truck weight regulations. This is done by calculating the safety index for this bridge at the critical section. The same failure function given in equation 9 is used to calculate the safety index. The random variables are, R, D and L. R can be evaluated based on the actual capacity of the critical section, D can be evaluated based on the dead load moment at the critical location and L can be evaluated as illustrated in equation 21.

To perform the safety index calculation, one needs the mean value and the C.O.V. in addition to the type of the probability distribution of every variable in the failure function. For the positive bending moment capacity of the composite beam section, reference [17] suggests using a mean maximum moment capacity equal to 1.04 times the calculated design moment capacity. This is also associated with a C.O.V. equal to 14 percent. The mean dead load moment values are assumed to be equal to the calculated dead load values with a C.O.V. of 9 percent. The live load moment is calculated using equation 21. Reference [46] uses a deterministic variable a=18.4 kip-ft (24.95 kN-m) for a vehicle with 1 kip (4.448 kN) total weight. The mean of m is given as 0.95 with a C.O.V. of 7 percent. H has a mean value of 2.80 with a C.O.V. of 7 percent. The mean of the load distribution factor g is assumed to be 0.9 of the AASHTO value and the C.O.V. is 8 percent. The impact factor is assumed to be 1.2 with a C.O.V. of 8 percent. The mean growth factor is assumed to be 1.15 with a C.O.V. of 10 percent. The $W_{.95}$ value under current truck weight regulations has a mean value of 75 kips (333.6 kN) with a C.O.V. of 10 percent. For the projected traffic under the proposed truck weight formula, the mean $W_{.95}$ value is assumed to be 101.3 kips (450.58 kN). This latter value is obtained based on a comparison between the measured $W_{.95}$ and the maximum permissible truck weights under current regulations. In calculating the 101 kips (450.58 kN) value, we assume that the bias between the proposed maximum permissible loads and $W_{.95}$ remains the same as currently observed. The safety index calculation was performed assuming that all the random variables follow lognormal distributions, this produced a value of 4.96 under current loading conditions and 3.86 for the projected loading if the proposed truck weight formula is adopted.

For the negative moment capacity, the section is non-compact and non-composite. The bias between the calculated mean moment capacity and the nominal elastic capacity is 1.03 and the C.O.V. is 12 percent.[17] For the moment at the support, reference [39] recommends using a=11.01 kip-ft (14.93 kN-m), mean of m=0.97 with a C.O.V. of 6 percent and a mean of H equal to 2.91, the rest of the data is the same as the data used for the calculation of the safety index for the critical positive moment. Calculations of the safety index for current loading conditions gives a value of 4.43. Under the projected loading conditions assuming that the new truck weight formula is in effect, the safety index reduces to 3.44. The input data and the results of the safety index calculations are given in table 30 below. If a safety index value equal to 2.5 is used here as a measure of acceptable risk, this bridge is then considered very safe for both the current loading conditions and the projected loading if the proposed truck weight formula is adopted. A summary of the input data and the results of the safety index calculations are given in table 30.

Table 30. Summary of safety index calculation for bridge # 1.

Variable	Positive bending		Negative bending	
	mean	COV	mean	COV
R (kip-ft)	7287	14%	5150	12%
D (kip-ft)	1058	9%	1496	9%
a (kip-ft/kip)	18.4	-	11.01	-
m	0.95	7%	0.97	6%
H	2.80	7%	2.91	7%
i	1.2	8%	1.2	8%
g	0.65	8%	0.65	8%
Gr	1.15	10%	1.15	10%
$W_{.95}$ (current - kips)	75	10%	75	10%
$W_{.95}$ (projected - kips)	101	10%	101	10%
current safety index	4.96	-	4.43	-
projected safety index	3.86	-	3.44	-

Summary for bridge 1

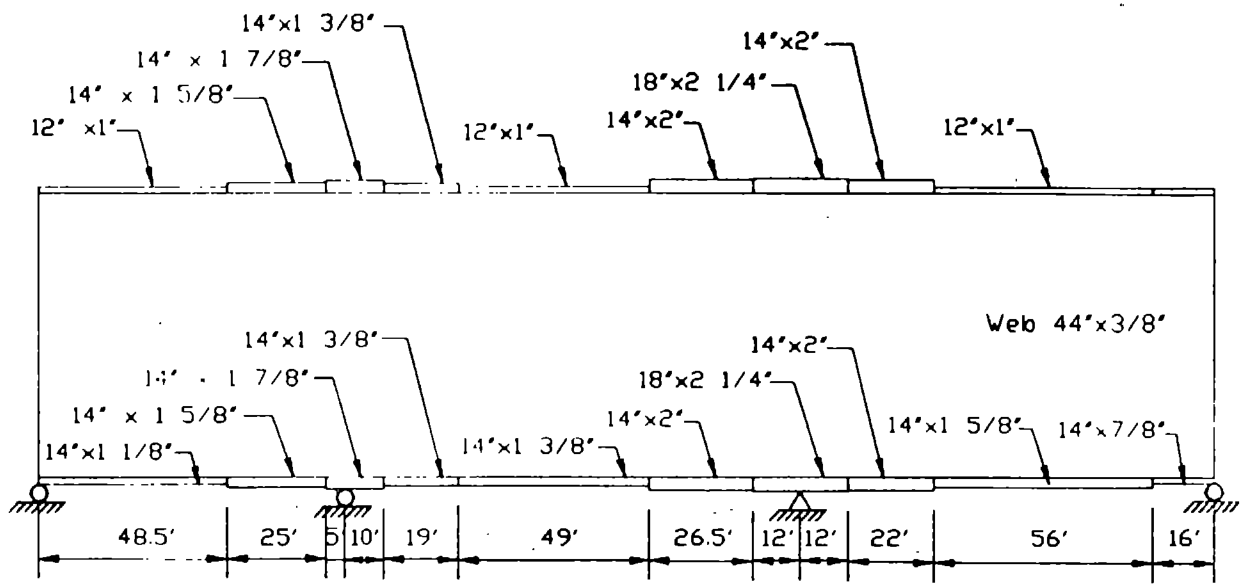
The results of the rating calculations for bridge #1 are given in table 29. The table gives a summary of the WSD and LFD rating calculations. The results show that adopting the proposed truck weight formula is safe for this bridge. This is true whether the evaluation is performed using current deterministic methods (LFD or WSD ratings) or using probabilistic methods (safety index calculations as shown in table 30). In fact, both inventory and operating rating produced factors above 1.0. Using one or two vehicles in one lane does not change the rating using WSD criteria. This is expected since the critical section is in the positive bending region and the span length is relatively short. Using the LFD approach, the critical section is over the pier and the effect of the second vehicle becomes important reducing the inventory rating from 1.46 to 1.03. This bridge provided high rating values under AASHTO's HS loading: 1.47 for inventory rating using WSD criteria and 1.66 using LFD. Adopting the new vehicles proposed in figure 2 will lead to a decrease in the rating factors to 1.18 and 1.46 for one vehicle under WSD and LFD criteria respectively and down to 1.03 for two vehicles with the LFD criteria. In all the cases considered, the rating values remain above 1.0. In addition, the probabilistic evaluation produced safety index values above the minimum target value of 2.5 for the two critical points of the bridge.

2. I-95 over Ramp A

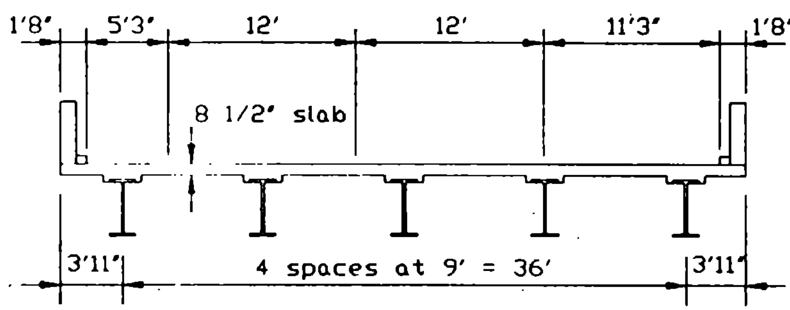
This bridge, also owned by Virginia Department of Highways and Transportation, is located in Prince George County in Virginia. The structure shown in figure 4, is a three-span continuous steel stringer bridge and is designed to be composite. The beams are spaced 9 ft (2.74 m) center to center. The spans are 78.5 ft (23.9 m), 116.5 ft (35.5 m) and 106 ft (32.3 m) long. The beams have webs which are 44 in (1118 mm) by 3/8 in (9.52 mm) and variable flanges (see figure 4-a). The skew angle is 57°.

A WSD rating with one truck in each lane produced an inventory rating factor of 1.14 and an operating rating factor of 1.90. These values drop to 0.82 and 1.52 for the inventory and operating ratings using two vehicles in one lane. The LFD rating factors drop from 1.35 to 0.91 for inventory stresses and from 2.25 to 1.52 for the operating stresses. Under HS loading, the bridge is quite satisfactory with rating factors above 1.30 (HS-26) using inventory stress criteria and above 2.39 (HS-47.8) using operating stresses. The results indicate that if the proposed truck weight formula is adopted, this bridge would be considered safe using operating stress criteria. The bridge would be overstressed under inventory stress levels when considering two vehicles in each lane since the rating factors are 0.82 and 0.91 respectively for the WSD and LFD methods. This latter observation is, however, conservative since it implies that the bridge will be loaded by two legal vehicles in all lanes simultaneously and it assumes that the bridge section is noncomposite at the piers. A summary of the ratings is given in table 31.

The results of the safety index calculations are given in table 32. They show safety indices above 2.5 for all the cases considered. The high dead to live load ratio insures that, for this particular bridge, the risk of failure is small as indicated by the high safety indices. These remain above 3.25 even when the truck loads are increased under the proposed truck weight formula.

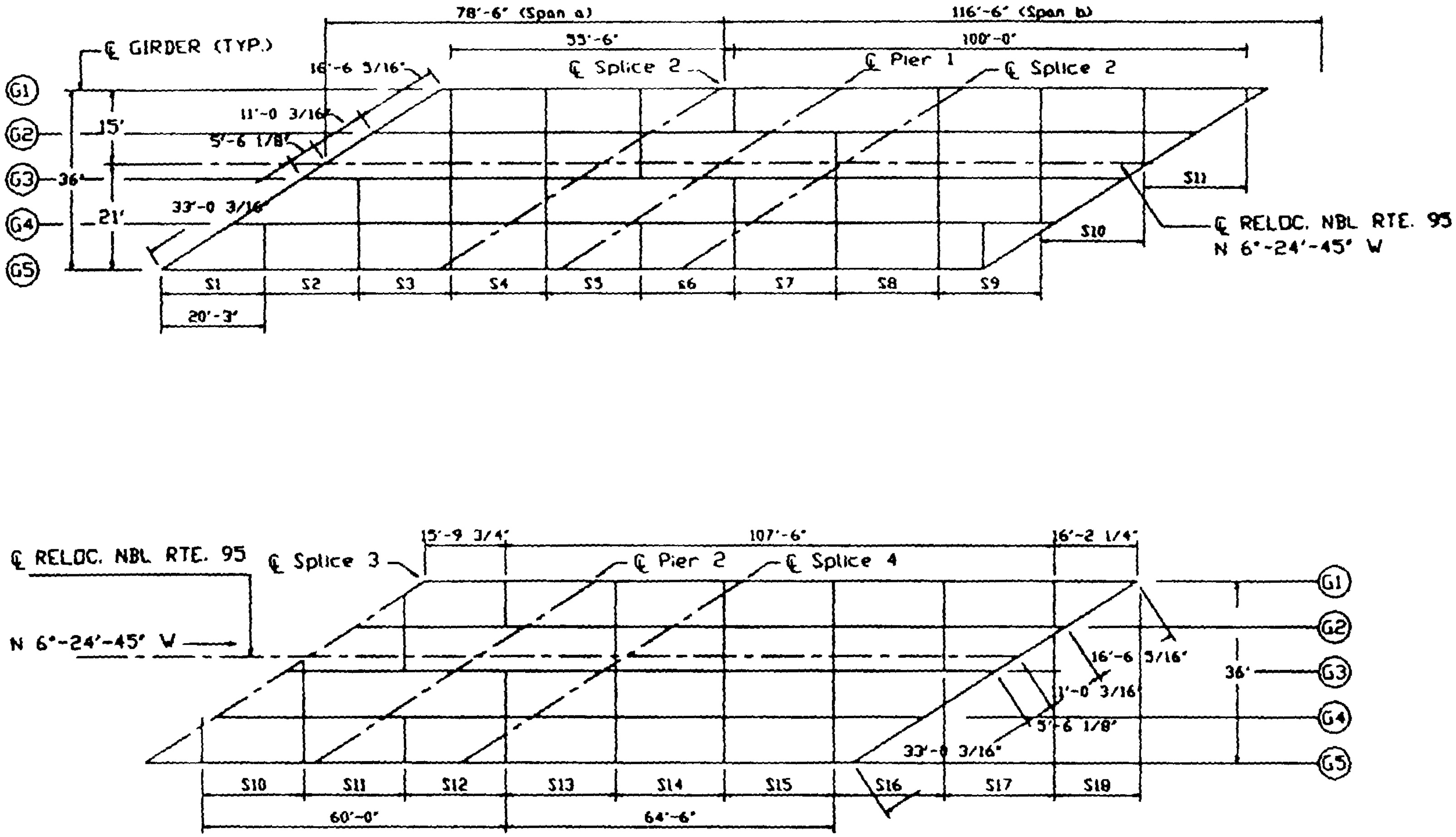


a. Beam Elevation



b. Cross Section

Figure 4. Layout of bridge #2.



c. Framing Plan

Figure 4. Layout of bridge #2 (continued).

Table 31. Rating summary for bridge # 2.

Rating method	WSD	WSD	LFD	LFD
No. of vehicles considered	1	2	1	2
critical vehicle type	5	5	5	5
critical section:	0.6 L_3	over pier 2	0.6 L_3	over pier 1
description of section:	web 44x3/8	web 44x3/8	web 44x3/8	web 44x3/8
	t. fl. 1x12	t. fl. 2 1/4x18	t. fl. 1x12	t. fl. 1 7/8x14
	b. fl. 1 5/8x14	b. fl. 2 1/4 x18	b. fl. 1 5/8x14	b. fl. 1 7/8x14
dead load moments (kip-ft)	970 + 118	2264 + 232	970 + 118	1317 + 145
special vehicle moment (kip-ft)	1594	2263	1594	1711
HS moment (kip-ft)	1265	1425	1265	1082
section type	composite	noncomposite	composite	noncomposite
section modulus (in^3)	1459	1897	-	-
ultimate moment capacity (kip-ft)	-	-	6079	5287
special vehicle inventory rating	1.14	0.82	1.35	0.91
special vehicle operating rating	1.90	1.52	2.25	1.52
HS inventory rating	(HS-28.8) 1.44	(HS-26.0) 1.30	(HS-34.0) 1.70	(HS-28.8) 1.44
HS operating rating	(HS-47.8) 2.39	(HS-48.2) 2.41	(HS-56.8) 2.84	(HS-48.2) 2.41

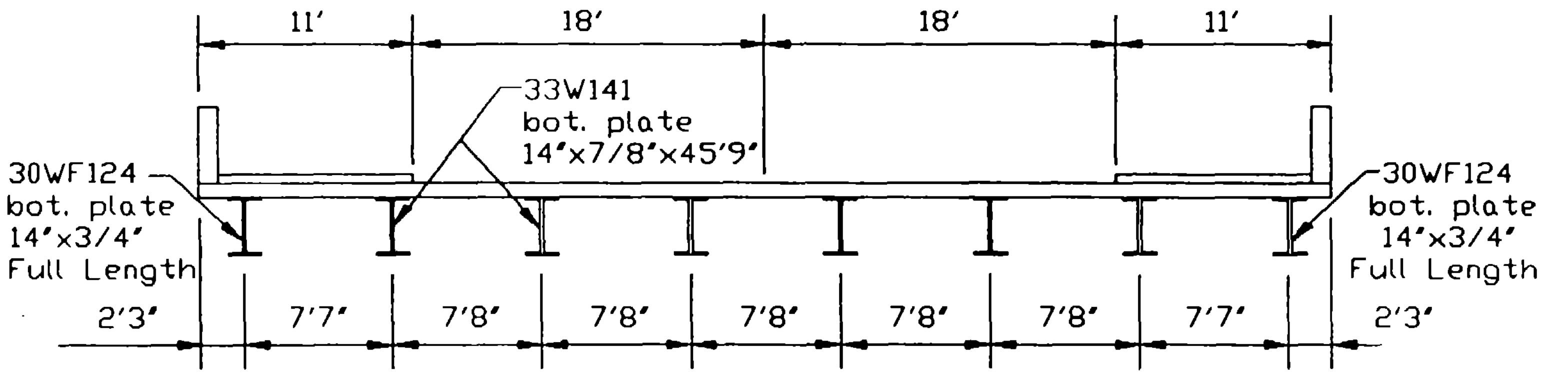
Table 32. Summary of safety index calculation for bridge # 2.

Variable	Positive bending		Negative bending	
	mean	COV	mean	COV
R (kip-ft)	6322	14%	5446	12%
D (kip-ft)	1088	9%	1462	9%
a (kip-ft/kip)	15.6	-	11.01	-
m	0.96	7%	0.97	6%
H	2.82	7%	2.91	7%
i	1.2	8%	1.2	8%
g	0.74	8%	0.74	8%
Gr	1.15	10%	1.15	10%
$W_{.95}$ (current - kips)	75	10%	75	10%
$W_{.95}$ (projected - kips)	101	10%	101	10%
current safety index	4.34	-	4.43	-
projected safety index	3.27	-	3.34	-

3. Delaware Avenue/Catskill Thruway

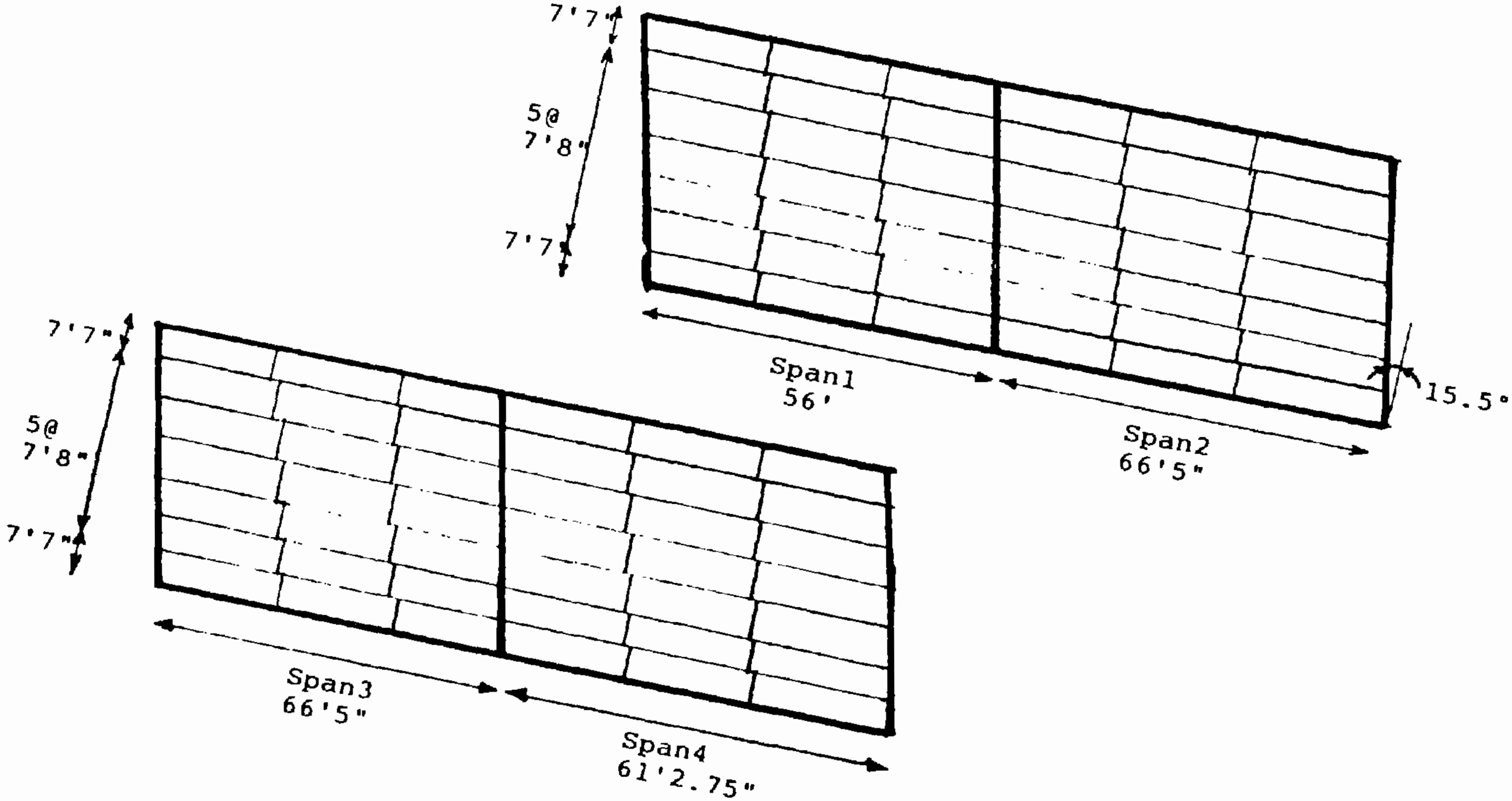
This bridge, owned by the New York State Thruway Authority, has four simple spans of I-beam composite design. The spans are 56, 66.5, 66.5 and 61 ft (17.1, 20.3, 20.3, 18.6 m). Interior stringers are W33x141 or W33x152 and exterior stringers are W30x124 or W30x132. The skew angle is 15° 30'. This bridge provides an example of the old AASHTO designs using lighter exterior stringers.

WSD and LFD operating and inventory ratings were performed for all critical sections of the bridge. The bridge is found to be deficient under current AASHTO HS-20 loading



Exterior beam of simply supported span 4 is critical.
Stringers are W30x124 plus bottom plate 14" x 3/4".

Figure 5. Layout of bridge #3.



Framing Plan

using WSD and inventory stress criteria. The inventory rating factor (RF) is equal to 0.72 (HS-14.4). The WSD inventory RF drops to 0.69 under the proposed new truck weight formula. But, it remains above 1.41 if the operating stress levels are used. LFD rating indicates that the bridge is satisfactory under current HS-20 loads as well as under the proposed new vehicles regardless of whether inventory or operating stresses are used.

The high dead load to live load ratio of this bridge produce high safety index values. Despite the low WSD inventory rating, the safety index calculations show this bridge capable of withstanding current loads as well as the additional loads expected under the proposed truck weight formula.

Table 33. Rating summary for bridge # 3.

Rating method	WSD	WSD	LFD	LFD
No. of vehicles considered	1	2	1	2
critical vehicle type	5	5	7	5
critical section (ext. beam):	0.5 L_4	0.5 L_4	0.5 L_4	0.5 L_4
description of section:	W30x124	W30x124	W30x124	W30x124
	pl. 14x3/4	pl. 14x3/4	pl. 14x3/4	pl. 14x3/4
dead load moments (kip-ft)	440 + 187	440 + 187	440 + 187	440. + 187
special vehicle moment (kip-ft)	690	690	690	690
HS moment (kip-ft)	663	663	663	663
section type	composite	composite	composite	composite
section modulus (in^3)	932	932	-	-
ultimate moment capacity (kip-ft)	-	-	3223	3223
Special vehicle inventory rating	0.69	0.69	1.61	1.61
Special vehicle operating rating	1.41	1.41	2.69	2.69
HS inventory rating	(HS-14.4) 0.72	(HS-14.4) 0.72	(HS-33.4) 1.67	(HS-33.4) 1.67
HS operating rating	(HS-29.4) 1.47	(HS-29.4) 1.47	(HS-55.8) 2.79	(HS-55.8) 2.79

Table 34. Summary of safety index calculation for bridge # 3.

Variable	mean	COV
R (kip-ft)	3352	14%
D (kip-ft)	627	9%
a (kip-ft/kip)	13.57	-
m	0.94	6%
H	2.75	10%
i	1.2	8%
g	0.58	8%
Gr	1.15	10%
$W_{.95}$ (current - kips)	47	15%
$W_{.95}$ (projected - kips)	62	15%
current safety index	4.72	-
projected safety index	3.84	-

4. I-57 over ILLINOIS RTE 17

The Illinois Department of Transportation bridge carries I-57 over Rte 17 (Sect. 139 HBR) in Kankakee County. The three continuous spans are 50, 62.5 and 50 ft (15.2, 19.1, 15.2 m) and are composite for positive moment only. The beams are W30x124, except for one of the exterior beams which is W33x130. The skew angle is 1°. This bridge was widened and has a larger fascia beam on one side. It is a good example of a continuous beam bridge with optimal span ratios (see figure 6).

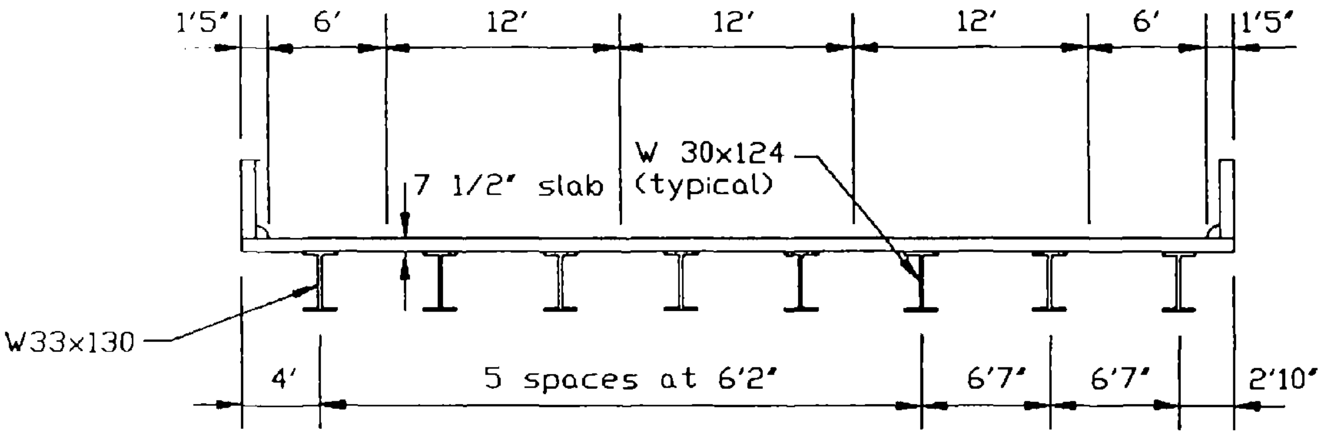


Figure 6. Layout of bridge #4.

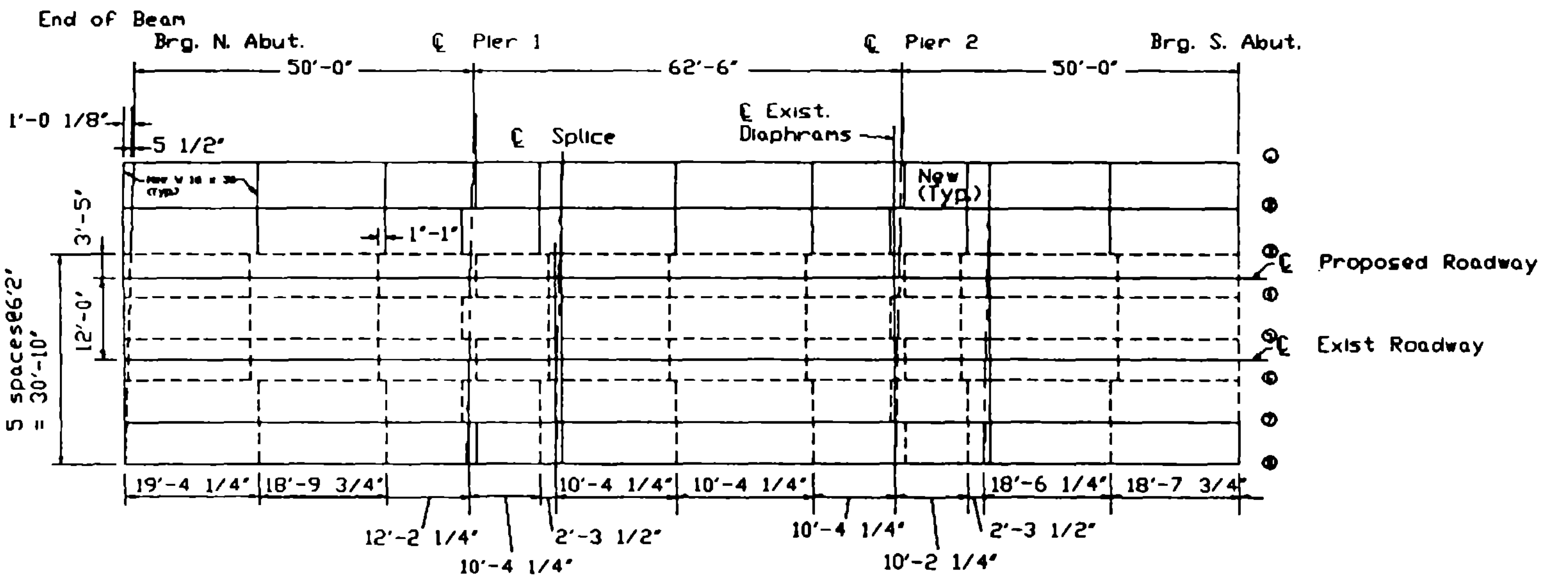


Figure 6. Layout of bridge #4 (continued).

As see in table 35, the WSD rating for HS loading produced factors of 0.75 (HS-15) for inventory and 1.41 (HS-28.2) for operating stress levels. This would indicate that the bridge is considered deficient under current loading for inventory stress. For typical vehicle #7 under the proposed truck weight formula, the WSD rating factors are 0.61 (inventory) and 1.14 (operating). WSD rating for two type 3 vehicles gave an inventory rating factor of 0.52 and an operating rating factor of 0.98 over the piers. Thus, the inventory rating by WSD indicates that this bridge is deficient under the current HS loading, and also, under the proposed truck weight formula.

Table 35. Rating summary for bridge # 4.

Rating method	WSD	WSD	LFD	LFD
No. of vehicles considered	1	2	1	2
critical vehicle type	7	3	7	3
critical section:	over pier 1	over pier 1	over pier 1	over pier 1
description of section: (F_y=33 ksi)	W30x124	W30x124	W30x124	W30x124
dead load moments (kip-ft)	281 + 33	281 + 33	281 + 33	281 + 33
special vehicle moment (kip-ft)	368	429	368	429
HS moment (kip-ft)	298	298	298	298
section type	noncomposite	noncomposite	noncomposite	noncomposite
section modulus (in^3)	355	355	-	-
ultimate moment capacity (kip-ft)	-	-	1122	1122
special vehicle inventory rating	0.61	0.52	1.05	0.90
special vehicle operating rating	1.14	0.98	1.75	1.51
HS inventory rating	(HS-15.0) 0.75	(HS-15.0) 0.75	(HS-25.6) 1.28	(HS-25.6) 1.28
HS operating rating	(HS-28.2) 1.41	(HS-28.2) 1.41	(HS-42.8) 2.14	(HS-42.8) 2.14

LFD rating indicates that the bridge is safe if one vehicle is assumed in each lane: The rating factors are 1.05 and 1.75 for inventory and operating stress respectively. The same is true under the HS loading: it produced rating factors equal to 1.28 (HS-25.6) and 2.14 (HS-42.8) for inventory and operating stress levels respectively. For two vehicles in one lane, the LFD ratings drop to 0.90 and 1.51. This shows that using the LFD method, the bridge will be considered deficient when all the lanes are loaded by at least two type 3 vehicles.

The safety index calculation (table 36) indicates that this bridge is safe under current loading conditions as represented by a safety index of 2.74. The safety becomes less probable when the new truck weight formula is implemented. This will produce a safety index of 1.68 which is below the target value of 2.5 adopted as the acceptable safety index target in this study. In order to support the loads projected under the proposed formula, this bridge needs to be rehabilitated.

Table 36. Summary of safety index calculation for bridge # 4.

Variable	mean	COV
R (Kip-ft)	1156	12%
D (kip-ft)	314	9%
a (kip-ft/kip)	4.49	-
m	1.02	8%
H	2.81	7%
i	1.2	8%
g	0.56	8%
Gr	1.15	10%
$W_{.95}$ (current - kips)	75	10%
$W_{.95}$ (projected - kips)	101	10%
current safety index	2.74	-
projected safety index	1.68	-

5. US-17 over Lockwood Dr.

This bridge located on U.S. 17 N.B. over Lockwood Drive is owned by South Carolina Department of Highways and Transportation. This is a seven span continuous composite steel stringer bridge (see figure 7). The spans are 105, 135, 135, 152, 135,135 and 105 ft (32, 41.1, 41.1, 46.3, 41.1, 41.1 and 32 m). The bridge is on tangent with a horizontal curve in span G. The bridge is superelevated with a cross slope of 0.0435 ft/ft (m/m) in span G. The 51 in (1295 mm) deep beams are spaced 8 ft 2 in (2.5 m) center to center. Span G is 57 in (1448 mm) deep. The web plates are 48 in (1219 mm) web plates, span G which has 54 in (1372 mm) plates. This bridge provides a good example of the behavior of bridges with relatively long spans.

The rating summary is given in table 37. WSD rating of the section over pier 6 under HS loading produced a rating factor of 0.64 which indicates that the bridge is deficient using inventory stress. The operating rating factor is 1.42. These values drop to 0.45 and 0.89 for two type 5 trucks in each lane. The LFD rating factors for two type 7 trucks are 0.59 and 0.98 for inventory and operating stress ratings respectively. These compare to 0.85 and 1.41 for the HS loading. These calculations indicate that this bridge may not be capable of supporting the proposed loads without strengthening unless the LFD operating stress ratings are adopted as the safety criteria. The value of 0.98 for LFD operating stress rating is within the allowable 5 percent tolerance which a number of States allow before a bridge is considered deficient. The rating factors obtained are on the conservative side since they assume that all lanes will be loaded by two typical vehicles which is unlikely in real life. Also, the assumption of no composite action at the piers is conservative.

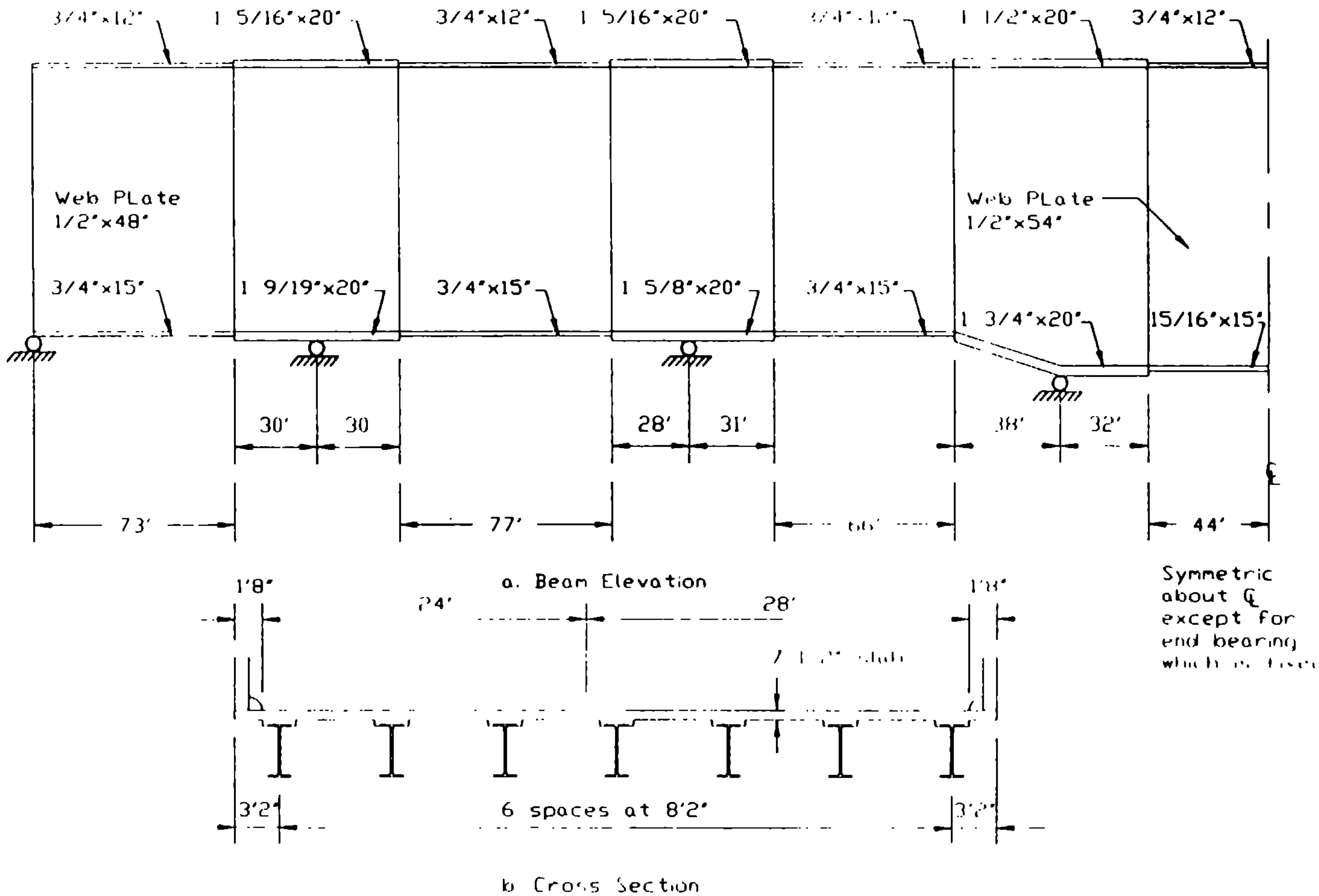
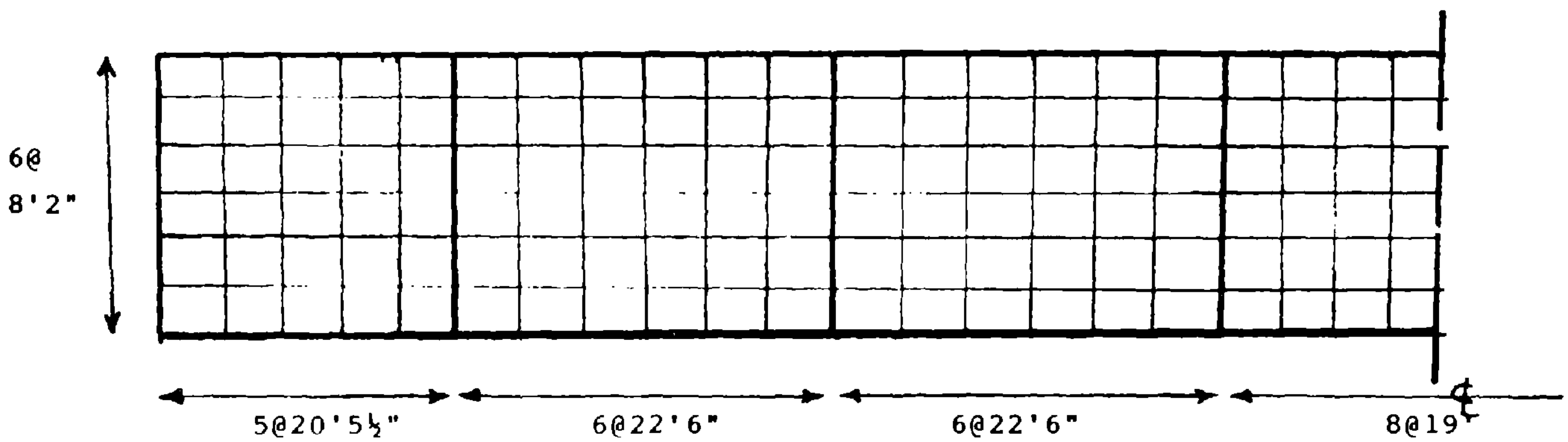


Figure 7. Layout of bridge #5.



Framing Plan

Figure 7. Layout of bridge #5 (continued).

Table 37. Rating summary for bridge # 5.

Rating method	WSD	WSD	LFD	LFD
No. of vehicles considered	1	2	1	2
critical vehicle type	7	5	5	7
critical section:	0.5 L_2	over pier 6	0.5 L_2	over pier 5
description of section:	web 1/2x48	web 1/2x48	web 1/2x48	web 1/2x48
	t. fl. 3/4x12	t. fl. 1 5/16x20	t. fl. 3/4x12	t. fl. 1 5/16x20
	b. fl. 3/4x15	b. fl. 1 9/16x20	b. fl. 3/4x15	b. fl. 1 5/8 x20
dead load moments (kip-ft)	706 + 94	2122 + 238	706 + 94	2009 + 232
special vehicle moment (kip-ft)	1445	2258	1445	2546
HS moment (kip-ft)	1093	1579	1093	1767
section type	composite	noncomposite	composite	noncomposite
section modulus (in^3)	1062	1472	-	-
ultimate moment capacity (kip-ft)	-	-	4425	6158
special vehicle inventory rating	0.85	0.45	0.91	0.59
special vehicle operating rating	1.45	0.89	1.52	0.98
HS inventory rating	(HS-22.6) 1.13	(HS-12.8) 0.64	(HS-24.0) 1.20	(HS-17.0) 0.85
HS operating rating	(HS-38.4) 1.92	(HS-28.4) 1.42	(HS-40.0) 2.00	(HS-28.2) 1.41

In this example, the reliability calculations show that the bridge is safe under current loading conditions with a safety index on the order of 3.5 (see table 38). The projected loading produced a safety index of 2.4 slightly lower than the target safety index of 2.5, this shows that the bridge is at some risk of failure, the risk however might still be acceptable since it will be localized at the midpoint of the second span. In fact, sections of maximum negative bending produced safety indices above 2.69 which is higher than the target safety index.

Table 38. Summary of safety index calculation for bridge # 5.

Variable	Positive bending		Negative bending	
	mean	COV	mean	COV
R (kip-ft)	4602	14%	6343	12%
D (kip-ft)	800	9%	2241	9%
a (kip-ft/kip)	15.06	-	13.5	-
m	1.00	6%	1.00	6%
H	2.87	7%	3.00	7%
i	1.2	8%	1.2	8%
g	0.67	8%	0.67	8%
Gr	1.15	10%	1.15	10%
$W_{.95}$ (current - kips)	75	10%	75	10%
$W_{.95}$ (projected - kips)	101	10%	101	10%
current safety index	3.51	-	3.56	-
projected safety index	2.42	-	2.69	-

STEEL TWO-GIRDER BRIDGES

6. I-279/ Ramp A in Alleghany County

This bridge is located in the Commonwealth of Pennsylvania on I 279 L.R. 1016-11 F. This is a right, dual four-span welded continuous steel two-girder bridge (figure 8). The spans are 69, 110, 110 and 74 ft (21, 33.5, 33.5 and 22.6 m). The girders have 72 in (1829 mm) by 7/16 in (11.1 mm) webs with variable flanges. The floor beams have 40 in (1016 mm) by 3/8 in (9.53 mm) webs with 12 in (12242 mm) by 1.5 in (38.1 mm) top and bottom flanges. Stringers are 24 in (635 mm) by 68 in (1727 mm) wide flange beams.

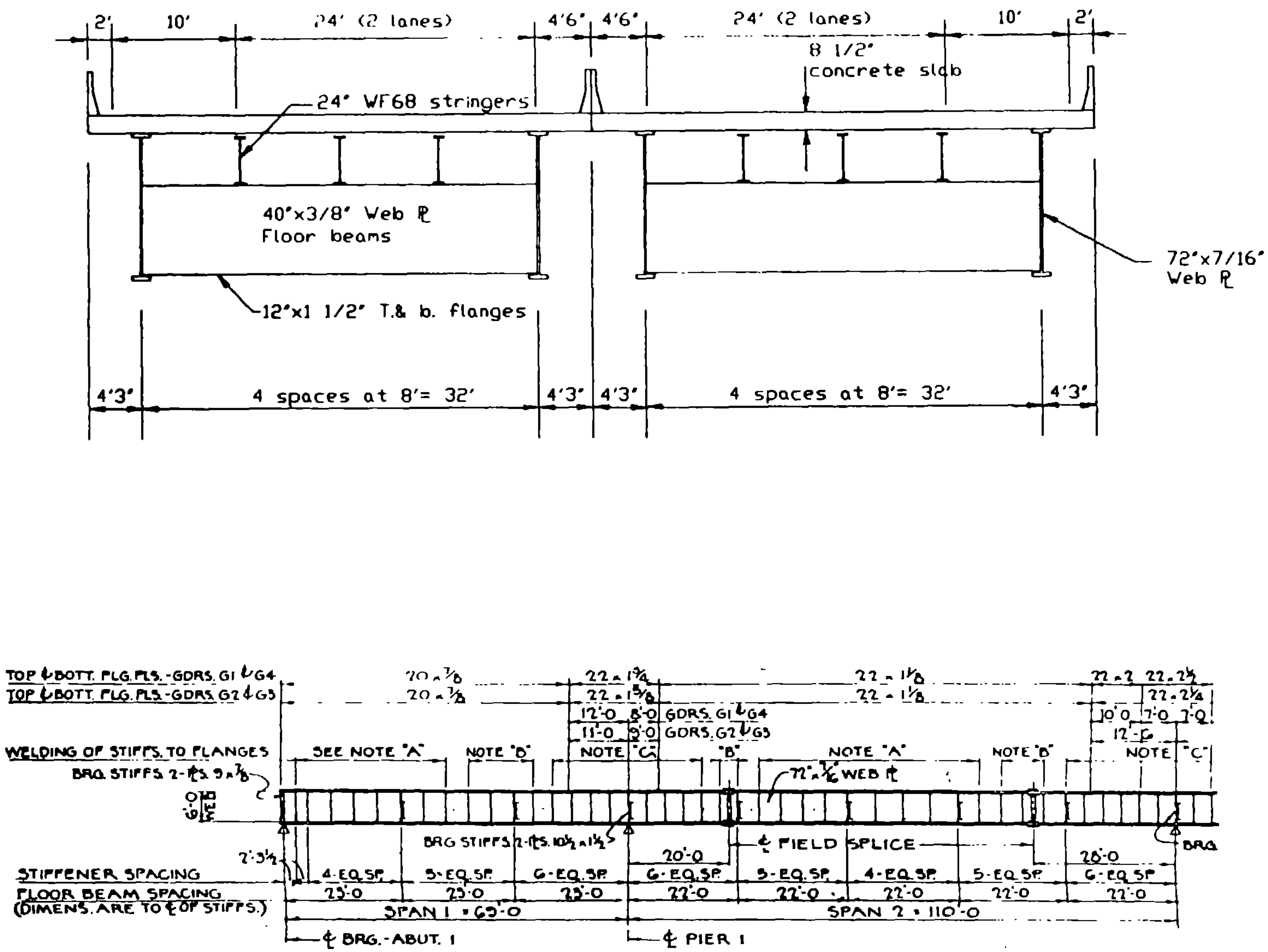


Figure 8. Layout of bridge #6.

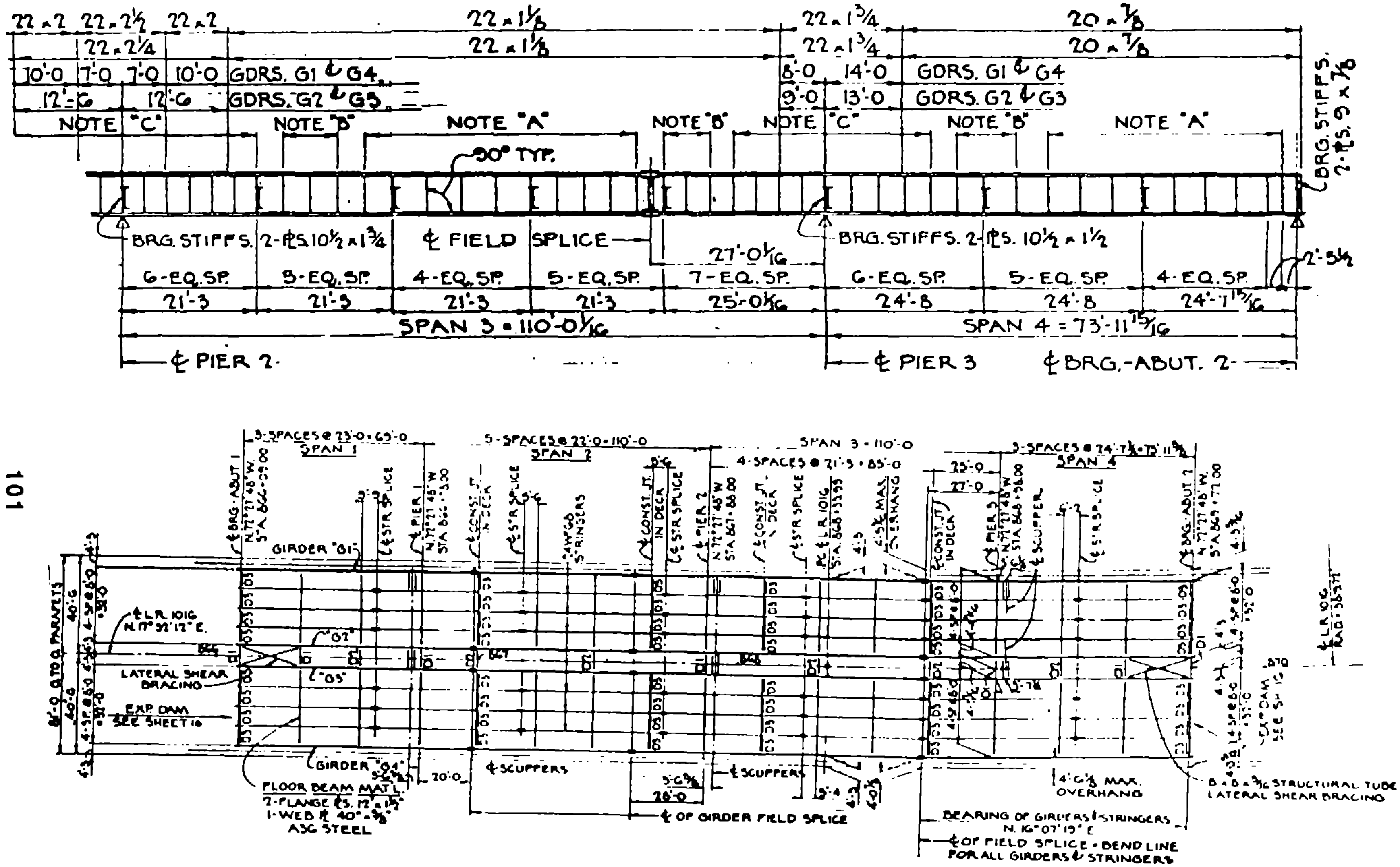


Figure 8. Layout of bridge #6 (continued).

LFD rating for one vehicle produced an inventory rating factor of 0.79 for span 2 (table 39). For two vehicles the rating factor was 0.62 at pier 2. The corresponding operating rating factors are 1.03 and 0.81. The WSD rating factors were 0.71 and 0.38 for one and two vehicles respectively with inventory ratings at span 2 and pier 2 respectively. Operating stress WSD rating factors were 1.26 and 0.98. All these values were for type 5 vehicles. Most bridge agencies prefer to use the inventory ratings for non-redundant bridges such as this one; the operating stresses, however, are presented here for comparison.

Table 39. Rating summary for bridge # 6.

Rating method	WSD	WSD	LFD	LFD
No. of vehicles considered	1	2	1	2
critical vehicle type	7	5	5	5
critical section:	pier 2	pier 2	0.5 L_2	pier 2
description of section:	web 7/16x72	web 7/16x72	web 7/16x72	web 7/16x72
	t. fl. 2 1/4x22	t. fl. 2 1/4x22	t. fl. 1 1/8x22	t. fl. 2 1/4x22
	b. fl. 2 1/4x22	b. fl. 2 1/4x22	b. fl. 1 1/8x22	b. fl. 2 1/4x22
dead load moments (kip-ft)	3666 + 1300	3666 + 1300	1415 + 502	3666 + 1300
special vehicle moment (kip-ft)	2143	3940	2306	3940
HS moment (kip-ft)	2465	2465	1850	2465
section type	noncomposite	noncomposite	noncomposite	noncomposite
section modulus (in^3)	3924	3924	-	-
ultimate moment capacity (kip-ft)	-	-	6447	11772
special vehicle inventory rating	0.70	0.38	0.79	0.62
special vehicle operating rating	1.80	0.98	1.03	0.81
HS inventory rating	(HS-12.2) 0.61	(HS-12.2) 0.61	(HS-19.8) 0.99	(HS-19.8) 0.99
HS operating rating	(HS-31.2) 1.56	(HS-31.2) 1.56	(HS-32.8) 1.64	(HS-32.8) 1.64

Using the HS trucks, rating factors of 0.61 (HS-12.2) are obtained for inventory stresses and 1.56 (31.2) for operating stresses. This indicates that this bridge is deficient according to the WSD inventory criteria under current loading. These factors drop to 0.38 and 0.61 for two type 5 trucks in each lane. The LFD method produces rating factors equal to 0.62 and 0.81 for two type 5 vehicles. This bridge is deficient and will not be able to sustain vehicles satisfying the proposed truck weight formula without strengthening. This two-lane girder bridge shows very high dead to live load ratios which produced high safety index values (above 4.6) despite the low deterministic ratings obtained for both the current and the projected loading conditions (see table 40).

Table 40. Summary of safety index calculation for bridge # 6.

Variable	mean	COV
R (kip-ft)	12125	12%
D (kip-ft)	4966	9%
a (kip-ft/kip)	9.5	-
m	0.97	6%
H	2.91	7%
i	1.2	8%
g	1.40	8%
Gr	1.15	10%
$W_{.95}$ (current - kips)	75	10%
$W_{.95}$ (projected - kips)	101	10%
current safety index	5.00	-
projected safety index	4.60	-

7. Lackawanna Bridge

The bridge shown in figure 9, is located on Pennsylvania Turnpike N.E. extension Lackawanna river Bridge and is owned by the Pennsylvania Turnpike Commission. This is a five-span continuous, riveted steel two-girder bridge with spans of 135, 155, 170, 167, 123 ft (41.1, 47.2, 51.8, 50.9, 37.5 m). Pin and link cantilever spans are found in spans 1 and 5. Girders are haunched with depths of 13 ft (4 m) maximum. The skew angle is 55^0. This bridge is currently being retrofitted to either remove or replace the fracture critical pin-link detail. For these calculations, the dead load forces are obtained by analyzing the existing bridge with pin and link. The live load forces are obtained by analyzing the bridge pin and link removed.

The WSD and LFD operating and inventory ratings given in table 41 were calculated for all critical sections of the structure. For this bridge, the AASHTO LFD procedure allows for an extra capacity equal to 14 percent of the ultimate moment capacity. This produced an actual maximum moment capacity equal to 15,118 kip-ft (20500 kN-m) at the critical section in span 1. The LFD procedure produced an inventory rating factor of 0.86 for two vehicles of type 5. The corresponding operating rating is 1.44. These are compared to the WSD inventory rating factor of 0.67 and the WSD operating rating factor of 1.42. WSD rating for HS loading produced a factor of 0.88 (HS-17.6) for inventory stresses and 1.88 (HS-37.6) for operating stresses. The LFD HS ratings are 1.19 and 1.98 for inventory and operating stresses. This means that the bridge is overstressed when rated by WSD or LFD inventory stress criteria. If operating ratings are used, the bridge is considered safe for both current HS loadings and for loads corresponding to the proposed truck weight limit formula. The safety index calculations show that this four-lane bridge is relatively unsafe under current loading conditions: The safety index of 2.27 is less than the minimum acceptable value of 2.5. The bridge however, will become deficient if the proposed truck weight formula is adopted. In the latter case the safety index obtained is 1.29 indicating that the probability of failure is on the order of 10 percent. Since the bridge has only two girders, a failure at any point in either girder means a high risk of total collapse. This bridge will need immediate rehabilitation if an increase in the truck weight formula is contemplated.

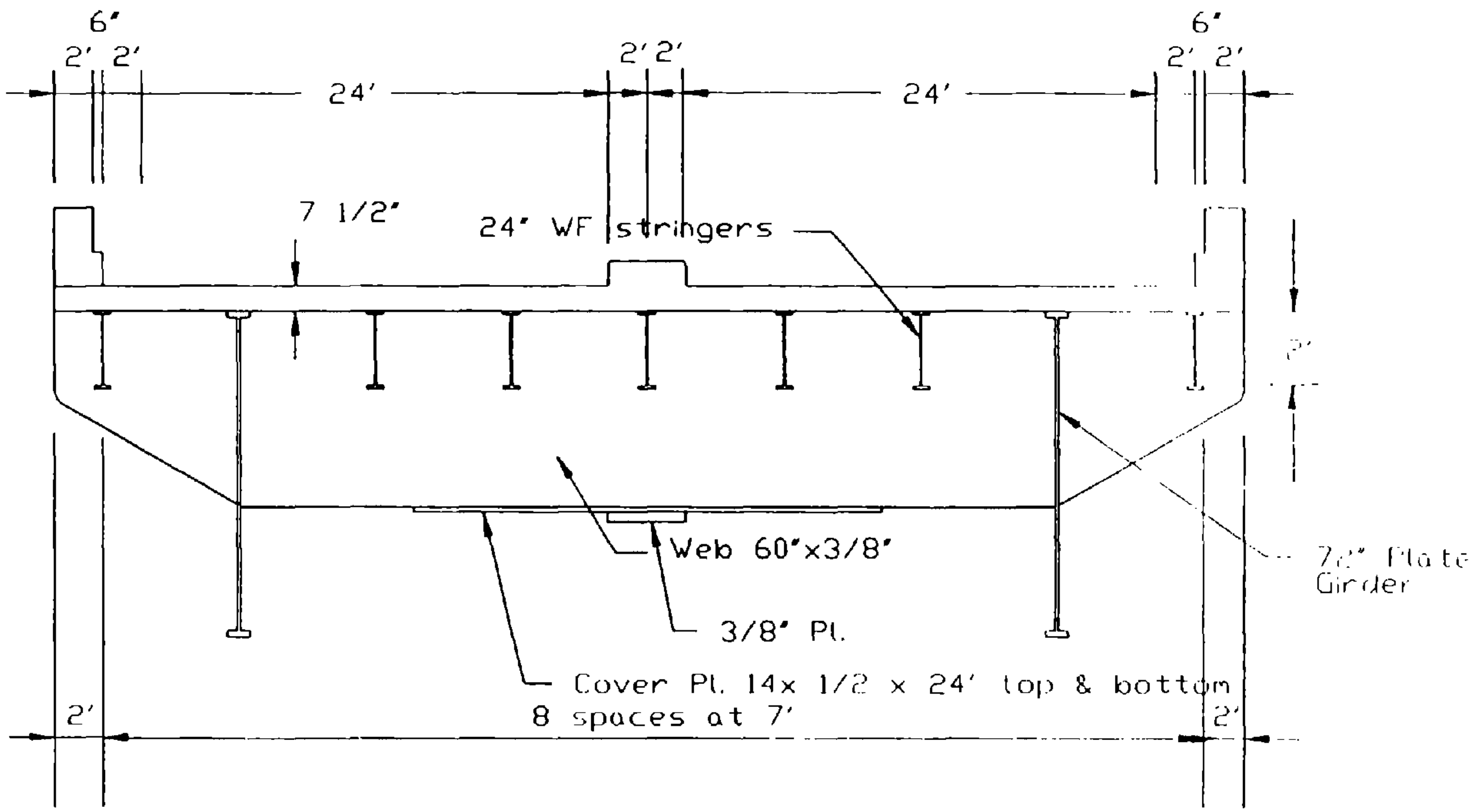


Figure 9. Layout of bridge #7.

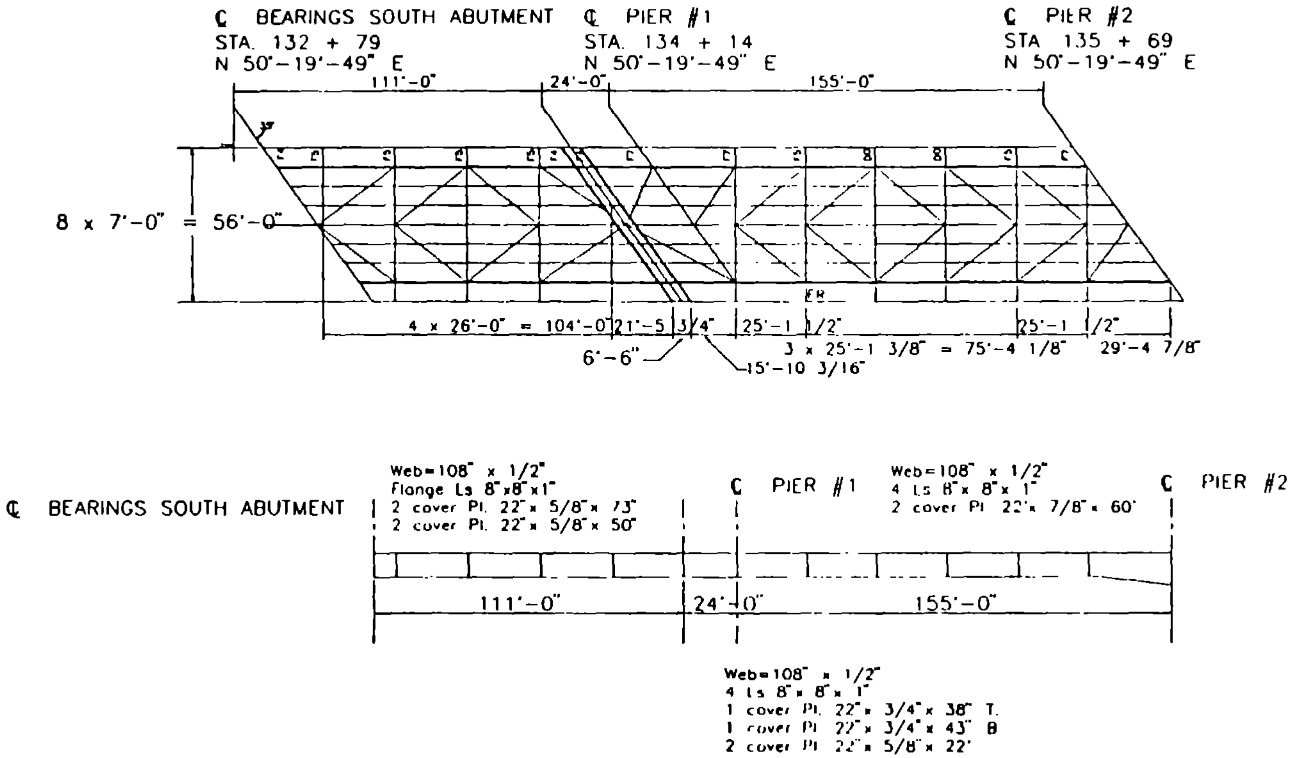


Figure 9. Layout of bridge #7 (continued).

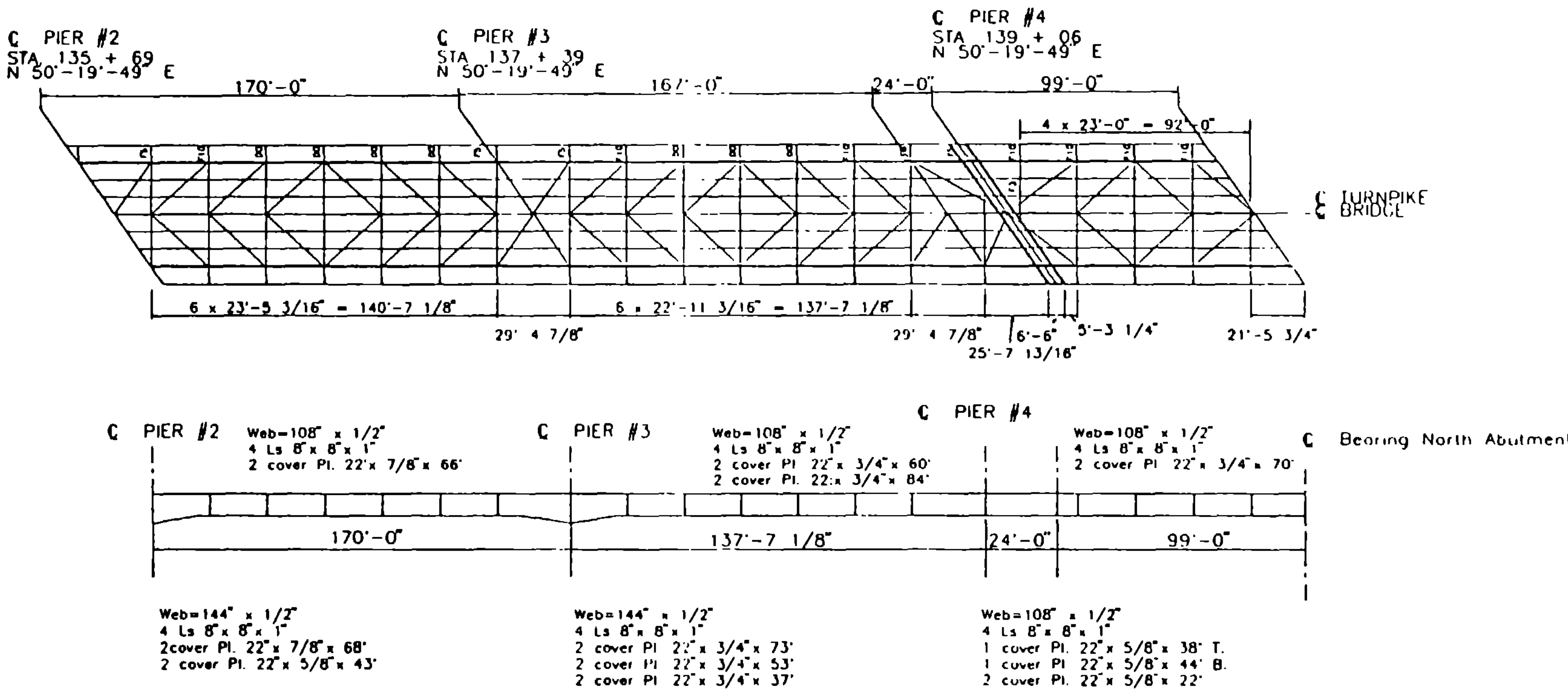


Figure 9. Layout of bridge #7 (continued).

Table 41. Rating summary for bridge # 7.

Rating method	WSD	WSD	LFD	LFD
No. of vehicles considered	1	2	1	2
critical vehicle type	5	4	5	4
critical section:	0.2 L_1	0.2 L_1	0.2 L_1	0.2 L_1
description of section:	web 1/2x108	web 1/2x108	web 1/2x108	web 1/2x108
	t. fl. 2 L8x8x1	t. fl. 2 L8x8x1	t. fl. 2 L8x8x1	t. fl. 2 L8x8x1
	b. fl. 2 L8x8x1	b. fl. 2 L8x8x1	b. fl. 2 L8x8x1	b. fl. 2 L8x8x1
dead load moments (kip-ft)	5644	5644	5644	5644
special vehicle moment (kip-ft)	4013	4186	4013	4186
HS moment (kip-ft)	3021	3021	3021	3021
section type	noncomposite	noncomposite	noncomposite	noncomposite
section modulus (in^3)	3944	3944	-	-
ultimate moment capacity (kip-ft)	-	-	15118	15118
special vehicle inventory rating	0.67	0.64	0.89	0.86
special vehicle operating rating	1.42	1.36	1.49	1.44
HS inventory rating	(HS-17.6) 0.88	(HS-17.6) 0.88	(HS-23.8) 1.19	(HS-23.8) 1.19
HS operating rating	(HS-37.6) 1.88	(HS-37.6) 1.88	(HS-39.6) 1.98	(HS-39.6) 1.98

Table 42. Summary of safety index calculation for bridge # 7.

Variable	mean	COV
R (kip-ft)	15571	12%
D (kip-ft)	5644	9%
a (kip-ft/kip)	18.2	-
m	1.00	7%
H	2.87	7%
i	1.2	8%
g	1.9	8%
Gr	1.15	10%
$W_{.95}$ (current - kips)	75	10%
$W_{.95}$ (projected - kips)	101	10%
current safety index	2.27	-
projected safety index	1.29	-

PRESTRESSED CONCRETE MULTI-BEAM BRIDGES

8. RTE 64 over RT 264

This bridge owned by Virginia Department of Highways and Transportation is located on Route 64 over Route 264 (WBL) in the city of Norfolk. This is a four-span prestressed concrete I beam bridge with spans of 34, 71, 71 and 42 ft (10.4, 21.6, 21.6 and 12.8 m). The beams are 45-in (1143 mm) AASHTO type III with a 7 in (178 mm) deck and 1.25 (31.8 mm) latex concrete overlay. The skew angle is 6^0 14'. Figure 8 gives a cross section and plans of the bridge.

Table 43. Rating summary for bridge # 8.

Rating method	WSD	LFD
No. of vehicles considered	1	1
critical vehicle type	5	5
critical section:	0.5 L_2	0.5 L_2
description of section:	AASHTO type III	AASHTO type III
	36 strands	36 strands
dead load moments (kip-ft)	816 + 41	816 + 41
prestressing moment (kip-ft)	800	800
special vehicle moment (kip-ft)	676	676
HS moment (kip-ft)	611	611
section type	composite	composite
section modulus (in^3)	10263	-
ultimate moment capacity (kip-ft)	-	4820
Special vehicle inventory rating	1.93	2.53
Special vehicle operating rating	4.08	4.22
HS inventory rating	(HS-42.8) 2.14	(HS-56.0) 2.80
HS operating rating	(HS-90.4) 4.52	(HS-93.4) 4.67

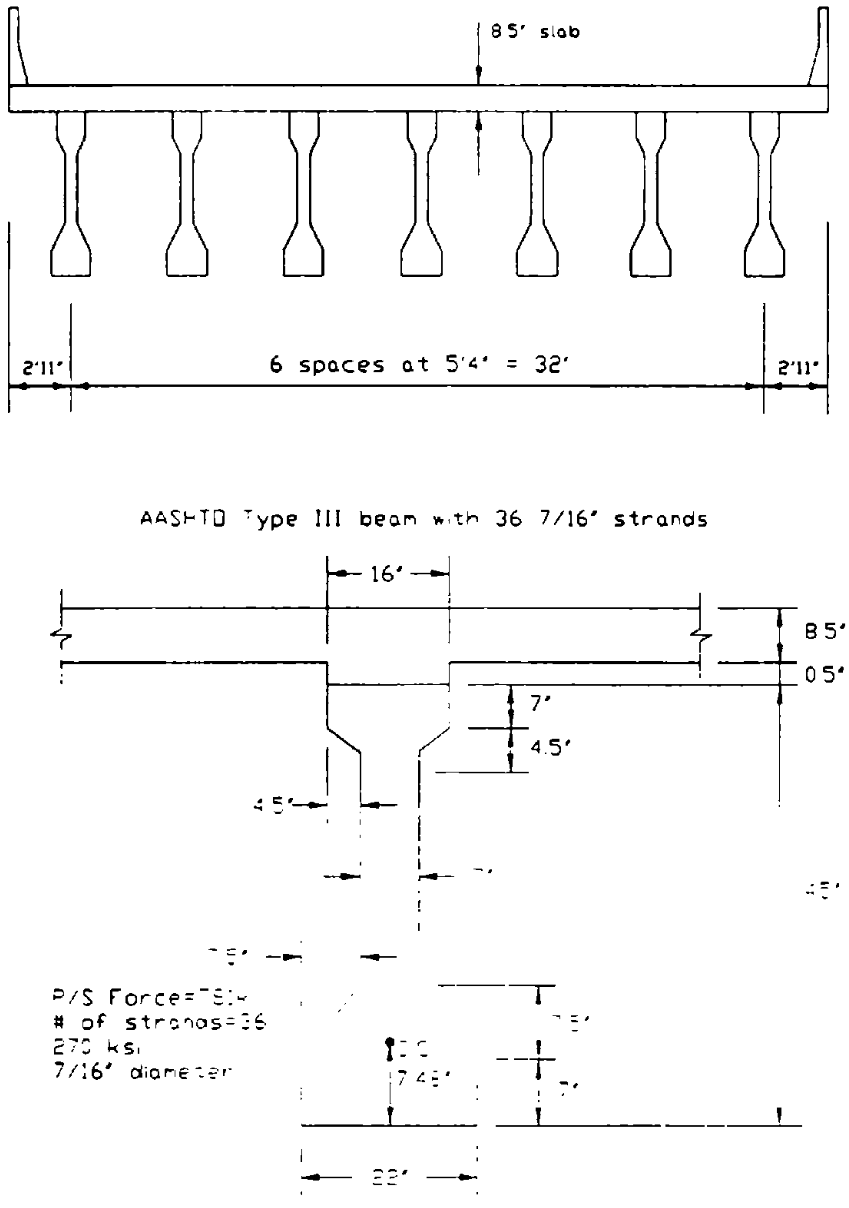


Figure 10. Layout of bridge #8.

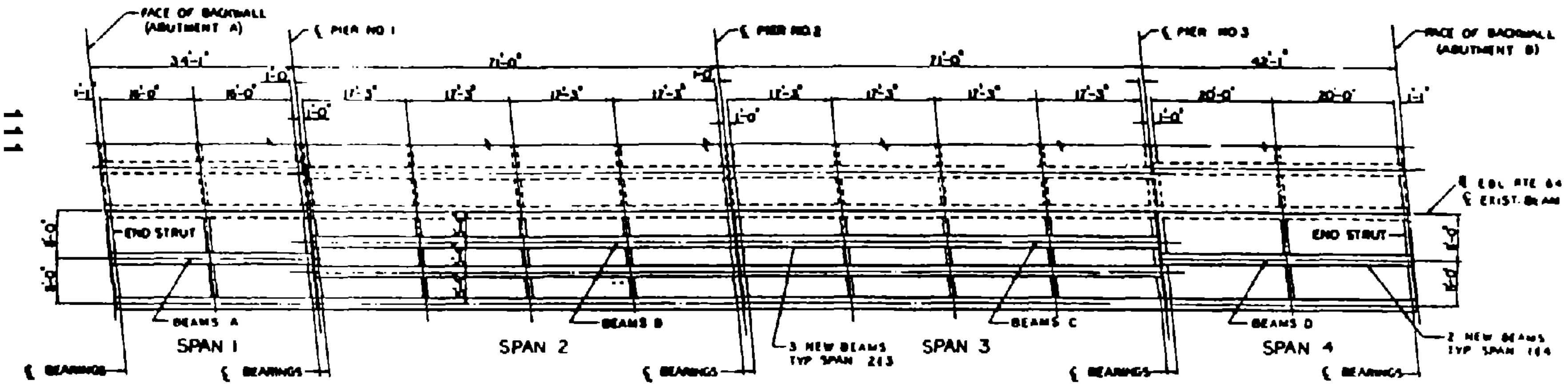


Figure 10. Layout of bridge #8 (continued).

Table 43 gives a summary of the bridge properties and ratings. The WSD operating and inventory ratings were calculated for all critical sections of the bridge for the seven typical vehicles given in figure 2. Since the spans are relatively short and simply supported, the ratings obtained using two vehicles in one lane are the same as those obtained for one vehicle. Vehicle 5 produced a WSD rating factor of 1.93 for inventory and 4.08 for operating. The corresponding LFD ratings were 2.53 and 4.22 for the two stress levels. Ratings for HS loadings were 2.14 (HS-42.28) and 4.52 (HS-90.4) using WSD and 2.80 (HS-56.0) and 4.67 (HS-49.33) using LFD. These numbers indicate that this bridge is overdesigned according to current AASHTO criteria and will be able to sustain the additional loads proposed in this study whether the LFD or WSD criteria are used for both the inventory and the operating stress levels. Confirming this observation are the results of the safety index calculations which produced extremely high safety index values under both the current and the projected loading conditions.

Table 44. Summary of safety index calculation for bridge # 8.

Variable	Positive bending	
	mean	COV
R (kip-ft)	5543	8%
D (kip-ft)	857	9%
a (kip-ft/kip)	10.4	-
m	0.94	8%
H	2.77	7%
i	1.2	8%
g	0.70	8%
Gr	1.15	10%
$W_{.95}$ (current - kips)	75	10%
$W_{.95}$ (projected - kips)	101	10%
current safety index	6.82	-
projected safety index	5.55	-

9. I 279 over Clever Road

This bridge on I 279 (L.R. 1016-10) over Clever Road is owned by the Commonwealth of Pennsylvania. This is a dual three-span bridge with simple spans that are 53, 110, and 95 ft (16.2, 33.5, 29 m) southbound and 47, 102 and 68 ft (14.3, 31.1, 20.7 m) northbound. The beams are keystone type 24/42 and 24/60 (similar to type V AASHTO beams) with an 8 in (203 mm) reinforced concrete deck slab. The skew angle is 50^0. The layout of this bridge is given in figure 11.

Table 45. Rating summary for bridge # 9.

Rating method	WSD	WSD	LFD	LFD
No. of vehicles considered	1	2	1	2
critical vehicle type	5	5	5	5
critical section:	midspan of S.B. span 3	midspan of S.B. span 3	midspan of S.B. span 3	midspan of S.B. span 3
dead load moments (kip-ft)	1887 + 188	1887 + 188	1887 + 188	1887 + 188
prestressing moment (kip-ft)	1248	1248	1248	1248
special vehicle moment (kip-ft)	1536	1577	1536	1577
HS moment (kip-ft)	1191	1191	1191	1191
section type	composite	composite	composite	composite
section modulus (in^3)	3563	3563	-	-
ultimate moment capacity (kip-ft)	-	-	8485	8485
special vehicle inventory rating	0.66	0.63	1.73	1.69
special vehicle operating rating	2.80	2.67	2.90	2.82
HS inventory rating	(HS-17.0) 0.85	(HS-17.0) 0.85	(HS-44.6) 2.23	(HS-44.6) 2.23
HS operating rating	(HS-72.2) 3.61	(HS-72.2) 3.61	(HS-74.4) 3.72	(HS-74.4) 3.72

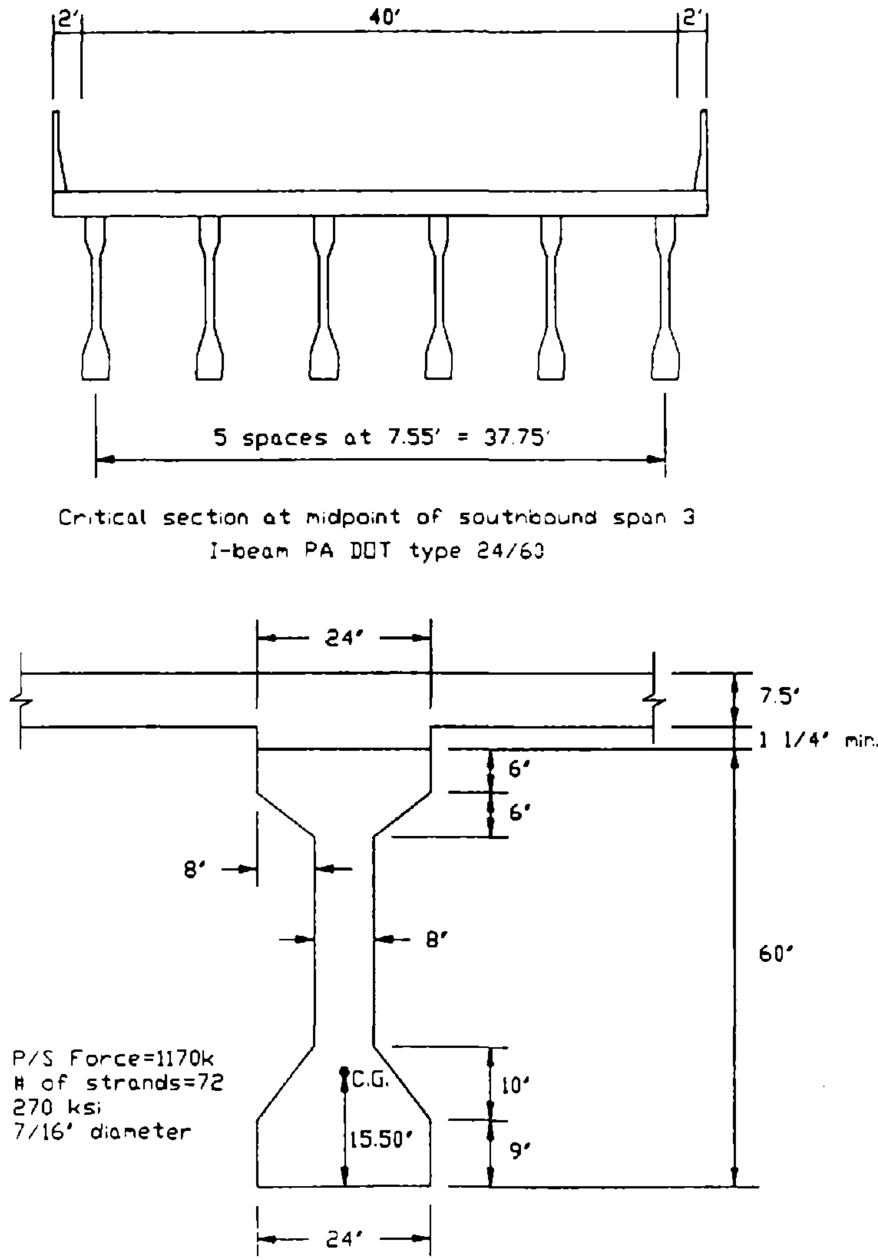


Figure 11. Layout of bridge #9.

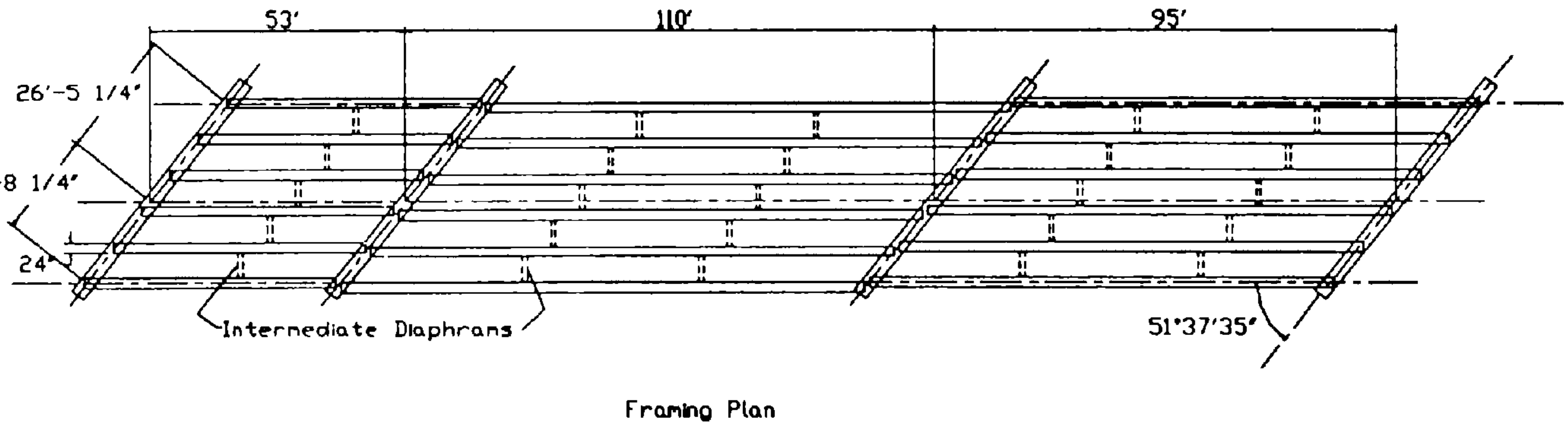


Figure 11. Layout of bridge #9 (continued).

The WSD ratings shown in table 45 produced inventory rating factors of 0.66 and 0.63 for one and two vehicles respectively on span 3 of the south bound bridge. The corresponding operating rating factors are 2.80 and 2.67. The LFD inventory ratings produced values quite different from the WSD approach; the rating factors being 1.73 and 1.69 for one and two vehicles respectively. The difference is due to the use of the ultimate moment capacity in LFD which is more realistic for concrete structures than the WSD approach. The ratings for HS loadings also show that this bridge will be overloaded according to WSD inventory stress criteria. With LFD, however, the bridge is very safe under HS loading. Even though this bridge seems deficient under a WSD inventory rating, the LFD rating shows that it will easily sustain HS trucks or those representing the proposed truck weight formula. Here again, the safety index calculations indicate very high safety levels for both the current and projected loading conditions (see table 46).

Table 46. Summary of safety index calculation for bridge # 9.

Variable	Positive bending	
	mean	COV
R (kip-ft)	9758	8%
D (kip-ft)	2075	9%
a (kip-ft/kip)	11	-
m	0.95	9%
H	2.77	7%
i	1.2	8%
g	0.70	8%
Gr	1.15	10%
$W_{.95}$ (current - kips)	75	10%
$W_{.95}$ (projected - kips)	101	10%
current safety index	8.43	-
projected safety index	7.23	-

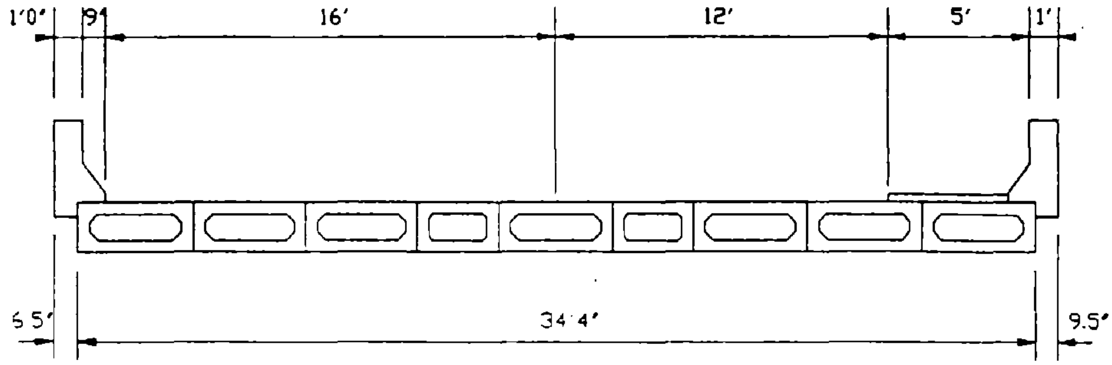
10. Mt. Pleasant St. over Jacks Run

Owned by the City of Greensburg, PA, the bridge is on Mt. Pleasant Street. This is a single span prestressed box beam bridge with a span length equal to 36 ft (11 m) and a skew angle of 52^0. The design follows PA DOT low-cost bridge standards. Figure 10 gives more information about the cross section and the layout of this bridge.

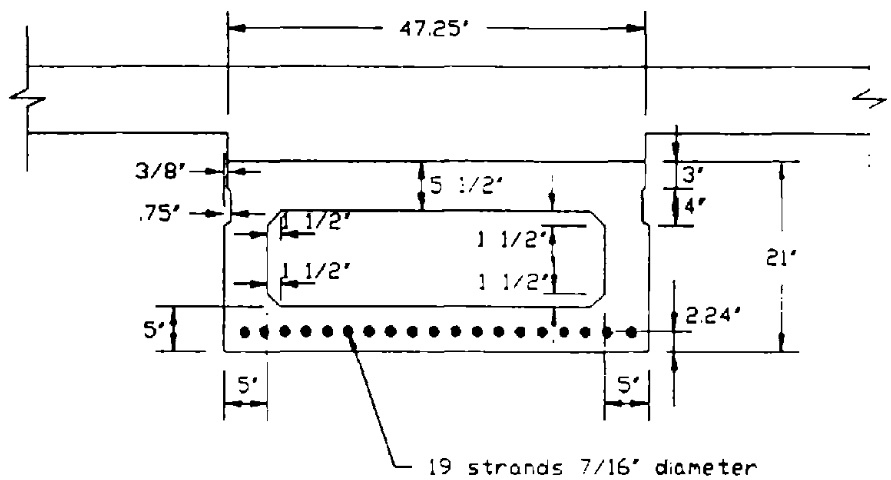
Table 47. Rating summary for bridge # 10.

Rating method	WSD	LFD
No. of vehicles considered	1	1
critical vehicle type	2	2
critical section:	midspan	midspan
description of section:	Box beam 21x48 and 5" thick	
dead load moments (kip-ft)	126 + 34	126 + 34
prestressing moment (kip-ft)	230	230
special vehicle moment (kip-ft)	167	167
HS moment (kip-ft)	179	179
section type	composite	composite
section modulus (in^3)	3173	-
ultimate moment capacity (kip-ft)	-	802
special vehicle inventory rating	1.61	1.64
special vehicle operating rating	2.65	2.74
HS inventory rating	(HS-30.0) 1.50	(HS-32.4) 1.52
HS operating rating	(HS-49.4) 2.47	(HS-51.0) 2.55

WSD operating and inventory ratings were performed for all critical sections of the bridge for the seven typical vehicles given in figure 2 of chapter two. As shown in table 47, vehicle type 2 produced a rating factor of 1.61 for inventory and 2.65 for operating



Critical box beam section is 48'x21'
5' thickness except on top 5 1/2'
19 strands 7/16' diameter
at c.g. 2.24' from bottom
P/S Force=412k



BOX BEAM - RECTANGULAR VOID

Figure 12. Layout of bridge #10.

rating. LFD rating produced factors of 1.64 and 2.74 for inventory and operating stress levels. The ratings obtained under the proposed vehicles are compared to HS ratings equal to 1.50 and 2.47 using the WSD inventory and operating rating criteria and 1.52 and 2.55 for the LFD inventory and operating criteria. These values indicate that this bridge will be safe under the proposed trucks. The safety index calculations confirm this observation by producing safety indices above 3.5 for both current and projected loading conditions. Since this is a short-span bridge the live load is governed by the single unit truck therefore the $W_{.95}$ used are those for the single unit trucks and are associated with a C.O.V. of 15 percent. Table 48 gives the input data and the results of the safety index calculations.

Table 48. Summary of safety index calculation for bridge # 10.

Variable	Positive bending	
	mean	COV
R (kip-ft)	922	8%
D (kip-ft)	160	9%
a (kip-ft/kip)	7.6	-
m	0.93	12%
H	2.69	7%
i	1.2	8%
g	0.35	8%
Gr	1.15	10%
$W_{.95}$ (current - kips)	47	15%
$W_{.95}$ (projected - kips)	58	15%
current safety index	4.74	-
projected safety index	3.93	-

REINFORCED CONCRETE TEE-BEAM BRIDGES

11. Rt. 49 at Rt. 276

Owned by South Carolina, the bridge shown in figure 13, is on the underpass Rt. 49 at U.S. Rt. 276 in Laurens County. The bridge has four simply supported R/C tee beam spans of 51, 56, 56 and 51 ft (15.5, 17, 17 and 15.5 m) with a skew of 12°. The beams are 6.5 ft (2 m) on centers with a cantilever of 2 ft, 4.5 in (0.72 m) from the face of the exterior beam to the edge of the Tee.

Table 49. Rating summary for bridge # 11.

Rating method	WSD	LFD
No. of vehicles considered	1	1
critical vehicle type	5	5
critical section:	midspan of 56 ft span	midspan of 56 ft span
description of section:	3.75 ft T beam	3.75 ft T beam
area of steel:	18.72 in^2	18.72 in^2
dead load moment (kip-ft)	612	612
special vehicle moment (kip-ft)	559	559
HS moment (kip-ft)	548	548
section type	composite	composite
moment capacity (kip-ft)	1090	2014
special vehicle inventory rating	0.86	1.00
special vehicle operating rating	1.64	1.68
HS inventory rating	(HS-17.4) 0.87	(HS-20.4) 1.02
HS operating rating	(HS-33.3) 1.67	(HS-34.2) 1.71

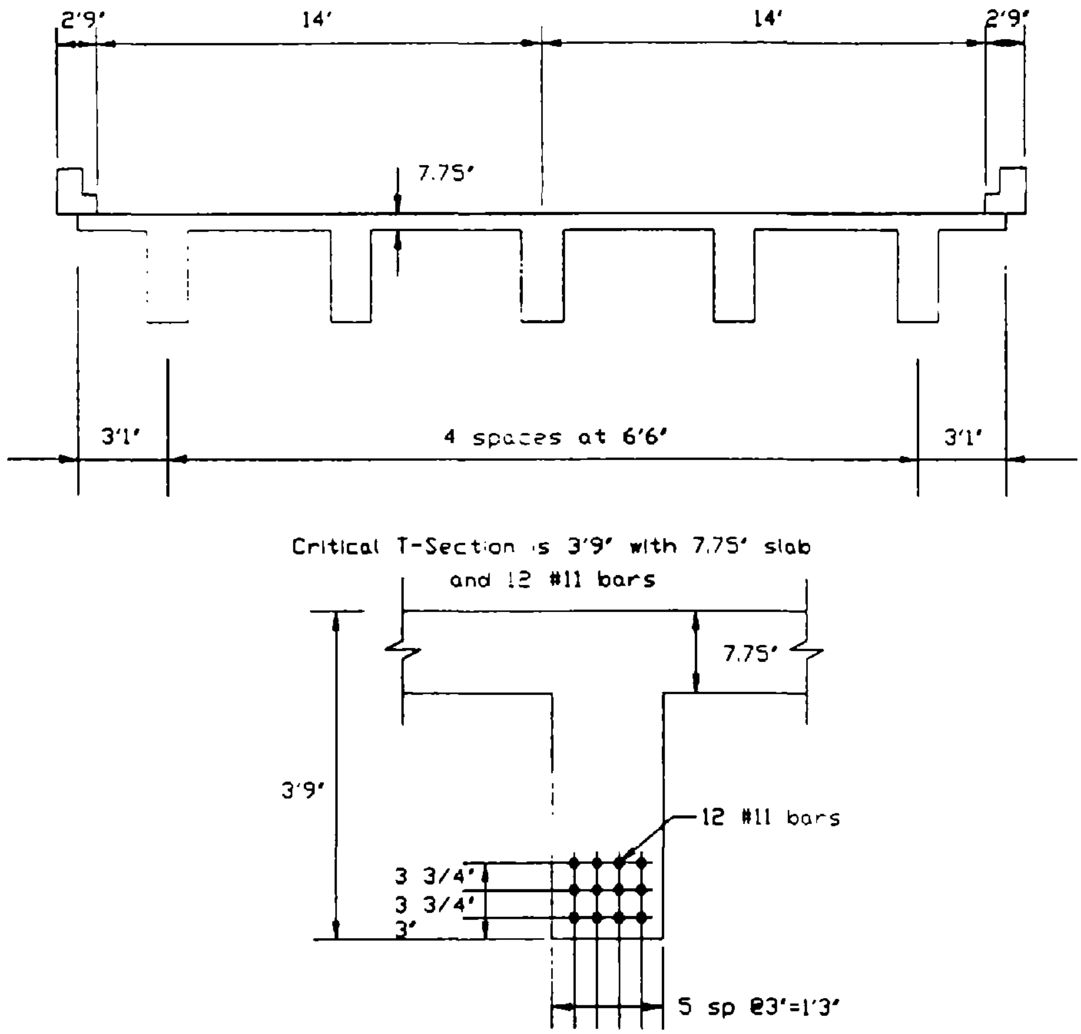
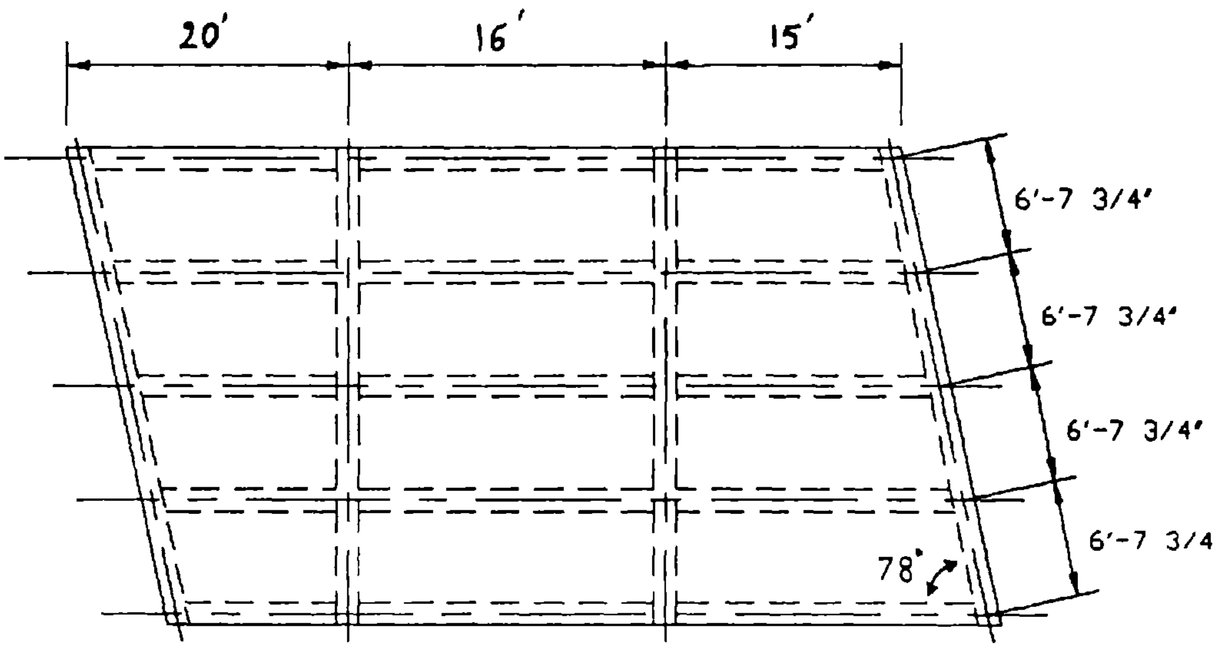


Figure 13. Layout of bridge #11.



c. Framing Plan

Figure 13. Layout of bridge #11 (continued).

WSD and LFD operating and inventory rating factors were calculated for all critical sections of the bridge for the seven typical vehicles. The results given in table 49 indicate that vehicle type 5 produced rating factors of 1.00 for LFD inventory and 1.68 for LFD operating for both one and two vehicles in one lane. A WSD rating produced factors of 0.86 and 1.64 for vehicle 5 for inventory and operating rating criteria respectively. For the HS truck, the rating factors obtained were 1.02 (HS-20.4) and 1.71 (HS-34.2) using the LFD criteria and 0.87 (HS-17.4) and 1.67 (HS-33.3) for the WSD criteria. The safety index calculations produced acceptable safety levels for both the current and the projected loadings.

Table 50. Summary of safety index calculation for bridge # 11.

Variable	Positive bending	
	mean	COV
R (kip-ft)	2215	12%
D (kip-ft)	612	9%
a (kip-ft/kip)	12.57	-
m	0.94	6%
H	2.75	10%
i	1.2	8%
g	0.60	5%
Gr	1.15	10%
$W_{.95}$ (current - kips)	47	15%
$W_{.95}$ (projected - kips)	58	15%
current safety index	3.60	-
projected safety index	2.52	-

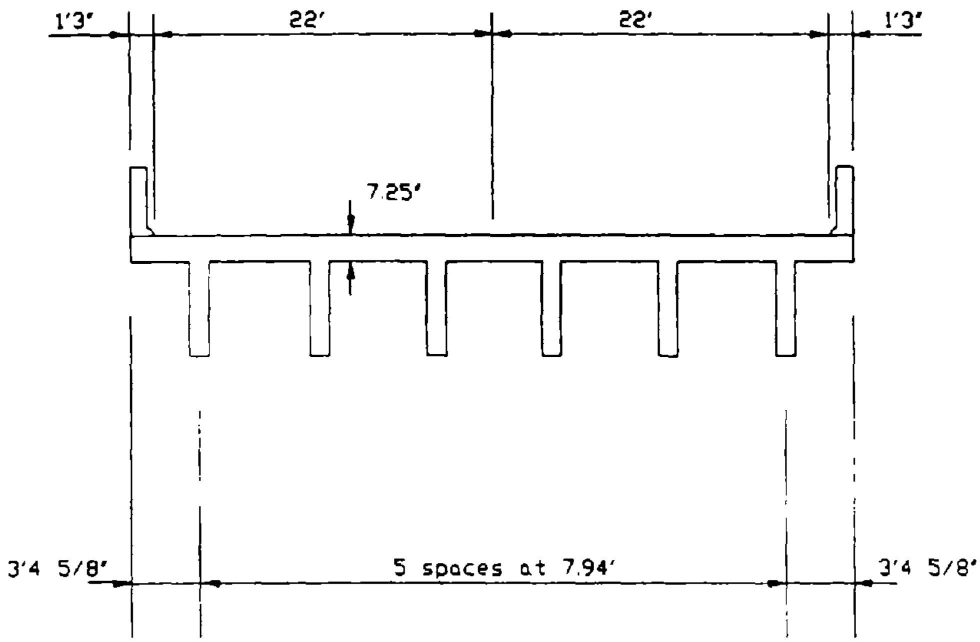
12. Oolenoy River Bridge

This bridge over Oolenoy River in Pickens County is in South Carolina. This is a right bridge with five R/C Tee-beam simple spans each equal to 30 ft (9.1 m). The beams are at 7 ft, 11.25 in (2.4 m) on centers, and the deck cantilevers 3 ft, 4 5/8 in (1 m) beyond the face of the exterior beam (see figure 14).

Table 51. Rating summary for bridge # 12.

Rating method	WSD	LFD
No. of vehicles considered	1	1
critical vehicle type:	1	1
critical section:	midspan	midspan
description of section:	3.125 ft T beam	3.125 ft T beam
area of steel:	9.37 in^2	9.37 in^2
dead load moment (kip-ft)	168	168
special vehicle moment (kip-ft)	246	246
HS moment (kip-ft)	264	264
section type	composite	composite
moment capacity (kip-ft)	477	899
special vehicle inventory rating	1.26	1.29
special vehicle operating rating	2.03	2.16
HS inventory rating	(HS-24.0) 1.20	(HS-23.8) 1.19
HS operating rating	(HS-38.4) 1.92	(HS-39.6) 1.98

Table 51 indicates that WSD rating produced an inventory rating factor equal to 1.26 and an operating rating factor of 2.03. The LFD rating factors were a little higher at 1.29 and 2.16 for inventory and operating ratings respectively. Ratings for the HS vehicle were lower than those obtained for the proposed vehicles but still above 1.0



Critical T-Section is 3.125' deep with 7.25" slab
and 6 #11 bars

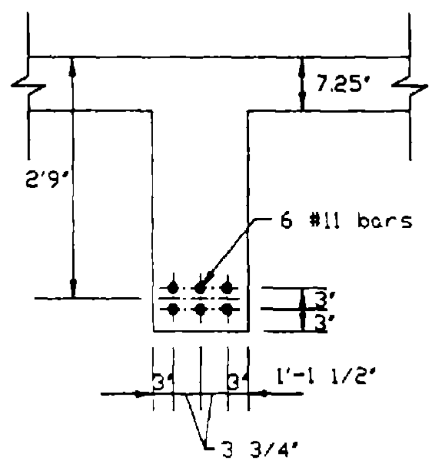


Figure 14. Layout of bridge #12.

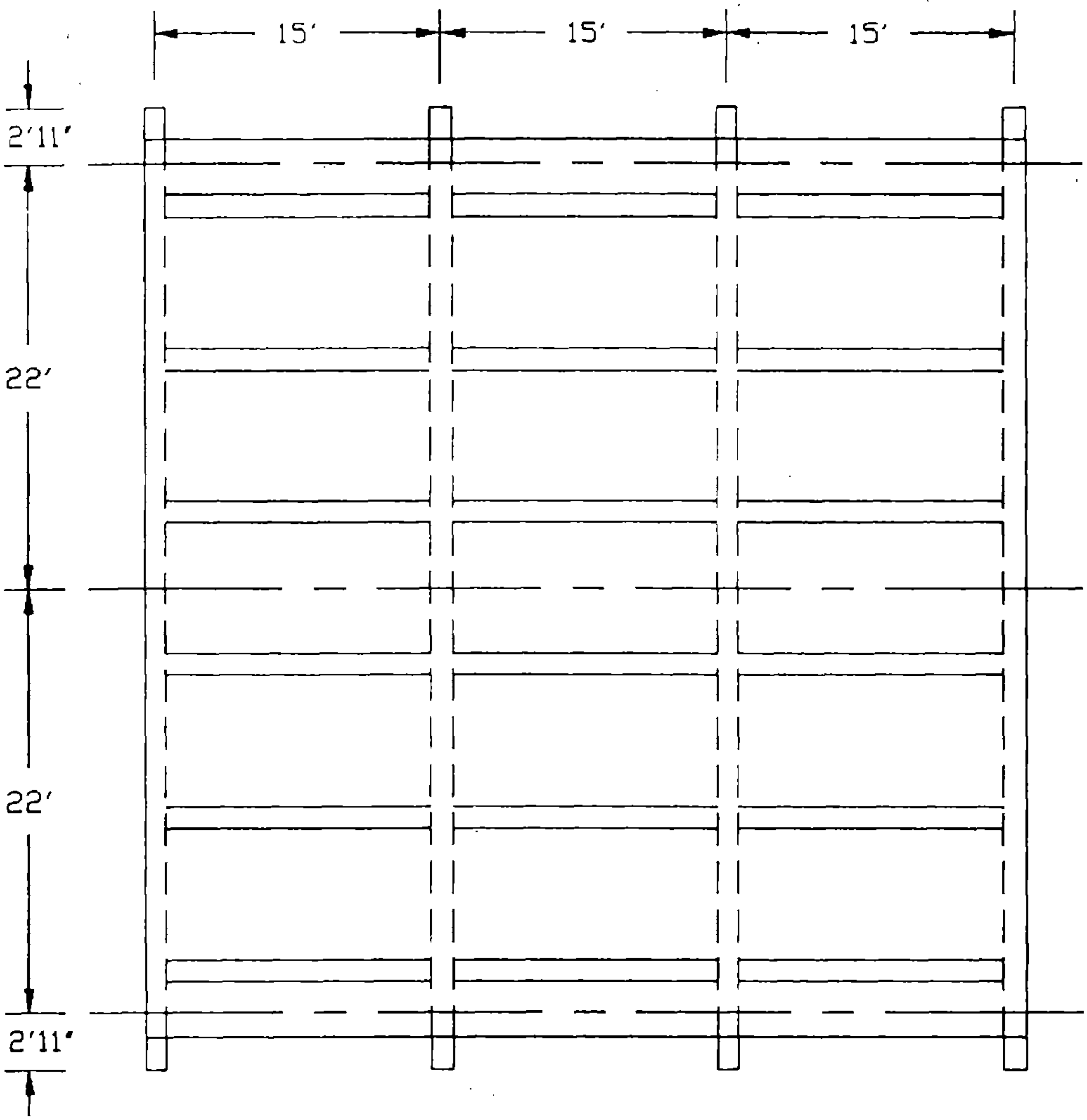


Figure 14. Layout of bridge #12 (continued).

(HS-20) meaning that this bridge is safe under current HS criteria and even safer under the proposed truck weight regulations. The safety index calculations show that the bridge provides acceptable safety levels for current loading conditions. The projected loading, however, produced a safety index value of 2.20 which is slightly lower than the target safety index chosen in this study to define adequate safety (see table 52). Strictly speaking, strengthening of this bridge may be required if the new truck weight formula is to be adopted. If, however, one is to account for the reserve strength of Tee-beam bridges, then no strengthening would be necessary.

Table 52. Summary of safety index calculation for bridge # 12.

Variable	Positive bending	
	mean	COV
R (kip-ft)	989	12%
D (kip-ft)	168	9%
a (kip-ft/kip)	6.07	-
m	0.92	15%
H	2.63	10%
i	1.2	8%
g	0.73	5%
Gr	1.15	10%
$W_{.95}$ (current - kips)	47	15%
$W_{.95}$ (projected - kips)	58	15%
current safety index	2.88	-
projected safety index	2.20	-

RATING SUMMARY

Table 53 gives a summary of the ratings for all the 12 bridges analyzed. One should note the large variation in the rating values for LFD and WSD procedures and inventory or operating stresses. If replacement or upgrading of bridges is based on WSD operating ratings then all the bridges except for bridge #5 will be classified as safe under the proposed new loadings. If LFD operating ratings are used to determine safety, then only bridge #6 would need upgrading. The number of bridges that will be in need of rehabilitation will increase to 8 if WSD inventory stresses are used as safety criteria. A total of 5 bridges will require upgrading if LFD inventory stress ratings are used. From this table we can easily observe that the proposed new truck weight limits will produce a large increase in the number of deficient bridges currently in existence if the inventory stress rating is used in the evaluation procedure. However, very few of the existing bridges will be considered deficient if the operating ratings are used.

Table 53. Summary of ratings for two vehicles in each lane.

Rating method:	LFD		WSD	
Stress level:	inventory	operating	inventory	operating
Bridge No.				
1	1.03	1.72	1.18	2.03
2	0.91	1.52	0.82	1.52
3	1.61	2.69	0.69	1.41
4	0.90	1.51	0.52	0.98
5	0.59	0.98	0.45	0.89
6	0.62	0.81	0.38	0.98
7	0.86	1.44	0.64	1.36
8	2.53	4.22	1.93	4.08
9	1.69	2.82	0.63	2.67
10	1.64	2.74	1.61	2.65
11	1.00	1.68	0.86	1.64
12	1.29	2.16	1.26	2.03

Table 54 gives the summary of the safety index calculations. The results indicate that three bridges will be considered deficient if the new truck weight limits are adopted. The safety index values of bridges No. 4, 7 and 12 fall below the 2.5 target value that is set as the required minimum safety index for bridge members. Bridge #7 in particular will represent very high risks since its safety index falls below 10 percent and is a two-girder bridge where a failure in one girder will likely induce a complete collapse. It is interesting to note that the results of the safety index calculations do not correspond to the results of the deterministic calculations. This would seem to indicate that a review of currently used rating procedures is in order.

Table 54. Summary of safety indices.

Bridge #	current	projected
1	4.43	3.44
2	4.34	3.27
3	4.72	3.84
4	2.74	1.68
5	3.51	2.42
6	5.00	4.60
7	2.82	1.29
8	6.82	5.55
9	8.43	7.23
10	4.74	3.93
11	3.60	2.52
12	2.88	2.20

COSTS OF UPGRADES

Based on the LFD inventory ratings given above and current practice, the steel bridges 2,4,5, 6 and 7 are unsatisfactory and need to be upgraded if the one vehicle ratings are used as criteria. No concrete bridges need to be upgraded if the LFD criteria are used. For WSD ratings, the bridges in need of rehabilitation are steel bridges 2,3,4,5,6 and 7, in addition to concrete bridges no. 9 and 11. The Ps/C bridge no. 9 and and the R/C Tee-beam bridge no. 11 will need complete replacement since no satisfactory cost-efficient upgrading method is available. The total replacement cost is estimated to be on the order of $2,100,000 for bridge no.9 and $730,000 for bridge no. 11 assuming a cost of $75.00 per square foot for construction and $25 per square foot to demolish the existing structure. For steel bridges, an estimated cost of $3.00 per pound of steel was used assuming that adding cover plates is the most reasonable method to upgrade existing steel bridges. If the negative moment capacity is the cause of the deficiencies then, removal of the deck will be necessary for rehabilitation. This will approximately cost about $75 per square foot for replacement and roughly $25 per square foot for deck removal. The costs obtained confirm the observation of Moses that most deficient bridges will need to be replaced due to the high costs of rehabilitation.[43] Table 55 gives a summary of the results obtained. Keep in mind that if WSD operating ratings are used then, only one of the analyzed bridges would need to be rehabilitated.

Table 55. Rehabilitation cost estimates for bridges with two-vehicle loading.

Bridge #	LFD	WSD
2	$155,000	$175,000
3	-	$75,000
4	$570,000	$570,000
5	$2,500,000	$2,500,000
6	$3,000,000	$3,000,000
7	$430,000	$480,000
9	-	$2,100,000
11	-	$730,000

COMPARISON WITH THREE-DIMENSIONAL ANALYSIS

A three-dimensional (3-D) finite element analysis was performed for bridges 1 and 2 to provide more precise results of the effect of using the proposed new truck weight formula. The analysis used BSDI's bridge analysis program. The length of the elements was approximately 10 percent of the span lengths to ensure adequate accuracy and the contributions of the slab were included using 3-D brick elements. Secondary elements such as bracing and diaphragms were also modeled. The analysis assumed four vehicles of the types shown in figure 1 of chapter two placing two vehicles in each lane in two adjacent lanes. The impact was calculated using AASHTO's specifications. Below is a summary of the results obtained for the two bridges.

Bridge 1

For this bridge, the channel diaphragms are defined as plate elements that extend the full depth of the girder with beam elements at the top and bottom. All connections are assumed to be completely rigid. In comparison with the conventional methods of analysis, the finite element approach showed a decrease in the stresses of the interior beams by up to 30 percent for certain cases. This is probably due to the contribution of the slab and diaphragms to load distribution. The stresses in the exterior beams were however higher than predicted. The maximum observed girder stress was 27 ksi (186 000 kPa) on one of the exterior beams. This is just equal to the allowable inventory stress assuming 50 ksi (345 000 kPa) steel. The diaphragms were found to be overstressed (using WSD) by up to 70 percent for certain cases; but these high stresses in the diaphragms might be partially due to modeling errors. In the model, the diaphragms were assumed to be rigidly connected to the stringers; in reality they are connected at the web only, producing partial rigidity and thus lower actual moments than the values that were calculated.

Bridge 2

In this analysis, the diaphragms were a combination of channels and K braces. Compared to the conventional analysis, the stresses in the interior beams obtained using the finite element method were up to 40 percent lower but the stresses in the external beams were up to 20 percent higher. With two adjacent lanes loaded, the maximum observed stress in the external stringer was 31 ksi (214 000 kPa). This is higher than the allowable inventory rating stress but is lower than the operating rating stress assuming 50 ksi (345 000 kPa) steel. The bracing elements were understressed but the channel diaphragms showed high stresses. The same modeling errors described for bridge 1 might have contributed to the high stress values observed.

Summary

The finite element analysis showed that traditional analysis usually overestimates the stresses in the interior beams but can underestimate the stresses in the exterior ones. The stresses observed were still acceptable according to the operating stress criteria using WSD approach. The bracing of multibeam bridges showed low stress levels but beam diaphragms were overstressed. The overstressing of the diaphragms might have been due to modeling errors and to the assumption of the rigidity of the connections and the stresses due to dead loads.

CONCLUSIONS

This chapter presented a summary of the results of the analysis of 12 typical bridges. The calculations indicated that if WSD operating stress criteria are used for the evaluation of these bridges, none of them will need to be rehabilitated. Rehabilitation costs are estimated if LFD and WSD criteria are used with inventory stresses for one vehicle and two vehicle in one lane. The assumption of one vehicle per lane produces three deficient bridges, two steel and one prestressed concrete bridge. The steel bridges can be easily and cheaply upgraded by adding cover plates. The prestressed concrete bridge will have to be completely replaced. If two vehicles are assumed to be in one lane, then many more bridges would need rehabilitation; also, the costs of upgrading would be much higher since most of the steel bridges will need to be upgraded at the support location which will require the removal of the deck. The finite element analysis showed that some secondary members might be overstressed if the proposed truck weight formula is adopted. This should not affect the safety of the bridges since the slab should able to distribute the loads efficiently even after the loss of some of the diaphragms. The finite element analysis however indicated that some of the outside stringers might be overstressed under certain extreme loading conditions when four vehicles cluster on one side of the bridge.

CHAPTER FIVE

FATIGUE ANALYSIS

INTRODUCTION

General Procedures

The purpose of the fatigue analysis in this chapter is to predict the effects of possible changes in truck weight and size regulations on the fatigue behavior of steel bridges. Specifically, the analysis is intended to evaluate the proposed truck weight formula (equation 25) and compare it with the TTI truck-weight (bridge) formula (equation 3).[6,74] Both of these truck-weight formulas define the maximum legal gross weight as a function of the wheelbase (distance between the outside axles), and do not restrict the sizes of trucks.

The TTI actually proposed two formulas: one is intended for highway systems that have bridges designed for either heavy (HS-20) loads or light (H-15) loads, and the other is intended for highway systems that have only bridges designed for heavy (HS-20) loads. Since most bridges on Interstate (National System of Interstate and Defense Highways) and primary highways are designed for heavy loads, the latter formula is used in the comparison.

The Surface Transportation Assistance Act (STAA) of 1982 specified legal sizes and weights of trucks permitted on a national system of highways referred to as the National Network for Trucks.[56,57] As a result, certain types of large trucks that were not previously permitted in many States can now operate on this system. Furthermore, the STAA mandated a study of the feasibility of permitting even larger trucks on this National Network.[66] Therefore, the STAA has significantly affected present truck traffic in the United States and is expected to have a major influence on future truck traffic.

To evaluate the effect of the TTI formula and the proposed truck weight formula on fatigue behavior, it is necessary to know the size and axle configurations of all of the major truck types that are operating now or that might be operating in the foreseeable future. These include (a) the major truck types operating before the STAA, (b) the new truck types permitted by the STAA, and (c) the new truck types being considered in the feasibility study mandated by the STAA. The major truck types operating before the STAA include only those numerous enough to significantly affect fatigue behavior. Most of the new truck types mentioned in (b) and (c) have been operated either with or without special permits in a few States, but are new in the sense that they were not permitted nationwide and were not prevalent enough to affect fatigue behavior.

The TTI and the proposed formulas, and applicable axle load limitations, are applied to the major truck types to determine corresponding practical maximum gross weights (PMGW), which are needed in the fatigue analysis. The PMGW is the maximum gross weight that can be achieved without violating legal axle or formula weights. The present 80-kip (356 kN) gross-weight limit is not applied in conjunction with TTI nor the proposed truck-weight formulas. This 80-kip (356 kN) limit is not required to assure adequate strength for bridges or adequate performance (service life) for pavement.

For the fatigue analysis, it is also necessary to predict how present truck traffic will be affected by changes in weight and size regulations. Specifically, it is necessary to estimate the frequency of occurrence of various truck types before and after a particular regulation change. The pre-STAA traffic consists of six main truck types; data on the weight spectrum, axle configurations, and percentage of the total truck traffic are available for each of these six types.[24,77,40]

As a result of regulation changes, new types will replace a portion of some, or all, of the old types. The resulting new spectrums are predicted by judgment for various scenarios. Several factors are considered in making such predictions: (a) the beneficial increases in PMGW and/or volume capacity resulting from the various regulation changes, (b) the relative costs of modifying highways to accommodate various new truck types, (c) studies of the effects of past changes in truck regulations in Ontario and elsewhere, and (d) recent studies of the effects of proposed regulation changes by various agencies.

Analytical Methods

The new fatigue design and evaluation procedures developed in NCHRP Project 12-28(3) provide a satisfactory means of determining the total or remaining fatigue life for present traffic spectrums.[11] More refined procedures, however, are required to accurately assess the effect of changes in weight and/or size regulations that permit vehicles with substantially different axle configurations than are now used. Such methods were used in a recent study of the effects of alternative truck configurations on bridges.[43,68,69]

To show the effects of replacing a portion of the trucks of a particular pre-STAA type with a particular new type, the relative fatigue damage caused by the old and new types is calculated for both the TTI and the proposed truck-weight formulas. Generally, a new double or triple combination replaces a portion of the present five-axle semitrailers. The calculated relative fatigue damage includes the following effects:

(a) The effect of a change in the gross weight on the magnitude of the fatigue stresses.

(b) The effect of a change in the axle spacings, and percentages of the gross weight carried by these axles, on the magnitude of the fatigue stresses.

(c) The effect of a change in the axle spacings on the equivalent number of stress cycles caused by the passage of the truck across the bridge.

(d) The effect of a change in the permissible gross weight or volume capacity on the number of truck passages required to transport a given amount of freight, and hence on the number of stress cycles caused by hauling a given amount of freight.

The second and third effects depend on (a) the span length of the bridge under consideration, (b) the location of the fatigue detail along the span, and (c) the presence or absence of continuity (simple or continuous spans.)

To provide a more comprehensive assessment of the effects of regulation changes, the relative fatigue damage is also calculated for each new truck traffic scenario. This calculation is made for both the TTI and the proposed truck-weight formulas and includes the effects of all changes in the complete truck spectrum.

Organization of Chapter

Present and future truck traffic is discussed first to provide a basis for the fatigue analysis. In this discussion, the influence of the 1982 Surface Transportation Assistance Act (STAA) is explained, present and proposed truck-weight formulas are given, present and future truck types are described, and future truck traffic scenarios are developed. Next, four fatigue characteristics required for the analysis are determined for each of the truck types being considered: effective weight, stress range ratio, cycles per truck passage, and trips per freight hauled.

The relative fatigue damage caused by various individual trucks and traffic scenarios is then calculated. Next, the actual steel bridges analyzed in chapter 4 are checked for fatigue under some of the assumed traffic scenarios. Finally, general conclusions are presented regarding the fatigue effects of the TTI and the proposed truck-weight formulas used in conjunction with size and axle-load regulations imposed by the STAA or proposed for future consideration.

PRESENT AND FUTURE TRUCK TRAFFIC

1982 Surface Transportation Assistance Act

National Network. The 1982 Surface Transportation Assistance Act (STAA), which became effective on January 6, 1983, mandated a National Network for Trucks and specified the sizes and weights of trucks that must be permitted to use this highway system.[56,57] The National Network includes almost all of the Interstate system plus a substantial length of Federal-aid Primary highways. The STAA also requires the States to permit reasonable access from the National Network to facilities for food, fuel, etc.

Weight Limits. The STAA specifies the maximum legal gross weights to be 20, 34, and 80 kips (89, 151, 356 kN), respectively, on single axles, double tandem axles, and vehicle combinations of five or more axles.[57,74] In addition, the gross vehicle weight (GVW), and all subsets of contiguous axles, may not exceed the gross weight from the original truck-weight (bridge) formula except that two consecutive sets of tandem axles may carry a gross weight of 34 kips (151 kN) each if the distance between the outer axles is 36 ft (11 m) or more.[57,74] No mention is made of triple tandem axles so they are controlled by the other limitations.

Size Limits. The STAA size regulations mandate the use of 48-ft (14.6 m) semitrailers in tractor-semitrailer combinations, and 28-ft (8.5 m) semitrailers or trailers in tractor-semitrailer-trailer or tractor-semitrailer-semitrailer combinations. The specified maximum lengths are for the individual trailers or semitrailers and the overall lengths of the combination units is not specified. A truck consisting of a tractor followed by two 28-ft (8.5 m) trailers or semitrailers will be referred to as a twin to distinguish it from other types of double-trailer vehicles that will be discussed later. The maximum width for all trucks was specified to be 8.5 ft (2.6 m).

Future Limits. The STAA also required a study of the feasibility of using even longer and heavier trucks than are now permitted by this act. Three types were considered in this STAA study: (a) western (Rocky Mountain) doubles, which consist of a 48-ft (14.6 m) semitrailer or trailer followed by a 28-ft (8.5 m) semitrailer or trailer, (b) turnpike doubles, which consist of two 48-ft (14.6 m) semitrailers or trailers, and (c) triples, which consist of three 28-ft (8.5 m) semitrailers or trailers.[66]

For all three types, it was assumed that the present 80-kip (356 kN) GVW limit would be eliminated so that the permissible GVW would depend only on the truck-weight formula and on the sum of the legal axle weights. Elimination of the 80-kip (356 kN) limit would be a very important relaxation of present regulations since it would permit doubles and triples with considerably higher legal gross vehicle weights than are now permitted.

The study estimated the costs to upgrade the Interstate system to accommodate the longer trucks.[66] The main costs are for modifying the interchanges to accommodate the larger turning radii of the longer trucks. These costs are proportional to the turning radius, which depends on the wheelbase of the individual semitrailer or trailer units. Specifically, the average costs per State to upgrade all interchanges were estimated to be as shown in table 56.[66]

Table 56 . Average cost per State to upgrade intrerchanges after the elimination of the 80-kip weight limit.

	Rural	Urban
western double	\$37,000,000	\$57,000,000
turnpike double	\$50,000,000	\$89,000,000
triple	\$32,000,000	\$48,000,000

1 kip = 4.48 kN

The relative costs to upgrade the interchanges for different types of trucks are useful in predicting likely future truck-traffic scenarios.

Comparison with Past Limits. Before the STAA was passed, Federal legislation prohibited trucks exceeding the present weight limits from using Interstate highways, but did not require the States to permit trucks of a particular size or weight to use any portions of their highways.[57,74] Some States had weight limits less than those Federal limits; single axle, tandem axle, and gross vehicle weights of 18, 32, and 73 kips (80, 142, 325 kN) were fairly common. Other States had weight limits, especially single axle limits, exceeding the Federal limits; these State limits were applicable to highways not included in the Interstate system.

All States had overall length limits on tractor-semitrailer combinations, and several had length limits of less than 48-ft (14.6 m) for the semitrailers. As a result, 45 ft (13.7 m) was the most common length for semitrailers in five-axle semitrailer-trailer combinations.[57] Many States prohibited, or severely limited, double combinations that included two semitrailers or trailers. In those States permitting doubles, their overall length was often limited to 65 ft (19.8 m).[78] Triple combinations that included three semitrailers or trailers were permitted in only a few States.[66,88] Almost all States limited the width to 8 ft (2.4 m) instead of 8.5 ft (2.6 m).

Impact of STAA. Changes in the truck traffic composition due to the STAA have been gradual.[78] The 48-ft (14.6 m) length is gradually becoming the standard for five-axle semitrailers. Initially, this added length was achieved by merely extending the overhang beyond the wheelbase previously used for 45-ft (13.7 m) semitrailers, but now a longer wheelbase is generally used with the 48-ft (14.6 m) length.[65]

The use of twins is increasing at a rather slow pace, but twins are expected to eventually replace 10 to 20 percent of the five-axle semitrailers under present regulations and considerably more if the 80-kip (356 kN) GVW limit is eliminated.[78] Under present regulations the main advantages of the twins over the five-axle semitrailers are: (a) greater volume capacity and (b) greater operational flexibility (the twin can be split into two parts for local shipping without reloading). Elimination of the present 80-kip (356 kN) GVW limit would provide considerable additional benefits by permitting substantially higher gross vehicle weights and corresponding payloads. The advantages of the other double and triple units that are being considered are similar; they would be substantially greater if the 80-kip (356 kN) limit were eliminated.

The present STAA size and weight regulations, and the possible relaxation of these regulations to permit other doubles and triples and to eliminate the 80-kip (356 kN) GVW limit, are not expected to result in unusual axle configurations such as developed after the adoption of the Ontario truck-weight (bridge) formula.[48,52] This is because the maximum GVW allowed by the TTI and the proposed formulas increases with the wheelbase, but is independent of the axle configuration; in contrast, the Ontario formula depends on both the wheelbase and axle configuration. [48,52] Instead of unusual configurations, the present STAA regulations, and possible future modifications, are expected to result mainly in standard units that have semitrailers and trailers of the maximum permitted lengths and axles spaced at the maximum distances permitted within these lengths. In addition, it is expected that tridem axles will be used in four-axle single units and six-axle semitrailers.

Tridem axles can be utilized more effectively under the TTI and the proposed formulas than under the present truck-weight (bridge) formula. Because of the 20-kip (89 kN) individual axle limit, the maximum load on a tridem axle cannot exceed 60 kips (267 kN). Minimum wheelbases (outer axle spacing) of 32, 17, and 18.3 ft (9.8, 5.2 and 5.6 m), respectively, are required to reach this limit under the present, TTI, and the proposed truck-weight formulas. The tandem axle limit of 34 kips (151 kN) applies instead of the TTI or the proposed formula limit to any two axles spaced at less than 8 ft (2.4 m).[6] Thus, the tridem axle limit is 51 kips (227 kN) or less for wheelbases less than 16 ft (4.9 m) under these two truck-weight formulas.

Some other interesting innovations also are being tried within the STAA regulations.[65] Perhaps the most important of these is the use of twin-steering axles designed to carry the full 20-kip (89 kN) load permitted by the regulations. In conventional truck designs, enough weight cannot be shifted to the front (steering) axle to reach this allowable 20-kip (89 kN) limit.[86,87] As a result, the practical maximum gross vehicle weight may be reduced as discussed later.

Truck-Weight Formulas

Four alternative truck-weight formulas are presented below; all give the maximum permissible gross weight, W, in kips for a full vehicle or contiguous subset of axles. In all formulas, the wheelbase, B, is defined as the distance in feet between the outer axles of the full vehicles or the subset of axles. These formulas are intended to be applied to all contiguous subsets of axles. These formulas already discussed in chapter one are repeated below.

Present Formula. The original truck-weight formula, which first related the permissible weight for a vehicle to its wheelbase, is specified in the STAA.[67,74,80] This formula is often called the bridge formula or Formula B. It is:

$$W = .5\left(\frac{NB}{N-1} + 12N + 36\right) \qquad (26)$$

in which N is the number of axles in the vehicle or subset. This formula has four main disadvantages: (a) it is based on arbitrary stress limits, (b) it yields unreasonably high permissible weights if it is applied without the 80-kip (356 kN) cap, (c) it includes N in an irrational way, and (d) it is considered by some to be too complicated.[6,74]

TTI Formulas. Two truck-weight formulas have been proposed to correct the last three of these disadvantages.[6,74] The new formulas, however, are based on the same arbitrary stress limits as the present formula. The first TTI formula is intended for highway systems that have bridges designed for both heavy (HS-20) and light (H15) loads. It is:

$$W = B + 34 \qquad \text{for } B<56$$
$$W = .5\,B + 62 \qquad \text{for } B>56 \tag{27}$$

The second formula is intended for highway systems that have only bridges designed for heavy (HS-20) loads. It is:

$$W = B + 34 \qquad \text{for } B<8$$
$$W = 2B + 26 \qquad \text{for } 8<B<24$$
$$W = .5B + 62 \qquad \text{for } B>24 \tag{28}$$

For B<56 ft (17.1 m), the second formula gives higher permissible loads that are appropriate for HS-20 bridges. Since the bridges on the National Network are generally designed for HS-20 or higher loads, this second formula is considered appropriate for the present study.

The Proposed Formula. The formula proposed in this study (also referred to as the City College of New York (CCNY) formula) was developed to avoid the arbitrary stress limits that were used as the basis for earlier formulas. Instead, it is based on a rational reliability analysis. It is given as:

$$W = 1.64B + 30 \qquad \text{for } B<50$$
$$W = .8B + 72 \qquad \text{for } B>50 \tag{29}$$

Pre-STAA Truck Types

Basic Types. Extensive data have been obtained on the types of trucks using the highway system in the United States, and on the characteristics of these types. The data were obtained from nationwide truck surveys and weight-in-motion studies on Interstate, primary, and secondary highways, and reflect the characteristics of pre-STAA truck traffic.[63,64,24,77,40,87]

The main truck types that were prevalent enough to affect fatigue behavior are described in tables 57 to 59. There are three main configurations: single, semitrailer (semi), and twin. There are a total of six variations of these three configurations with different numbers of axles. In table 57, the standard FHWA code (3S2, 2S1-2, etc.) is used to identify axle configurations. Since this code does not indicate the size or basic type (western double, triple, etc.), a new (CCNY) code is also used. This code indicates the basic type and total number of axles. For example, WD5 is a five-axle western double and TD5 is a five-axle turnpike double. Both have an FHWA code of 2S1-2. For truck types that have both pre- and post-STAA versions, an A or B is included at the end of the CCNY code to distinguish between the two. A means after STAA and B means before STAA. Various pre and post-STAA combination trucks are illustrated in figure 15.

About 4.5 percent of the five-axle semitrailers observed in the nationwide WIM study had spread tandem axles.[40] The maximum legal weight for such axles is 40 kips (178 kN) compared with 34 kips (151 kN) for conventional tandem axles. The tables do not include a separate listing for the semitrailers with spread tandem axles. Similarly, four-axle singles are not included because only 0.1 to 0.2 percent of the trucks observed in nationwide studies were of that type.[40,87]

Axle Spacings and Loads. The average axle spacings measured in the studies are listed in table 58.[24,40] Axles that are spaced 4 ft (1.2 m) apart are tandem axles. The average measured axle loads, expressed as a percentage of the gross vehicle weight,

Table 57. Weight and volume characteristics of various truck types.

Category	Type	Code FHWA	Code CCNY	Trailer Lengths	PMGW TTI	PMGW CCNY	PMP TTI	PMP CCNY	Cv
Pre-STAA	single	2D	SU2		30	30	19	19	500
		3A	SU3		47	47	27	27	800
	semi	2S1	ST3	40	51	51	28	28	2148
		2S2	ST4B	40	64	64	37	37	2148
		3S2	ST5B	45	78	78	48	48	2417
	twin	2S1-2	TW5B	26/26	80	80	49	49	2792
Post-STAA	single	4A	SU4		58	59	33	34	900
		4A	SU4S		78	80	48	50	1500
	semi	2S2	ST4A	48	64	64	37	37	2741
		3S2	ST5A	48	78	78	47	47	2741
		3S3	ST6	48	89	105	57	73	2741
		3S2	ST5S	48	88	88	57	57	2741
	twin	2S1-2	TW5A	28/28	90	90	55	55	3198
		3S1-2	TW6	28/28	93	105	58	70	3198
		3S2-2	TW7	28/28	93	116	57	80	3198
		3S2-3	TW8	28/28	93	122	57	86	3198
	western double	2S1-2	WD5	48/28	90	90	46	46	4340
		3S1-2	WD6	48/28	105	105	61	61	4340
		3S2-2	WD7	48/28	105	117	60	72	4340
		3S2-3	WD8	48/28	105	131	60	.86	4340
		3S2-4	WD9	48/28	105	139	59	93	4340
	turnpike double	2S1-2	TD5	48/48	90	90	37	37	5482
		3S1-2	TD6	48/48	105	105	52	52	5482
		3S2-2	TD7	48/48	115	117	61	63	5482
		3S2-3	TD8	48/48	115	131	61	77	5482
		3S2-4	TD9	48/48	115	139	60	84	5482
	triple	2S1-2-2	TP7	28/28/28	109	130	61	82	4796
		3S1-2-2	TP8	28/28/28	109	145	61	97	4796
		3S2-2-2	TP9	28/28/28	109	146	60	97	4796

PMGW = practical maximum gross weight in kips
PMP = practical maximum payload in kips
Cv = volume capacity in cubic feet
TTI = TTI truck-weight formula without a GVW cap
CCNY = CCNY truck-weight formula without a GVW cap
PMGW and PMP values listed for pre-STAA types are based on the original truck-weight (bridge) formula with an 80-kip GVW cap, but these value are the same as those for the TTI and CCNY formulas except for TW5B

Table 58. Axle spacings and loads for various truck types.

	Feet to Axle Number:								Percent of Gross Vehicle Weight on Axle Number:								
Type	2	3	4	5	6	7	8	9	1	2	3	4	5	6	7	8	9
SU2	16								40.0	60.0							
SU3	16	20							30.0	35.0	35.0						
ST3	12	44							27.0	40.0	33.0						
ST4B	12	38	42						23.0	35.0	21.0	21.0					
ST5B	12	16	44	48					18.0	22.5	22.5	18.5	18.5				
TW5B	10	30	40	60					16.0	25.0	21.0	19.0	19.0				
SU4	14	18	22						25.0	25.0	25.0	25.0					
SU4S	14	23	32						25.0	25.0	25.0	25.0					
ST4A	13	50	54						23.0	35.0	21.0	21.0					
ST5A	13	17	50	54					18.0	22.5	22.5	18.5	18.5				
ST6	13	17	36	45	54				16.0	19.5	19.5	15.0	15.0	15.0			
ST5S	10	14	47	51					21.0	21.0	21.0	18.5	18.5				
TW5A	10	31	40	62					16.0	25.0	21.0	19.0	19.0				
TW6	10	14	31	40	62				16.0	12.5	12.5	21.0	19.0	19.0			
TW7	10	14	27	31	40	62			16.0	12.5	12.5	10.5	10.5	19.0	19.0		
TW8	10	14	27	31	40	58	62		16.0	12.5	12.5	10.5	10.5	19.0	9.5	9.5	
WD5	13	54	63	85					16.0	25.0	21.0	19.0	19.0				
WD6	13	17	54	63	85				16.0	12.5	12.5	21.0	19.0	19.0			
WD7	13	17	50	54	63	85			16.0	12.5	12.5	10.5	10.5	19.0	19.0		
WD8	13	17	50	54	63	81	85		16.0	12.5	12.5	10.5	10.5	19.0	9.5	9.5	
WD9	13	17	50	54	63	67	81	85	16.0	12.5	12.5	10.5	10.5	9.5	9.5	9.5	9.5
TD5	13	54	63	105					16.0	25.0	21.0	19.0	19.0				
TD6	13	17	54	63	105				16.0	12.5	12.5	21.0	19.0	19.0			
TD7	13	17	50	54	63	105			16.0	12.5	12.5	10.5	10.5	19.0	19.0		
TD8	13	17	50	54	63	101	105		16.0	12.5	12.5	10.5	10.5	19.0	9.5	9.5	
TD9	13	17	50	54	63	67	101	105	16.0	12.5	12.5	10.5	10.5	9.5	9.5	9.5	9.5
TP7	10	31	40	62	71	93			13.0	16.0	15.0	14.0	14.0	14.0	14.0		
TP8	10	14	31	40	62	71	93		13.0	8.0	8.0	15.0	14.0	14.0	14.0	14.0	
TP9	10	14	27	31	40	62	71	93	13.0	8.0	8.0	7.5	7.5	14.0	14.0	14.0	14.0

Axle loads are average values for the full spectrum of loaded and unloaded trucks.

Table 59. Practical maximum gross weights and payloads for old (pre-STAA) truck types.

Type	Wheel Base	Front Axle Load	Additional Axles S	Additional Axles T	Gross Weight Limit Axle	Gross Weight Limit Cap	Gross Weight Limit OBF	Gross Weight Limit TTI	Gross Weight Limit CCNY	Practical Maximum Gross Weight Present	Practical Maximum Gross Weight TTI	Practical Maximum Gross Weight CCNY	Empty Wt	Practical Maximum Payload Present	Practical Maximum Payload TTI	Practical Maximum Payload CCNY
SU2	16	10	1	0	30		46	58	56	30	30	30	11	19	19	19
SU3	20	13	0	1	47		51	66	63	47	47	47	20	27	27	27
ST3	44	11	2	0	51	80	69	84	102	51	51	51	23	28	28	28
ST4B	42	10	1	1	64	80	70	83	99	64	64	64	27	37	37	37
ST5B	48	10	0	2	78	80	78	86	109	78	78	78	30	48	48	48
TW5B	60	10	4	0	90	80	86	92	120	80	90	90	31	49	59	59

All weights and loads are in kips; all lengths are in feet.
GVW = gross vehicle weight B=wheelbase N=number of axles S=single T=tandem
Axle Gross Weight Limit = front axle load plus the sum of the legal axle loads for all additional single and tandem axles
OFB Gross Weight Limit = GVW from original truck-weight (bridge) formula:
GVW=.5(N)(B)/(N-1)+6(N)+18
TTI Gross Weight Limit = GVW from following TTI truck-weight formulas:
GVW=2(B)+26 for 8<B<=24 GVW=.5(B)+62 for B>24
CCNY Gross Weight Limit = GVW from following CCNY truck-weight formulas:
GVW=1.64(B)+30 for B<=50 GVW=.8(B)+72 for B>50
Practical Maximum Gross Weight = max legal weight as limited by axle loads or truck-weight formula

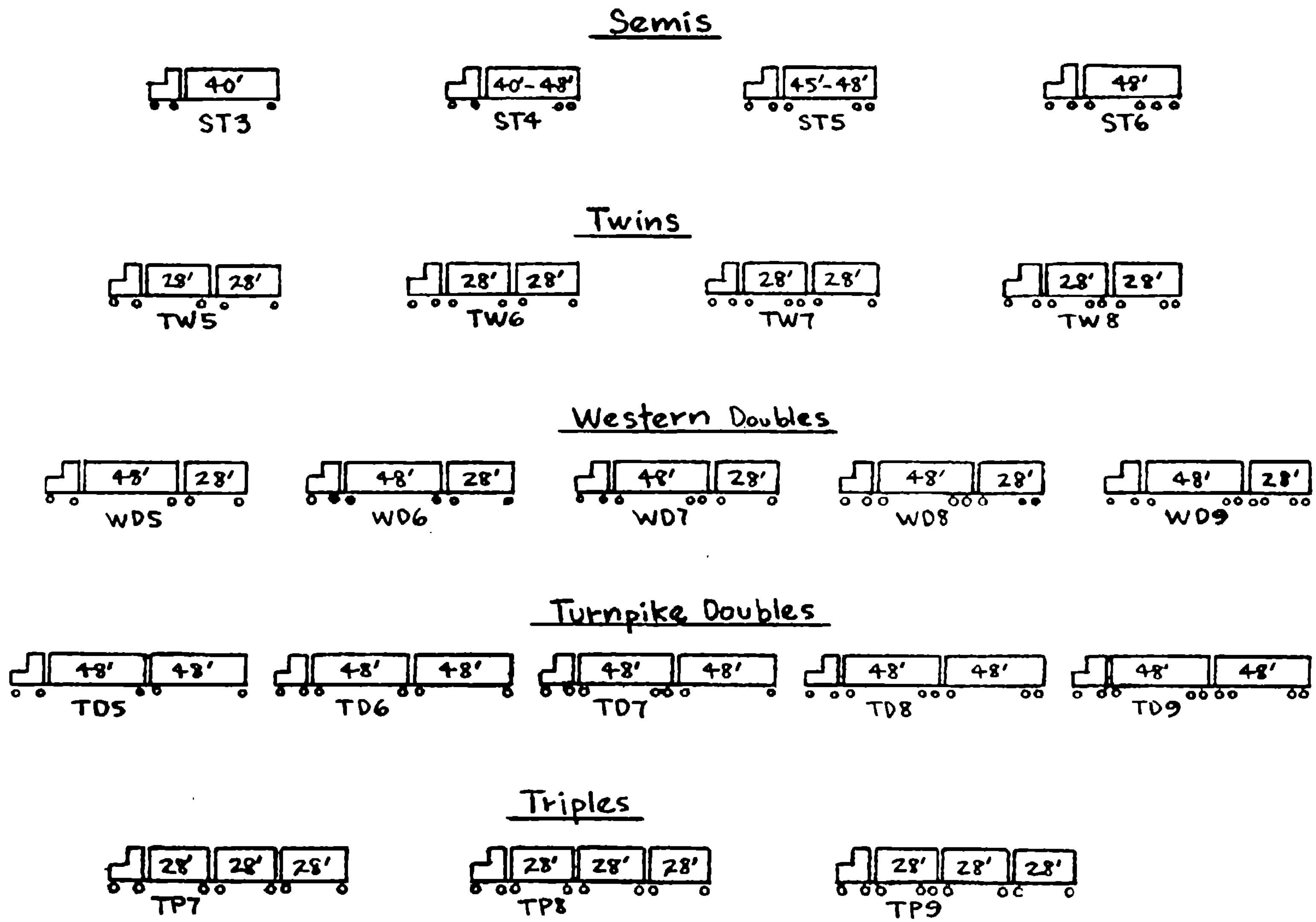


Figure 15. Typical combination trucks.

are also listed in table 58. These are average values for the observed composition of traffic, which included both empty and loaded trucks.

Practical Maximum Gross Weights and Payloads. The practical maximum gross weight (PMGW) for a truck is the maximum gross weight that can be carried without violating legal weight limits.[85,86,87] The PMGW is limited to the smallest weight obtained from: (a) the sum of the practical maximum front-axle load and the maximum legal loads for all other axles, (b) the maximum gross weight allowable by the applicable truck-weight formula, or (c) any applicable gross weight cap, such as the present 80-kip (356 kN) limit. In a few cases, the legal loads on subsets of contiguous axles may be limited by the applicable truck-weight formula and must be considered in calculating the PMGW.

The front-axle (steering-axle) load is usually limited to less than the maximum legal single-axle load by practical and safety considerations; enough weight cannot be shifted to the front axle to reach the legal axle limit.[86,87] The front-axle load increases with the gross vehicle weight for any particular truck type. Curves defining this relationship for the main truck types have been developed from a nationwide truck survey.[87] Also, empirical equations defining the relationships have been developed from a nationwide weigh-in-motion study.[40] These curves and equations can be used to develop front-axle loads corresponding to particular gross vehicle weights. The results from these two methods are almost identical and are consistent with front-axle loads estimated in other studies.[85,86,88] One recent study of different truck scenarios assumed a value of 12 kips (53.4 kN) for all truck types.[81]

Values of the PMGW, and parameters used in calculating these values, are given in table 59 for the main pre-STAA truck types. The listed front-axle load for each truck was obtained by using its PMGW with the curves and formulas mentioned above; since PMGW depends on the front-axle load, iteration was required. Three PMGW values are given for each truck type: (a) one based on the present STAA weight limits including the present truck-weight formula (Formula B) and the 80-kip (356 kN) cap, (b) one based on the TTI truck-weight formula without a GVW cap and (c) one based on the the proposed truck-weight formula without any cap (this formula is labeled CCNY formula in the table). The present single and tandem axle limits of 20 and 34 kips (89 and 151.2 kN) , respectively, apply to all three cases.

The first case (Formula B) represents the actual pre-STAA traffic. The limits for this case were in effect on Interstate highways when the studies used to develop table 57 were made. Hence, these PMGW values are based on the proper legal limits for most of the trucks in the studies. However, some of the trucks in the studies were limited by different (usually slightly lower) legal limits. The other two cases give the expected PMGW values if either the TTI or the proposed formula is applied without a GVW cap to the pre-STAA truck types.

Practical maximum gross payloads (PMP) are also given in table 59; these were obtained by subtracting the empty weight for each type from the corresponding PMGW. The empty weights were obtained from nationwide truck survey data.[63,87]

Volume Capacity. The volume capacity for each pre-STAA truck type is listed in table 57. For the semitrailer combinations, the volume capacity was determined from the following formula:

$$C_V = 53.7\ L \qquad (30)$$

in which C_V is the volume capacity in cubic feet and L is the total length of the one or more trailers or semitrailer. This equation was derived from cargo stowage figures for specific trucks.[86] It is based on an 8 ft (2.4 m) width and a 13.5 ft (4.1 m) height.

Volume capacities for single units vary widely and no comprehensive data were found on these capacities. Consequently, the capacities were estimated from expected sizes. These volume capacities are intended to be average values for each type of single.

The listed volume capacities apply specifically to van-type trucks. Width and height clearance requirements, however, limit volume capacities for flatbed and bulk-hauling trucks to about these same values. Volume capacities of tank trucks are limited by the same width, height, and length limitations as van-type trucks, but are generally smaller because the tanks are cylindrical.

Post-STAA Truck Types

Basic Types. Several new truck types related to the STAA are described in tables 57,58 and 60; they include types that have resulted from the present regulations and types that could result from regulation changes being considered. These new types, together with the pre-STAA types described in the preceding section, are the main types that are expected to be prevalent enough to affect fatigue behavior now and in the foreseeable future. Most of the new truck types have been operated either with or without special permits in a few States, but are new in the sense that they were not permitted nationwide and were not prevalent enough to affect fatigue behavior. Various pre and post-STAA combination trucks are illustrated in figure 15.

The first type is a semitrailer (semi) unit consisting of a tractor and a 48-ft (14.6 m) semitrailer; similar semitrailer units were common before the STAA but had 45-foot trailers. The second type is a twin consisting of a tractor and two 28-ft (8.5 m) semitrailers or trailers. This type was also included in the pre-STAA types but with different permissible weights and sizes. These first two types are permitted under present STAA regulations provided the GVW is limited to 80 kips (356 kN).

Since tridem axles can be utilized more effectively under regulation changes being considered, four-axle singles and six-axle semitrailers with tridem axles have been included in truck traffic scenarios being considered by others.[43,81] Two types of four-axle singles with tridem axles are included in the present study: one has an 8 ft (2.4 m) (overall spacing) tridem with a permissible load of 42 or 43 kips (186.8 or 191.3 kN) (as limited by the TTI or the proposed (CCNY) formula) and the other has an 18 ft (5.5 m) tridem with a permissible load of 60 kips (267 kN).

Table 60. Practical maximum gross weights for new (post-STAA) truck types.

Type	Wheel Base	Front Axle Load	Additional Axles		Subset Reduction		Gross Weight Limit			PMGW		Empty Wt	PMP	
			Single	Tandem	TTI	CCNY	Axle	TTI	CCNY	TTI	CCNY		TTI	CCNY
SU4	22	16	3	0	18	17	76	70	66	58	59	25	33	34
SU4S	32	20	3	0			80	78	82	78	80	30	48	50
ST4A	54	10	1	1			64	89	115	64	64	27	37	37
ST5A	54	10	0	2			78	89	115	78	78	31	47	47
ST6	54	11	3	1	11		105	89	115	89	105	32	57	73
ST5S	51	20	0	2			88	88	113	88	88	31	57	57
TW5A	62	10	4	0			90	93	122	90	90	35	55	55
TW6	62	11	3	1			105	93	122	93	105	35	58	70
TW7	62	12	2	2	2	4	120	93	122	93	116	36	57	80
TW8	62	12	1	3	2	4	134	93	122	93	122	36	57	86
WD5	85	10	4	0			90	105	140	90	90	44	46	46
WD6	85	11	3	1			105	105	140	105	105	44	61	61
WD7	85	12	2	2	2	3	120	105	140	105	117	45	60	72
WD8	85	12	1	3	2	3	134	105	140	105	131	45	60	86
WD9	85	13	0	4	8	10	149	105	140	105	139	46	59	93
TD5	105	10	4	0			90	115	156	90	90	53	37	37
TD6	105	11	3	1			105	115	156	105	105	53	52	52
TD7	105	12	2	2	2	3	120	115	156	115	117	54	61	63
TD8	105	12	1	3	2	3	134	115	156	115	131	54	61	77
TD9	105	13	0	4	8	10	149	115	156	115	139	55	60	84
TP7	93	10	6	0			130	109	146	109	130	48	61	82
TP8	93	11	5	1			145	109	146	109	145	48	61	97
TP9	93	12	4	2	2	4	160	109	146	109	146	49	60	97

All weights and loads are in kips; all lengths are in feet.
Axle Gross Weight Limit = front axle load plus sum of legal loads on all other axles
PMGW = practical maximum gross weight; lowest of (a) axle limit minus subset reduction or (b) truck-weight formula (TTI or CCNY) limit
PMP = practical maximum payload; PMGW minus empty weight

A six-axle semitrailer with an 18 ft (5.5 m) tridem is also included; such a truck cannot be effectively utilized under present regulations because of the 80-kip (356 kN) GVW cap.

Under STAA regulation changes being considered, a significantly higher GVW would be permitted for twins. Three additional new types of combinations are being considered. The first is a western (Rocky Mountain) double consisting of a tractor, a 48-ft (14.6 m) semitrailer or trailer, and a 28-ft (8.5 m) semitrailer or trailer. The next is a turnpike double consisting of a tractor and two 48-ft (14.6 m) semitrailers or trailers. The last is a triple consisting of three 28-ft (8.5 m) semitrailers or trailers. In subsequent discussions, the term trailer will generally be used to apply to both full trailers and semitrailers since the difference between the two (method of coupling) has little or no effect on the fatigue analysis.

For each basic type, trucks with a progressively increasing number of axles are listed. The first of these consists of a two-axle tractor pulling a one-axle trailer followed by as many two-axle trailers as occur in that basic type. The second consists of a three-axle tractor pulling the same trailers. The additional trucks are three-axle tractors pulling a two-axle trailer followed by trailers with two, three or four axles. For each basic type, the table lists trucks with enough axles to permit the full GVW from the higher of the two truck-weight formulas.

One unusual truck is listed. This is the ST5S truck, which is a five-axle semi with a twin-steering axle. It is similar to the standard five-axle semitrailer (ST5), but has a front-axle weight of 20 kips (89 kN) instead of 10 kips (44.8 kN). The distance to the second axle is also assumed to be less in the ST5S semi.

The special hauling vehicles (SHV) being considered by others are not included in the present study because they are intended to be used only as permit vehicles and would not be governed by the truck-weight formulas being evaluated in the present study.[43,81]

Axle Spacings and Loads. The expected axle spacings for the new truck types are listed in table 58; axles spaced 4 ft (1.2 m) apart are tandem axles. The listed spacings were developed from the expected configurations for these types.[78,88] All trailers were assumed to have the maximum lengths permitted by the STAA, and the axles were assumed to be spaced as far apart as possible within these lengths as illustrated in figure 16. Specifically, the first axle under the first trailer in a combination was assumed to be 4 ft (1.2 m) from the front of that trailer, and the outermost axles for all other trailers were assumed to be 3 ft (0.9 m) inside of an end. The distance between successive trailers was taken as 3 ft (0.9 m). The distances between the first and second axles of conventional and cab-over-engine tractors were assumed to be 13 and 10 ft (4 and 3.05 m), respectively. The solid circles in the figure represent the minimum number of axles that can be used. The open circles represent additional axles that can be used; these are spaced 4 ft (1.2 m) from the others.

Expected average axle loads, expressed as a percentage of gross vehicle weight, are also listed in table 58. These were estimated from measured axle loads on similar types of trucks and are intended to represent average values for the expected future composition of this truck type, which includes both empty and loaded trucks. Thus, these percentage axle loads cannot be determined directly from the legal axle loads.

Practical Maximum Gross Weights and Payloads. Values of the PMGW, and parameters used in calculating these values, are given in table 60 for the new post-STAA truck types. Two PMGW values are given for each truck type: one based on the TTI truck-weight formula and the other based on the the proposed truck-weight (CCNY) formula. The front-axle loads were estimated as explained previously and the other axle loads were taken as the maximums permitted for single or tandem axles under the STAA namely, 20 and 34 kips (89 and 151 kN), respectively.

Permissible gross weights for various subsets of axles were calculated by the two truck-weight formulas and are listed in table 61. These subsets included (a) individual tractors, (b) individual trailers, and (c) linkages between successive trailers. The linkage subsets include one single or tandem axle from each trailer

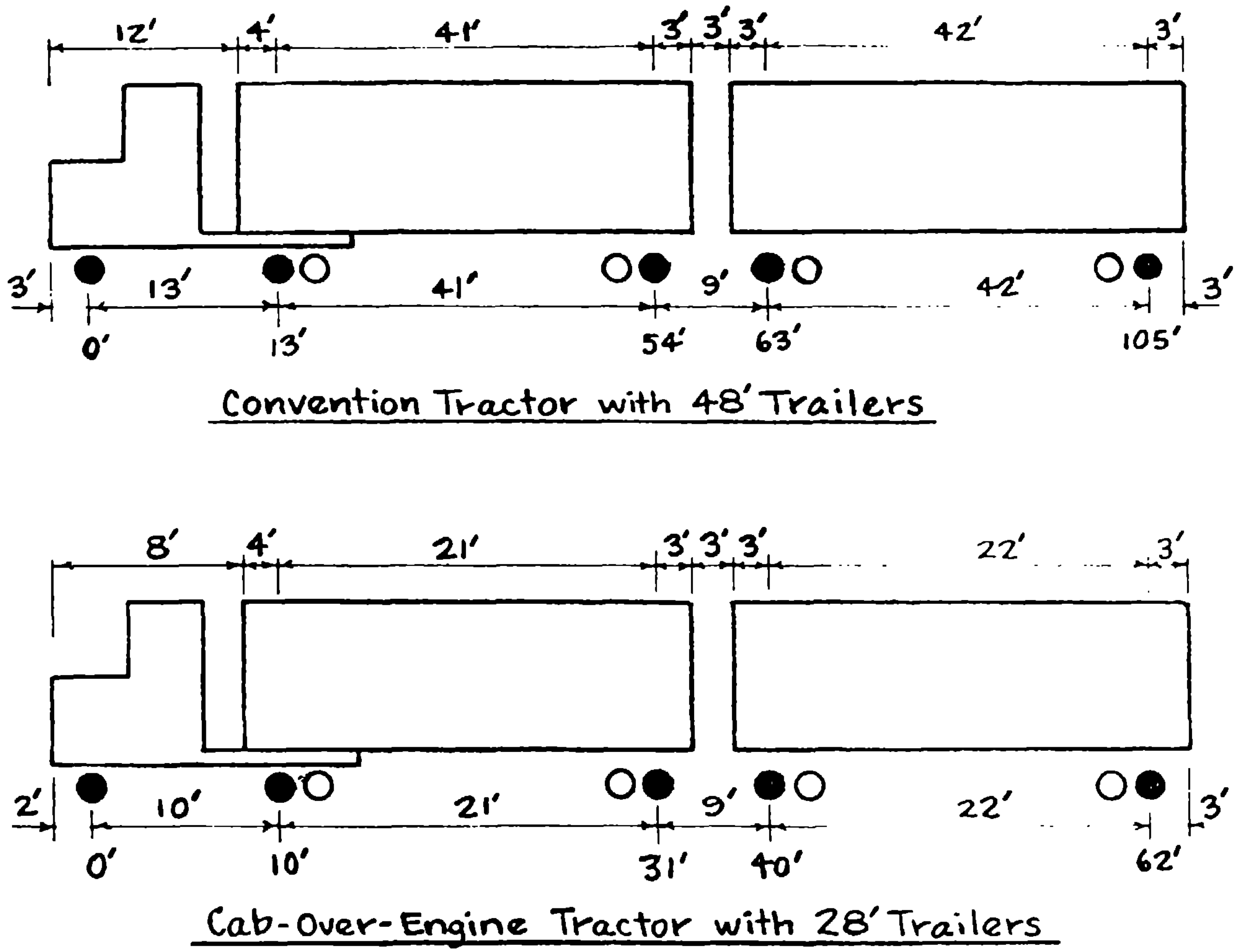


Figure 16. Typical dimensions for post-STAA combination trucks.

Table 61. Gross weight limits for axle subsets of new (post-STAA) trucks.

Subset	B	Number of Axles: Front	Single	Tandem	Gross Weight Limit: Axle	TTI	CCNY	Occurs In:
Tridem	8	0	3	0	60	42.0	43.1	SU4
Tridem	16	0	3	0	60	62.0	59.5	SU4S,ST6
Tractor	10	1	1	0	33	46.0	46.4	TW5A,TP7
Tractor	14	1	0	1	47	54.0	53.0	TW6,TW7,TW8,TP8,TP9
Linkage	9	0	2	0	40	44.0	44.8	TW5A,TW6,WD5,WD6 TD5,TD6,TP7,TP8,TP9
Linkage	13	0	1	1	54	52.0	51.3	TW7,TW8,WD7,WD8,TD7 TD8,TP9
Linkage	17	0	0	2	68	60.0	57.9	WD9,TD9
Trailer	21	0	2	0	40	68.0	64.4	TW5,TP7
Trailer	21	0	1	1	54	68.0	64.4	TW6,TP8
Trailer	21	0	0	2	68	68.0	64.4	TW7,TW8,TP9
Trailer	22	0	2	0	40	70.0	66.1	TW5,TW6,TW7,WD5,WD6 WD7,TP7,TP8,TP9
Trailer	22	0	1	1	54	70.0	66.1	TW8,WD8
Trailer	22	0	0	2	68	70.0	66.1	WD9
Trailer	41	0	2	0	40	82.5	97.2	WD5,TD5
Trailer	41	0	1	1	54	82.5	97.2	ST4A,WD6,TD6
Trailer	41	0	0	2	68	82.5	97.2	ST5A,WD7,WD8,WD9 TD7,TD8,TD9
Trailer	41	0	3	1	94	82.5	97.2	ST6
Trailer	42	0	2	0	40	83.0	98.9	TD5,TD6,TD7
Trailer	42	0	1	1	54	83.0	98.9	TD8
Trailer	42	0	0	2	68	83.0	98.9	TD9

All weights and loads are in kips and all lengths are in feet.
GWL = gross weight limit for subset
B = wheelbase for subset

Axle GWL = front axle load plus the sum of the legal axle loads for all single and tandem axles in the subset

TTI GWL = GWL from following TTI truck-weight formulas:
GWL=2(B)+26 for 8<B<=24 GWL=.5(B)+62 for B>24

CCNY GWL = GWL from following CCNY truck-weight formulas:
GWL=1.64(B)+30 for B<=50 GWL=.8(B)+72 for B>50

separated by a 9-ft (2.74 m) clear distance. The types of trucks that include each type of subset are listed in the table.

Generally, the sum of the legal axle loads for each subset is less than the permissible gross weight from the formula for that subset. In such cases, the subset limit does not affect the PMGW. In a few cases, however, the subset limit governs. In these cases, the difference between the sum of the legal axle weights and the subset limit is called the subset reduction, and this reduction is listed in table 60. It is subtracted from the sum of the axle loads to get the Axle Gross Weight Limit given in the table.

The practical maximum payloads for the various truck types are also listed in table 60. Again, these payloads were obtained by subtracting the average empty weight for each type from the corresponding PMGW. The average empty weights were obtained from available data or estimated from data for other related truck types.[63,87,88]

Volume Capacity. The volume capacity of each post-STAA truck type is listed in table 57. For combinations, the volume capacity was determined from the following formula:

$$C_V = 57.1\ L \tag{31}$$

in which C_V is the volume capacity in cubic feet and L is the total length of the one or more trailers. This equation was derived from the same cargo stowage data as equation 30. It is based on an 8.5-ft (2.6 m) width and a 13.5-ft (4.1 m) height, which was the predominant height before the STAA and remains the predominant height after the STAA.[57] The volume capacities for the singles were estimated from the expected sizes as discussed for pre-STAA trucks. The volume capacities are specifically for van-type trucks, but apply approximately to other important types as discussed earlier.

Traffic Scenarios

The compositions of the truck traffic (percentages of various truck types) for various possible scenarios are given in table 62. These compositions were estimated from the base composition (scenario A) by assuming that certain percentages of each type in the base scenario are replaced by other types. This process is illustrated in table 63 for scenario G.

Since each new type generally requires a different number of trips to haul a given amount of freight than the type it replaces, the assumed new compositions were adjusted accordingly. A factor T, which defines the trips per freight hauled for a particular truck type, was used to make this adjustment. Specifically, the percentage of each original type replaced by a new type was multiplied by the ratio of T factors for the two types. The resulting modified percentages were then summed for the entire spectrum. The sum was usually less than 100 percent, which means that the number of trips required to haul a given amount of freight with the new scenario is less than that required with the base scenario. The modified percentage for each type was then divided by the sum for the spectrum to give the correct percentage of that type in the new scenario; these corrected percentages, of course, total to 100 percent.

The sum of the modified percentages for a new scenario is called the relative volume for that scenario. It is used later in calculating the relative fatigue damage for the scenario. In the next section, the factor T is discussed in more detail and values of T are given for all truck types. These values are different for the TTI and the proposed formulas. The replacements expected with each scenario also differ for the two formulas because these formulas often permit different GVW's and thus provide different incentives to use certain types of trucks. All of the new scenarios represent traffic on the National Network for Trucks.

Scenario A. This is the base scenario and represents pre- STAA traffic on the system of Interstate, primary, and secondary highways in the United States. It is based on comprehensive data from nationwide truck-survey and weigh-in-motion

Table 62. Composition of truck traffic for various scenarios.

	Percentage of Trucks												
	Base	Scenario B		Scenario C		Scenario D		Scenario E		Scenario F		Scenario G	
Type	A	TTI	CCNY	TTI	CCNY	TTI	CCNY	TTI	CCNY	TTI	CCNY	TTI	CCNY
SU2	12.30	12.98	12.98	12.02	12.80	12.20	13.16	12.36	13.17	12.24	13.32	12.48	13.47
SU3	6.50	6.87	6.87	7.51	8.00	7.62	8.22	7.72	8.23	7.65	8.32	7.80	8.42
ST3	3.00	3.16	3.16	3.19	3.39	3.23	3.49	3.28	3.49	3.25	3.53	3.31	3.57
ST4B	11.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ST5B	62.90	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
TW5B	3.80	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SU4	0.00	0.00	0.00	5.03	5.30	5.10	5.45	5.17	5.46	5.12	5.52	5.22	5.58
SU4S	0.00	0.00	0.00	1.13	1.18	1.14	1.22	1.16	1.22	1.15	1.23	1.17	1.25
ST4A	0.00	10.86	10.86	6.95	5.27	6.56	4.74	6.11	4.74	6.54	5.01	5.50	4.28
ST5A	0.00	53.27	53.27	36.86	27.15	34.13	24.34	31.32	24.37	34.25	24.65	28.35	21.43
ST6	0.00	0.00	0.00	14.56	15.92	14.77	16.37	14.96	16.39	14.82	16.57	15.11	16.76
TW5A	0.00	10.29	10.29	8.55	5.90	4.05	3.09	3.92	3.09	5.43	4.00	3.30	2.64
TW6	0.00	2.57	2.57	4.20	5.27	3.95	2.76	3.82	2.76	5.30	3.57	3.21	2.36
TW7	0.00	0.00	0.00	0.00	4.98	0.00	2.61	0.00	2.61	0.00	3.37	0.52	2.23
TW8	0.00	0.00	0.00	0.00	4.84	0.00	2.54	0.00	2.54	0.00	3.29	0.53	2.17
WD5	0.00	0.00	0.00	0.00	0.00	3.91	2.98	0.00	0.00	0.00	0.00	2.68	1.59
WD6	0.00	0.00	0.00	0.00	0.00	3.32	2.53	0.00	0.00	0.00	0.00	2.27	1.35
WD7	0.00	0.00	0.00	0.00	0.00	0.00	2.32	0.00	0.00	0.00	0.00	0.00	1.23
WD8	0.00	0.00	0.00	0.00	0.00	0.00	2.13	0.00	0.00	0.00	0.00	0.00	1.13
WD9	0.00	0.00	0.00	0.00	0.00	0.00	2.06	0.00	0.00	0.00	0.00	0.00	1.09
TD5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	4.06	3.20	0.00	0.00	2.87	1.70
TD6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.22	2.54	0.00	0.00	2.27	1.35
TD7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.91	2.25	0.00	0.00	0.00	1.20
TD8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.01	0.00	0.00	0.00	1.07
TD9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.92	0.00	0.00	0.00	1.02
TP7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	4.26	2.67	2.59	1.10
TP8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.47	0.41	1.01
TP9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.47	0.42	1.01

studies.[63,24,40] This was the latest comprehensive nationwide data available to the authors.

Scenario B. This scenario represents post-STAA traffic on the National Network for Trucks after the trucking industry has fully responded to the changes in regulations in the STAA. As mentioned earlier, the response to these changes has been gradual.[57,78] Since comprehensive data taken long enough after passage of the STAA were not available, scenario B was estimated by modifying scenario A. The assumed modifications reflect changes in size, rather than weight, regulations because the STAA truck-weight (bridge) formula and 20/34/80 weight limits were previously in effect on Interstate highways. Since the truck weights for this scenario are based on this truck-weight formula rather than the TTI or the proposed formula, the compositions listed under the TTI and CCNY headings in table 62 are the same.

The specific modifications made to scenario A are (a) replacement of some five-axle semitrailers (ST5B) by twins, (b) replacement of all other pre-STAA five-axle semitrailers (ST5B) by post-STAA five-axle semitrailers with a longer cargo length and wheelbase, and (c) replacement of all pre-STAA twins (TW5B) with post-STAA twins (TW5A or TW6) with a longer cargo length and wheelbase. For this scenario only, the post-STAA twins are subject to the present 80-kip (356 kN) GVW cap; thus, the PMGW and PMP values listed in table 57 for these twins are not applicable for this scenario. For all subsequent scenarios, a GVW cap is not applied to these twins and the PMGW and PMP values listed in table 57 are applicable.

It is assumed that 12 percent of the ST5B semis are replaced with TW5A twins and 3 percent of ST5B semis are replaced with TW6 twins. This is consistent with estimates by others that 10 to 20 percent of present semitrailers will eventually be replaced with twins because the twins provide greater volume capacity and operational flexibility.[78] Observations of present traffic suggest that some TW6 twins will be used even though they are limited to the same weight and volume capacities as the TW5A twins. The TW6 twins generally use the same trailers as the TW5 twins, but with a three-axle tractor rather than a two-axle tractor.

No singles (SU2 and SU3) are replaced since the STAA regulations do not alter the permissible weight and volume capacities for these types. The percentages for these types, however, change slightly because of changes in the relative volume resulting from replacements of other types.

Scenario C. This scenario represents traffic under the present STAA size and axle-load regulations with the present 80-kip (356 kN) GVW cap removed and the present truck-weight (bridge) formula replaced by either the TTI or the proposed truck-weight formula.

The expected composition of singles is the same for TTI and the proposed formulas. The main change from scenarios A and B is the inclusion of four-axle singles (SU4 and SU4S), which can be utilized effectively if these truck-weight formulas are applied without a GVW cap as discussed earlier. Specifically, 2.2 percent of the trucks are shifted from ST4A semis to SU4 singles and 0.1 percent of the SU3 singles are shifted to SU4 singles. In addition, 0.8 percent of the trucks are shifted from ST4B semis to SU4S singles. Thus, it is assumed that about 1/4 of the four-axle singles are SU4S singles; it is expected that these large capacity singles will be required much less frequently than the smaller SU4 singles. Also, 1 percent of the trucks are shifted from SU2 singles to SU3 singles.

With the TTI formula, 22 percent of the four and five-axle semitrailers (ST4B and ST5B) are replaced with 6-axle semitrailers (ST6) and 15 percent are replaced with twins; three-axle semitrailers (ST3) are not replaced. As discussed earlier, both six-axle semitrailers and twins permit higher weight capacities and, therefore, are expected to be used in these percentages. Since the TW6 twin provides a slightly higher weight capacity than the TW5A twin, 1/3 of the twins are expected to be TW6's rather than 1/5 as assumed in scenario B. Twins with seven or more axles do not permit higher weight or volume capacities than 6-axle twins and, therefore, are not included in this spectrum. With these assumptions regarding combinations and singles, the resulting composition for the TTI formula approximates that used in a recent study of regulation changes.[81] That study did not include SU2 singles so the percentages differ accordingly.

The proposed formula permits significantly higher vehicle weights for 6-axle semitrailers and twins (except TW5A) than the TTI formula; consequently, a higher percentage of the four and five-axle semitrailers is replaced by these types. Specifically, 25 percent of the four and five-axle semitrailers are replaced with six-axle semitrailers, and 30 percent of these semitrailers are replaced with twins. Under the the proposed formula, permissible vehicle weights for twins increase progressively with the number of axles. This tends to promote the use of twins with larger numbers of axles. On the other hand, some equipment and operating costs increase with the number of axles.[81,87] Furthermore, the percentage of the traffic that requires a given GVW capacity decreases as this capacity increases. Therefore, it is assumed that the twins are equally distributed among vehicles with different numbers of axles ranging from six to nine.

Scenario D. This scenario includes the truck types in scenario C plus western doubles. The composition of singles for this scenario is assumed to be the same as that for scenario C except for small shifts due to changes in relative volume resulting from other replacements. Similarly, the percentage of four and five-axle semitrailers replaced by six-axle semitrailers is assumed to be the same as for scenario C. Again, the three-axle semitrailers are not changed from scenario A.

Under the TTI formula, the WD6 permits a higher GVW than the TW6; therefore, 20 percent rather than 15 percent, of the four and five-axle semitrailers are replaced by doubles (twins and western doubles). For the reasons discussed previously, it is assumed that this 20 percent is equally distributed among the four applicable doubles (TW5A, TW6, WD5, WD6). Twins and western doubles with more axles are not included because they do not provide larger weight or volume capacities.

Under the the proposed formula, permissible GVW's for western doubles progressively increase with the number of axles up to nine, and often exceed those for the twins. Therefore, 35 percent, rather than 30 percent, of the four and five-axle semitrailers are replaced by doubles (twins and western doubles) and this 35 percent is equally distributed among the nine applicable doubles (TW5A to TW8 and WD5 to WD9).

Scenario E. This scenario includes the truck types in scenario C plus turnpike doubles. The composition of singles for this scenario is assumed to be the same as that for scenario C except for small shifts due to changes in relative volume resulting from other replacements. Similarly, the percentage of four and five-axle semitrailers replaced by six-axle semitrailers is assumed to be the same as for scenario C. Again, the three-axle semitrailers are not changed from scenario A.

Under the TTI formula, a higher GVW is permitted for the TD7 than for any of the western doubles. Therefore, 25 percent of the four and five-axle semitrailers are replaced by doubles (twins and turnpike doubles). For the reasons discussed earlier, it is assumed that this 25 percent is equally distributed among the five applicable doubles (TW5A, TW6, WD5, WD6, WD7). Again, twins and turnpike doubles with more axles are not included because they do not provide larger weight or volume capacities.

Since the permissible GVW's under the the proposed formula are the same for both turnpike and western doubles, the percentage of semis replaced with turnpike doubles in this scenario is the same as the percentage replaced with western doubles in scenario D. Specifically, 35 percent of the four and five-axle semitrailers are replaced by twins and turnpike doubles, and this 35 percent is equally distributed among the nine applicable types.

Scenario F. This scenario includes the truck types in scenario C plus triples. Again, the composition of singles, six- axle semitrailers, and three-axle semitrailers is the same as for scenario C except for slight shifts due to change in relative volume.

Under the TTI formula, the permissible GVW is slightly greater for the TP7 triple than for the WD6 double, but it is more difficult to utilize triples effectively in many applications. Therefore, the percentage of four and five-axle semitrailers replaced by twins and triples in this scenario is assumed to be the same (20 percent) as the percentage replaced by twins and western doubles in scenario D. Again, this 20 percent is equally distributed among the three applicable types (TW5A, TW6, TP7). Triples with more axles are not included because they do not provide an increased weight or volume capacity under the TTI formula.

Under the the proposed formula, the permissible GVW's for the TP8 and TP9 triples are higher than for any other truck type. Since triples are more difficult to utilize effectively than doubles, however, the percentage of four and five-axle semitrailers replaced by twins and triples in this scenario is assumed to be the same (35 percent) as the percentage of semis replaced by twins and turnpike doubles in scenario E. This 35 percent is equally distributed among the seven applicable types (TW5A, TW6, TW7, TW8, TP7, TP8, TP9).

Scenario G. This scenario includes all of the new truck types except the ST5S, which is considered to be experimental. Again, the composition of singles, six-axle semitrailers, and three-axle semitrailers is the same as for scenario C. Since this scenario provides the greatest number of choices for replacing semitrailers, the highest replacement percentage is assumed. Specifically, the percentage of four and five-axle semitrailers replaced by twins, doubles, and triples is assumed to be 30 percent under the TTI formula and 40 percent under the the proposed formula. These percentages are equally distributed among eight applicable truck types under the TTI formula and 17 applicable types under the the proposed formula. As in the other scenarios, twins, doubles, and triples that do not provide a weight advantage over the same type with less axles are omitted.

FATIGUE CHARACTERISTICS OF TRUCKS

Four truck characteristics that affect fatigue behavior are discussed in subsequent paragraphs: (a) effective weight, (b) trips per freight hauled, (c) stress range ratio, and (d) cycles per truck passage. Values of these parameters are developed for each truck type and listed in table 64. The symbols We, T, S, and C, respectively, are used for these parameters. The first two parameters depend on the truck-weight formula

Table 63. Calculation of scenario G for CCNY (proposed) truck weight formula.

Type	T	Pr	Pm	Pn		Replacement Spectrum SU2	SU3	ST3	ST4B	ST5B	TW5B	Modified Replacement Spectrum SU2	SU3	ST3	ST4B	ST5B	TW5B	
					P	12.3	6.5	3.0	11.5	62.9	3.8	12.3	6.5	3.0	11.5	62.9	3.8	P
					T	3.98	2.58	1.48	1.27	1.06	0.97	3.98	2.58	1.48	1.27	1.06	0.97	T
SU2	3.98	11.30	11.29	13.47		11.30	0.00	0.00	0.00	0.00	0.00	11.29	0.00	0.00	0.00	0.00	0.00	
SU3	2.58	7.40	7.06	8.42		1.00	6.40	0.00	0.00	0.00	0.00	0.65	6.41	0.00	0.00	0.00	0.00	
ST3	1.48	3.00	2.99	3.57		0.00	0.00	3.00	0.00	0.00	0.00	0.00	0.00	2.99	0.00	0.00	0.00	
ST4B	1.27	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
ST5B	1.06	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
TW5B	0.97	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
SU4	2.21	2.30	4.68	5.58		0.00	0.10	0.00	0.00	2.20	0.00	0.00	0.09	0.00	0.00	4.59	0.00	
SU4S	1.38	0.80	1.04	1.25		0.00	0.00	0.00	0.00	0.80	0.00	0.00	0.00	0.00	0.00	1.04	0.00	
ST4A	1.14	4.01	3.58	4.28		0.00	0.00	0.00	4.01	0.00	0.00	0.00	0.00	0.00	3.58	0.00	0.00	
ST5A	1.00	19.04	17.96	21.43		0.00	0.00	0.00	0.00	19.04	0.00	0.00	0.00	0.00	0.00	17.96	0.00	
ST6	0.82	18.60	14.05	16.76		0.00	0.00	0.00	2.90	15.70	0.00	0.00	0.00	0.00	1.88	12.17	0.00	
ST5S	0.91	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
TW5A	0.86	2.70	2.22	2.64		0.00	0.00	0.00	0.27	1.48	0.95	0.00	0.00	0.00	0.18	1.19	0.84	
TW6	0.76	2.70	1.98	2.36		0.00	0.00	0.00	0.27	1.48	0.95	0.00	0.00	0.00	0.16	1.07	0.75	
TW7	0.72	2.70	1.87	2.23		0.00	0.00	0.00	0.27	1.48	0.95	0.00	0.00	0.00	0.15	1.01	0.71	
TW8	0.70	2.70	1.82	2.17		0.00	0.00	0.00	0.27	1.48	0.95	0.00	0.00	0.00	0.15	0.98	0.69	
WD5	0.83	1.75	1.33	1.59		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.18	1.15	0.00	
WD6	0.70	1.75	1.13	1.35		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.15	0.98	0.00	
WD7	0.64	1.75	1.03	1.23		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.14	0.90	0.00	
WD8	0.59	1.75	0.95	1.13		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.13	0.82	0.00	
WD9	0.57	1.75	0.91	1.09		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.12	0.79	0.00	
TD5	0.89	1.75	1.42	1.70		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.19	1.24	0.00	
TD6	0.70	1.75	1.13	1.35		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.15	0.98	0.00	
TD7	0.62	1.75	1.00	1.20		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.13	0.87	0.00	
TD8	0.56	1.75	0.89	1.07		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.12	0.78	0.00	
TD9	0.53	1.75	0.85	1.02		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.11	0.74	0.00	
TP7	0.57	1.75	0.92	1.10		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.12	0.80	0.00	
TP8	0.53	1.75	0.85	1.01		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.11	0.74	0.00	
TP9	0.53	1.75	0.85	1.01		0.00	0.00	0.00	0.27	1.48	0.00	0.00	0.00	0.00	0.11	0.74	0.00	
		100.00	83.82	100.00		12.30	6.50	3.00	11.50	62.90	3.80	11.94	6.49	2.99	7.86	51.55	2.98	
			83.82			Sum for All Trucks:			100.00			Sum for All Trucks:			83.82			

T = trips per freight hauled (relative to ST5A)
Pp = percentage of this truck type in pre-STAA truck traffic
Pn = percentage of this truck type in new truck traffic scenario
Pr = percentage of this truck type replacing original truck type in pre-STAA traffic

Table 64. Fatigue characteristics of various truck types.

		We		T		S				C	
Type	We/Wm	TTI	CCNY	TTI	CCNY	L=30	L=60	L=90	L=180	L=30	L=60
SU2	0.55	16.5	16.5	3.98	3.98	0.59	0.77	0.84	0.91	1.00	1.00
SU3	0.78	36.7	36.7	2.58	2.58	0.57	0.76	0.83	0.90	1.00	1.00
ST3	0.61	31.1	31.1	1.48	1.48	0.38	0.50	0.64	0.81	1.27	1.00
ST4B	0.69	44.2	44.2	1.27	1.27	0.35	0.51	0.64	0.81	1.21	1.00
ST5B	0.78	60.8	60.8	1.06	1.06	0.36	0.49	0.63	0.81	1.39	1.00
TW5B	0.82	65.6	65.6	0.97	0.97	0.24	0.42	0.59	0.77	1.08	1.00
SU4	0.80	46.4	47.2	2.23	2.21	0.55	0.77	0.83	0.90	1.00	1.00
SU4S	0.80	62.4	64.0	1.40	1.38	0.39	0.64	0.75	0.86	1.00	1.00
ST4A	0.69	44.2	44.2	1.14	1.14	0.38	0.43	0.55	0.76	1.19	1.00
ST5A	0.78	60.8	60.8	1.00	1.00	0.37	0.44	0.58	0.78	1.57	1.00
ST6	0.78	69.4	81.9	0.91	0.82	0.27	0.47	0.63	0.79	1.08	1.00
ST5S	0.78	68.6	68.6	0.91	0.91	0.39	0.44	0.59	0.79	1.46	1.00
TW5A	0.80	72.0	72.0	0.86	0.86	0.24	0.41	0.58	0.77	1.13	1.00
TW6	0.80	74.4	84.0	0.83	0.76	0.23	0.40	0.58	0.77	1.04	1.00
TW7	0.80	74.4	92.8	0.84	0.72	0.23	0.40	0.58	0.77	1.04	1.00
TW8	0.80	74.4	97.6	0.84	0.70	0.22	0.41	0.59	0.77	1.01	1.00
WD5	0.80	72.0	72.0	0.83	0.83	0.30	0.33	0.42	0.67	1.10	1.04
WD6	0.80	84.0	84.0	0.70	0.70	0.27	0.32	0.42	0.68	1.14	1.03
WD7	0.80	84.0	93.6	0.71	0.64	0.25	0.32	0.42	0.68	1.36	1.02
WD8	0.80	84.0	104.8	0.71	0.59	0.24	0.33	0.43	0.68	1.13	1.02
WD9	0.80	84.0	111.2	0.71	0.57	0.24	0.34	0.42	0.68	1.14	1.01
TD5	0.80	72.0	72.0	0.89	0.89	0.30	0.27	0.35	0.64	1.50	1.06
TD6	0.80	84.0	84.0	0.70	0.70	0.29	0.27	0.35	0.63	1.68	1.03
TD7	0.80	92.0	93.6	0.64	0.62	0.27	0.26	0.36	0.64	1.78	1.02
TD8	0.80	92.0	104.8	0.64	0.56	0.27	0.26	0.36	0.64	1.69	1.02
TD9	0.80	92.0	111.2	0.64	0.53	0.24	0.26	0.36	0.64	1.53	1.01
TP7	0.80	87.2	104.0	0.67	0.57	0.17	0.28	0.41	0.67	1.30	1.00
TP8	0.80	87.2	116.0	0.67	0.53	0.17	0.28	0.41	0.67	1.29	1.00
TP9	0.80	87.2	116.8	0.68	0.53	0.16	0.28	0.41	0.67	1.26	1.00

We = effective gross weight
Wm = practical maximum gross weight
T = trips per freight hauled (relative to ST5A)
S = stress range ratio at the 0.75L point in a continuous span
C = stress cycles per truck passage
L = span length in feet
TTI = TTI truck-weight formula without a GVW cap
CCNY = CCNY truck-weight formula without a GVW cap

so separate values based on the TTI and the proposed (CCNY) formulas are given for the post-STAA truck types. The corresponding values for the pre-STAA truck types are based on the present truck-weight formula and 20/34/80 weight limits, but would have been the same under the other truck-weight formulas for all types except the TW5B.

The last two parameters depend on (a) the span length of the bridge, (b) the location of the fatigue detail along the span, and (c) whether the bridge has simple or continuous spans. Therefore, stress range ratios are listed for four different span lengths: 30, 60, 90, and 180 ft (9.1, 18.3, 27.4 and 54.9 m). Cycles per truck passage are listed for the first two of these span lengths; C is 1.0 for all truck types for the two longer spans. Stress range ratios were calculated for both simple and continuous spans, but only factors for continuous spans are listed in the tables to avoid an excessive amount of data. This is deemed appropriate because continuous spans are generally more critical for fatigue and because the factors for the two cases are fairly close.[11,24] The most critical locations along the span are 0.50L for simple spans and 0.75L for continuous spans; L is the span length.[11,24] Therefore, the listed factors are for the 0.75L location.

Effective Weight

It has been shown that for fatigue calculations a spectrum (histogram) of different truck weights can be represented by an effective weight defined as:

$$W_e = \left(\sum W_i^3 \alpha_i\right)^{\frac{1}{3}} \tag{32}$$

in which W_e is the effective weight, α_i is the fraction of weights within an interval i, and W_i is the midwidth of that interval.[11,24,77] A given number of passages of a truck with this effective weight causes the same fatigue damage as an equal number of passages of the different trucks in the spectrum. W_e depends only on the truck spectrum and not on the characteristics of the bridge under consideration.

The ratio of effective weight, W_e, to the average weight, W_a, has been calculated from nationwide data for various truck types.[24,77] This ratio depends on the distribution of empty and loaded truck weights for the particular type and is assumed to remain constant if truck regulations change. It has been reported that the ratio of the average weight, W_a, to the practical maximum gross weight, W_m, also remains approximately constant with changes in truck regulations.[85] Values of this W_a/W_m ratio for various truck types were determined from nationwide truck-survey data.[63,87]

These W_a/W_m values were combined with the W_e/W_a ratios to obtain W_e/W_m ratios, which are listed in table 64. It is assumed that these ratios will not change significantly as a result of possible changes in truck regulations. The ratios were then applied to the practical maximum gross weights for the two different truck-weight formulas to get the effective weight of each truck type for each formula. The results are listed in table 64.

Trips Per Freight Hauled

The number of trips required to haul a given amount of freight depends on the weight capacity, C_w, and the volume capacity, C_v, of the truck. The resulting fatigue damage to a bridge depends on the number of passages of the truck across the bridge, or in other words, on the number of trips. Thus, the trips per freight hauled depends only on the characteristics of the individual trucks or the truck traffic; it does not depend on the characteristics of the bridge under consideration.

Approximately 2/3 of the semitrailers and 1/2 of singles on the highway system are loaded; the rest are empty.[24,77,87] The amount of freight carried by the loaded trucks is generally limited by either the weight capacity, C_w, or the volume capacity, C_v. If F_w is the fraction of loaded trucks limited by C_w and F_v is the fraction of loaded trucks limited by C_v, a factor T defining the trips per freight hauled can be obtained from the following equation:

$$T = \frac{47F_W}{C_W} + \frac{2741F_V}{C_V} \tag{33}$$

The constants in the numerators in the first and second terms are the weight and volume capacities, respectively, of the 48-ft (14.6 m) five-axle semitrailers (ST5A). Thus, T actually defines the trips required with a particular truck type relative to the trips required with this five-axle semitrailer. If T is greater than 1.0, more trips are required with this type than with the five-axle semitrailers. The fraction of trucks that are empty does not influence equation 33 since it is assumed that if the number of loaded trips is changed (due to a change in C_W or C_V) by a certain percent, the corresponding empty (return) trips will be changed by the same percent.

The fractions of loaded trucks controlled by weight or volume capacity probably vary with many factors including (a) type of truck, (b) type of highway, and (c) geographical region. Comprehensive data on these fractions were not found; consequently, a factor of 0.5 is used for both F_W and F_V for all truck types. This means that nationwide the number of loaded trucks controlled by weight and volume capacities are assumed to be equal. This is consistent with one estimate quoted in a study of truck regulations, but differs from another.[86]

Values of C_W and C_V are given in table 57 for various truck types; C_W is equal to the practical maximum payload, PMP, and C_V is equal to the listed volume capacity. These values were used in equation 33 to get the values listed in table 64. Even with the uncertainty in F_W and F_V, and other inherent assumptions, these values are considered more accurate than values provided by another approach used previously.[43,68,69] In this latter approach, the trips per freight hauled were assumed to be proportional to the maximum legal gross weight and independent of the volume capacity. Since the factor T has a much smaller effect on fatigue damage than W_e, a lower accuracy is tolerable for this parameter.

Stress Range Ratio

The stress range ratio, S, is the stress range caused by a truck as it crosses the bridge divided by the stress range caused by the passage of a concentrated load equal to the truck weight. It accounts for the effects of different axle spacings on the bending moment (and stress range) caused by a given gross truck weight. The stress range ratio is always less than 1.0 since the multiple axles of the truck distribute the weight over a length and thereby reduce the moment. The stress range ratio depends on (a) the span length, (b) the location along the span, and (c) whether the bridge has simple or continuous spans. It does not depend on the GVW.

For simple spans, the stress range ratio is the same as the moment ratio obtained by calculating the moment at a point along the span when the truck is at its worst position and dividing it by the moment at that point when the concentrated load is at its worst position. For noncomposite continuous spans, the stress range ratio is equal to the moment range ratio obtained by placing the truck and concentrated load at their worst positions in the span under consideration and in adjacent spans. For composite continuous spans, the stress range ratio differs from the moment range ratio because the stress for positive bending (load in the span under consideration) is based on the composite section while the stress for negative bending (load in an adjacent span) must be based on the noncomposite section.[11,60] Therefore, the stress range ratio depends on the section modulus ratio; that is, the section modulus for negative bending (noncomposite) divided by that for positive bending (composite).

In the present study, the stress range ratio was calculated for each truck type and for the four span lengths mentioned earlier. The values are specifically for the 0.75L point on a continuous span, but the comparable values for simple spans are not much different as discussed earlier. Because most steel girder bridges are composite, the values are based on a section modulus ratio of 0.8; this is typical for the critical location along the span and within the cross section.[60] The calculated stress range ratios are given in table 64.

Cycles per Truck Passage

When a truck crosses a short-span bridge it causes a complex stress cycle with several peaks and valleys corresponding to the individual axles. This complex cycle causes more fatigue damage than a simple cycle with only one peak and, therefore, can be considered to be equivalent to more than one simple cycle.[11,24,77] The equivalent number of cycles depends on the axle spacings and weight distribution for the truck and on various characteristics of the bridge. It does not depend on the GVW.

The equivalent number of stress cycles per truck passage, C, was calculated by a computer program that utilizes the widely accepted "rain flow" method of counting cycles.[11,24,77] The equivalent number of simple cycles causes the same fatigue damage as the single complex cycle actually produced by the truck. C is 1.0 for all truck types for the two longer spans; therefore, only the values for a 30 and 60-ft (9.1 and 18.3 m) spans are listed in table 64.

RELATIVE FATIGUE DAMAGE

General Procedures

Fatigue Damage Factor. Relative fatigue damage was calculated for different truck types and traffic scenarios. This relative damage is defined as the fatigue damage caused in hauling a given amount of freight with a particular truck type, or with the different truck types in a particular traffic scenario, divided by that caused by hauling the same amount of freight with the base truck or with the different truck types in the base scenario. Relative fatigue life is the reciprocal of relative fatigue damage; it compares the fatigue lives in years for two different truck types, or traffic scenarios (spectrums of different truck types), used to haul a given amount of freight annually.

It has been shown that fatigue damage is directly proportional to (a) the number of stress cycles and (b) the stress range for these cycles raised to the third power.[11,24,60] Stress range, in turn, is proportional to the product W_e times S, and the number of stress cycles is proportional to C. Thus, a fatigue damage factor, D, can be defined as:

$$D = \frac{(W_e S)^3 C}{1000} \qquad (34)$$

The constant 1000 is included merely to provide convenient numbers for the damage factor D. Since the factor defines relative rather than absolute damage, then, any value of the constant is permissible. The D factor was used in calculating relative fatigue damage for both individual truck types and traffic scenarios. The number of stress cycles that result from hauling a given amount of freight is also proportional to T. However, T is not included in equation 34 because it is applied in different ways in calculating relative damage for individual truck types and traffic scenarios.

Effect of Fatigue Limit. Comparisons of fatigue damage and fatigue lives based on the calculated relative fatigue damage apply if the actual magnitudes of the stress ranges involved in the comparisons are above the fatigue limit. However, if the effective stress range corresponding to the effective truck weight for a spectrum is below the variable-amplitude fatigue limit for the detail under consideration, it is assumed that no fatigue damage occurs and the fatigue life is infinite.[11,24,60] For such cases, changes in truck regulations have no effect unless they raise the effective stress range above the variable-amplitude fatigue limit; then the fatigue life is changed from infinity to some finite value.

In many cases, stress ranges in actual bridges are low enough so that proposed changes in truck regulations will not cause detrimental fatigue effects, but this influence of the fatigue limit cannot be generalized and can be accounted for only by analyzing individual bridges. Thus, it is convenient and conservative to consider the effects of truck regulation changes in terms of relative fatigue damage, which does in fact apply to many actual bridges.

Selected Parameters. The relative fatigue damage for either individual truck types or traffic scenarios depends on all of the parameters that affect We, T, S, and C; these parameters were discussed earlier. In the present study, relative damage values were calculated for the same span lengths (30, 60, 90, and 180 feet) and truck-weight formulas (TTI and the proposed) as the fatigue characteristics listed in table 64. These relative damage values, like the S and C values discussed earlier, are specifically for the 0.75L point in a continuous span bridge, which is generally the most critical location for fatigue.[11,24] Comparable values for other locations and for simple spans, however, are expected to be reasonably close to these values.[11,24]

Truck Types

Calculation Procedures. The relative fatigue damage for various truck types compared with that of the post-STAA five-axle semitrailer (ST5A) are given in table 65. For all post-STAA truck types, these values are based on either the TTI or the proposed truck- weight formula applied without a GVW cap but with the 20/34 axle limits. The values for pre-STAA truck types are based on the present truck-weight (bridge) formula and 20/34/80 weight limits. All relative damage values were calculated by dividing D times T for a truck type by that for the ST5A semi. The fatigue damage for the the proposed (CCNY) formula divided by that for the TTI formula, which is the same for all spans, is also listed for each truck type.

Results. Some general trends are apparent from the data in table 65. These trends show the effect of substituting various types of trucks for five-axle semitrailers. Substituting four-axle singles for five-axle semis causes considerably more fatigue damage over the entire range of spans because the singles have a much shorter wheelbase. Substituting six-axle semis for five-axle semis results in less fatigue damage for very short spans, but more fatigue damage for all longer spans because of the higher PMGW's allowed with the six-axle semi.

Table 65. Relative fatigue damage for various truck types.

Type	TTI L=30	TTI L=60	TTI L=90	TTI L=180	CCNY L=30	CCNY L=60	CCNY L=90	CCNY L=180	CCNY/TTI
SU2	0.211	0.437	0.244	0.122	0.211	0.437	0.244	0.122	1.000
SU3	1.357	3.022	1.681	0.852	1.357	3.022	1.681	0.852	1.000
ST3	0.182	0.290	0.268	0.220	0.182	0.290	0.268	0.220	1.000
ST4B	0.330	0.767	0.667	0.535	0.330	0.767	0.667	0.535	1.000
ST5B	0.855	1.457	1.381	1.146	0.855	1.457	1.381	1.146	1.000
TW5B	0.228	1.056	1.287	1.157	0.228	1.056	1.287	1.157	1.000
SU4	2.121	5.352	2.965	1.497	2.211	5.581	3.092	1.561	1.043
SU4S	1.121	4.767	3.279	1.974	1.193	5.072	3.489	2.100	1.064
ST4A	0.352	0.413	0.376	0.391	0.352	0.413	0.376	0.391	1.000
ST5A	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
ST6	0.386	1.706	1.734	1.405	0.570	2.524	2.565	2.079	1.479
ST5S	1.507	1.360	1.369	1.330	1.507	1.360	1.369	1.330	1.000
TW5A	0.270	1.132	1.430	1.323	0.270	1.132	1.430	1.323	1.000
TW6	0.256	1.199	1.497	1.419	0.338	1.582	1.975	1.872	1.319
TW7	0.263	1.218	1.542	1.451	0.438	2.030	2.570	2.419	1.667
TW8	0.200	1.292	1.606	1.468	0.376	2.434	3.027	2.766	1.884
WD5	0.528	0.631	0.519	0.853	0.528	0.631	0.519	0.853	1.000
WD6	0.565	0.782	0.686	1.175	0.565	0.782	0.686	1.175	1.000
WD7	0.496	0.748	0.697	1.202	0.623	0.940	0.876	1.509	1.256
WD8	0.398	0.822	0.751	1.234	0.644	1.330	1.215	1.995	1.617
WD9	0.360	0.887	0.719	1.211	0.665	1.639	1.328	2.236	1.847
TD5	0.772	0.360	0.336	0.782	0.772	0.360	0.336	0.782	1.000
TD6	0.985	0.436	0.423	0.973	0.985	0.436	0.423	0.973	1.000
TD7	0.992	0.494	0.510	1.168	1.025	0.510	0.526	1.206	1.033
TD8	0.931	0.467	0.521	1.190	1.203	0.603	0.673	1.537	1.292
TD9	0.572	0.451	0.524	1.185	0.834	0.657	0.763	1.728	1.458
TP7	0.170	0.500	0.680	1.203	0.245	0.723	0.985	1.741	1.447
TP8	0.151	0.530	0.703	1.225	0.279	0.981	1.303	2.270	1.852
TP9	0.145	0.552	0.702	1.229	0.272	1.033	1.315	2.301	1.873

Relative Fatigue Damage = damage for this truck/damage for ST5A
TTI = TTI truck-weight formula without a GVW cap
CCNY = CCNY truck-weight formula without a GVW cap
L = span length in feet

Substituting twins for five-axle semis also results in more fatigue damage except for very short spans; again, this is due to the higher PMGW's for the twins. By spreading the weight over a longer wheelbase the western doubles extend the range of short spans for which the fatigue damage is reduced. By spreading the weight even more, turnpike doubles further extend this range of spans for which the fatigue damage is reduced. Triples also reduce fatigue damage within a range of short spans, but increase fatigue damage on longer spans. Compared with doubles, they benefit from longer wheelbases but suffer from larger PMGW's.

For a given truck configuration (TW, WD, etc.), the relative fatigue damage generally increases with the number of axles if higher PMGW's are permitted because of the additional axles. For a given truck type (TW6, TW7, ST6, etc.), the relative fatigue damage usually increases with the span length because the effects of higher PMGW's is greater on longer spans.

For 15 out of 23 post-STAA truck types, the proposed formula permits higher PMGW's than the TTI formula. For these types, the fatigue damage associated with the proposed formula ranges from 1.03 to 1.88 times that associated with the TTI formula because the reduction in trips per freight hauled for the proposed trucks does not fully compensate for the detrimental effects of their higher W_e values. For 7 out of 23 post-STAA truck types, the PMGW's and fatigue damage are the same for the two formulas.

Traffic Scenarios

Calculation Procedures. The relative fatigue damage for various traffic scenarios compared with scenario A, which represents pre-STAA traffic, is given in table 66. These values were calculated by first multiplying the damage factor, D, for each truck type in the scenario by the fraction of this type in the traffic. Then the resulting weighted damage factors were summed to get an overall damage factor for the scenario. The relative damage for a particular scenario is equal to this overall damage factor times the relative volume for that scenario divided by the overall damage factor for scenario A. The relative volume for scenario A is 1.0.

Table 66. Relative fatigue damage for various scenarios.

Span	Weight Formula	Relative Volume B	C	D	E	F	G	Relative Damage B	C	D	E	F	G
30	TTI	94.69	93.93	92.59	91.41	92.27	90.52	1.026	0.949	0.952	1.011	0.901	0.916
	CCNY	94.69	88.25	85.84	85.74	84.79	83.82	1.026	0.913	0.945	1.039	0.862	0.957
60	TTI	94.69	93.93	92.59	91.41	92.27	90.52	0.699	0.968	0.939	0.898	0.941	0.898
	CCNY	94.69	88.25	85.84	85.74	84.79	83.82	0.699	1.287	1.211	1.132	1.217	1.153
90	TTI	94.69	93.93	92.59	91.41	92.27	90.52	0.745	0.990	0.934	0.897	0.964	0.899
	CCNY	94.69	88.25	85.84	85.74	84.79	83.82	0.745	1.408	1.248	1.189	1.327	1.203
180	TTI	94.69	93.93	92.59	91.41	92.27	90.52	0.871	1.053	1.036	1.032	1.063	1.044
	CCNY	94.69	88.25	85.84	85.74	84.79	83.82	0.871	1.491	1.439	1.380	1.541	1.454

Scenario A: pre-STAA traffic; base scenario
Scenario B: post-STAA traffic
Scenario C: post-STAA traffic without GVW cap
Scenario D: post-STAA traffic plus western doubles; without GVW cap
Scenario E: post-STAA traffic plus turnpike doubles; without GVW cap
Scenario F: post-STAA traffic plus triples; without GVW cap
Scenario D: post-STAA traffic plus western doubles; without GVW cap
Scenario G: post-STAA traffic plus doubles and triples; without GVW cap
Relative Volume = truck volume for a scenario as a percentage of truck volume for Scenario A
Relative Damage = fatigue damage for a scenario divided by fatigue damage for Scenario A

The relative volume accounts for the different number of trips required to haul a given amount of freight with a particular scenario compared with that required to haul the same freight with scenario A. Values of relative volume for the scenarios are listed in the table; they reflect the T values of the individual truck types in the scenario as discussed earlier. The values of D were calculated from the values of W_e, S, and C in table 64 with one exception. The W_e values used for the TW5A and TW6 twins in scenario B are based on the present 80-kip (356 kN) GVW cap as discussed earlier.

Results. Some general trends are apparent from the data in table 66. Except on very short spans, the post-STAA traffic (scenario B) causes less fatigue damage than the pre-STAA traffic. This is because some of the semis have been replaced by twins, which have a longer wheelbase but only a slightly higher (80 vs. 78) PMGW because of the 80-kip (356 kN) GVW cap. The other scenarios, which are controlled by more liberal weight regulations, result in less fatigue damage than scenario A over a range of short spans, but more damage for longer spans. This is because most of the new truck types have higher PMGW's than the truck types they replace. Except on some very short spans, the proposed truck-weight formula results in more fatigue damage than the TTI formula because it allows higher PMGW's for many truck types.

Plots of the relative fatigue damage for scenarios C and G compared with scenario B are given in figures 17 and 18, respectively. They show how the relative fatigue damage varies with the span length for the two truck-weight formulas. They also show that the relative damage is greater for the proposed formula over most of this range. Table 67 shows the percentage of the fatigue damage done by the various truck types in scenarios C and G. Most of the damage is done by the five and six-axle semis and by the twins.

Table 67. Percentage of fatigue damage caused by various truck types.

	Scenario C								Scenario G							
	TTI				CCNY				TTI				CCNY			
Type	30	60	90	180	30	60	90	180	30	60	90	180	30	60	90	180
SU2	1.07	1.21	0.72	0.41	1.11	0.90	0.50	0.29	1.11	1.30	0.79	0.42	1.06	1.00	0.56	0.30
SU3	6.63	8.05	4.76	2.78	6.86	5.97	3.30	1.94	6.88	8.68	5.25	2.80	6.54	6.67	3.74	2.02
ST3	0.66	0.57	0.56	0.53	0.68	0.43	0.39	0.37	0.68	0.62	0.62	0.54	0.65	0.48	0.44	0.39
ST4B	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ST5B	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
TW5B	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SU4	8.02	11.03	6.49	3.78	8.65	8.54	4.70	2.75	8.32	11.90	7.17	3.81	8.25	9.53	5.32	2.86
SU4S	1.51	3.51	2.56	1.78	1.67	2.77	1.89	1.32	1.57	3.78	2.83	1.79	1.59	3.09	2.14	1.37
ST4A	3.62	2.31	2.24	2.68	2.67	1.22	1.11	1.33	2.86	1.90	1.89	2.06	1.96	1.05	0.97	1.07
ST5A	61.95	33.75	35.86	41.33	44.33	17.34	17.23	19.96	47.60	26.98	29.34	30.91	31.69	14.51	14.62	15.58
ST6	9.82	23.67	25.55	23.86	18.04	31.23	31.52	29.59	10.18	25.53	28.21	24.08	17.20	34.86	35.68	30.83
TW5A	4.54	10.36	13.90	14.82	3.04	4.98	6.25	6.70	1.75	4.15	5.70	5.56	1.23	2.37	3.01	2.97
TW6	2.17	5.54	7.35	8.03	3.80	6.96	8.64	9.49	1.66	4.40	5.97	5.97	1.54	3.31	4.16	4.20
TW7	0.00	0.00	0.00	0.00	4.93	8.94	11.24	12.25	0.27	0.71	0.98	0.97	2.00	4.24	5.41	5.43
TW8	0.00	0.00	0.00	0.00	4.23	10.71	13.24	14.01	0.21	0.77	1.04	1.00	1.72	5.09	6.38	6.21
WD5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.87	1.95	1.74	3.02	1.50	0.82	0.68	1.19
WD6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.08	2.41	2.30	4.15	1.60	1.02	0.90	1.64
WD7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.83	1.22	1.15	2.11
WD8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.83	1.73	1.59	2.79
WD9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.89	2.13	1.74	3.12
TD5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	4.20	1.11	1.13	2.76	2.19	0.47	0.44	1.09
TD6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.36	1.34	1.42	3.44	2.80	0.57	0.55	1.36
TD7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.91	0.66	3.94	0.13
TD8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.41	0.78	0.88	2.15
TD9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.37	0.85	1.00	2.41

Scenario C: post-STAA traffic without GVW cap
Scenario G: post-STAA traffic plus twins, doubles, and triples; without GVW cap

ANALYSIS OF ACTUAL BRIDGES

General Procedures

Bridges. A fatigue analysis was made for each of the seven steel bridges studied in chapter 4; a detailed description of the bridges is given in that chapter. Most of the bridges are continuous-span WF stringer bridges, and most of the beams have partial or full-length coverplates. One of the bridges is a riveted two-girder, continuous-span bridge and another utilizes simple-span rolled beams. Most of the bridges are composite, at least in the positive-bending regions. The bridges have been in service for periods ranging from 2 to 39 years.

Calculation Procedures. A list of 82 fatigue details, and pertinent data for each, was obtained from the static analysis described in chapter 4. This list was screened to determine the most critical detail of each type (fatigue category) listed for each bridge. These critical details were then analyzed in two steps. First, the effective stress range was calculated and compared with the limiting stress range for infinite life. Then, if the effective stress range significantly exceeded that limiting stress range, the remaining fatigue life was calculated. The specific calculation procedures are based on those developed in NCHRP Project 12-28(3) and utilized in AASHTO fatigue design and evaluation procedures.[60,11]

Truck Traffic. In the analysis, it was assumed that scenario A was applied to the bridge from the time it was put into service to the present, and that either scenario A or C will be applied in the future. No other scenarios were investigated. In applying scenario C, however, it was assumed that either (a) the TTI truck-weight formula or (b) the the proposed truck-weight formula will be in effect.

It was also assumed that an annual traffic-volume growth rate of 3 percent will occur at each bridge. The present truck traffic volumes were estimated from available information whenever possible and assumed at reasonable values when not. Since the purpose of the study is to show the effects of the different truck-weight formulas, it is not essential to know the actual truck volume precisely.

Effective Stress Range

Bridge Parameters. Table 68 summarizes the effective- stress-range calculations for 15 details selected from the original list of 82. For each detail, the stress range, S_{ru}, caused by the passage of a 100-kip (444.8 kN) concentrated load over the bridge was calculated from the unit-load moment range and section moduli from the static analysis. In this calculation, all moment was assumed to be applied to one beam and the portions of the stress range in positive and negative bending were treated separately as required in the evaluation procedures.[11,60] Details with the highest S_{ru} values were selected in the screening. Details at which the compressive dead load stress (at the expected crack initiation location) was sufficient to counteract the tensile portion of the stress range were excluded in line with the fatigue evaluation procedures.[11,60]

Factors determined in accordance with the evaluation procedures were then applied to the S_{ru} values to account for the effects of (a) lateral distribution, (b) impact, and (c) beneficial effects not normally included in static design. These factors are intended to represent "average" effects that influence fatigue rather than the extreme effects considered in static design.[11,60] The factors, and the value of S_{ru}, depend only on the characteristics of the bridge and not on the truck traffic scenario being applied to the bridge.

The lateral distribution factor, LDF, for multibeam bridges depends on the beam spacing and distance between points of contraflexure; it is much smaller than the factor normally used in static design.[11,60] For the two-girder bridge number 7, the lateral distribution was determined by simple-beam action (in the lateral direction) with load placed at the center of the outer traffic lane in accordance with the evaluation procedures.[11,60] The impact factor, IF, of 1.10 suggested for the evaluation of existing bridges was used in all cases.[11] A section modulus factor, SMF, of 1.15 was applied to account for the beneficial effects not normally included

Table 68. Fatigue analysis of actual bridges - Effective stress range.

	Bridge		Detail							SF			Sre				
Case	No.	Type	No.	Type	Span	Sru	LDF	IF	SMF	SA	SCT	SCC	SA	SCT	SCC	Srl	Srl/Rs
1	1	CRB	4	E'	2	15.1	0.411	1.10	1.15	37.7	39.1	43.8	2.24	2.32	2.60	0.9	0.67
2			6	C	1	8.1	0.411	1.10	1.15	56.1	58.6	66.4	1.80	1.87	2.12	3.7	2.74
3			11	Cs	2	15.3	0.411	1.10	1.15	23.6	24.1	27.0	1.42	1.46	1.63	4.4	3.26
4	2	CRB	3	B	1	26.1	0.457	1.10	1.15	31.4	32.0	36.4	3.59	3.65	4.16	5.9	4.37
5			4	Cs	1	23.3	0.457	1.10	1.15	31.4	32.0	36.4	3.21	3.26	3.71	4.4	3.26
6			5	C	1	8.8	0.464	1.10	1.15	40.7	42.6	48.3	1.59	1.66	1.88	3.7	2.74
7	3	SRB	22	B	4	19.2	0.383	1.10	1.15	29.6	30.5	33.8	2.08	2.14	2.38	5.9	4.37
8			23	Cs	4	19.2	0.383	1.10	1.15	29.6	30.5	33.8	2.08	2.14	2.38	4.4	3.26
9	4	CRB	6	A	1	29.4	0.349	1.10	1.15	23.0	23.1	25.3	2.26	2.27	2.49	8.8	6.52
10	5	CWG	32	C	7	8.7	0.397	1.10	1.15	58.4	60.8	69.8	1.94	2.02	2.32	3.7	2.74
11			35	Cs	7	33.2	0.396	1.10	1.15	35.3	36.1	41.3	4.44	4.54	5.20	4.4	3.26
12			36	B	7	35.7	0.396	1.10	1.15	35.3	36.1	41.3	4.78	4.88	5.59	5.9	4.37
13	6	CWG	12	B	4	13.6	0.461	1.10	1.15	30.1	30.6	34.8	1.81	1.84	2.09	5.9	4.37
14			13	Cs	4	13.2	0.461	1.10	1.15	30.1	30.6	34.8	1.75	1.79	2.03	4.4	3.26
15	7	CRG	1	D	1	7.0	0.976	1.10	1.15	29.4	30.3	34.6	1.91	1.97	2.25	2.6	1.49

All values in kips and inches.
Sru = stress range for a 100-kip concentrated load applied to one girder
Sre = effective stress range for a scenario
Srl = variable-amplitude fatigue-limit stress range
Rs = reliability factor; 1.35 for redundant members and 1.75 for nonredundant members
LDF = lateral distribution factor
IF = impact factor
SMF = section modulus factor
SF = scenario factor; "average" WeS for a scenario
SA = Scenario A
SCT = Scenario C(TTI)
SCC = Scenario C(CCNY)
Cs = Category C for transverse stiffeners
SRB = simple-span rolled-beam bridge
CRB = continuous-span rolled-beam bridge
CWG = continuous-span welded-girder bridge
CWG = continuous-span riveted-girder bridge
Sre = (Sru)(SF)(LDF)(IF)/(SF)/100

in static design; this factor increases the effective section modulus and decreases the stress range.[11,60]

Scenario Parameters. Three different scenario factors, SF, were applied to get the effective stress ranges corresponding to the three scenarios being considered: scenario A, scenario C (TTI) and scenario C (proposed truck weight formula (CCNY). Each scenario factor accounts for the cumulative effects of the different truck types in the scenario. Specifically, it accounts for the different effective weights, We, and stress range ratios, S. Both factors were discussed earlier. The correct moment range for a given truck type was obtained by multiplying the unit-load moment range, $S_{ru}/100$, by W_eS.

The scenario factor, then, is given by:

$$SF = \left(\sum W_{ei}^3 S_i^3 \alpha_i\right)^{\frac{1}{3}} \tag{35}$$

in which W_{ei} and S_i are the effective weight and stress range ratio, respectively, for truck type i, and α_i is the fraction of that type in the truck traffic. Thus, SF is similar to the effective spectrum weight defined by equation 32, but accounts for differences in axle spacings as well as gross weights.

The scenario factor depends on (a) the span length, (b) the location along the span, and (c) whether the bridge is simple or continuous. SF values for different values of these parameters were calculated from the data in previous tables and are listed in table 69. The SF values used in table 68 were obtained from table 69 by interpolation.

The values of the effective stress range, S_{re}, were determined by combining the various factors in the following equation:

$$S_{re} = \frac{(S_{ru})(SF)(LDF)(IF)}{100\ (SMF)} \tag{36}$$

Table 69. Scenario factors.

Span Type	Scenario	Span Length	.1L	.2L	.3L	.4L	.5L	.6L	.7L	.8L	.9L	1.0L
			Location Along Span									
SS	A	30	27.1	26.1	24.9	24.0	23.4	24.0	24.9	26.1	27.1	
		60	32.1	30.9	30.4	30.0	29.4	30.0	31.2	30.9	32.1	
		90	39.4	38.4	38.9	38.1	37.0	38.1	38.9	38.4	39.4	
		180	47.1	46.6	46.9	46.5	45.9	46.5	46.9	46.6	47.1	
	C(TTI)	30	26.9	25.8	24.6	23.7	23.0	23.7	24.6	25.8	26.9	
		60	33.5	32.2	31.8	31.2	30.3	31.2	31.8	32.3	33.5	
		90	40.2	39.3	39.5	38.6	37.5	38.6	39.5	39.3	40.2	
		180	49.3	48.8	48.9	48.4	47.9	48.4	48.9	48.9	49.3	
	C(CCNY)	30	28.4	27.2	25.9	25.0	24.3	25.0	25.9	27.2	28.4	
		60	37.7	35.8	35.4	34.8	33.6	34.8	35.4	35.9	37.7	
		90	45.6	44.7	44.8	43.7	42.8	43.7	44.8	44.7	45.6	
		180	56.4	55.9	56.0	55.5	55.0	55.5	56.0	56.0	56.4	
CS	A	30	34.5	16.9	18.0	18.8	19.2	19.1	19.2	20.3	26.7	53.1
		60	43.0	20.9	23.7	25.1	25.6	26.6	27.6	26.8	25.7	57.9
		90	46.4	26.3	29.9	31.6	32.6	34.1	35.2	34.9	33.1	55.5
		180	55.3	32.5	36.5	38.8	40.6	42.4	43.9	44.9	45.1	64.3
	C(TTI)	30	28.2	17.0	18.2	19.0	19.5	19.5	19.4	20.1	25.3	50.4
		60	45.2	21.6	23.9	25.2	26.1	27.1	27.8	27.6	27.5	61.8
		90	47.0	26.8	30.2	32.0	33.2	34.7	35.8	35.5	34.2	57.5
		180	57.8	33.8	37.9	40.4	42.3	44.1	45.7	46.6	46.4	67.6
	C(CCNY)	30	31.6	17.7	19.0	19.8	20.3	20.2	20.2	20.8	26.8	57.7
		60	50.2	23.8	26.6	28.1	29.1	30.5	31.3	30.7	31.0	68.0
		90	53.9	30.4	34.3	36.3	38.0	39.7	41.1	40.8	39.2	65.8
		180	66.5	38.8	43.4	46.3	48.6	50.6	52.4	53.4	53.1	78.0

Scenario Factors = cube root of SUM((Pi)(Wei)(Wei)(Wei)(Si)(Si)(Si))
Pi = fraction of truck type i in traffic
Wei = effective weight of truck type i
Si = stress range ratio for truck type i
The Scenario Factor is an "average" WeS value for the spectrum.

where S_{re} is the stress range, SF is the scenario factor, LDF is the distribution factor, IF is the impact factor and SMF is the section modulus factor. The cycles per truck passage, C, were not involved because the bridges were long enough so that C was essentially 1.0. The relative volume for the various scenarios does not influence the effective stress range, but is involved in the subsequent calculation of remaining fatigue life.

Results. The calculated effective stress ranges are listed in table 68. They are quite low, and are consistent with measured stresses in actual bridges.[24,77] For comparison, the limiting stress range, S_{rl}, divided by a reliability factor, R_s is also listed for each case. If S_{re} is less than S_{rl}/R_s, the fatigue life is expected to be infinite as explained in more detail elsewhere.[24,60,77]

The degree of certainty associated with the limiting value, S_{rl}/R_s, is controlled by R_s.[11] The design and evaluation procedures suggest R_s values of 1.35 and 1.75 for redundant and nonredundant members, respectively, and these values were used in the present analysis.[11,60] The 1.75 value applies only to case 15. With R_s=1.35, there is about a 97.7 percent probability that the actual limiting value will exceed S_{rl}/R_s; this approximates the reliability provided by the present AASHTO fatigue design specifications. With R_s=1.75, there is about a 99.9 percent probability that the actual limiting value will exceed S_{rl}/R_s.

As expected, S_{re} is less than S_{rl}/R_s for all three scenarios in many cases. For these cases, the fatigue life is expected to be infinite so differences in the truck-weight formulas have no effect.

Remaining Life

Calculation Procedures. The remaining safe fatigue lives were calculated for the few cases where S_{re} significantly exceeded S_{rl}/R_s. The important parameters used in the calculations, are given in table 70. Equations from appendix

Table 70. Fatigue analysis of actual bridges - Remaining safe life.

Case	Yp	ADTT Present	ADTT Limit	Detail	K	Rs	Future Scenario	Vr	Sre Past	Sre Future	Yt	Dp	Yl	Dl	Dp+Dl	Yf	Df
1	7	720	3600	E'	1.1	1.35	A	1.000	2.24	2.24	55.2	6.2	54.4	133.3	139.6	30.6	49.0
	7	720	3600	E'	1.1	1.35	C(TTI)	0.937	2.24	2.32	55.2	6.2	56.7	150.5	156.7	29.8	49.0
	7	720	3600	E'	1.1	1.35	C(CCNY)	0.883	2.24	2.60	55.2	6.2	58.7	214.6	220.8	24.5	49.0
11	2	360	1800	Cs	12.0	1.35	A	1.000	4.44	4.44	154.8	1.9	54.4	133.3	135.2	58.4	152.9
	2	360	1800	Cs	12.0	1.35	C(TTI)	0.937	4.44	4.54	154.8	1.9	56.7	144.8	146.7	58.2	152.9
	2	360	1800	Cs	12.0	1.35	C(CCNY)	0.883	4.44	5.20	154.8	1.9	58.7	220.5	222.4	48.8	152.9
12	2	360	1800	B	33.0	1.35	A	1.000	4.78	4.78	341.1	1.9	54.4	133.3	135.2	95.6	339.2
	2	360	1800	B	33.0	1.35	C(TTI)	0.937	4.78	4.88	341.1	1.9	56.7	144.1	146.0	93.3	339.2
	2	360	1800	B	33.0	1.35	C(CCNY)	0.883	4.78	5.59	341.1	1.9	58.7	219.5	221.4	73.6	339.2
15	35	2520	7200	D	6.0	1.75	A	1.000	1.91	1.91	63.8	21.5	35.5	61.9	83.4	27.7	42.3
	35	2520	7200	D	6.0	1.75	C(TTI)	0.937	1.91	1.97	63.8	21.5	37.7	70.2	91.7	27.2	42.3
	35	2520	7200	D	6.0	1.75	C(CCNY)	0.883	1.91	2.25	63.8	21.5	39.7	107.6	129.1	21.3	42.3

All lives and damage (Y and D) are in years and all stresses are in ksi.
The annual growth rate in traffic volume was assumed to be 3%.
K = detail constant
Rs = reliability factor
Vr = relative volume from table 10
Sre = effective stress range from table 13
Yt = total life based on present traffic conditions
Yp = life for past period; current age of the bridge
Yf = life for future period; remaining life
Yl = life to reach a limiting ADT of 20000 vehicles/hour/lane
Dp = damage for past period; converted to present traffic conditions
Df = damage for future period; converted to present traffic conditions
Dl = damage to reach a limiting ADT of 20000 vehicles/hour/lane

C of reference 11, and from reference 60, were used with some modifications in the calculations. The equations involve (a) projections of present traffic volumes backward and forward at a 3 percent growth rate and (b) ratios of the past and future S_{re} values. The R_s values mentioned previously were applied in all cases; specifically, the calculated S_{re} values were divided by R_s before they were used in the various equations.

The damage terms, D_p, D_f, and D_l express the fatigue damage occurring during a particular time period in terms of the number of years present traffic must be applied (without growth) to cause the same damage. Hence, the fatigue life is reached when the total damage equals Y_t, which is the total life under present traffic (without growth) conditions. Damage for the past period is less than the actual time for that period because the traffic volume was smaller at that time than it is now. Damage for the future period is greater than the actual time for that period because the traffic volume is greater and the truck weights are heavier (for scenario C).

As explained in the evaluation procedures, a limiting traffic volume of 20,000 vehicles/hour/lane was used to represent the maximum capacity of the highway.[11] Traffic was assumed to grow 3 percent annually until it reached that level and then remain constant. The limiting truck volume was calculated from this limiting traffic volume by applying factors to account for the fraction of trucks in the traffic and the fraction of these trucks that travel in the outer (most critical) lane.

The relative volumes, V_r, for scenarios C(TTI) and C(CCNY) were obtained from table 66; these values account for the number of trips required to haul a given amount of freight with scenario C compared with that required to haul the same freight with scenario A. This factor was incorporated into the calculations by multiplying it times the present volume to get the starting volume for the future period.

Results. The calculated remaining safe fatigue lives, Y_f, are listed in table 70. Since the suggested R_s values were applied, the remaining lives are referred to as safe lives and provide probabilities of about 97.7 percent and 99.9 percent that the actual life will exceed the calculated life for redundant and nonredundant

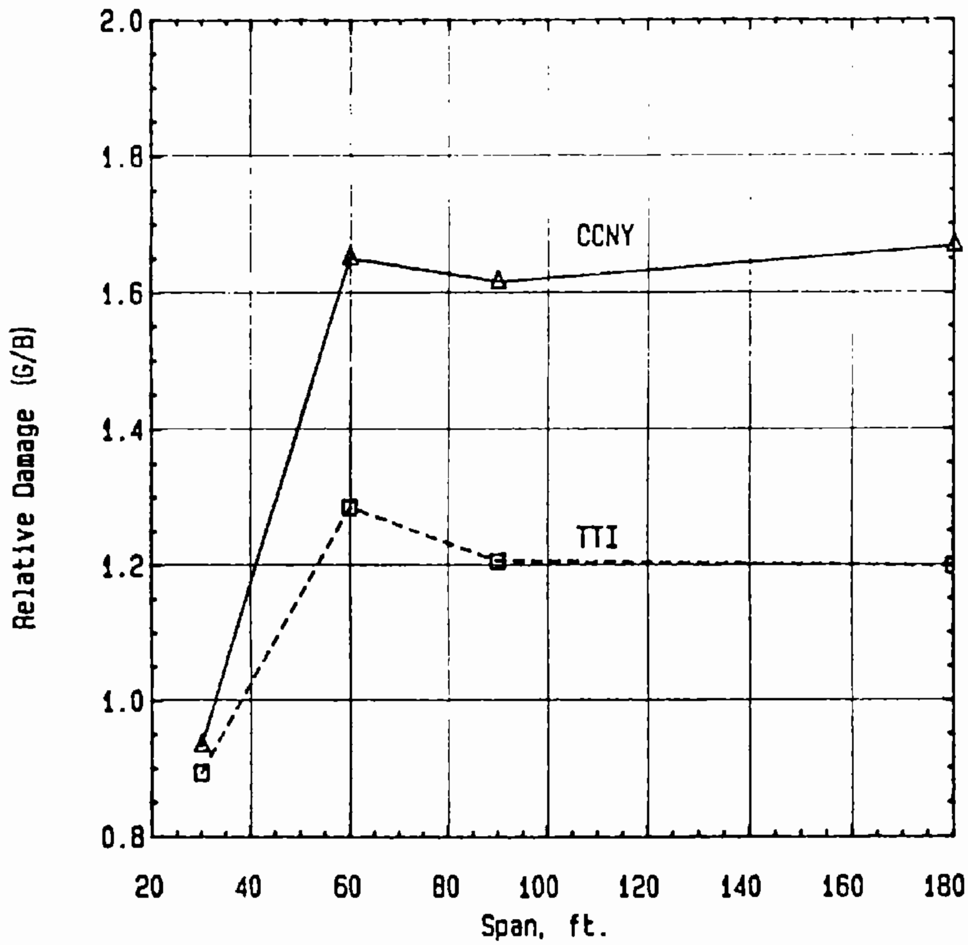


Figure 17. Relative fatigue damage (scenario G/scenario B).

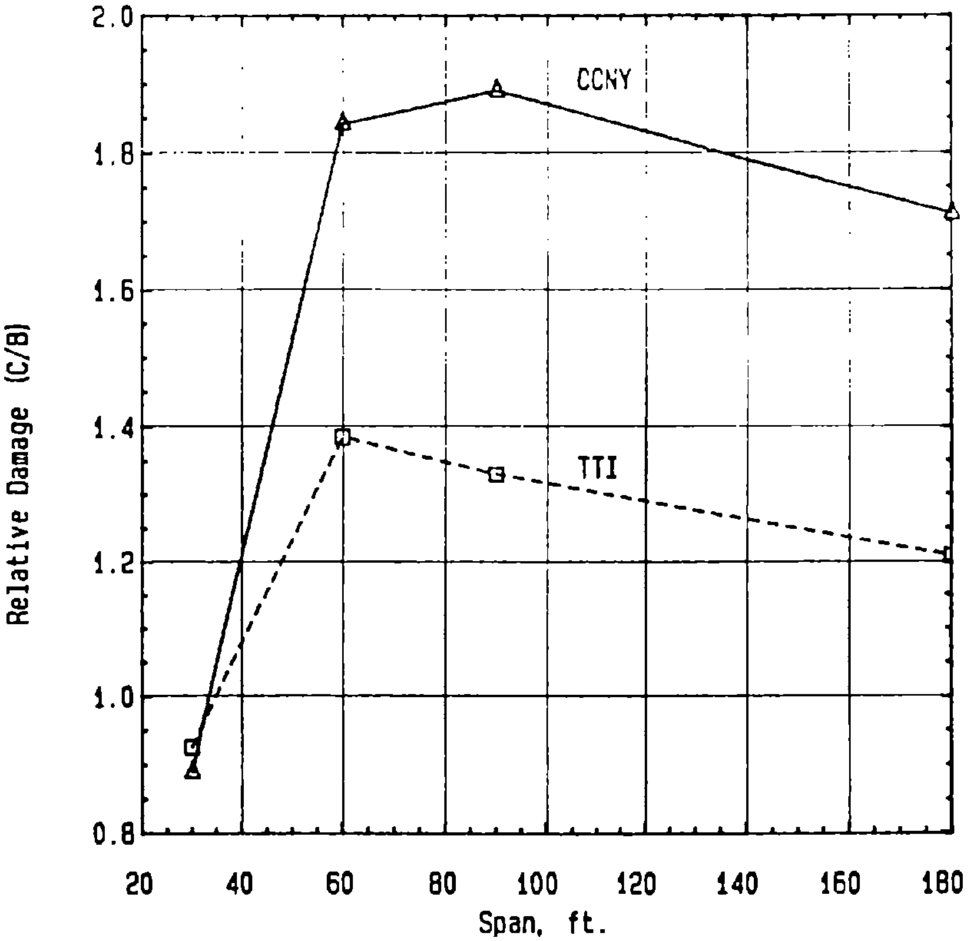


Figure 18. Relative fatigue damage (scenario C/scenario B).

members, respectively. As mentioned earlier, these are the average levels of safety provided by present AASHTO fatigue design specifications.[11] Mean life, which is the best estimate of the actual life, is about 5 times the safe life for redundant members and 10 times the safe life for nonredundant members.[11] These large differences result from the scatter inherent in fatigue data.

As expected from the previous calculations, the remaining safe lives for scenario C(TTI) were smaller than those for scenario A, and the remaining safe lives for scenario C(CCNY) were still smaller. In some of the cases, however, even these reduced lives were sufficient for practical requirements. For such cases, truck-weight formulas that permit heavier trucks do not cause fatigue problems. Furthermore, the remaining mean lives, which represent the best estimates of the remaining lives, are much greater; hence, fewer bridges would be affected by increases in legal truck weights if the mean lives were used as a basis of comparison. On the other hand, some bridges would be adversely affected by increases in truck weights and the number of these would increase if some of the other scenarios, which cause greater fatigue damage than scenario C, were applied.

CONCLUSIONS

The fatigue analysis in this chapter determined the relative fatigue damage caused by various new truck types and traffic scenarios that might result from changes in truck regulations, especially from the application of either the TTI or the proposed truck- weight formula without a gross weight cap but with the present axle load limits. The remaining safe fatigue lives were also calculated for several actual steel bridges to further evaluate the effects of truck regulation changes.

These relative-damage calculations showed that for many possible truck types and scenarios, fatigue damage would be increased by the possible changes. The calculated increases were generally greater for the the proposed formula, which permits heavier weights than the TTI formula for many truck types.

The calculations for actual bridges, and information from other sources, however, suggest that many existing bridges would not be affected by the possible truck regulation changes because the fatigue stresses in these bridges would be below the variable-amplitude fatigue limit. Even for bridges with fatigue stresses above the fatigue limit, the reduced fatigue lives with the new regulations may still be sufficient for practical requirements. Similarly, fatigue may not govern in the design of many new bridges even with the possible changes in regulations. Many other existing and new bridges, on the other hand, would be adversely affected by the increased fatigue damage. New bridges would require more material, and the lives of existing bridges would be reduced by significant amounts.

The influence of the fatigue limit on the practical effects of truck regulation changes cannot be generalized and can be accounted for only by analyzing individual bridges. Therefore, it would be very difficult to make precise quantitative estimates of nationwide costs from the present fatigue analysis, and such estimates are beyond the scope of the present project. Other investigators, however, have estimated the nationwide fatigue- related costs associated with certain truck regulation changes.[43,68,69] Assumed nationwide fatigue-related costs under present regulations were used as the starting point for these estimates.

CHAPTER SIX

CONCLUSIONS

A new truck weight formula that regulates the weight of heavy trucks and axle groups is developed based on rational safety criteria. The procedure utilizes a reliability analysis such that the projected truck load effect produces a uniform safety index for bridges designed according to current AASHTO criteria. The proposed formula is given as:

$$
\begin{aligned}
W &= (1.64\ B + 30)\ 1000 \qquad \text{for } B < 50 \text{ ft} \\
W &= (0.80\ B + 72)\ 1000 \qquad \text{for } B > 50 \text{ ft}
\end{aligned}
\tag{25}
$$

where W is the truck or axle group weight in pounds and B is the truck or axle group length in feet. The calibration of the proposed formula was executed to satisfy a safety index target of 2.5 accounting for possible growth in the truck traffic and truck weights with time. The proposed formula provides a uniform and rational approach to truck weight regulation with a uniform level of safety for all span lengths. The proposed formula is not sensitive to the assumed data base if the safety index criteria are changed accordingly. The 2.5 uniform safety index criteria used in this study is slightly more conservative than Moses' criteria used to develop load factors for the evaluation of capacity of existing bridges but is less conservative than the criteria suggested by Kulicki in the calibration of a new AASHTO bridge design code.[9,10]

If the proposed formula is adopted it will increase the number of bridge deficiencies. However, bridges that satisfy current AASHTO inventory stress ratings will not be affected. The expected number of bridge deficiencies is uniformly spread for all span lengths. The change in the expected number of deficiencies that would be obtained if different criteria are adopted was also presented in order to provide a comparison between the expected costs of rehabilitation if different criteria were used in the development of the truck weight formula.

Twelve bridges were analyzed in detail to check the effect of changing the truck weight regulation on typical existing structures. These bridges included steel girders, T-beam and prestressed concrete beams, continuous and simple spans. The calculations indicated that if WSD operating stress criteria are used for the evaluation of these bridges, only one of them will need to be rehabilitated. Some of the bridges analyzed were deficient even under the HS-20 loading for WSD inventory stress ratings.

Rehabilitation costs are estimated if LFD and WSD criteria are used with inventory stresses for one vehicle and two vehicle in one lane for the rating of the 12 typical bridges. The assumption of one vehicle per lane produces three deficient bridges: two steel and one prestressed concrete bridge. The prestressed concrete bridge will have to be completely replaced. The steel bridges can be easily and cheaply upgraded by adding cover plates. If two vehicles are assumed to be in one lane, then the costs of upgrading the steel bridges will be much higher because most will need to have stronger sections at the supports; to strengthen the beams over the supports requires removal of the deck.

A 3-D finite element analysis performed on two steel bridges showed that some secondary members might be overstressed if the proposed truck weight formula is adopted. This should not affect the safety of the bridges since the slab should able to redistribute the loads efficiently even after the loss of the some of the diaphragms. The finite element analysis however indicated that some of the external beams might also be overstressed under certain extreme loading conditions when four vehicles cluster on one side of the bridge.

The fatigue analysis determined the relative fatigue damage caused by various new truck types and traffic scenarios that might result from changes in truck regulations. The relative-damage calculations showed that for many possible truck types and scenarios fatigue damage would be increased by the possible changes. The fatigue calculations performed for actual bridges, however, suggest that many existing bridges would not be affected by the possible truck regulation changes because the fatigue stresses in these bridges would be below the variable-amplitude fatigue limit. Even for bridges with fatigue stresses above the fatigue limit, the reduced fatigue lives with the new regulations may still be sufficient for practical requirements.

REFERENCES

1. R.W. James, J.S. Noel, L.H. Furr and F.E. Bonilla, "Proposed New Truck Weight Formula," Publication No. FHWA/RD-85/088, Federal Highway Adminsitration, Washington, DC, June 1985.

2. Transportation Research Board, Truck Weight Limits: Issues and Options, Special Report 225, Washington, DC, 1990.

3. A.C. Agarwal, "Vehicle Weight Regulations Across Canada: A Technical Review with Respect to the Capacity of Highway Systems." Ontario Ministry of Transportation and Communications, RR214, Ontario, Canada, March 1978.

4. D.J., Harman, "Surveys of Commercial Vehicles Weights:1975 to 1982." Ontario Ministry of Transportation and Communications, RR236, Ontario, Canada, November 1985.

5. "Standard Specifications for Highway Bridges," 13th ed., AASHTO, Washington, DC, 1983.

6. R.W. James, J.S. Noel, L.H. Furr and F.E. Bonilla, "Proposed New Truck Weight Limit Formula", ASCE Journal of Structural Engineering, Vol. 112, No. 7, July 1986.

7. F. Moses and M. Ghosn, "Discussion on Proposed New Truck Weight Limit Formula", ASCE Journal of Structural Engineering, Discussion, November 1987.

8. P. Thoft-Christensen and M.J. Baker Structural Reliability Theory and Its Applications, Springer-Verlag, New York,1982.

9. J.M. Kulicki, "Development of Comprehensive Bridge Specification and Commentary", NCHRP 12-33, Transportation Research Board, Washington, DC, 1990.

10. F. Moses and D. Verma, "Load Capacity Evaluation of Existing Bridges," Report No. 301, Transportation Research Board, Washington, DC, December, 1987.

11. F. Moses, C.G. Schilling, and K.S. Raju, "Fatigue Evaluation Procedures for Steel Bridges," Report No. 299, Transportation Research Board, Washington, DC, November, 1987.

12. "Ontario Highway Bridge Design Code," and Commentary, Ontario Ministry of Transportation and Communications, Toronto, Ontario, Canada, 1983.

13. B. Ellingwood, T.V. Galambos, J.G. MacGregor and C.A. Cornell, "Development of a Probability Based Load Criteria for ANSI A58," National Bureau of Standards, NBS 577, Washington DC, June 1980.

14. P.F. Csagoly and R.A. Dorton, "Proposed Ontario BridgeDesign Load," RR 186, Ministry of Transportation and Communications, Ontario, Canada, November 1973.

15. A.S. Nowak and H.N. Grouni, "Safety Criteria in Calibration of the OHBD Code," Proc. Intl. Conf. on Short and Medium Span Bridges, Toronto, Canada, August 1982.

16. A.S. Nowak and J. Zhou, "Reliability Models for Bridge Analysis," Report No. UMCE 85-3, Department of Civil Engineering, University of Michigan, Ann Arbor, MI, 1985.

17. T.V. Galambos, et. al., Eight Collected Papers on Load and Resistance Factor Design, ASCE Journal of Structural Engineering, V. 104, ST 9, September 1978.

18. T.V. Galambos, R.T. Leon, C.W. French, M. Barker and B. Dishongh, "Inelastic Rating Procedures for Steel Beam and Girder Bridges," Final Report, NCHRP project 12-28(12), Transportation Research Board, Washington, DC, September 1990.

19. "Guide Specification for Alternate Load Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections," AASHTO, Washington, DC,1985.

20. F. Moses and M. Ghosn, "Instrumentation for Weighing Trucks-In-Motion for Highway Bridge Loads", Publication No. FHWA/OH-83/001, Ohio Department of Transportation, Columbus, OH, August 1983.

21. M. Ghosn, F. Moses and J. Gobieski, "Evaluation of Steel Bridges Using In-Service Testing," Transportation Research Records, TRR 1072, pp. 71-78, December 1986.

REFERENCES (CONTINUED)

22. P. Albrecht and K. Yamada, "Simulation of Service Fatigue Loads for Short-Span Highway Bridges," STP671, ASTM, 1979.

23. J.W. Fisher, et. al., "Fatigue Strength of Steel Beams with Welded Stiffeners and Attachments," NCHRP Report No.147, Transportation Research Board, Washington, DC, 1974.

24. C.G. Schilling, "Variable Amplitude Load Fatigue; Volume I - Traffic Loading and Bridge Response," Publication No. DTFH61-86-C-00036-I, Federal Highway Administration, Washington, DC, March 1987.

25. C. Kostem, "Overloading of Highway Bridges a Parametric Study," Fritz Engineering Laboratory, Report No. 378B.7, Lehigh University, Lehigh, PA, I976.

26. A. Wegmuller, "Overload Behavior of Composite Steel-Concrete Bridges," Journal of the Structural Division, ASCE, No. ST9, Proc. Paper 13227, September 1977.

27. "State of the Art Report on Redundant Bridge Systems", ASCE Journal of Structural Engineering, Vol. 111, No. 12, Dec. 1985.

28. B.O. Kuzmanovic and M.R. Sanchez, "Lateral Distribution of Live Loads on Highway Bridges," ASCE Journal of Structural Engineering, Vol. 112, NO. 9, August 1986.

29. B. Bakht and F. Moses, "Lateral Distribution Factors for Highway Bridges," ASCE Journal of Structural Engineering, Vol. 114, No. 8, August 1988.

30. P.W. Botzler and J. Colville, "Continuous Composite Bridge Model Tests," Journal of The Structural Division, ASCE No. ST9, Proc. Paper 1484, September 1979.

31. E.G. Burdett and D.W. Goodpasture, "Comparison of Measured and Computed Ultimate Strengths of Four Bridges," Hiqhway Research Record, No. 382, 1972..

32. B. Bakht and P.F. Csagoly, "Bridge Testing", Structural Research Report SRR-79-10. Ministry of Transportation and Communications, Ontario, Canada, 1979.

33. B. Bakht and P.F. Csagoly, "Strength Reserves in Highway Bridges", First International Symposium on Transportation Safety, San Diego, July 11-13, 1979.

34. "Non-Destructive Load Testing for Bridge Evaluation and Rating", Final Draft report, NCHRP 12-28(13), Transportation Research Board, Washington, DC, July 1988.

35. E.G. Burdette, and D.W. and Goodpasture, "Correlation of Bridge Load Capacity Estimates with Test Data," NCHRP Report 306, Transportation Research Board, Washington, DC, June 1988.

36. K.K.P. Lee, D. Ho, and H.W. Chung, "Static and Dynamic Tests of Concrete Bridges," ASCE Journal of Structural Engineering, Vol. 113, No. 1, January 1987.

37. P.F. Csagoly and R.A. Dorton, "The Development of OntarioHighway Bridge Design Code," Transportation Research Record, No. 665, Vol. 2, 1978.

38. M. Ghosn and F. Moses, "Markov Renewal Model for Maximum Bridge Loading," ASCE Journal of Engineering Mechanics, Vol. 111, No. 9, September 1985.

39. M. Ghosn and F. Moses, "Bridge Load Modelling and Reliability Analysis," Case Western Reserve University, Cleveland, OH, 1984.

40. R.E. Snyder, G.E. Likins and F. Moses, "Loading Spectrum Experienced by Bridge Structures in the United States," Publication No. FHWA/RD-85/012, Federal Highway Administration, Washington, DC, February, 1985.

41. C.A. Cornell, D. Liu, et. al., "Development of Site Specific Load Models for Bridge Rating," Interim Report to NCHRP project 12-28(11), Transportation Research Board, Washington, DC, 1988.

42. C.G. Schilling, et al., "Fatigue of Welded Steel Bridge Members Under Variable-Amplitude Loadings," NCHRP Report 188, Transportation Research Board, 1978.

43. F. Moses, "Effects on Bridges of Alternative Truck Configurations and Weights," Report to TRB Truck Weight Study Committee, Transportation Research Board, Washington, DC, December 1988.

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

45. R.A. Imbsen, D.W. Liu, R.A. Schamber and R.V. Nutt, "Strength Evaluation of Existing Reinforced Concrete Bridges," NCHRP report 292, Transportation Research Board, Washington, DC, June 1987.

46. F. Moses and M. Ghosn, "A Comprehensive Study of Bridge Loads and Reliability," Publication No. FHWA/OH-85/005, Ohio Department of Transportation, Columbus, OH, January 1985.

47. "Manual for Maintenance Inspection of Bridges", AASHTO, Washington, DC, 1983.

48. A.C. Agarwal and P.F. Csagoly, "Evaluation and Posting of Bridges in Ontario," Transportation Research Records No. 664, TRB, 1978.

49. "An Investigation of Truck Size and Weight Limits: Final Report," Office of the Secretary of Transportation, US Department of Transportation, Washington, DC, August 1981.

50. "Bridge Weight-Limit Posting Practice," NCHRP Synthesis 108, Transportation Research Board, Washington, DC, August 1984.

51. R.C. Cassano and R.J. LeBeau, "Correlating Bridge Design Practice with Overload Permit Policy," Transportation Research Records No. 664, TRB, 1978.

52. A.M. Clayton and F.P. Nix, "Effects of Weight and Dimension Regulations: Evidence from Canada," Transportation Research Record No.1061, TRB, 1986.

53. P.F. Csagoly and R.A. Dorton, "Truck Weights and Bridge Design Loads in Canada," Document 79-SRR-12, Ontario Ministry of Transportation and Communications, Downsview, Ontario, Canada, October 1979.

54. R.D. Desrosiers, "The Development of a Technique for Determining the Magnitude and Frequency of Truck Loadings on Bridges," Civil Engineering Department, University of Maryland, College Park, MD, April 1969.

55. "Effect of Truck Weights on Deterioration, Operations, and Design of Bridges and Pavements," Byrd, Tallamy, MacDonald and Lewis, Fall Church, VA, November 1987.

56. J.P. Eicher, "National Truck Weight Regulations," TR News, TRB, January-February, 1988.

57. Eicher, J. P., Klimek, T. E., & Strickland, S. G., "National Network for Trucks: Development, Performance, and Outlook," Transportation Research Record 1052, TRB, 1986.

58. C.F. Galambos and W.L. Armstrong, "Loading History of Highway Bridges," Highway Research Records No. 295, HRB, 1969.

59. M. Ghosn, "Bridge Overstress Criteria," Interim Report, Department of Civil Engineering, City College of New York, New York, NY, May, 1989.

60. "Guide Specifications for Fatigue Design of Steel Bridges," AASHTO, Washington, DC, 1989.

61. D.J. Harman and A.G. Davenport, "A Statistical Approach to Traffic Loading on Highway Bridges," Canadian Journal of Civil Engineering, Vol. 6, June, 1979.

62. C.P. Heins and R.C. Forbes, "Analysis Charts for Issuing Vehicle Permits," Transportation Research Records No. 507, TRB, 19

63. P.M. Kent and M.T. Robey, "1975-1979 National Truck Characteristics Report," Federal Highway Administration, Washington, DC, June 1981.

64. P.M. Kent and H. Bishop, "1974 National Truck Characteristics Report," Federal Highway Administration, Washington, DC, April, 1976.

65. R.A. Lill, "Geometric Design for Large Trucks - An Overview of the Issues from the Perspective of the American Trucking Association, Inc.," Transportation Research Records No. 1052, TRB, 1986.

66. J.W. March, "Findings of the Longer Combination Vehicle Study," Transportation Research Records No. 1052, TRB, 1986.

67. "Maximum Desirable Dimensions and Weights of Vehicles Operated on Federal-Aid Systems," House Document No. 354, U. S. Government Printing Office, Washington, DC, August 1964.

68. F. Moses, "Effect on Bridges of Alternative Truck Configurations and Weights," Second Interim Report, NCHRP Project HR2-16(b), Transportation Research Board, Washington, DC, May 1989.

69. F. Moses, "Effect on Bridges of Alternative Truck Configurations and Weights," Interim Report, NCHRP Project HR2-16(b), Transportation Research Board, Washington, DC, December1988.

70. "Motor Vehicle Size and Weight Regulations, Enforcement, and Permit Operations," NCHRP Synthesis 68, Transportation Research Board, Washington, DC, April 1980.

71. D.L. Neumann and P. Savage, "Truck Weight Case Study for the Highway Performance Monitoring System (HPMS)," Office of Highway Planning, Federal Highway Administration, Washington, DC, June 1982.

72. F.P. Nix and R. Schipizky, "The Effect of Vehicle Weight and Dimension Regulations," Proceeding of the Annual Conference of the Canadian Institute of Transportation Engineering, Ottawa, Ontario, Canada, 1984.

73. F.P. Nix, A.M. Clayton and B.G. Bisson, "Description and Analysis of Vehicle Weight and Dimension Regulations," Transportation Development Centre, Montreal, Quebec, Canada, 1985.

74. J.S. Noel, et. al., "Bridge Formula Development," Transportation Research Record No. 1072, TRB, 1986.

75. A.S. Nowak, A. S., snf Y.K. Hong, "Bridge Live Load Models," ASCE Journal of Structural Engineering, Vol. 117, 1991.

76. "Oversize-Overweight Permit Operation on State Highways," NCHRP Report 80, Transportation Research Board, Washington, DC, 1966.

77. C.G. Schilling, "Highway Structures Design Handbook, Chapter I/6 - Fatigue, Section I - Fatigue Loadings,"United States Steel Corporation, Pittsburgh, PA, January 1981.

78. R.E. Skinner, J. Morris and S. Godwin, "TRB's Study of Twin-Trailer Trucks," Transportation Research Record No. 1052, TRB, 1986.

79. H.K. Stephenson and A.A. Jakkula, "Highway Loads and Their Effect on Highway Structures Based on Traffic Data of 1942," Bulletin 116, Texas Engr. Experiment Station, Texas A & M College, College Station, TX, January, 1950.

80. H.K. Stephenson, et al., "Truck Weight Trends Related to Highway Structures," Bulletin No. 19, Texas Transportation Institute, College Station, TX, July, 1962.

81. J.R. Stowers, "Productivity Analysis for Truck Weight Study," submitted to the Transportation Research Board, Transportation Research Board, Washington, DC, September1989.

82. J.R. Stowers, et al., "Federal Truck Size and Weight Study," Transportation Research Record 920, TRB, Transportation Research Board, Washington, DC, 1983.

83. "Vehicle Classification and Truck Weight Survey: 1970-1985," New York Department of Transportation, Albany, NY, January 1986.

84. C.M. Walton and C. Yu, "Truck Size and Weight Enforcement: A Case Study," Transportation Research Records No. 920, TRB, 1983.

85. C.M. Walton, C. P. Yu and P. Ng, "Procedure for Assessing Truck Weight Shifts that Result from Changes in Legal Limits," Transportation Research Records No. 920, TRB, 1983.

86. R.E. Whiteside, et al., "Changes in Legal Vehicle Weights and Dimensions - Some Economic Effects on Highways," NCHRP Report 141, Transportation Research Board, Washington, DC, 1973.

87. R. Winfrey, P.D. Howell and P.M. Kent, "Truck Traffic Volume and Weight Data for 1971 and Their Evaluation," Report No. FHWA-RD-76-138, Federal Highway Administration, Washington, DC, December 1976.

88. C.P. Yu and C.M. Walton, "Characteristics of Double and Triple Trailer Truck Combinations Operating In the United States," Transportation Research Records No. 966, TRB, 1984.