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RATING BRIDGES BY NONDESTRUCTIVE PROOF-LOAD TESTING FOR CONSISTENT SAFETY

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16. Abstract

Because of the limitations of the analytical approach to rating bridges in the current AASHTO specifications, proof-load testing is often a desirable alternative rating method. The study reported here examined the feasibility of a proof-load testing program for New York, addressing both economic and technical aspects. Such a program with clearly defined application areas is found to be cost-effective. Two technical objectives are identified, based on review of current practice elsewhere: 1) prescription of the target proof load, and 2) development of a detailed procedure manual to guide typical proof-load tests. These tasks were completed and the results (including a draft text of the manual) are presented in this report.

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METRIC CONVERSION FACTORS

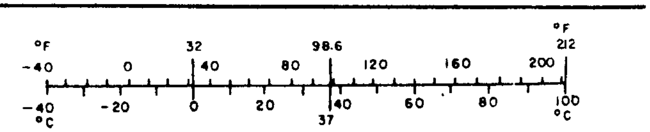
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in ≡ 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



CONTENTS

I. INTRODUCTION

A. Background

Highway bridge evaluation in the United States is currently guided by the 1983 AASHTO Manual for Maintenance Inspection of Bridges (1), referred to here as "the AASHTO Manual." The method specified is essentially analytical. In practice this analytical approach is often inapplicable or inadequate because 1) necessary information about the bridge is not always readily available (in the case of "unratable" bridges), or 2) resulting rating factors are lower than the required minimum of 1 (in the case of certain deficient bridges). In such cases, physical testing is often desirable to obtain a reliable rating. Proof load testing has been considered one of the preferred test methods, because the underlying concept is intuitively acceptable and results thus are conclusive (2).

New York State has about 17,000 bridges in its highway network. Limitations of the analytical approach in rating them have been observed in practice. A previous report (2) discussed bridge evaluation practice in New York State in some detail. Unratable and deficient bridges are two major groups presenting difficulties for analytical rating. A proof load test program has been proposed for rating these bridges, to meet federal requirements for bridge rating, and its feasibility in New York State is examined here.

B. Objective and Approaches

This study's objective was to examine feasibility of a proof load test program for highway bridge rating in New York, including both the economic and the technical aspects addressed here. The economic problem was to answer questions regarding cost effectiveness of such a program, and the technical problem was to address concerns about details in routine execution of proof load testing.

The technical aspect was addressed by examining details of available guidelines, and developing elements found missing in them for application in New York State. Two areas of needed research were identified: 1) prescribing target proof loads and using them in determining load ratings, and 2) developing a detailed procedure manual for proof load tests. The first subject was dealt with by using the criterion of a uniform structural safety level, consistent with current analytical rating practice. The second subject was addressed by writing a draft procedure manual to guide proof load testing practice, based in part on experience of the profession in this area.

The economic aspect was covered by an analysis describing such a program in terms of costs, benefits, and benefit cost ratios, to ensure cost effective investment and operation. This economic analysis was facilitated by technical findings earlier in the study, which suggested an outline for a proof load test program and steps for its implementation. Estimation over a life cycle was included in the economic analysis.

C. Organization of This Report

This report has five parts. Chapter II presents a brief review of current practice in proof load testing of civil structures, and identifies additional needed research for bridge rating in New York. Chapter III explains the prescription of target proof loads based on bridge safety criteria. Chapter IV introduces guidelines for bridge proof load testing, in a draft procedure manual included as Appendix A. This manual is recommended as a starting point for technical requirements in New York's proof load test program. It will be improved based on experience and data acquired in future operation. Chapter V presents results of the economic analysis, showing a proof load test program to be a viable approach in developing reliable bridge ratings, as required by the federal government. Chapter VI summarizes the findings and recommendations resulting from this study.

II. REVIEW OF CURRENT PRACTICE

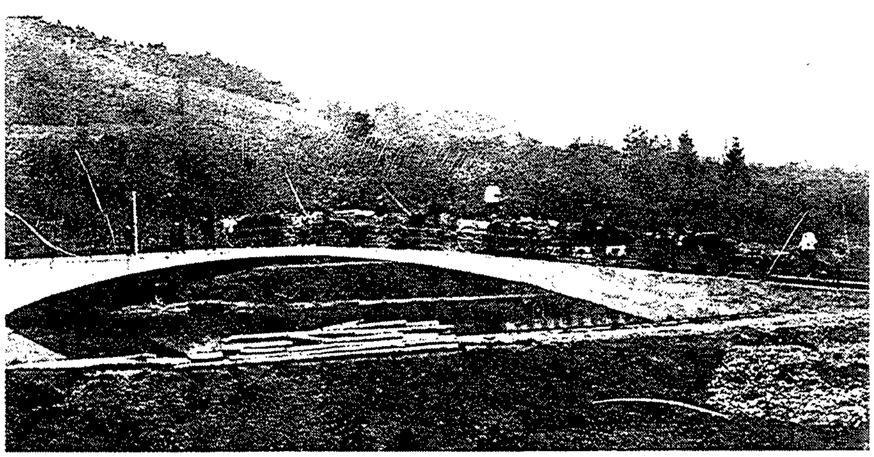
A. Proof-Load Test Practice

The concept of proof-load testing has been applied to various types of structure in many parts of the world. Originally, the test provided a means of verifying design assumptions and construction quality. Subsequently, it became an effective approach for assessing load-carrying capacity of existing structures. Its application to bridge structures also has become routine in several countries. The interested reader is referred to an unpublished NCHRP report (3) for detailed review of testing practice in the United States and elsewhere. Only a brief overview is presented here as an introduction to the technology available, supplementing the NCHRP report. This overview covers several European countries, Canada, Australia, New Zealand, and the United States.

Reinforced concrete bridges were load-tested in Switzerland as early as the late-19th century (4). Figure 1 shows one of the earliest bridge proof-load tests. The main goal of load testing then was to ensure that bridges would not collapse under service loads. Load tests on new bridges in Switzerland are now usually carried out as acceptance tests in fulfillment of code requirements (4). The elastic nature of the structure is judged mainly by recovery from deformations after unloading. Load tests are also required to quantify defects or weaknesses of existing structures, and may be used as acceptance tests after subsequent strengthening of defective elements.

The Italian building code requires that every bridge be proof-tested before opening to traffic (5,6). The intent is to verify conformance of as-built

Figure 1. Load test in 1890 on a reinforced-concrete bridge in Switzerland.



stiffness and resistance with superstructure design specifications and applicable standards. The structure is usually quasi-statically loaded with heavy trucks, arranged longitudinally to maximize load effects. Transversely, trucks are often placed with maximum eccentricity to check the structure's torsional stiffness and transverse redistribution of the load, and to produce stresses close to the design values at critical sections. Proof-load-tested bridges have included 12- to 40-m spans having reinforced-concrete slabs, I-beams, simply supported-box girders, and continuous box girders (5,6).

In Canada, by 1979 the Ontario Ministry of Transportation and Communications had successfully proof-tested over 200 bridges (7). Most possessed much higher load-carrying capacities than could be analytically predicted. Figure 2 shows loading trucks in Ontario. Note that the Ontario Highway Bridge Design Code (8) includes the following general provisions in Article 14-8 ("Load Testing") regarding physical testing of bridges:

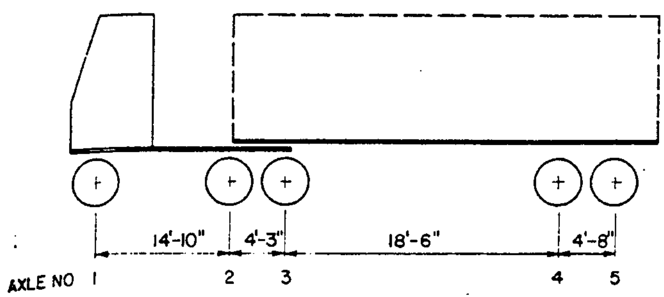
> Bridges shall be considered for load testing if analytical evaluations are deemed to be unsatisfactory. When a load test is proposed as part of the evaluation procedure, such a test, including details of loads, instrumentation, condition survey and analysis, is subject to approval.

Proof-load testing has been performed on concrete bridges in New Zealand since 1977 (9), and has been considered an economical method of raising or removing bridge weight restrictions, and thus deferring their replacement and saving the cost of long detours for heavy traffic. New Zealand Standard NZS 3101-1982 ["Code of Practice for the Design of Concrete Structures" (10)] includes detailed provisions to guide bridge load-testing for strength evaluation. New Zealand Standard NZS 4203-1976 ["Code of Practice for General Structural Design and Design Loading for Building" (11)] also contains a general clause regarding load testing for buildings.

By 1981, 94 bridges had been load tested in Britain (12). Only deflections were measured, except for a few cases where strains were also recorded. It was concluded that bridge load tests are beneficial when associated with inspection and analysis. Such testing has advanced engineering design and contributed to diagnosis and assessment of obsolete and distressed bridges. Note that British bridge testing standards include sections to guide bridge evaluation by load testing (13).

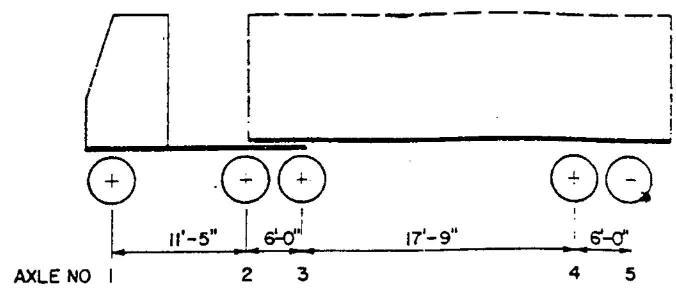
In the United States, Florida has the most advanced program of proof-load testing (14). Figure 3 shows the configuration and appearance of Florida loading trucks. A principal purpose of their load test program is to examine structures of questionable strengths. It has been estimated that about 85 percent of bridges with load restrictions in Florida actually have adequate load-carrying capacities and consequently do not need posting or replacement. Testing usually commences with a site survey to examine feasibility of such work. Plans and details of instrumentation are then established, and theoretical analysis is also performed. If test results at each load increment compare favorably with theoretical predictions and no apparent distress is observed, the bridge is accepted as safe.

Figure 2. Configuration and appearance during test of loading trucks of the Ontario Ministry of Transportation and Communications.

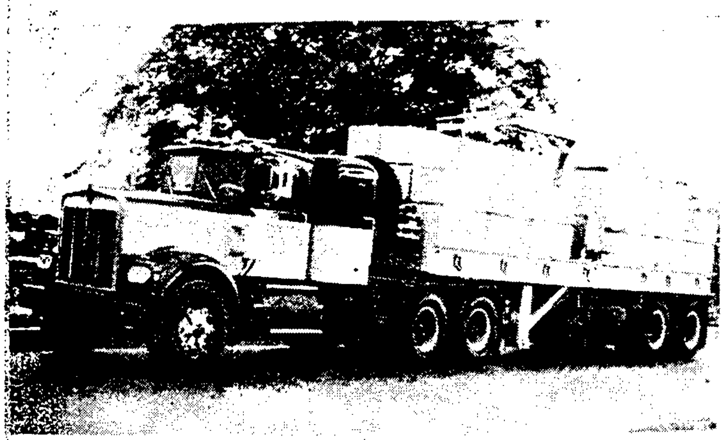


1 ft. = 0.3048 m
1 in. = 2.54 cm

Test Vehicle 1



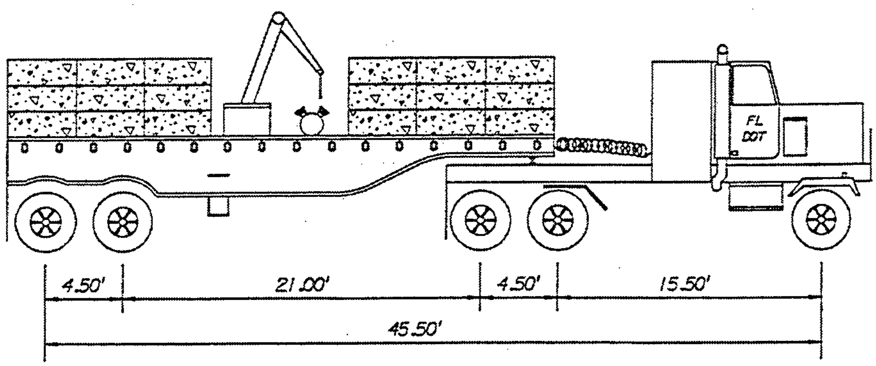
Test Vehicle 2



Test Vehicle 1

Test Vehicle 2

Figure 3. Configuration and appearance during test of loading trucks of the Florida Department of Transportation.



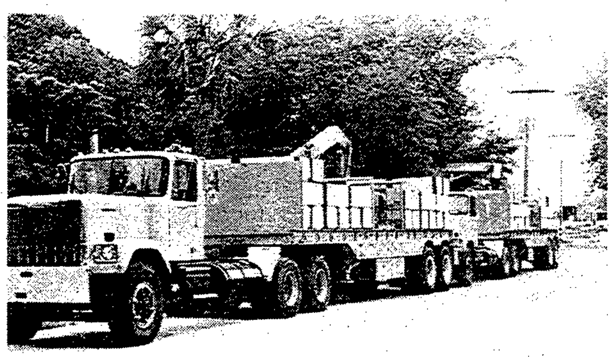
WEIGHTS

72 ballast blocks	154,800 lb.
Equipment	8,200 lb.
Trailer	24,000 lb.
Tractor	17,000 lb.
Total	204,000 lb.

LOAD TRANSFER

5th wheel	82,350 lb.
Steering axle	15,630 lb.
Drive tandem	83,720 lb.
Trailer tandem	104,650 lb.

Note: All weights and dimensions are approximate and for information only.



Although significant experience does exist, well-documented procedures for bridge rating by proof testing were not found. Thus, development of a detailed procedure manual was identified as a major task for the present study. This manual is intended to include experience of others in meeting New York's specific needs.

For the countries and jurisdictions discussed, differences in procedures for proof-load testing on structures were noted: 1) load configuration (mechanisms, location, cycles, durations, etc.), 2) instrumentation type, 3) evaluation criteria, and 4) requirements for load intensity. Load configuration is often determined to maximize structural response and facilitate load application during testing. Instrumentation type depends on practicality of measurement and evaluation criteria. Reasons for these differences are either well explained or may be intuitively understood. On the other hand, required intensities of load are critically important and their basis has not been well documented, which is also the case for safety factors adopted in many design codes for civil structures. This issue will now be discussed separately.

B. Target Proof-Load Requirements in Codes and Guidance Manuals

Nondestructive proof-load testing examines the structure's capability to perform intended service functions. A clearly defined target proof load thus should be decided before testing, along with a set of evaluation criteria for partial or full acceptance. This section briefly reviews specifications for target load given by codes and "guidance" manuals (the latter not intended to be mandatory) of various countries, for an overview of current practice. Table 1 compares target proof-load levels given by these specifications. Table 2 continues this comparison for corresponding acceptance criteria. For reference purposes, codes and manuals for general structures are also noted.

Table 1 shows that many design/evaluation codes or guidance manuals permit proof-load testing on civil structures constructed of steel, reinforced concrete, and prestressed concrete (8,10,13,15,16,17,18). ACI Code 318 for concrete buildings (21) also includes such provisions, although the AASHTO codes for bridges (1,19) do not. Note that provisions vary regarding target proof load. Some codes do not definitely specify such a load, although proof load testing is permitted. Specified target proof-loads include separate parts for factored or unfactored design dead and live loads. Load factors for these two parts vary from 0.85 to 1.0 for dead load, and 0.5 to 1.19 for live load including dynamic effects.

Criteria listed in Table 2 are intended to guide decision-making for acceptance. "No visible evidence of failure" or "able to sustain with strength limit state test load" are often listed as part of an acceptance criterion, which suggests the importance of visual observation in acceptance decision-making. Although requirements for target proof-load level in Table 1 do not differentiate between steel and concrete structures, acceptance criteria differ noticeably for these two major structural construction materials. Table 2 indicates that steel structures need to be loaded for shorter periods -- for example, Australian 4100 requires a load duration of only 15 minutes. Concrete structures usually must undergo relatively longer loading, and their acceptance criteria are accordingly

Table 1. Comparison of target proof-load requirements.

Codes or Manuals (ref.) (and Applicability)	Proof Load Test Permitted? (Article)	Target Proof Load	Remarks
Codes:			
Australia:			
AS 3600 1988 (15) (Concrete Structures)	Yes (21.1)	$D_d + L_d$	
AS 4100 1990 (16) (Steel Structures)	Yes (17.1 to 17.3)	$D_d + L_d$	
Canada:			
Ontario Bridge Code (8) (Bridges)	Yes (14 8)	N/S	
New Zealand:			
NZS 3101: 1982 (10) (Concrete Structures)	Yes (15.4)	For Buildings: $0.85\ (D_d + L_d)$ For Bridges: $D_L + 1.19\ L_L$	Non flexural members are preferably investigated analytically
NZS 4203: 1976 (11) (General Structures)	No (N/A)	N/A	
United Kingdom:			
BS 5400: 1978 (13) (Bridges)	Yes (A.1, A.6)	$D_d + L_d$	
USA:			
AASHTO Manual (1) (Bridge Evaluation)	No (N/A)	N/A	
AASHTO Standard Code (19) (Bridge Design)	No (N/A)	N/A	Non flexural members are preferably investigated analytically
ACI 318 (20) (Concrete Buildings)	Yes (20.3 to 20.5)	$0.85\ (D_d + L_d)$	
ACI 437R 67: 1982 (21) (Concrete Buildings)	Yes (2.2, 3.4, 4.2, 5.2)	$0.85\ (D_d + L_d)$	
Manuals:			
United Kingdom:			
Appraisal of Existing Structures (17) (General Structures)	Yes (3.2.6, 4.2.3)	N/S	
Czechoslovakia:			
Testing Bridges in Situ (18) (Bridges)	Yes (30.1, 30.2)	$D_d + (0.5\ \text{to}\ 1.1)\ L_d$	

Notes:
- N/A - Not applicable
- N/S - Not Specified
- D_d - Required strength for dead load (factored design dead load)
- L_d - Required strength for live load including dynamic effects (factored design live load)
- D_L - Unfactored design dead load
- L_L - Unfactored design live load

Table 2. Comparison of acceptance criteria for proof-load testing.

Codes or Manuals (applicability)	Load Duration	Acceptance Criteria
Codes:		
Australia:		
AS 3600-1988 (15) (Concrete Structures)	D_d: 72 hr L_d: 24 hr	δ<12/20,000h or 75% recovery after 24 hr
AS 4100-1990 (16) (Structures)	$D_d + L_d$: 0.25 hr	Able to sustain with strength limit state test load
Canada:		
Ontario Bridge Code (8) (Bridges)	N/S	N/S
New Zealand:		
NZS 3101: 1982 (10) (Concrete Structures)	for buildings: 0.85 $(D_d + L_d)$: 24 hr	1. No visible evidence of failure 2. δ<12/20,000h or 75% recovery after 24hr
	for bridges: N/S	1. No visible evidence of failure 2. Flexural crack < 1.5 appropriate values 3. δ's nonlinearity < 20% 4. 75% recovery after 1 hr
NZS 4203: 1976 (11) (General Structures)	N/S	N/A
United Kingdom:		
BS 5400: 1978 (13) (Bridges)	N/S	1. Crack width < 2/3 of limit state requirements for R/C; No visible cracks for P/C 2. $\delta < 1^2/25{,}000h$
USA:		
AASHTO Manual (1) (Bridge Evaluation)	N/A	N/A
AASHTO Standard Code (19) (Bridge Design)	N/A	N/A
ACI 318 (20) (Concrete Buildings)	N/A 0.85 D_d: 72 hr 0.85 L_d: 24 hr	1. No visible evidence of failure 2. $\delta<1^2/20{,}000h$ or 75% recovery after 24 hr for R/C; 80% recovery after 24 hr for P/C
ACI 437R-67: 1982 (21) (Concrete Buildings)	0.85 $(D_d + L_d)$: 24 hr	1. No visible evidence of failure 2. $\delta<1^2/20{,}000h$ or 75% recovery after 24 hr for R/C; 80% recovery after 24 hr for P/C
Manuals:		
United Kingdom:		
Appraisal of Existing Structures (17) (General Structures)	N/S	N/S
Czechoslovakia:		
Testing Bridges in Situ (18) (Bridges)	Steel D_d: 3hr Others D_d: 24hr (0.5 to 1.1)L_d: N/S	N/S

based on response and recovery over longer periods. Nevertheless, bridge structures are not subject to the same requirements, perhaps because of the transient nature of their live (traffic) loads. (Examples are BS 5400 and NZS 3101.)

The following observations result from this review: 1) AASHTO codes for bridges do not include provisions for proof-load testing; 2) target proof-load levels vary from not definitely specified to those with dead-load factors of 0.85 to 1.0 and live load factors of 0.5 to 1.19 (for factored or unfactored design loads); and 3) the bases of these requirements are not well documented. From these observations, it was decided to develop a procedure to determine target proof loads for bridge rating. These target loads are intended to be consistent (in both format and assured safety) with current practice for analytical rating guided by the AASHTO codes. This work is presented in the next chapter and the results are included as requirements in the draft procedure manual (Appendix A).

III. TARGET PROOF LOAD AND BRIDGE RATING

A. Proposed Proof-Load Formula

The following proof-load formula is proposed for bridge rating:

$$\phi\ Y_p = \alpha_L\ L_n\ g_n\ I_n + \alpha_D\ D_a \tag{1}$$

where Y_p = target proof-load effect,
L_n = nominal static live-load effect,
g_n = nominal load distribution factor, and
I_n = an impact factor accounting for dynamic effect of vehicular loading.

L_n, g_n, and I_n are specified in three AASHTO specifications (1,19,22). D_a is additional dead-load effect expected on the structure after the proof-load test, such as that of an overlay. ϕ, α_L, and α_D are resistance reduction factor and live- and dead-load factors, respectively. The load and resistance factors determined in this chapter are based on a structural reliability criterion. In addition to determining target proof load, the proposed formula is also intended for use in rating by proof-load testing:

$$\text{Rating Factor} = (\phi\ R_p - \alpha_D\ D_a)/\alpha_L\ L_n\ g_n\ I_n \tag{2}$$

where R_p = proved capacity equal to or lower than the target level Y_p.

This rating methodology is consistent with the current rating method given by the 1983 AASHTO Manual (1) in concept as well as format. Note that only bending moment as load effect is considered in this study. This chapter's Section B describes models of structural reliability used to determine resistance and load factors. Live-load factor α_L is determined first without additional dead load ($D_a = 0$). Dead-load factor α_D is then found, with α_L given. For simplicity of presentation, only structural reliability models for determination of α_L are given, since the case of $\alpha_D \neq 0$ is a straightforward extension of these models.

B. Structural Reliability Models, Safety Index, and Database

1. Structural Reliability Model

Consider a limit-state function Z for a typical primary member of a bridge (e.g., a girder):

$$Z = R - D - L = R' - L \tag{3}$$

where R, D, and L are true values of resistance, existing dead load, and live load effects, respectively, and $R' = R - D$ is the resistance margin for live load. (No additional dead load is considered here for determination of α_L, and it will be included for determination of α_D with α_L given.) R′ and L are modeled by independent random variables, which are assumed to be of lognormal distribution. The random variables are used to model the uncertainties attributed to fluctuation of vehicular load, variation of material properties and construction quality, approximation due to simplified analysis methods, etc. The mean and standard deviation of R′($M_{R'}$ and $\sigma_{R'}$) are given by means and standard deviations of R and D:

$$M_{R'} = M_R - M_D \; ; \; \sigma_{R'}^2 = \sigma_R^2 + \sigma_D^2 \tag{4}$$

based on the assumption that R and D are independent of one another. Z equal to or less than 0 indicates failure of the member, and higher than 0 means survival. The live-load effect is further modeled by a combination of the following factors (23):

$$L = a\, HW_{.95}\, m\, g\, I \tag{5}$$

where all variables are modeled by independent random variables, except a which is a deterministic coefficient correlating truck weight to bending moment as load effect, based on the AASHTO rating vehicles (1). H is a factor accounting for presence of multiple vehicles on the bridge, and $W_{.95}$ is a characteristic value of the vehicle weight spectrum. Their product is treated here as a single variable. m covers effect of vehicle configuration variation on the load effect. g is lateral distribution factor, and I is impact factor for dynamic effect.

2. Safety Index

Structural reliability is often measured by the failure probability of the component P_f:

$$P_f = \text{Probability } [Z \leq 0] \tag{6}$$

If Z were a normal random variable, which can be a linear combination of normal variables, then

$$P_f = 1 - \Phi(\beta) = 1 - \Phi(M_Z/\sigma_Z) \tag{7}$$

where $\Phi(\;)$ is the cumulative probability function of the standard normal variable, and β is called the "safety index." M_Z and Φ_Z are respectively the mean and standard deviations of Z. In this study, Z as defined in Eq.3 is not a linear combination of normal variables, but can be linearized (by a polynomial series expansion of first order), and variables R′, m, $HW_{.95}$, g, and I can be transformed (equivalently in failure probability) to normal variables at a point known as the design point in the variable space. At this point, the joint probability distribution of the random variables realizes its maximum in the failure region. Eq. 6 can then be used to calculate the safety index after the linearization and transformation described in more detail by Ang and Tang (24).

It has been estimated that current bridge evaluation practice using the AASHTO Manual assures a safety index of about 2.3 for primary components [23,32]. $\beta = 2.3$ is thus elected as the target reliability level for this study to prescribe the target proof-load level. This criterion is consistent with that used in developing the AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges (22). The same target safety index has also been employed in developing a permit overload checking method (25).

3. Proof-Load Testing for Deficient Bridges

Proof-load testing has an important application in verifying or improving an existing rating obtained by analytical methods. In New York State, it is thus applicable to bridges with low load ratings. Since an existing rating contains information on capacity of the structural component, it is used advantageously in modeling that component's resistance. Nominal resistance R_n related to the existing rating factor RF is obtained by an analytical evaluation (1,19,22).

Resistance R is assumed to have a mean M_R related to its nominal value R_n by a bias B_R as follows:

$$M_R = B_R R_n \tag{8}$$

A proof-load test eliminates possibilities that the true resistance margin R' is lower than the applied proof load Y_p. This is shown in Figure 4 by truncating and then normalizing the probability density function of R' at Y_p. Thus, the limit function takes the following form:

$$Z = R' - a\ m\ HW_{.95}\ g\ I \tag{9}$$

where R' has a truncated lognormal distribution. A method developed by Fujino and Lind (2,26) is used here to calculate the safety index for this case. This method transforms R' to a standard normal variable by equivalence in probability. The limit function is transformed accordingly for calculating the safety index. This has been compared with a direct integration method over a wide range of parameter variation for practical applications. Consistency has been observed in results obtained by the two methods (2).

4. Proof-Load Testing for Unratable Bridges

Proof-load testing is also desirable for evaluating a bridge when it is not suitable for rating by analytical methods. This occurs when necessary information on the bridge or reliable analysis methods are not available, and is applicable to unratable bridges in New York State. R' is assumed to be equal to the applied proof load effect Y_p if the bridge survives the proof test, since no further information on R' is available. Thus,

$$Z = Y_p - L = Y_p - a\ m\ HW_{.95}\ g\ I \tag{10}$$

Figure 4. Structural reliability model for proof-load testing of deficient bridges with existing ratings.

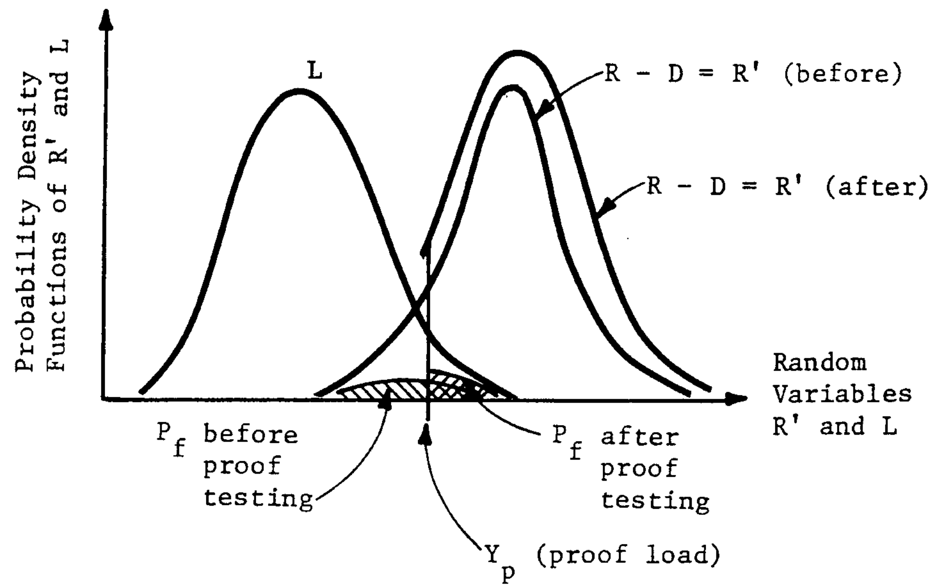
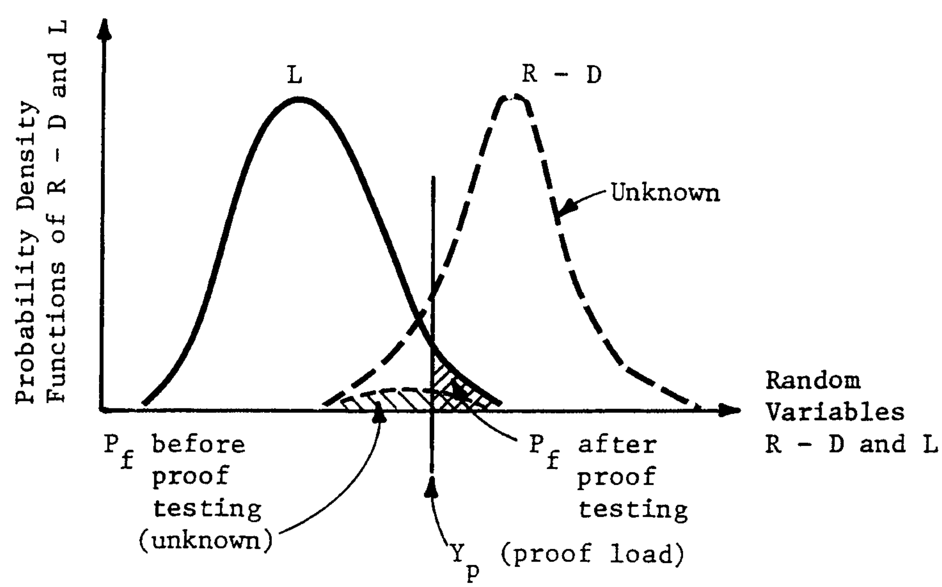


Figure 5. Structural reliability model for proof-load testing of unratable bridges without existing ratings.



is used to calculate the safety index β for this case. Note that the assumption of $R' = Y_p$ underestimates the resistance conservatively, because R' certainly can be higher than Y_p. A graphic demonstration of the proof-load test effect on structural reliability for this case is shown in Figure 5. It is observed that one can find such a Y_p to satisfy a target structural reliability; this is the mechanism used here to determine the resistance and load factors.

5. Statistical Database

Substantial statistical data have been collected for bridge structural reliability assessment (23,27,28,29,30,31). Table 3 presents a comprehensive database used in this study as input to these models, with information sources identified. It includes mean or bias (ratio of mean to nominal value) and coefficient of variation (COV, as ratio of standard deviation to mean). Three major construction materials are considered here: steel, reinforced concrete (R/C), and prestressed concrete (P/C). This covers a reasonably wide range of highway bridges in this country. The live-load parameters cover traffic load variation for a period of 2 years, for consistency with the current maximum inspection interval. The four traffic conditions characterizing the live (vehicular) load are defined by AASHTO (22). This classification permits evaluation engineers to take site-specific loading condition into account in rating bridges. It is noted that the nominal dead load effect D_n is estimated by its empirical relation to live-load effect L_{HS20} based on the HS-20 loading. These relations are also listed in Table 3, with sources identified. It is noted that the coefficients of variation of R in Table 3 are intended to cover structural component deterioration due to steel corrosion, concrete spalling, prestress loss, etc. They are used in β calculation only for the application case of proof testing with analytical rating, where it is usually desired to take deterioration into account for a more reliable rating by proof testing. They are higher than those for components in good condition (23,30) and are selected here based on subjective estimates, since no data are available. Implications of their use are examined later in a sensitivity analysis in Section D of this chapter.

The database in Table 3 is a summary of data collected by a variety of techniques, including field measurement for member dimensions, weigh-truck-in-motion for the real load spectrum, empirical relation for live-to-dead load ratio, etc. It covers variation in current traffic loading and practice in bridge design and construction in the United States.

Note that the live-load parameters in this database include data collected in New York State. In addition, two parameters (namely the vehicle-configuration parameter m and the characteristic value of vehicle-weight spectrum $HW_{.95}$), were re-evaluated in this study using data collected in 1989 by the NYSDOT Data Services Bureau. Statistical parameters of m from these data were found to be almost exactly the same as those in the national database. The mean value of $HW_{.95}$ was found to be slightly higher than that in the national database, with the coefficient of variation (COV) being consistent. This is perhaps because these New York sites were selected for their heavier weights and higher volumes of traffic. Implications of the slight difference between the New York and national data are examined in a

Table 3. Statistical database for structural reliability assessment.

RANDOM VARIABLE		MEAN		COV (%)		
	SPAN (ft)					
m (23)	30	1.0		11.0		
	40	1.0		11.0		
	50	0.95		11.0		
	60	0.92		11.0		
	70	0.90		11.0		
	80	0.91		8.2		
	90	0.92		7.5		
	100	0.93		5.7		
	120	0.95		5.4		
	140	0.95		4.6		
	160	0.96		3.4		
	180	0.97		3.9		
	200	0.97		3.2		
	TRAFFIC CONDITION	COMBINATIONS	SINGLES	COMBINATIONS	SINGLES	
HW_{95} (23)	1	170 kips	92 kips	5.0	8.0	
	2	180 kips	100 kips	6.0	8.0	
	3	210 kips	120 kips	10.0	10.0	
	4	225 kips	125 kips	10.0	10.0	
	SURFACE ROUGHNESS					
I (23)	smooth	1.1		10.0		
	medium	1.2		10.0		
	rough	1.3		10.0		
RANDOM VARIABLE	BIAS			COV (%)		
	STEEL	R/C	P/C	STEEL	R/C	P/C
R	1.05 (23)	1.05	1.0	16 (23)	14	11
D	1.0 (23)	1.0 (23)	1.0 (23)	10 (23)	10 (30)	10 (23)
g	0.9 (23)	0.97 (31)	0.96(23)	13 (23)	11 (31)	8 (23)

NOTES: COV = STANDARD DEVIATION/MEAN
BIAS = MEAN/NOMINAL

$$\frac{D_n}{L_{HS20}\, I_n} = \begin{cases} 0.0132 \times SL \text{ (STEEL)} & (33) \\ 0.6967 - 0.00762 \times SL + 0.0002554 \times SL \times SL \text{ (R/C)} & (30) \\ 0.014 \times SL \text{ (P/C)} & (34) \end{cases}$$

SL = SPAN LENGTH IN FT.

sensitivity study in Section C of this chapter. It is concluded that this will not affect the proof-load factors proposed later, because of consistency with the target safety level determined by the national database.

C. Proposed Load and Resistance Factors

To be consistent with bridge evaluation practice by current analytical methods (1) and the recently developed method of load and resistance factors (22), the resistance reduction factor ϕ is proposed to be 0.95, 0.9, and 0.95, respectively, for steel, R/C, and P/C materials. Thus α_L is the only factor to be determined in the proof-load formula (Eq. 1), to reach the target safety index of 2.3. Given an α_L, Y_p determined by Eq.1 is used in limit functions in Eqs. 10 and 11 for the two application cases of proof-load testing. Their safety indices are then calculated for comparison with the target value of 2.3. This mechanism allows selection of α_L to satisfy the requirement of structural reliability. It is noted that for a given α_L, β varies with traffic condition, span length, and material type. Thus, for each traffic (live-load) condition, α_L is selected by minimizing the variation of β due to other factors.

1. Proof-Load Testing of Deficient Bridges

For the application case of proof-load testing when an analytical rating exists, Figure 6 shows the relation of required α_L to the target safety index 2.3 and existing rating factor RF; Traffic Conditions 1 through 4 are defined in Table 4 (22). It is observed that the lower the original rating, the higher is the proof load needed to reach the same target safety level. This is expected, since a higher proof load is required to reduce the greater failure risk characterized by a lower rating factor. It is also seen in Figure 6 that when rating factor RF is equal to or lower than 0.7, variation of α_L with RF becomes less significant. An RF equal to 0.7 thus is selected as a threshold for whether to take the existing rating into account in determining target proof load. In other words, when an existing rating factor is higher than 0.7, that is considered an important piece of information to be included in selecting the target proof-load level, but not worth consideration if lower than 0.7. According to this criterion, the proof load factor α_L is proposed in Table 4 for the case of proof-load testing with an analytical rating factor of 0.7 or more. Figure 7 shows safety index β using the proposed load and resistance factors for this case. They produce a relatively uniform safety level of 2.3. It is noted that reinforced concrete bridges have significantly different live-to-dead load ratios than steel and prestressed concrete structures. This is a major factor causing a little higher reliability than the other two types of bridge, especially for longer spans. Reliability is not assessed for spans longer than 100 ft, since the available empirical ratio of dead to live load is considered valid only up to this span length, and few R/C highway bridges exceed this span length in the United States.

Figure 6. Required α_L for existing rating factor RF.

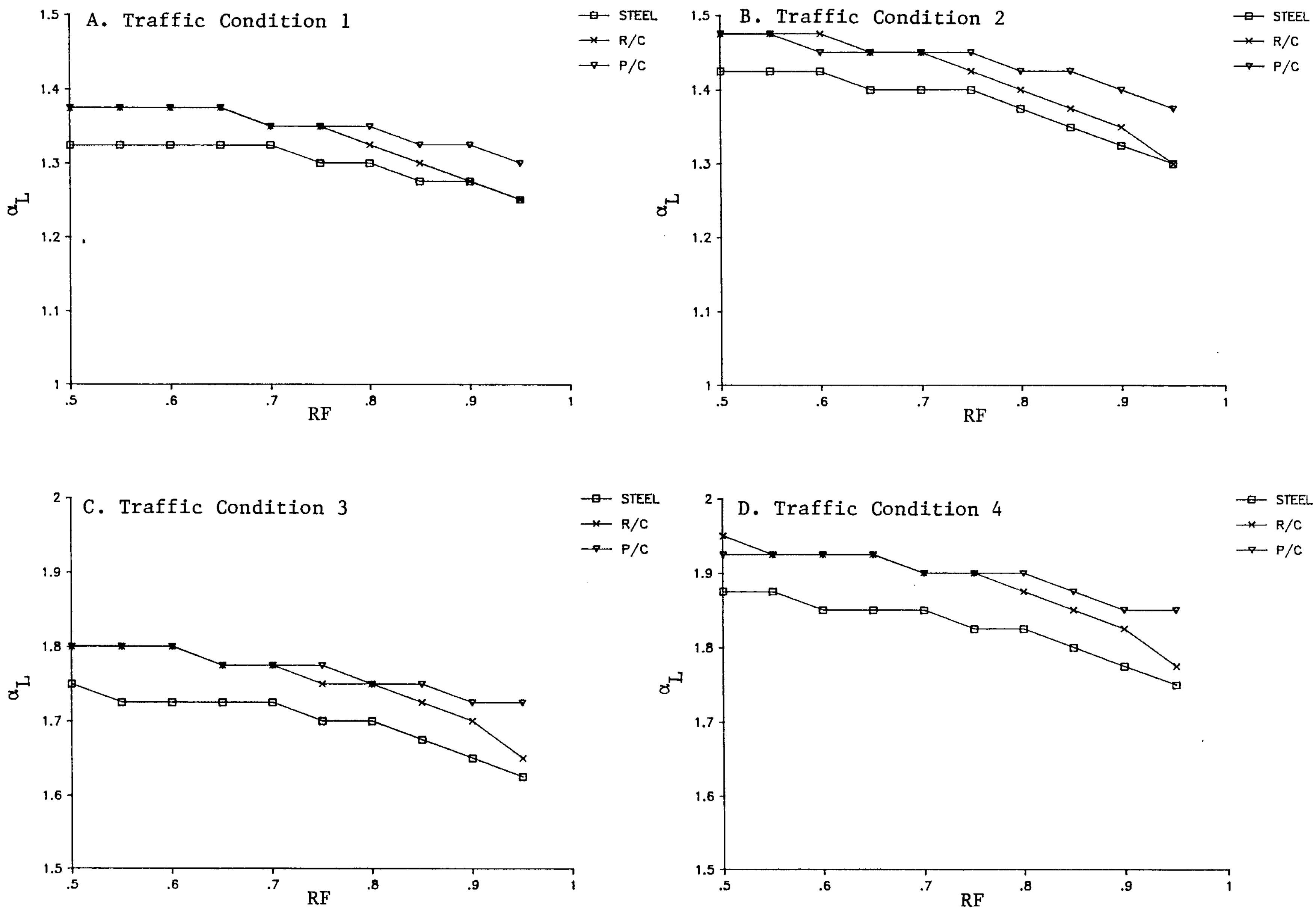


Figure 7. Structural safety based on proposed proof-load formula for deficient bridges.

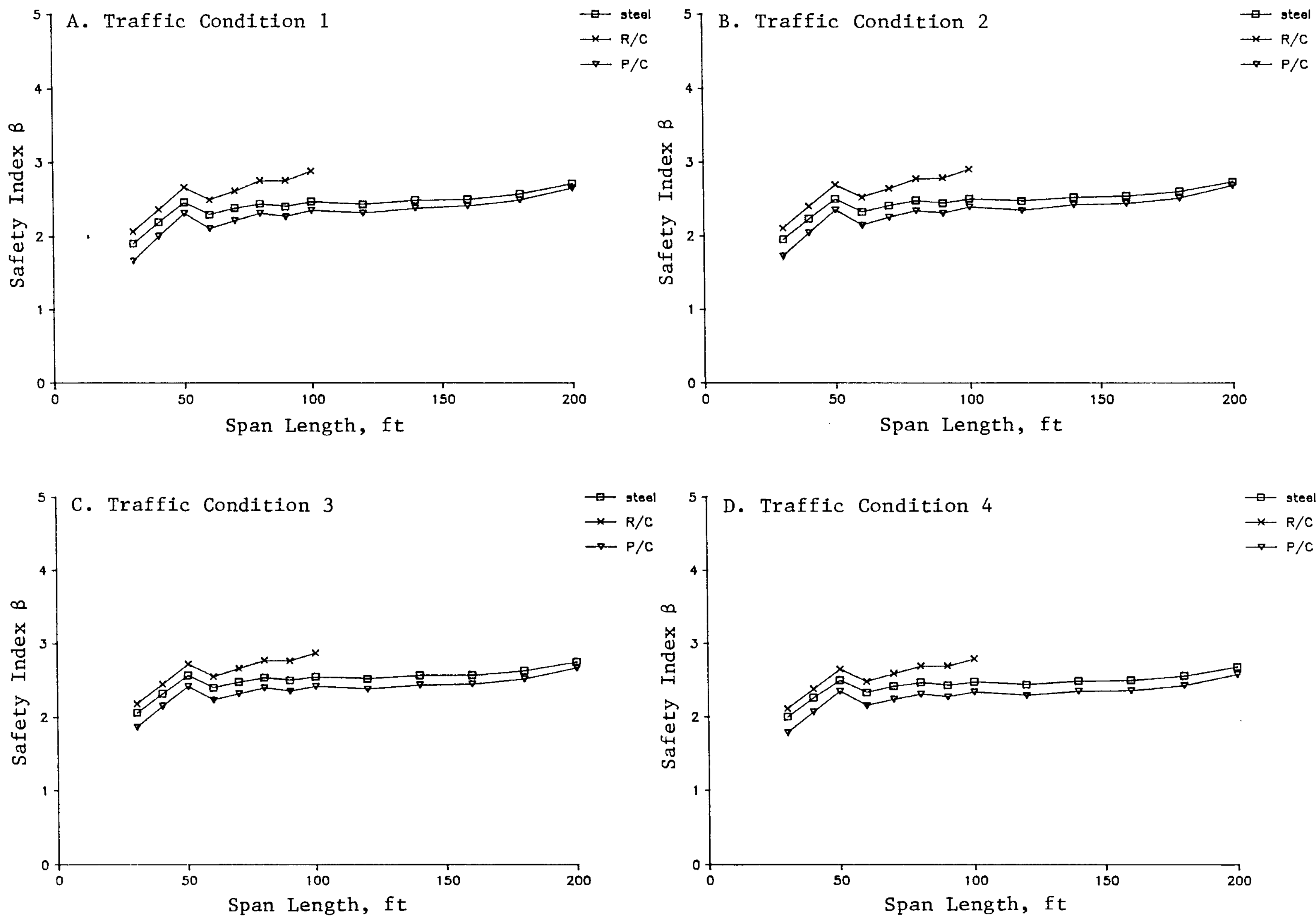


Table 4. Proposed live-load factor α_L for proof-load testing with existing analytical rating factor RF ≥0.7.

Traffic Condition	Live-Load Category	Proposed α_L
1	Low-volume roadways (ADTT less than 1000), reasonable enforcement and apparent control of overloads	1.35
2	High-volume roadways (ADTT greater than 1000), reasonable enforcement and apparent control of overloads	1.45
3	High-volume roadways (ADTT less than 1000), significant sources of overloads without effective enforcement	1.80
4	High-volume roadways (ADTT greater than 1000), significant sources of overloads without effective enforcement	1.90

Note: ADTT - Average Daily Truck Traffic

Table 5. Proposed live-load factor α_L for proof-load testing without analytical rating or existing rating factor RF <0.7.

Traffic Condition	Live Load Category	Proposed α_L
1	Low volume roadways (ADTT less than 1000), reasonable enforcement and apparent control of overloads	1.45
2	High volume roadways (ADTT greater than 1000), reasonable enforcement and apparent control of overloads	1.55
3	Low volume roadways (ADTT less than 1000), significant sources of overloads without effective enforcement	1.90
4	High volume roadways (ADTT greater than 1000), significant sources of overloads without effective enforcement	2.00

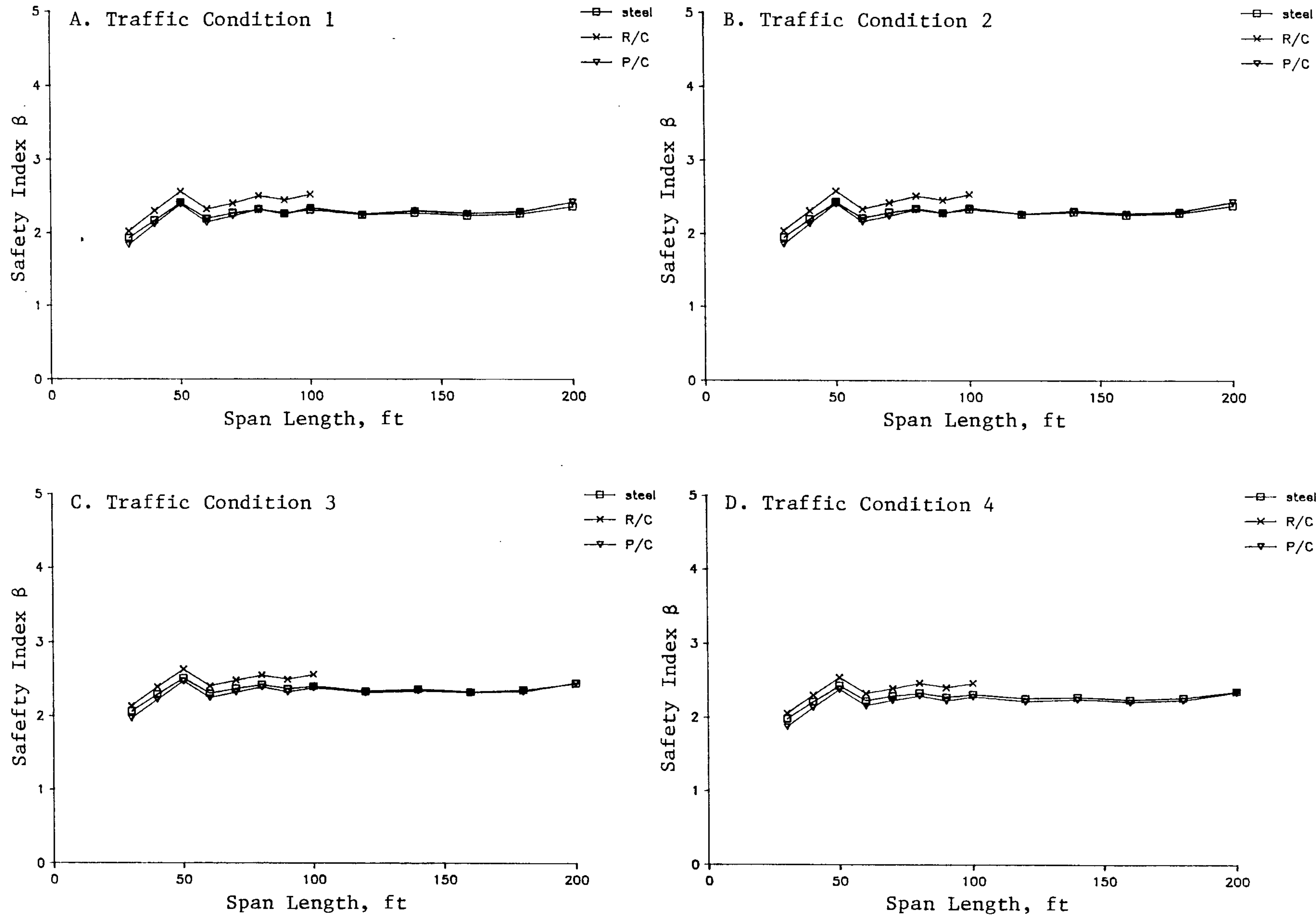
Note: ADTT - Average Daily Truck Traffic

2. Proof-Load Testing of Unratable Bridges

For the application case when a rating factor is not available, Table 5 gives the proposed proof-load factor α_L for the four categories ("traffic conditions") of live-load traffic. These are higher than those for the previous application case, because less information is required than for deficient bridges. Figure 8 shows the safety index assured by the proposed proof-load factor for this case. It is seen that a relatively uniform safety level of 2.3 is realized with respect to span length. In this application case, difference in β for various materials is a little lower than in the previous case. This is because the dead-load influence on β is eliminated, since it no longer appears in the limit function (Eq.10).

Note that when an existing rating factor is lower than 0.7, the live-load factors in Table 5 can be used to determine the required target proof load and load rating by proof-load testing. They are the maximum proof-load

Figure 8. Structural safety based on proposed proof-load formula for unratable bridges.



levels needed for bridge rating to ensure the target safety level, which do not depend on any a priori information about bridge capacity.

3. Non-Redundant Structures

As has been shown, the results in Tables 4 and 5 are derived for redundant structures associated with the selected target safety level of 2.3. For non-redundant structures, a resistance reduction factor $\phi = 0.8$ is used (22) for a corresponding requirement of higher safety level. Figures 9 and 10 show the respective safety indices assured by the proposed proof-load formula for non-redundant structures that are deficient and unratable (with and without an analytical rating). It is noted that a relatively uniform safety level of 3.2 is realized with respect to span length, consistent with the target level found as the average across the nation (22,23,32). Thus, the proposed proof-load formula can also be applied to non-redundant structures.

4. Bridge Posting

When a rating obtained by proof-load testing is lower than 1.0, the bridge should be posted, according to Eq.2. Assuming linear decrease in the projected live-load effect (23), the required safety index β is shown to have been achieved using the proposed rating formula. Figures 11 and 12 show safety index β with respect to proposed bridge rating after proof testing for an 80-ft span bridge. Note that β does not change in the case of proof-load testing for unratable bridges. For proof-load testing of deficient bridges, β increases as RF decreases and is consistently higher than the target safety level of 2.3. It thus is considered conservatively satisfactory.

5. Additional Dead Load

Additional dead load has been included in the general form of the proposed proof-load formula (Eq. 1). Reliability modeling of this case is similar to those described in Section B of this chapter. Since asphalt overlay represents a case of dead load with higher variations, it is included here for conservative prescription of α_D. The bias and COV of the asphalt overlay are taken to be 1.0 and 0.25, respectively (28). The additional dead-load factor α_D is found to be 1.25, to reach the target safety level. Figure 13 shows the safety index β considering the additional dead load as being uniform over spans and near the target level of 2.3.

6. Continuous Bridges

To this point, determination of resistance and load factors for proof-load testing has been based on single-span bridges. Several typical continuous bridges of two and three spans are used here to examine the safety levels resulting from the proposed proof-load formula. The safety-index calculation is essentially the same as for simple-span bridges, the only difference being that more than one control point ought to be considered for continuous bridges. Thus, safety indices are calculated for all the

Table 6. m for continuous bridges (23).

2 Span Bridges:

Span Lengths (ft)	CHECKPOINT Midspan MEAN	Midspan COV(%)	Support MEAN	Support COV(%)
30-40	.99	10	.97	6
50-75	.90	9	.90	6
70-105	.95	5	.95	3

3 Span Bridges:

Span Lengths (ft)	CHECKPOINT 1.4 MEAN	1.4 COV(%)	2.0 MEAN	2.0 COV(%)	2.5 MEAN	2.5 COV(%)
30-45-30	.97	8	.96	4	.93	8
50-75-50	.99	8	.97	4	.89	8
70-105-70	.99	8	.98	4	.91	8

checkpoints, including the interior support point 2.0 between Spans 1 and 2, the center point 2.5 of the longer span assumed to be Span 2, and 40-percent point 1.4 of the exterior span assumed to be Span 1. Table 6 gives the statistical parameters of m used for several typical continuous bridges. The rest of the data are taken from Table 3, and are the same as for simple-span bridges. Tables 7 and 8 show safety indices for these typical continuous bridges. The safety indices are between 2.0 and 3.5, maintaining the same levels as by the analytical rating approach (23).

D. Sensitivity Analysis

Changes in input data and certain assumptions in modeling and calculating structural reliability may influence the obtained safety index, and in turn may affect the proof-load factors being proposed. Thus, a sensitivity analysis is warranted to ensure that reasonable changes in input data and assumptions will not adversely affect the proof-load formula proposed here. Figures 7 and 8 show that the safety index for prestressed concrete is always slightly lower than that of steel or reinforced concrete. Prestressed concrete thus was selected here for the sensitivity analysis.

The assumption of lognormal distribution for the random variables is examined first. Figure 14 displays safety index β if one of the random variables is modeled by a normal rather than lognormal variable for the case of bridges under Traffic Condition 1. It shows that a different probability distribution assumption has little influence on the safety index, and thus the assumption of lognormal distribution is not critical to the obtained results.

Figure 15 demonstrates sensitivity of the safety index to bias and COV of R (B_R and V_R) for the application case of deficient bridges with existing rating factors equal to or higher than 0.7. Traffic Conditions 3 and 4 are considered here, as they are more critical than the other two conditions. It is seen in Figures 15C and 15D that higher V_R leads to higher β for shorter spans. This is because the higher scatter in R increases scatter of $R' = R - D$ for shorter spans where the dead-load effect is insignificant, and truncating distribution of R' by proof-load testing is more effective in reducing failure risk. For longer

Figure 9. Structural safety based on proposed proof-load formula for nonredundant deficient bridges.

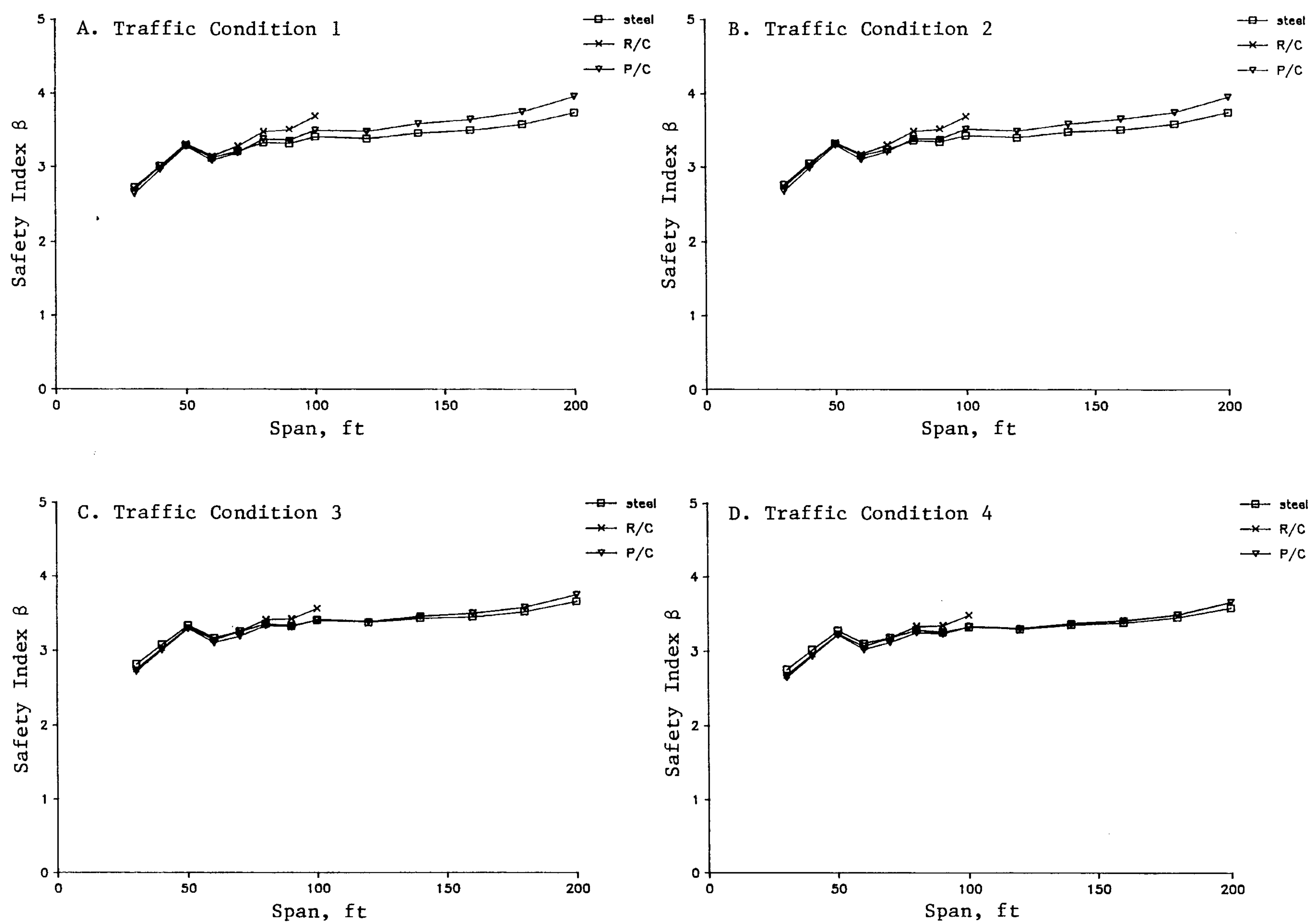


Figure 10. Structural safety based on proposed proof-load formula for nonredundant unratable bridges.

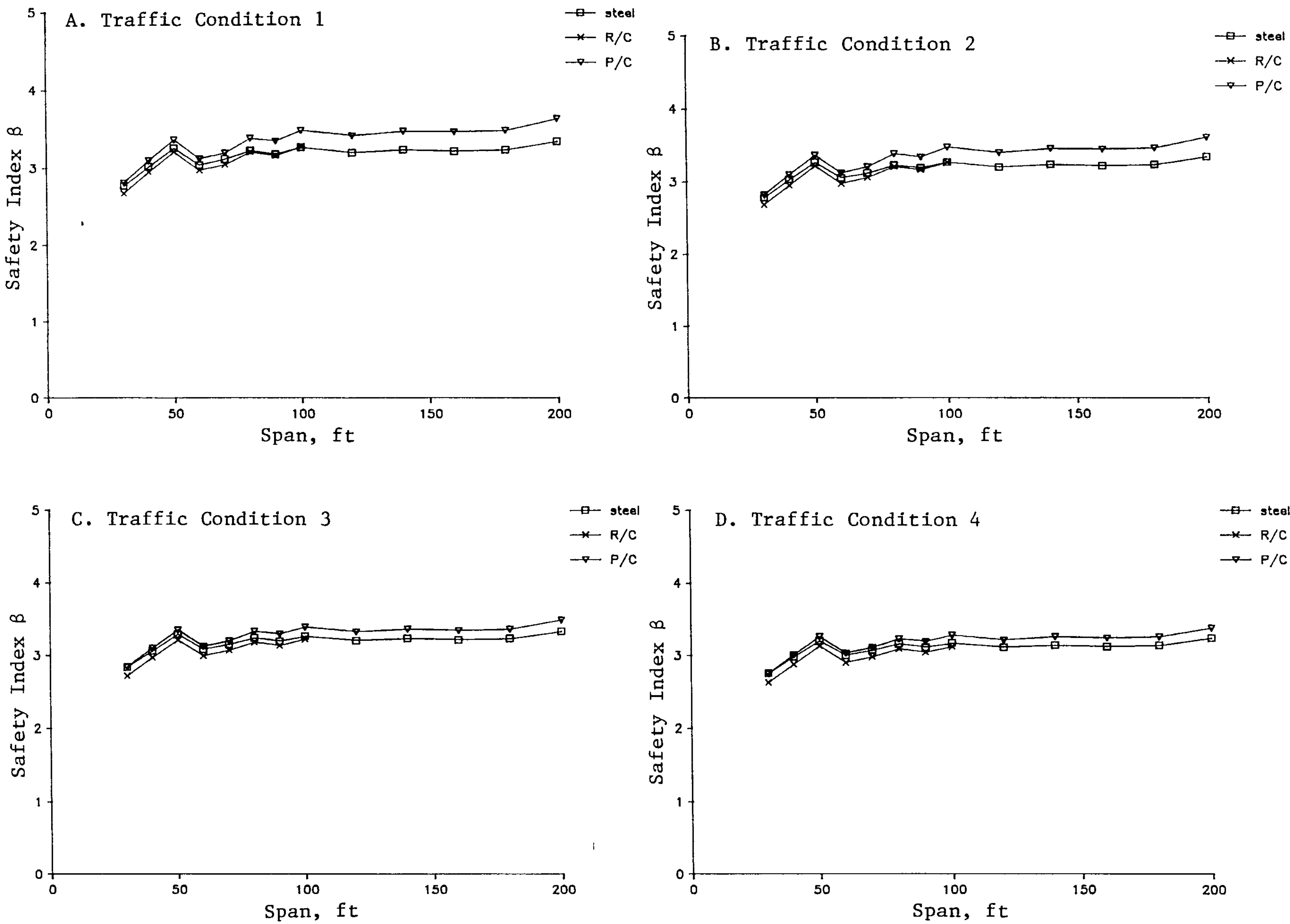


Figure 11. Structural safety for load-posted bridges originally unratable (80-ft span).

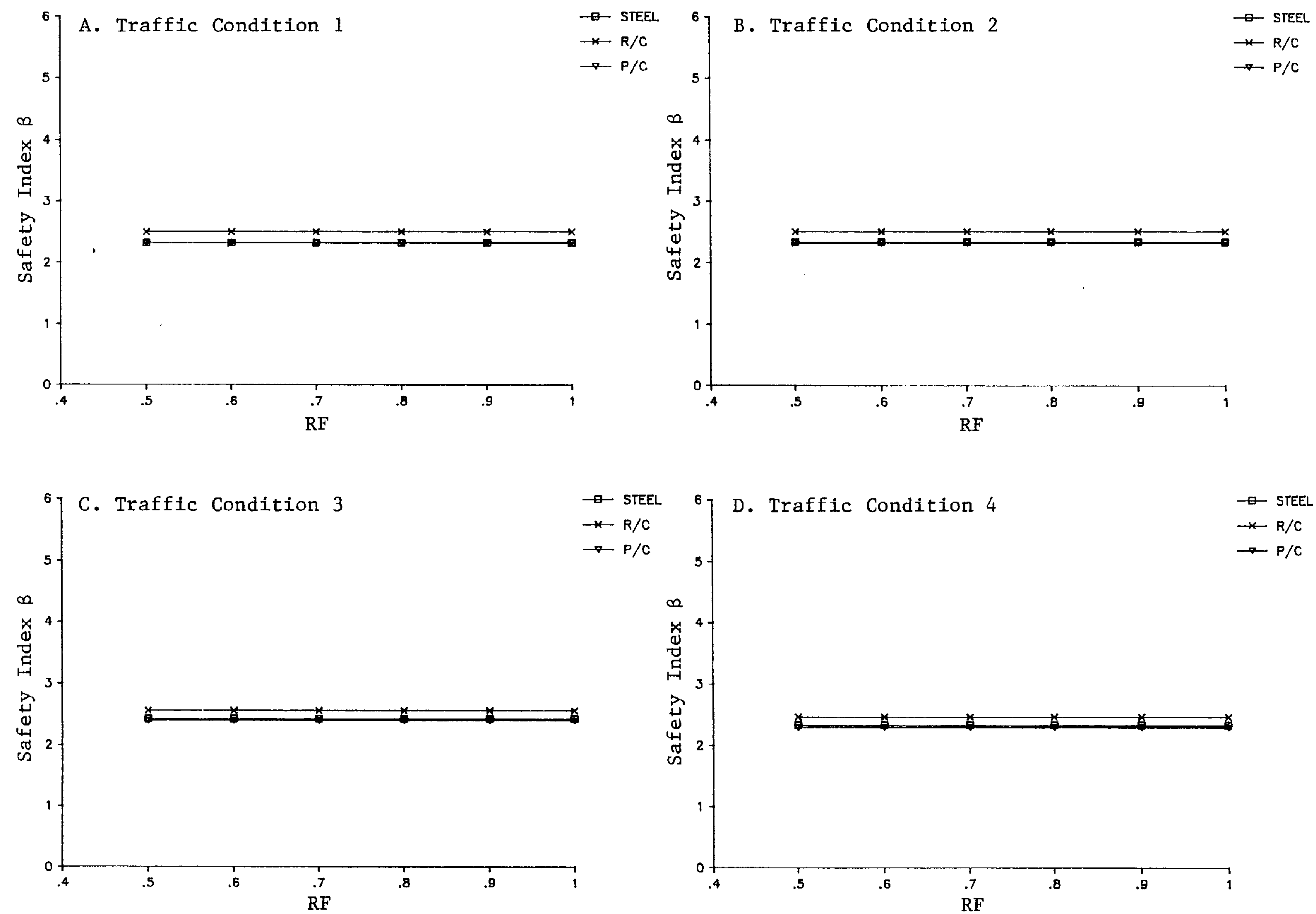


Figure 12. Structural safety for load-posted bridges originally deficient (80-ft span).

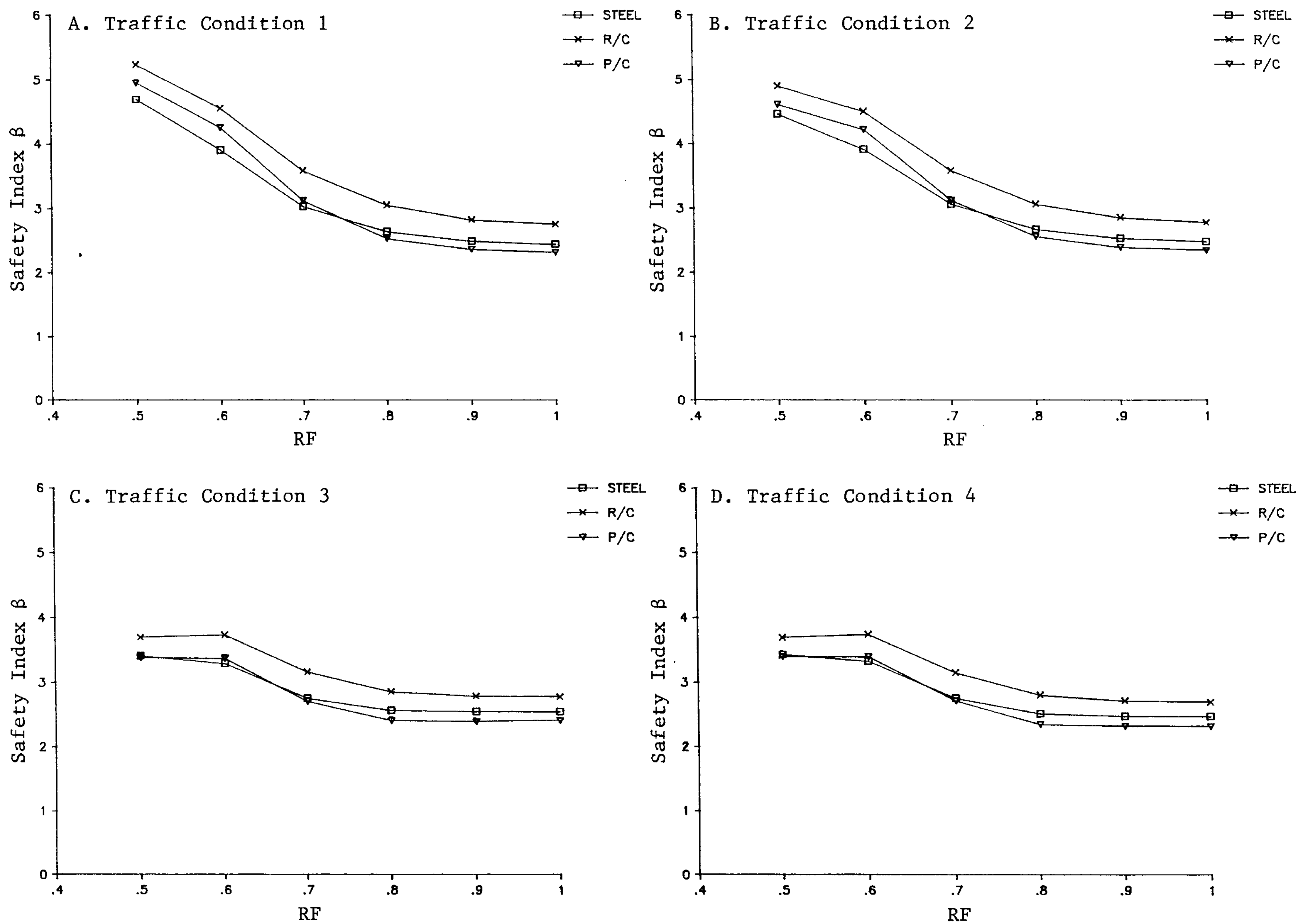


Figure 13. Structural safety with additional dead load, based on proposed proof-load formula.

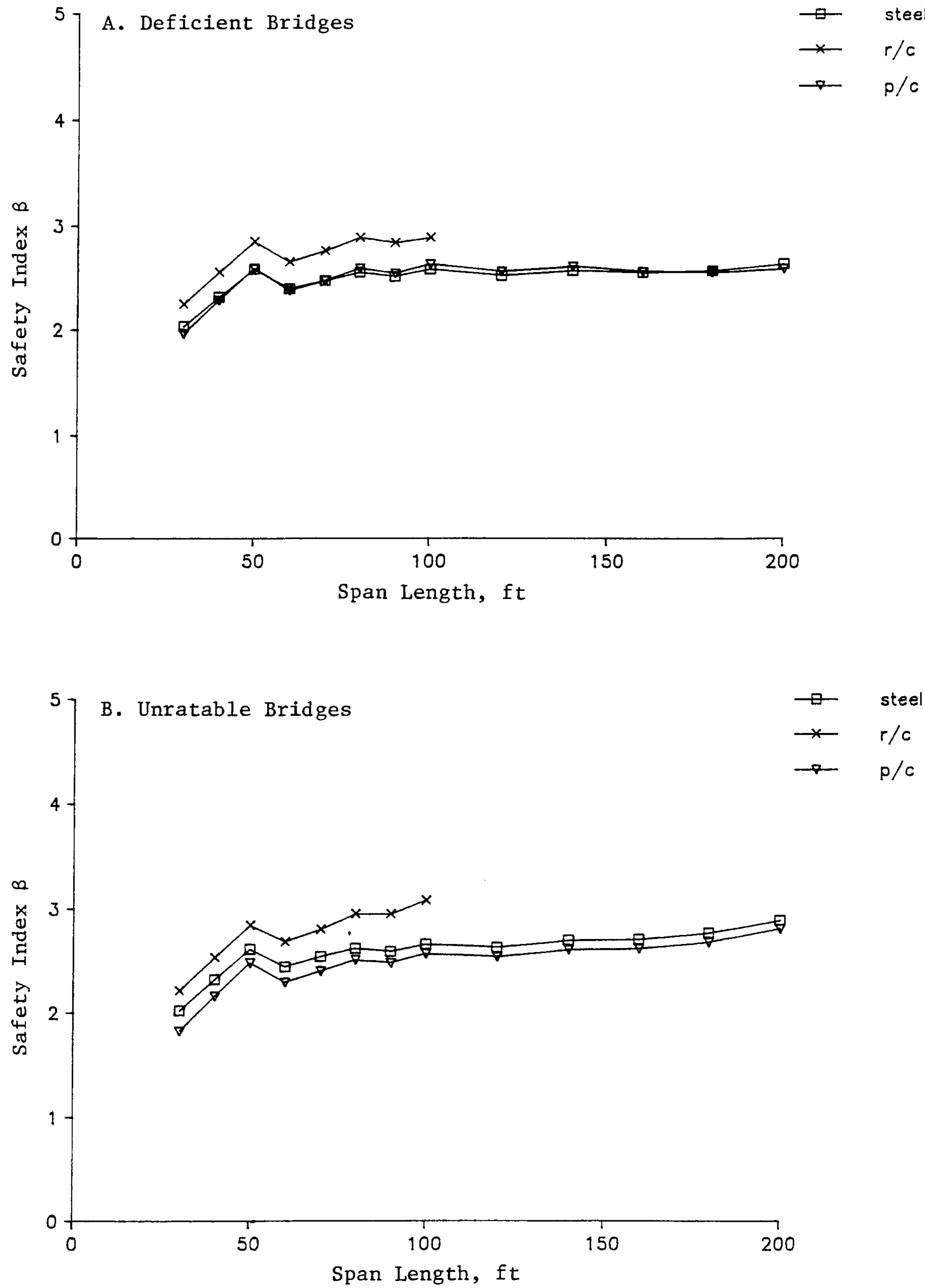


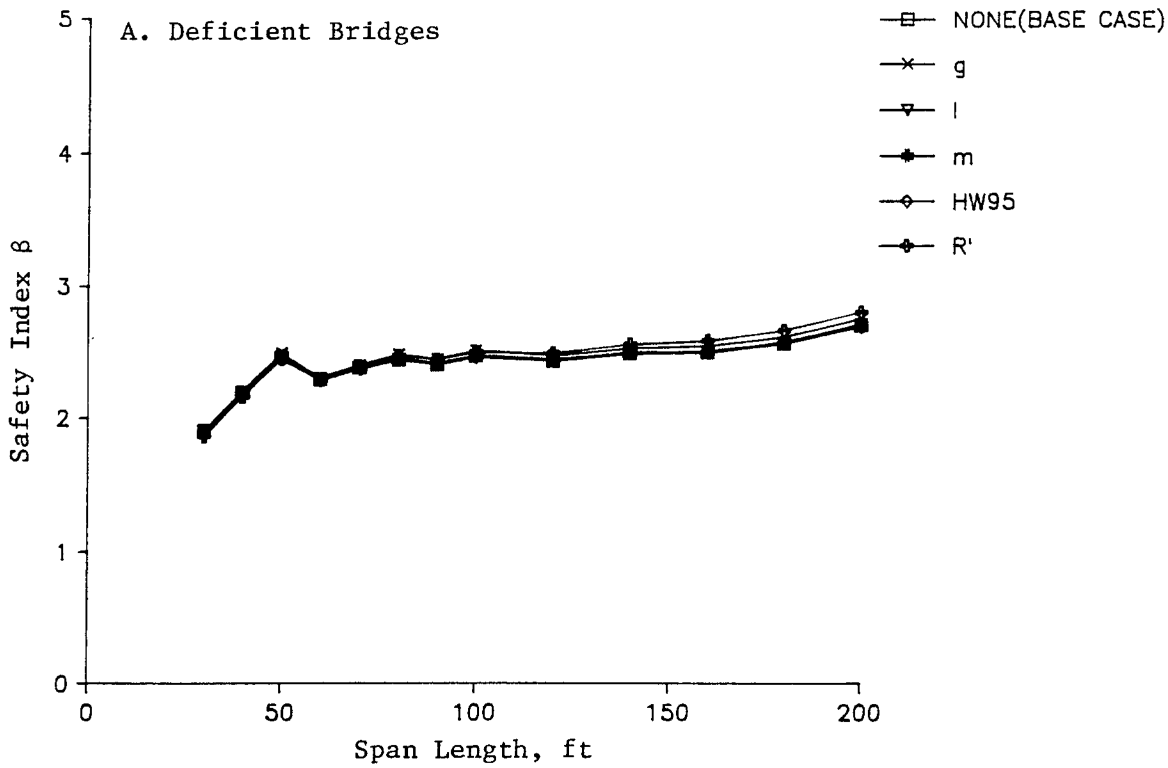
Table 7. Safety Index β for two-span continuous bridges using proposed proof-load formula.

SPAN LENGTHS		MIDSPAN CHECKPOINT TRAFFIC CONDITION 1	2	3	4	SUPPORT CHECKPOINT TRAFFIC CONDITION 1	2	3	4
UNRATABLE BRIDGES									
34-40	STEEL	2.33	2.35	2.43	2.35	2.48	2.5	2.55	2.47
	R/C	2.49	2.5	2.55	2.46	2.71	2.72	2.73	2.63
	P/C	2.31	2.33	2.39	2.3	2.54	2.54	2.56	2.46
50-75	STEEL	2.52	2.53	2.6	2.51	2.67	2.68	2.73	2.64
	R/C	2.71	2.72	2.75	2.65	2.93	2.93	2.92	2.82
	P/C	2.54	2.55	2.59	2.49	2.77	2.77	2.76	2.66
70-105	STEEL	2.41	2.43	2.49	2.4	2.77	2.78	2.81	2.72
	R/C	2.64	2.65	2.66	2.56	3.09	3.08	3.04	2.93
	P/C	2.46	2.47	2.49	2.39	2.93	2.92	2.88	2.77
DEFICIENT BRIDGES									
34-40	STEEL	2.32	2.36	2.44	2.38	2.29	2.35	2.45	2.39
	R/C	2.51	2.55	2.59	2.52	2.52	2.56	2.62	2.55
	P/C	2.15	2.2	2.29	2.22	2.19	2.25	2.35	2.27
50-75	STEEL	2.58	2.61	2.67	2.6	2.49	2.54	2.63	2.56
	R/C	2.82	2.85	2.86	2.79	2.72	2.76	2.8	2.73
	P/C	2.46	2.5	2.55	2.47	2.43	2.48	2.56	2.48
70-105	STEEL	2.52	2.56	2.61	2.54	2.55	2.6	2.68	2.61
	R/C	2.88	2.91	2.88	2.8	2.81	2.85	2.87	2.79
	P/C	2.43	2.46	2.5	2.41	2.53	2.57	2.62	2.54

Table 8. Safety Index β for three-span continuous bridges using proposed proof-load formula.

CHECKPOINT		FOR UNRATABLE BRIDGES TRAFFIC CONDITION 1	2	3	4	FOR DEFICIENT BRIDGES TRAFFIC CONDITION 1	2	3	4
SPAN LENGTHS 30-45-30									
2.5	STEEL	3.18	3.18	3.19	3.11	3.05	3.09	3.13	3.07
	R/C	3.48	3.47	3.42	3.32	3.36	3.39	3.37	3.3
	P/C	3.34	3.33	3.28	3.18	3.07	3.1	3.12	3.04
2.0	STEEL	2.38	2.39	2.45	2.36	2.18	2.24	2.35	2.28
	R/C	2.61	2.62	2.63	2.53	2.41	2.46	2.52	2.44
	P/C	2.43	2.44	2.46	2.35	2.07	2.13	2.24	2.16
1.4	STEEL	2.39	2.41	2.48	2.39	2.29	2.33	2.43	2.37
	R/C	2.58	2.59	2.63	2.53	2.51	2.55	2.6	2.53
	P/C	2.41	2.42	2.46	2.36	2.15	2.2	2.3	2.22
SPAN LENGTHS 50-75-50									
2.5	STEEL	2.63	2.65	2.7	2.61	2.6	2.64	2.71	2.64
	R/C	2.86	2.86	2.87	2.77	2.84	2.87	2.89	2.81
	P/C	2.69	2.69	2.71	2.61	2.51	2.55	2.61	2.53
2.0	STEEL	2.33	2.34	2.41	2.32	2.16	2.21	2.32	2.25
	R/C	2.56	2.56	2.58	2.48	2.36	2.41	2.48	2.4
	P/C	2.37	2.38	2.4	2.3	2.03	2.09	2.2	2.12
1.4	STEEL	2.64	2.65	2.7	2.62	2.6	2.64	2.71	2.64
	R/C	2.87	2.87	2.88	2.78	2.84	2.88	2.89	2.82
	P/C	2.7	2.7	2.72	2.62	2.51	2.55	2.61	2.53
SPAN LENGTHS 70-105-70									
2.5	STEEL	2.52	2.53	2.59	2.5	2.54	2.58	2.64	2.57
	R/C	2.72	2.73	2.75	2.65	2.79	2.82	2.83	2.75
	P/C	2.55	2.56	2.59	2.49	2.43	2.47	2.52	2.44
2.0	STEEL	2.65	2.66	2.7	2.61	2.5	2.55	2.63	2.56
	R/C	2.94	2.93	2.91	2.81	2.75	2.79	2.81	2.73
	P/C	2.77	2.77	2.75	2.64	2.45	2.49	2.55	2.47
1.4	STEEL	2.1	2.12	2.22	2.13	2.26	2.3	2.36	2.29
	R/C	2.25	2.27	2.33	2.24	2.57	2.6	2.6	2.52
	P/C	2.06	2.08	2.16	2.06	2.08	2.12	2.19	2.11

Figure 14. Sensitivity of Safety Index β to probability distribution assumption (RF = 0.7, Steel, Traffic Condition 1).



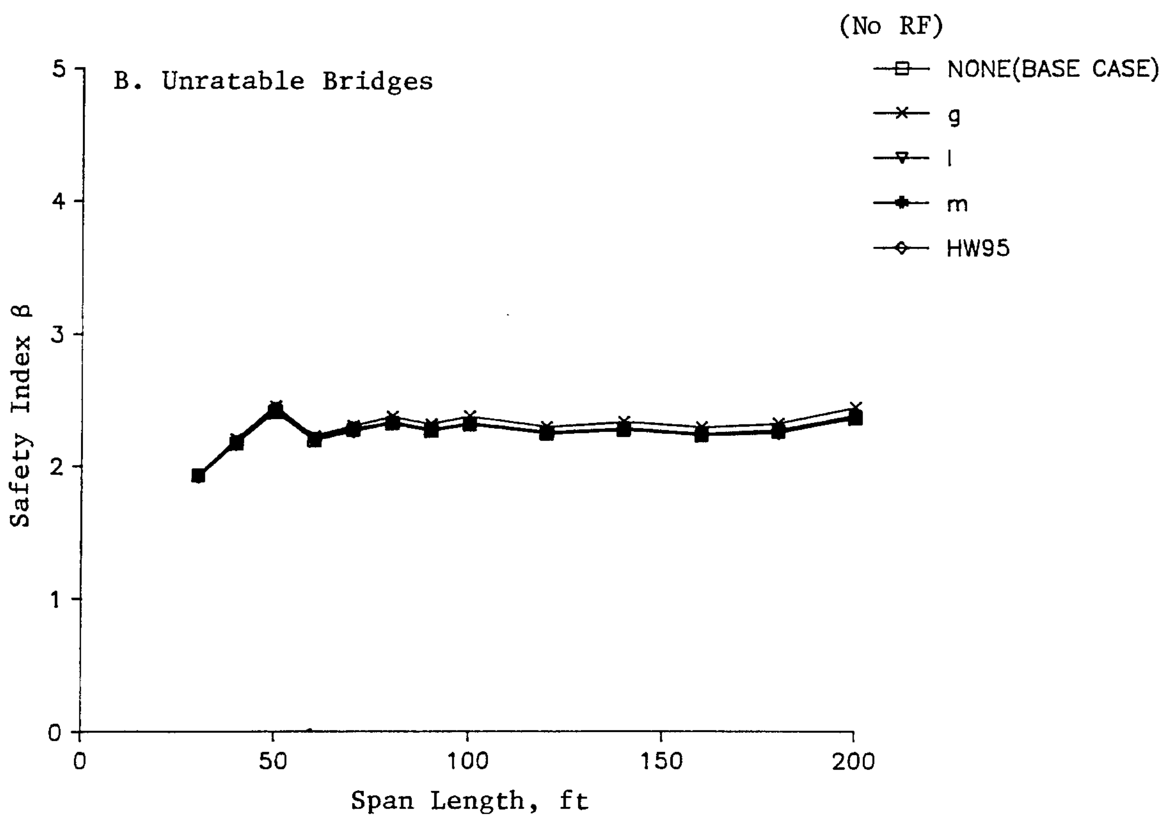
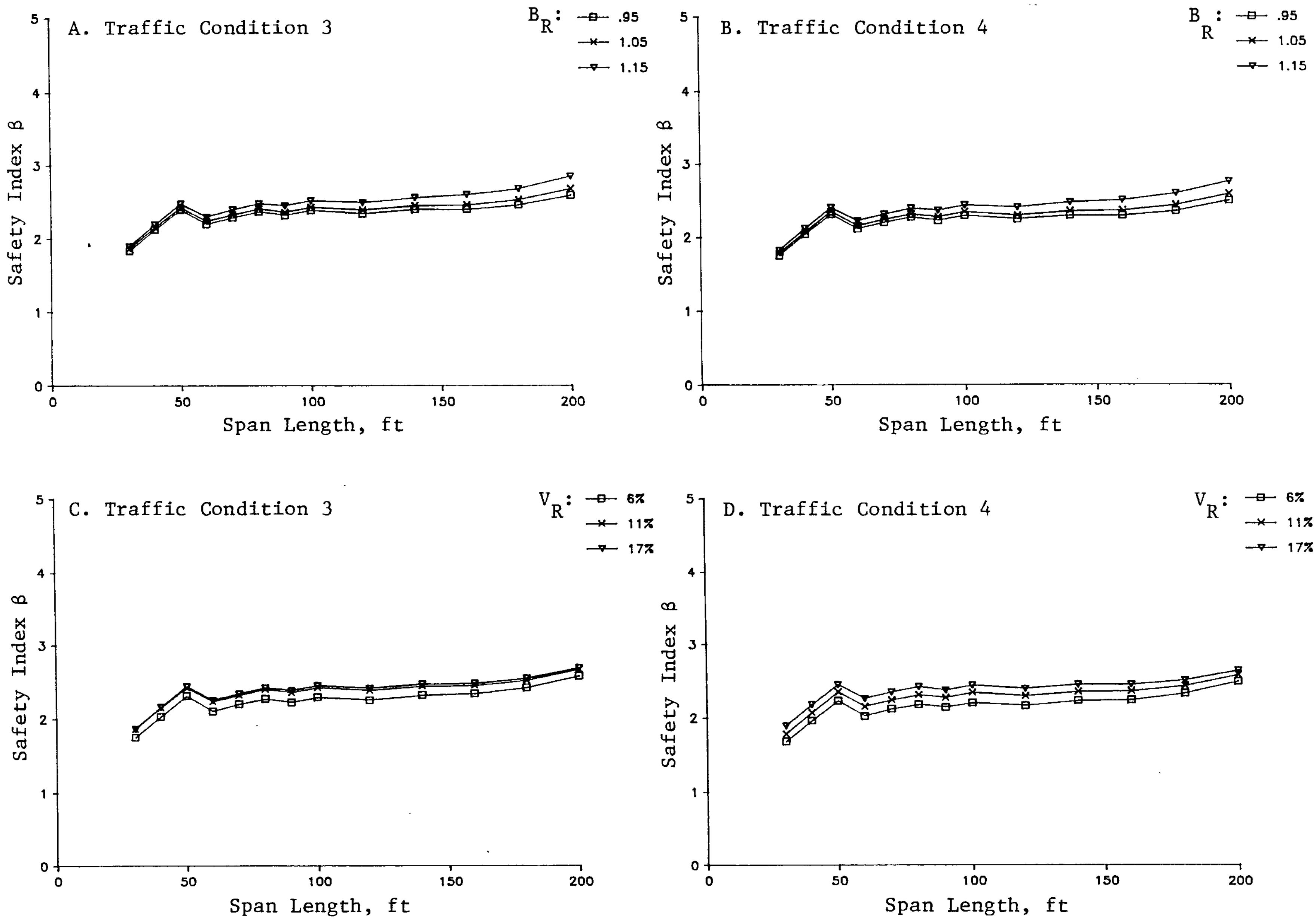


Figure 15. Sensitivity of Safety Index β to bias and COV of R (RF = 0.7, P/C).



spans, the dead-load effect is more dominant, and higher scatter of R thus does not significantly increase scatter of R′. Proof-load testing thus is less effective than for shorter spans. Figure 15 also shows that neither B_R nor V_R affect the safety level significantly. Figure 16 shows that the same conclusion applies to the bias and COV of dead-load effect D (B_D and V_D), since they have even lower influence on β than B_R and V_R, respectively. It is noted that parameters of R and D do not affect β at all in the second application case of unratable bridges without analytical rating, because information on them is not needed for β calculation using Eq.10.

Figure 17C, 17D, 17E and 17F show safety indices for the two application cases of proof-load test with bias of g (B_g) perturbed. It is observed that the change of B_g has little influence on the uniformity assured by the proposed proof-load formula. On the other hand, this does affect the absolute reliability level. Figure 17A and 17B exhibit β produced by the current working-stress evaluation method at the operating level, for two traffic conditions. Figure 17 shows that targets set by the current working-stress level are correspondingly either reached or exceeded by the proposed formula.

Note that the changes in B_g considered here are the base case times 0.9 or 1.1. The changes in biases B_I, B_m, and $B_{HW.95}$ will produce the same curve, as long as these changes are the base case times 0.9 or 1.1. This is because the product of lognormal random variables is again lognormal, and the mean of the resulting random variable equals the product of the means of these four variables. Thus, 10-percent change in B_g will cause the same change in β as 10-percent change in B_I , B_m, and $B_{HW.95}$.

Figure 18 shows the results of sensitivity examination for COV of g (V_g). β still remains uniform if V_g is changed, for both unratable and deficient bridges. Reliability levels reached by the proposed formula are higher than those by the current working-stress evaluation method at the operating level under the changed parameter.

More cases of input-data change were examined in this sensitivity analysis. Figures 19 to 21 show the results of sensitivity analyses for COVs V_I , V_m, and $V_{HW.95}$. It is interesting to note that the change in V_m will cause more significant changes of β in short spans (Figure 20), because truck configuration is more critical for short spans. It is concluded that possible changes of input data will not affect the proof-load formula proposed here, with respect to producing uniform β and satisfying the target levels.

Figure 16. Sensitivity of Safety Index β to bias and COV of D (RF = 0.7, P/C).

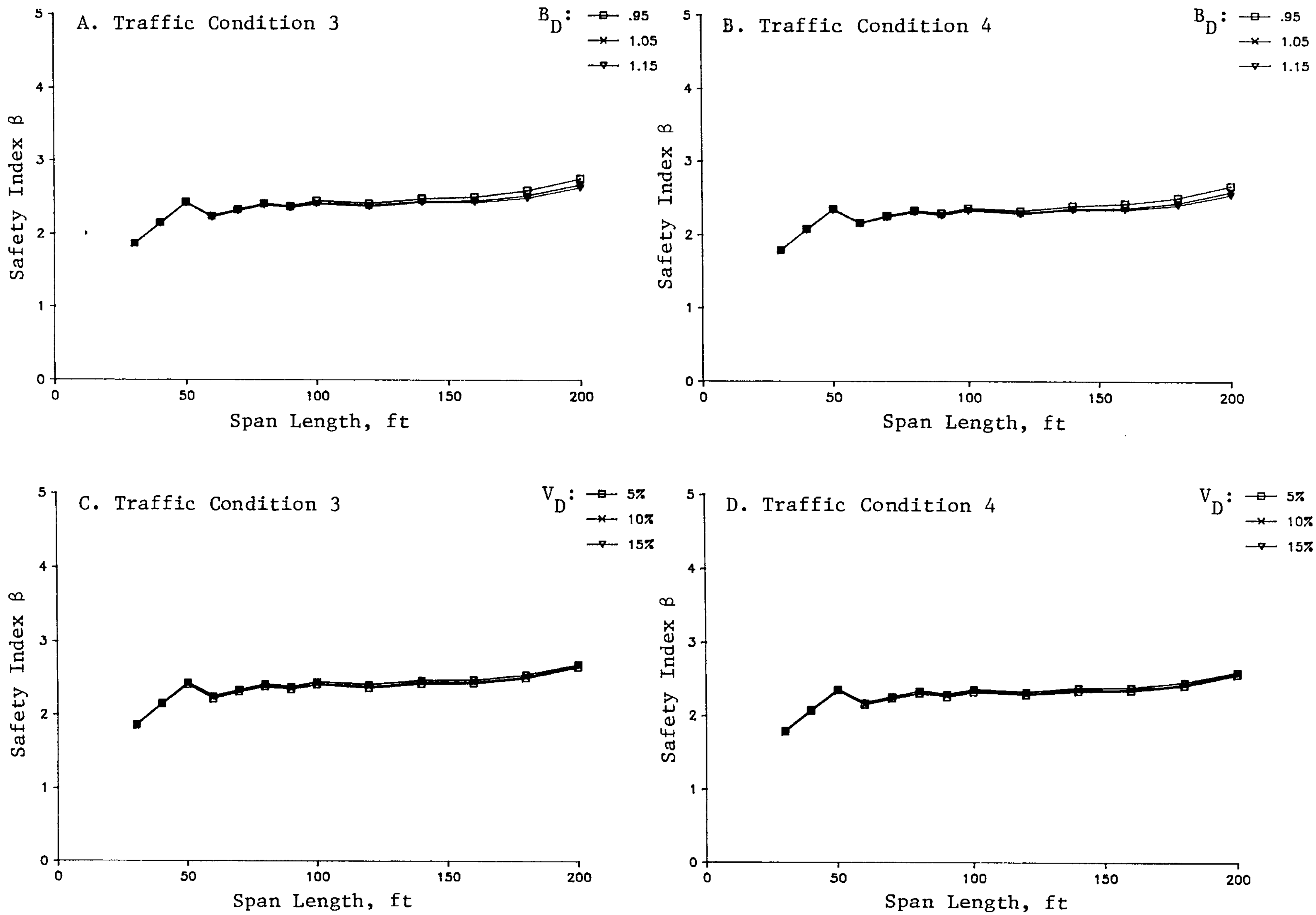


Figure 17. Sensitivity of Safety Index β to bias of g (P/C).

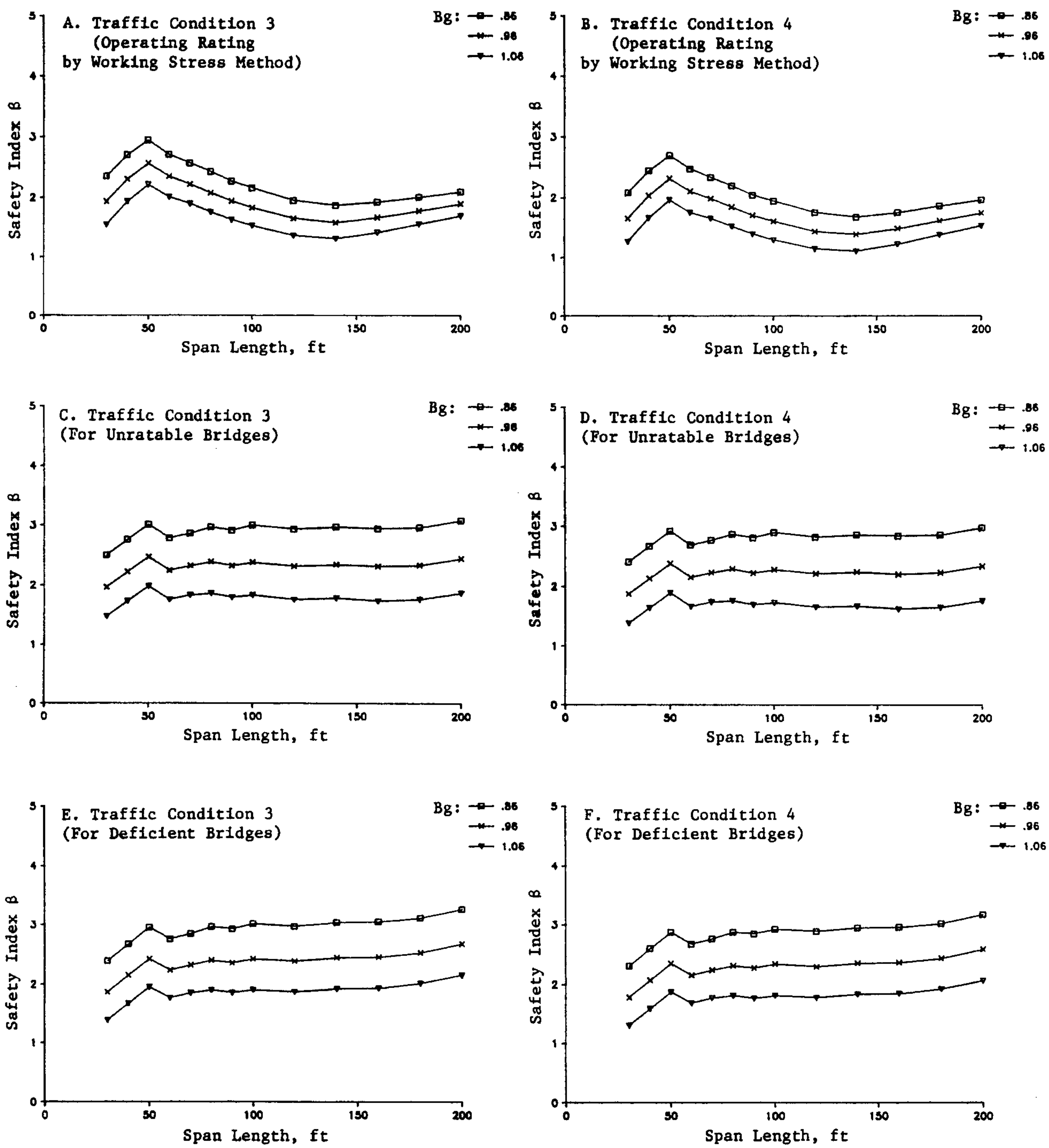


Figure 18. Sensitivity of Safety Index β to COV of g (P/C).

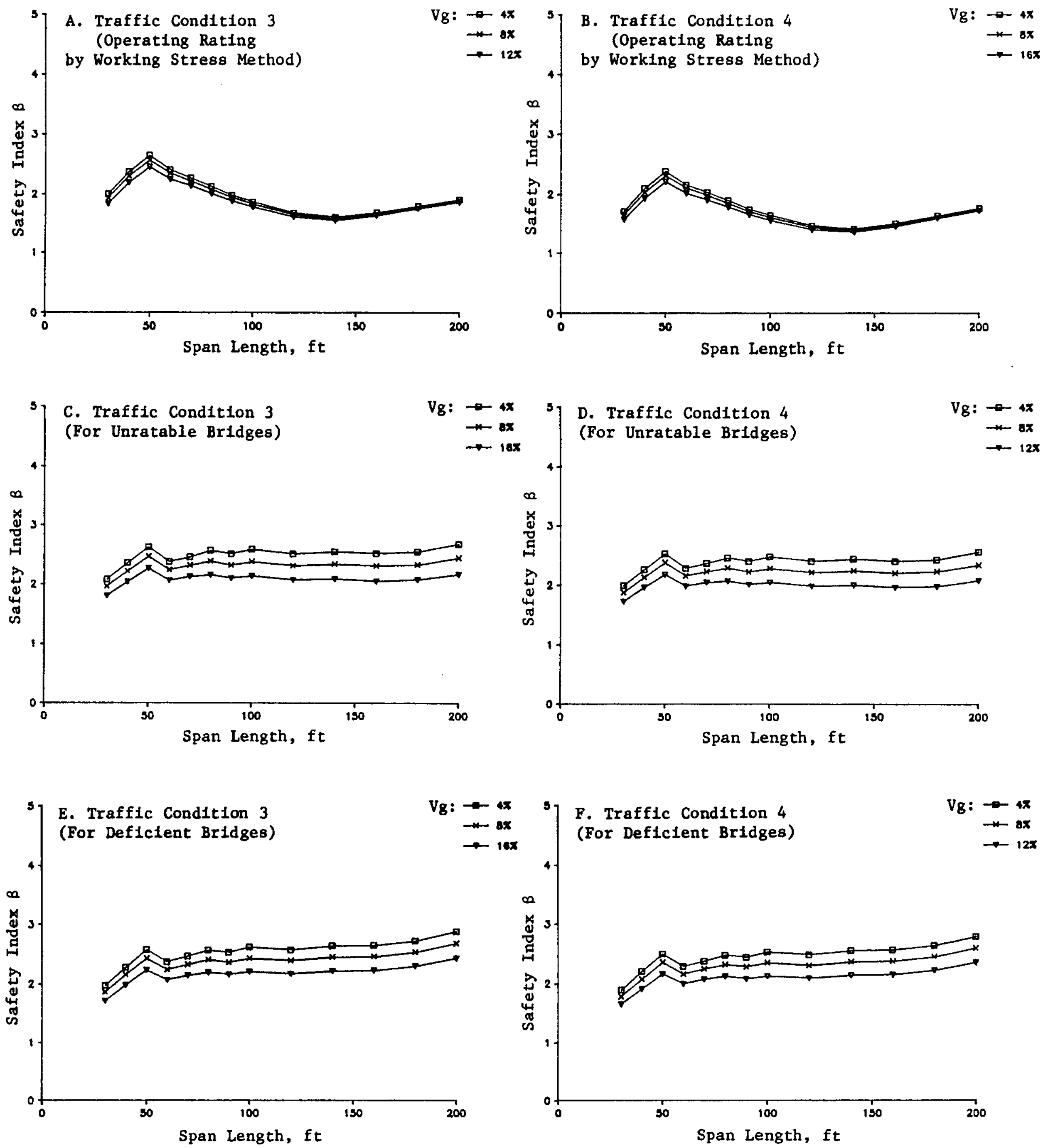


Figure 19. Sensitivity of Safety Index β to $V_I(P/C)$.

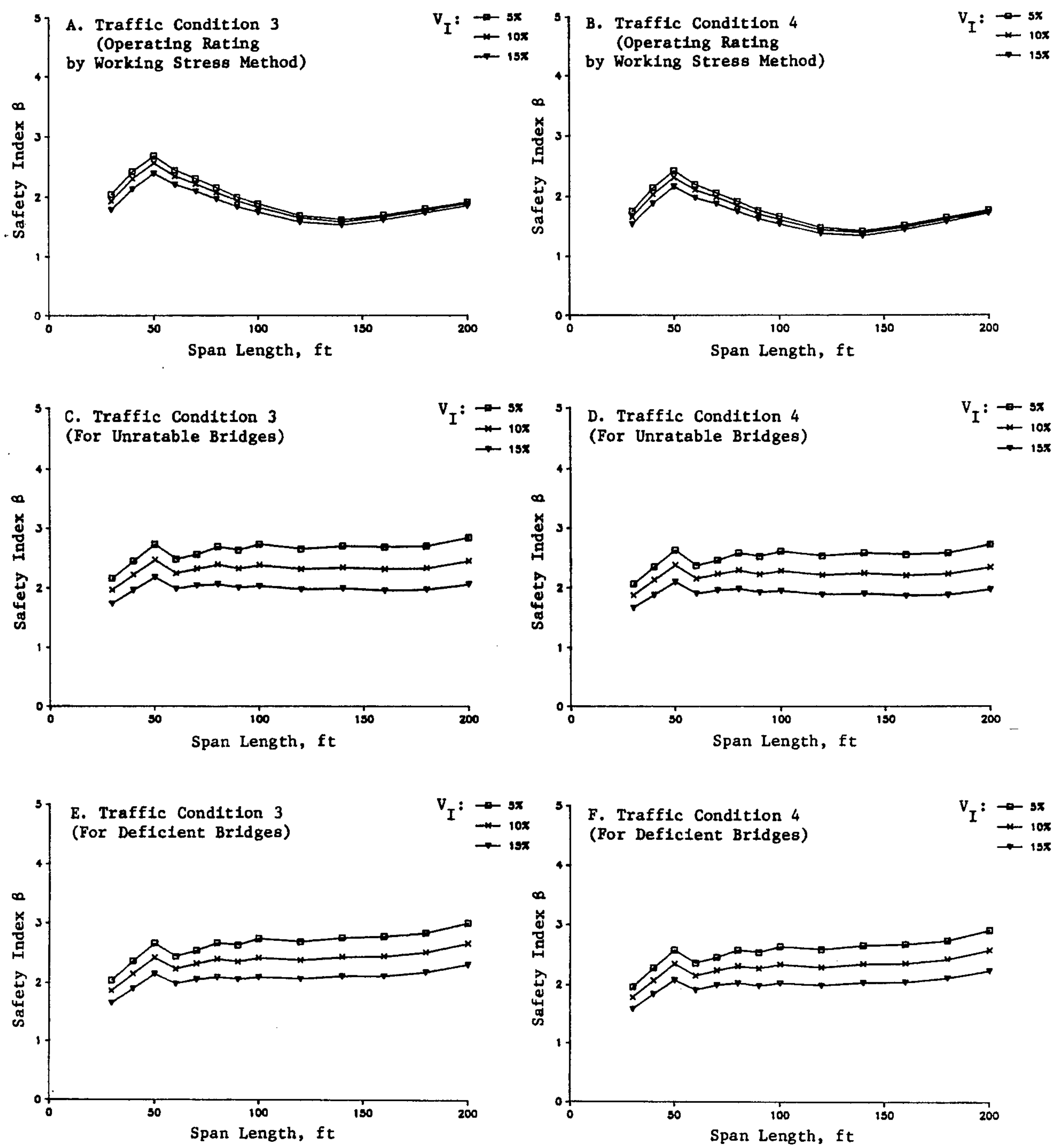


Figure 20. Sensitivity of Safety Index β to V_m (P/C).

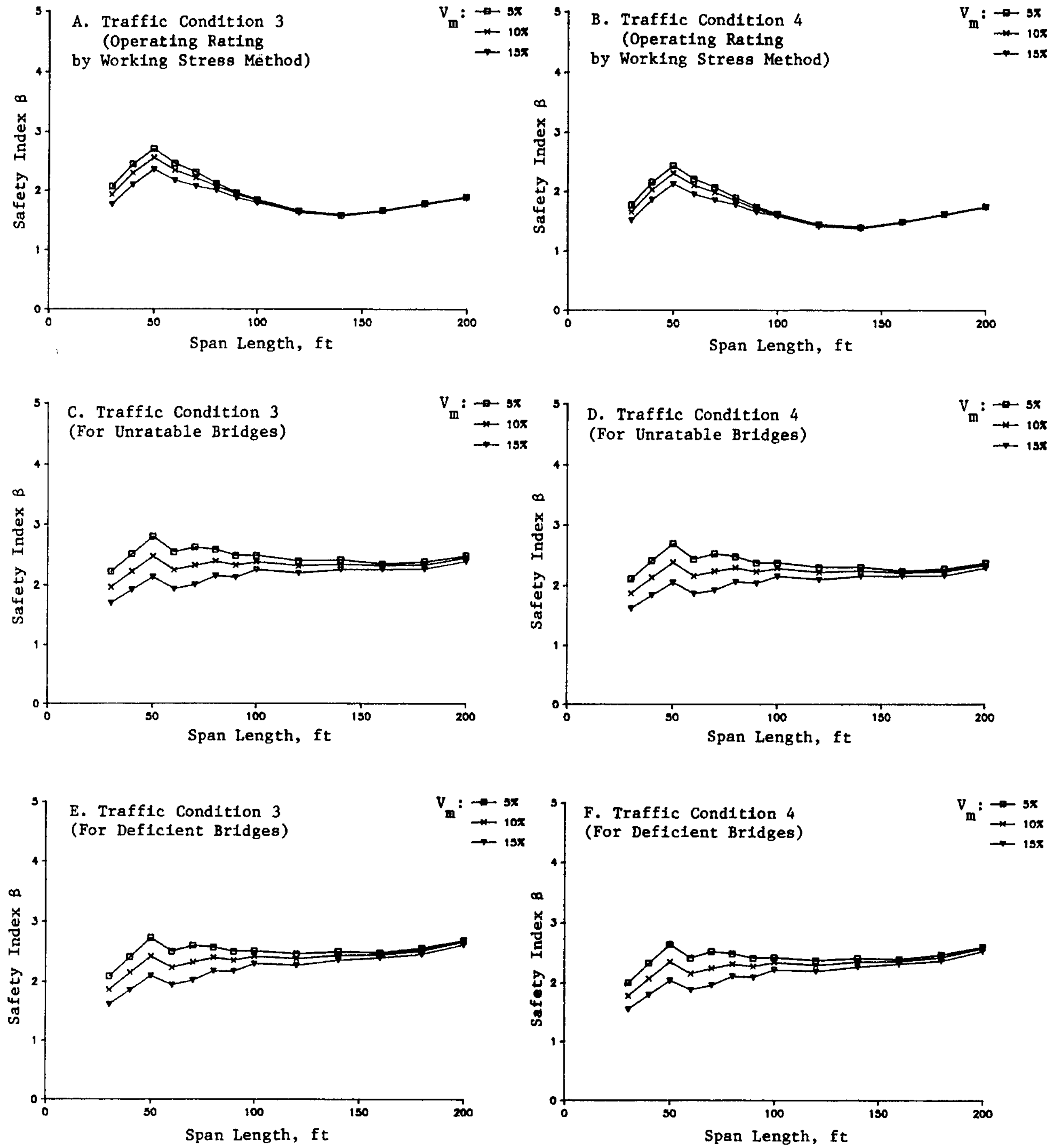
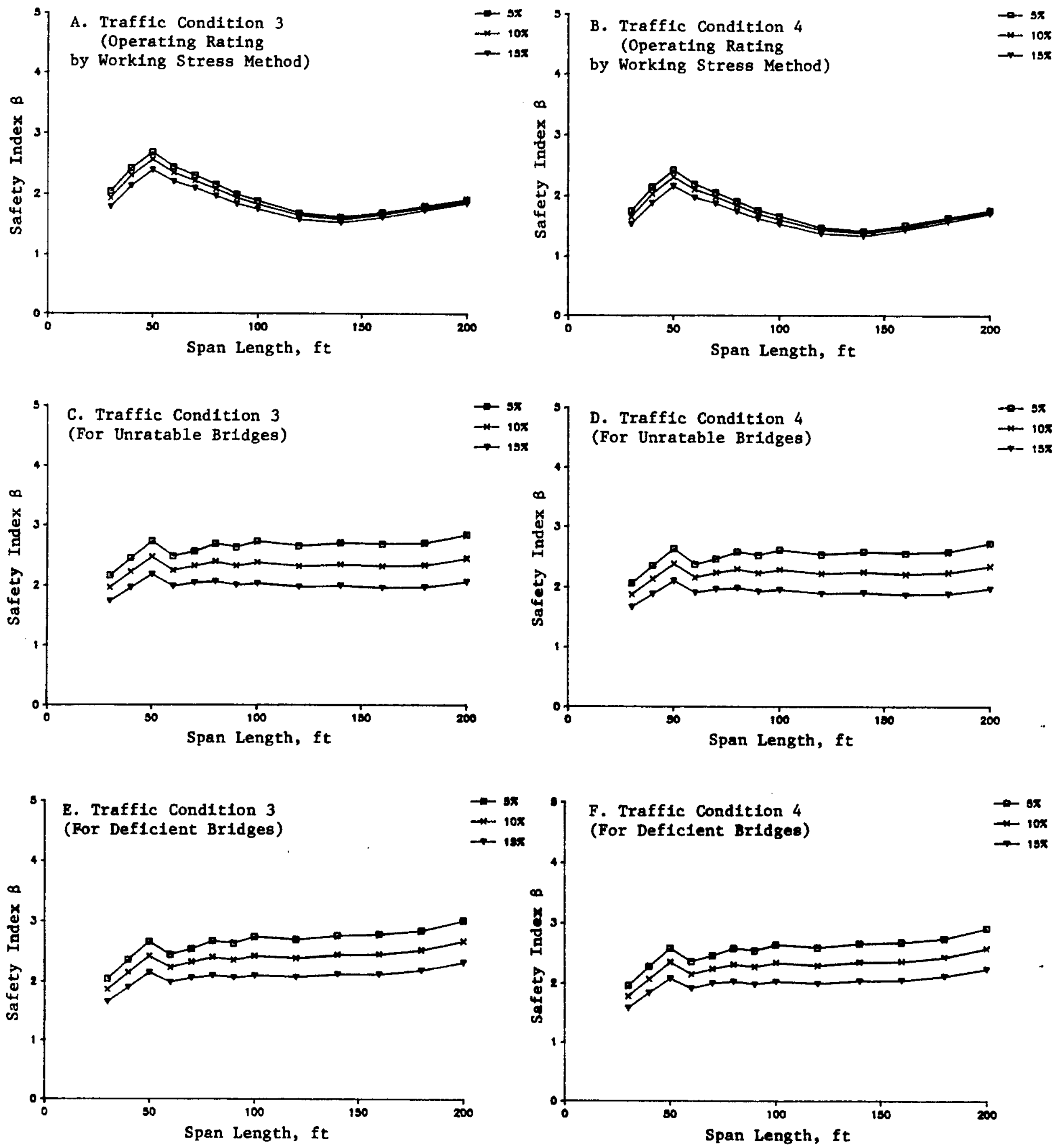


Figure 21. Sensitivity of Safety Index β to $V_{HW.95}$ (P/C).



IV. GUIDELINES FOR PROOF-LOAD TESTING OF HIGHWAY BRIDGES

It was pointed out in Chapter II that the AASHTO codes do not provide guidance for proof-load testing of bridge structures. Further, no detailed procedure manuals are available to assist the test engineer in such work. For the proposed proof-load test program, such a document is essential for quality assurance. As a result of this study, a draft procedure manual has been developed and is included here as Appendix A. This draft may be further modified with greater knowledge and experience to be obtained by application.

This document was prepared in two steps: 1) a quantitative study to develop the target proof-load requirements suitable for bridges in the United States, based on the criterion of uniform structural safety, and 2) integration of professional experience elsewhere in load-test practice, including that documented in the codes, guidance manuals, and elsewhere in the literature. Step 1 has been presented in detail in Chapter III, and Step 2 was carried out by consulting and summarizing specifications and recommendations included in the literature.

Table 9. R-permit bridges in New York State.

Restriction Reason	Number of Bridges
Low Operating Rating (Unratable)	76
Low Operating Rating (Condition Rating < 4)	216
Low Operating Rating (Condition Number ≥ 4)	220
Posted for Load Restriction	22
Design Load < H20	34
Below Minimum Width	24
Low Rating for Primary-Member Condition	105
Low Rating for Deck Condition	12
Total	709

V. ECONOMIC ANALYSIS

The proof-load test program is intended to apply to two groups of structures: unratable and deficient bridges. Nevertheless, the technology developed here may be applied by other state and local transportation agencies.

Of 7751 state-owned bridges in New York, 1735 are unratable due to lack of reliable analysis methods, structural components inaccessible for inspection, or lack of required information (e.g., design plans). To meet federal rating inventory requirements, a reliable rating procedure must be developed for these bridges. One possible solution is developing improved analytical procedures, but these must use approximations based on necessary assumptions that have to be verified. On the other hand, proof-load tests subject bridge structures to real loads, and their ratings thus can be more reliably determined based on observed responses.

The other group of structures to benefit from a proof-load test program includes some of the 709 R-permit bridges in the state highway system. Various restrictions on traffic apply to them. In New York's current rating system the following criteria are used to designate R-permit bridges:

1. Low operating rating: below H29 upstate and H33 downstate
2. Load-posted
3. Low design load: less than H20
4. Narrow bridge width: below 24 ft upstate and 28 ft downstate
5. Low primary-member condition rating: rated 3 or less
6. Low deck-condition rating: rated 1
7. Region prerogative: any reason

Reasons for restriction of these R-permit bridges are listed in Table 9. Proof-load testing may increase load ratings of the bridges with low operating ratings, particularly the 220 bridges with low operating ratings and condition ratings of 4 or more. It is highly probable that these bridges will survive proof-load tests and have their restrictions removed, permitting reduction or elimination of load limits.

A total of 1955 bridges are suggested for proof-load tests, this being the sum of the 1735 unratable and 220 R-permit bridges. It is assumed that on the average, each bridge will take two days for testing and that a calendar year includes 9 testing months or 195 testing days. The total of 1955 bridges would require about 20 years,-testing 98 bridges annually. Economic examination of the

Table 10. Costs of proof-load test program (analysis based on a 20-year program).

	UNIT COST x QUANTITY	TOTAL PRESENT WORTH	LIFE SPAN (years)	ANNUAL COST Year 1	ANNUAL COST Year 20
1. EQUIPMENT (uniform annual worth):					
Loading Trucks	$200,000x2	$400,000	15	$46,240	$46,240
Concrete Blocks	$200x100	10,000	30	1,156	1,156
Transport Truck	$50,000x1	50,000	15	5,780	5,780
Transducers	$1,000x20	20,000	10	2,944	2,944
Data Processing System	$25,000x1	25,000	20	1,840	1,840
Van	$30,000x1	30,000	15	3,468	3,468
Traffic Control Trucks	$75,000x3	225,000	15	26,010	26,010
Miscellaneous Items				4,372	4,372
Subtotal		$750,000		$91,810	$91,810
2. PERSONNEL (non-uniform annual worth):					
Proof Load Test Crew					
1 Grade 24 Test Engineer				$53,340	$112,379
2 Grade 20 Assistant Engineers				86,866	183,014
3 Grade 8 Engineering Technicians				68,355	144,014
Traffic Control Crew					
8 Grade 6 Technicians				122,884	258,898
Subtotal				$331,445	$698,305
GRAND TOTAL COSTS				$423,255	$790,115

Note: See Appendix B for computation details.

proof-load test program in this report consists of cost-benefit analyses, which explore cost-effectiveness of the program.

In the analysis presented here, an annual discount rate of 4 percent is assumed in the calculations. Note that the estimated monetary values and numerical figures given here without reference sources were based on either best knowledge of research personnel or conversations with experienced personnel within or outside the Department.

A. Costs

Total costs of the proof-load test program consist of expenses for equipment and personnel needed for its operation. Both items are summarized in Table 10 and their calculations are included in Appendix B.

1. Equipment Costs

As included in Table 10, two specially built vehicles are needed for loading, each equipped with a built-in crane for loading and unloading as well as a remote control system. Estimated present cost for each vehicle is \$200,000, with a life span of 15 years. A total of 100 concrete 1/2-yd^3 blocks weighing 2025 lb each will be needed at \$200 each, resulting in a total present cost of \$10,000, with an assumed 30-year life span. These blocks will be used as movable loads for convenient loading and unloading. An additional truck is needed for transportation of the concrete blocks to complement the two loading vehicles. Its estimated present cost of \$50,000 for a 15-year life span results in a \$5,780 annual cost.

Also shown in Table 10 is a data acquisition and processing system to consist of: 1) 20 displacement and strain transducers, at an annual cost of \$2,944, assuming a 10-year life span; 2) a data-processing system of \$25,000, for a life span of 20 years; and 3) a van at \$30,000 to house the data-processing system for a 15-year life span. Additionally, three specially equipped trucks will be needed for traffic control at an annual cost of \$26,010 for a 15-year life span. Miscellaneous items, such as cables for transducers and tools for instrumentation are estimated at 5 percent of the total equipment cost (Table 10).

Initial purchase cost of all the equipment needed for implementation of the proof-load test program will be \$750,000. Additionally, after 15 years all vehicles involved will need to be replaced for a \$1.22 million projected cost. If all equipment costs for the 20-year program are uniformly distributed over the 20 years, an annual cost of \$91,810 is estimated as shown in Table 10.

2. Personnel Costs

A test crew to operate the program is expected to consist of a test engineer (Grade 24), two assistant engineers (Grade 20), and three technicians (Grade 8). The personal service cost (including fringe benefits) is estimated at \$208,561 for the first year. For traffic control, it is estimated that eight technicians (Grade 6) will be needed at a cost (including fringe benefits) of \$122,884 for a 9-month test period in the first year (Table 10). It is suggested that an engineer from the region with jurisdiction for the tested bridge join the crew during testing to minimize test operation cost and maximize benefits of technology transfer within the Department. Total personnel costs are estimated at about \$331,000 for Year 1 and \$700,000 for Year 20.

3. Summary

Table 10 shows that a grand total cost of \$423,255 is estimated for the first year of the proposed program. The cost for each bridge is about \$4,319, assuming that 98 bridges will be tested.

B. Benefits

The proof-load test program will result in benefits for the Department and highway users in terms of cost reduction and safety enhancement. These benefits are estimated, using a discount rate of 4 percent over the lifetime of 20 years.

Table 11. Benefits of proof-load test program (analysis based on a 20-year program).

	ANNUAL BENEFIT Year 20	ANNUAL BENEFIT Year 1
1. BENEFITS TO THE DEPARTMENT		
Annual cost for rating by consultants	$1,099,775	$522,000
2. BENEFITS TO HIGHWAY USERS		
Annual reduced mileage for restricted trucks = 1,087,680 miles		
Reduction in travel time	$5,628,405	$133,574
Reduction in fuel consumption	4,995,634	118,557
Reduction in lubricant consumption	152,620	3,622
Total user benefits	$10,776,660	$255,753
3. SAFETY BENEFITS		
Annual reduced mileage for restricted		
Annual reduced accidents = 6.336		
Reduction in fatal accident costs	$2,899,783	$68,818
Reduction in personal injury costs	2,095,978	49,742
Total safety benefits	$4,995,761	$118,560

Note: See Appendix C for computation details.

This benefit analysis is summarized in Table 11, and detailed calculations are provided in Appendix C. Discussion of analysis details follows.

1. <u>Benefits to the Department</u>

Without the proposed proof-load test program, consultants will have to be hired to rate 1735 unratable state-owned bridges; costs for hiring them are estimated here as benefits to the Department. On the average, this would now cost about $6,000 per bridge. For 87 bridges annually over the 20-year program life, benefits are estimated at $522,000 for Year 1 and $1,099,775 for Year 20, as shown in Table 11 and detailed in Appendix C.

2. <u>Benefits to Highway Users</u>

Highway users will also benefit from the proposed proof-load test program, due to the expected removal of load restrictions on the 220 R-permit bridges and resulting reduction in travel time, cost, and mileage. These benefits are estimated based on an annual reduced-mileage for restricted trucks as follows:

$$\left(\begin{array}{l}\text{Annual}\\ \text{reduced}\\ \text{mileage}\\ \text{for}\\ \text{restricted}\\ \text{trucks}\end{array}\right) = \left(\begin{array}{l}\text{Annual}\\ \text{total}\\ \text{mileage}\\ \text{traveled}\\ (\underline{35})\end{array}\right) \times \left(\begin{array}{l}\text{Ratio of}\\ \text{trucks}\\ \text{to all}\\ \text{vehicles}\\ (\underline{36})\end{array}\right) \times \left(\begin{array}{l}\text{Ratio of}\\ \text{restricted}\\ \text{trucks to}\\ \text{total}\\ \text{truck}\\ \text{traffic}\end{array}\right) \times \left(\begin{array}{l}\text{Annual ratio}\\ \text{of reduced}\\ \text{detour}\\ \text{mileage}\\ \text{to total}\\ \text{road}\\ \text{mileage}\end{array}\right)$$

$$= 103 \text{ billion miles} \times 10\% \times 12\% \times 0.00088 = 1{,}087{,}680 \text{ miles} \qquad (11)$$

where annual ratio of reduced detour mileage to total road mileage = detour road mileage 8 miles per bridge x 11 bridges (expected to have load restrictions removed) per year/total road mileage of 100,000 miles = 0.00088. Travel time cost for commercial vehicles is now estimated at $7 per hour (35), or $0.1228 per mile for an average speed of 57 mph (36). (This cost includes driver's wage.) Thus, the saving from reduced travel time is $133,574 for the first year over the annual reduced-mileage for restricted trucks.

Reduced operating costs include major savings in fuel and lubricant consumption. Fuel consumption of a two-axle, six-tire truck for a speed of 57 mph on a level road is estimated to be 0.109 gal/mi. Using such a truck as a typical case and a gasoline price of $1 per gal, fuel saving is $118,557 per year, for the 1,087,680 annual reduced-travel-miles of restricted trucks. According to the Institute of Transportation Engineers (37), engine oil consumption is 3.33 qt/1000 mile on the average. Assuming an average oil cost of $1/qt the resulting saving is about $3622 per year, for the same annual reduced-mileage for restricted trucks.

Another cost saving to the public is reduced pollution caused by operating vehicles, including air pollution from exhaust, water pollution from oil and gasoline leakage, etc. Reduction in travel mileage by restricted trucks will result in decrease of these pollutants and thus a benefit to the public. Due to lack of reliable data, this benefit is not quantified here.

Total user benefits estimated at this time are $255,753 for the first year, as shown in Table 11 and detailed in Appendix C. They will increase cumulatively by a factor of $n \times 1.04^{n-1}$ for Year n. For example, these benefits will be $10.8 million for the 20th year as shown there.

3. Safety Benefits

Due to reduction of restricted-truck detouring, resulting from expected removal of load restrictions after proof-load testing, fewer traffic accidents are expected as well. Because both the Department and the public may benefit from this increased safety, these benefits are estimated separately from the first two categories of benefit.

Reduction in accidents is expected due to less mileage traveled, assuming that the number of accidents is a linear function of travel mileage. The reduced number of accidents by restricted trucks is estimated as follows:

$$\begin{pmatrix}\text{Annual}\\ \text{reduction}\\ \text{in}\\ \text{accidents}\end{pmatrix} = \begin{pmatrix}\text{Total annual}\\ \text{accidents}\\ \text{involving}\\ \text{trucks}\end{pmatrix} \times \begin{pmatrix}\text{Ratio of}\\ \text{restricted}\\ \text{truck traffic}\\ \text{to total truck}\\ \text{traffic}\end{pmatrix} \times \begin{pmatrix}\text{Ratio of}\\ \text{reduced detour}\\ \text{mileage to total}\\ \text{road mileage}\end{pmatrix}$$

$$= 60{,}000 \times 12\% \times 0.00088 = 6.336 \tag{12}$$

Table 12. Summary of economic analysis (based on a 20-year program).

	Annual Worth	
	Year 1 ($1,000)	Year 20 ($1,000)
For the Department		
Costs	$423	$790
Benefits	522	1,100
Savings (= benefits - costs)	99	310
For public users	250	10,777
Savings (= benefits)		
For safety	119	4,996
Savings (= benefits)		
Benefit-to-Cost Ratio		
For the Department	1.2	1.4
For the Department, public users, and safety	2.1	21.4

About 1 percent of accidents involving trucks are fatal (35), costing $1,086,147 per accident (35), and 60 percent of these result in personal injuries (36), costing $13,090 per accident (37). Using these figures and an annual number of reduced accidents of 6.336 (Eq. 12) the total safety benefit is estimated to be $118,557 for the first year and near $5 million for the 20th year, based on a discount rate of 4 percent and accumulation of savings (Table 11 and Appendix C).

C. Comparison of Costs and Benefits

Costs and benefits associated with the proposed proof-load test program have been estimated as summarized in Table 12. Implementing the program requires $750,000 for equipment, and $1.22 million will be needed 15 years later for replacing the required vehicles. Annual benefit-to-cost ratio for the Department is estimated at 1.2 to 1.4 over the 20-year lifetime of the program. Table 12 also shows an increasing benefit-to-cost ratio to public users of the highway system, from 2.1 for Year 1 to 21.4 for Year 20, mainly due to accumulation of benefits over time. Combined with expected higher reliability of rating by proof-load testing, these results present implementation of the proof-load test program as an attractive investment. It is noted that other benefits of the proof-load test program are not included, such as pollution reduction and technology advancement, due to lack of reliable statistical data in terms of monetary values.

VI. CONCLUSIONS

A proof-load test program operated by the Department will be a cost-effective approach to meeting federal requirements for reliable load rating of all bridges in New York State. The estimated annual benefit-to-cost ratio for the Department varies from 1.2 to 1.4 over the proposed testing period of 20 years. When public users and safety are also included, the annual benefit-to-cost ratio is estimated at 2.1 to 21.4 over the program life of 20 years. The proof-load test program will also provide a technical advancement over analytical approaches, in providing more reliable ratings. Technical details of implementing such a program have been examined here, and a draft procedure manual has been developed as a first step of quality assurance.

ACKNOWLEDGMENTS

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APPENDIX A

DRAFT PROCEDURE MANUAL FOR PROOF LOAD TESTING OF HIGHWAY BRIDGES

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CONTENTS

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1. INTRODUCTION

1.1 Purpose

This procedure manual establishes guidelines for load rating of existing highway bridges by nondestructive proof load testing.

1.2 Scope

This procedure shall be applied to determine load ratings of existing highway bridge structures, in conjunction with the AASHTO Manual for Maintenance Inspection of Bridges.

1.3 Applicability

This procedure is applicable to short and medium span highway bridges of steel, prestressed concrete, and reinforced concrete. A test engineer shall control the test, having at least the minimum qualifications for inspection personnel defined by the AASHTO Manual of Maintenance Inspection of Bridges and specializing in bridge structure behavior, field testing, and evaluation. A proof load test shall consist of three steps: planning, testing, and analysis and reporting, executed as required here. Because of the complexity of the subject and the numerous parameters involved, engineering judgment may be required in applying this manual.

2. SYMBOLS AND DEFINITIONS

Y_p = Target proof load effect

R_p = Final proof load effect or strength

RF = Rating factor

D_a = Additional dead-load effect to be added to the structure after proof-load test

L_n = Nominal live-load effect

I_n = Nominal impact factor

ϕ = Resistance reduction factor

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α_D = Dead-load factor for additional dead load effect

α_L = Live-load factor for target proof load effect

3. PLANNING

3.1 General

The proof load test is a full scale, nondestructive examination of safe load capacity of bridges. It can be used either to enhance an existing rating or provide a reliable rating that cannot be given by other analytical methods. The test must be carefully planned to maximize the probability of its success and minimize risk of bridge damage due to the test load.

3.2 Preliminary Investigation

A preliminary investigation shall be conducted to collect necessary information for pertinent decisions, including but not limited to:

1. Collection and review of background information, such as identification of the bridge, traffic carried, maintenance history, inspection records, existing load rating, etc.;

2. Field inspection of the bridge structure and identification of deficient and critical members, potential failure modes, and temporary features that may affect the load-response relationship of the structure (e.g., frozen bearings);

3. Identification of significant changes in structural behavior and traffic demand since the latest inspection;

4. Evaluation of structural soundness and condition; and

5. Determination of a load rating (if not available) based on best knowledge on the structure and available calculation techniques.

As a result of this preliminary investigation, bridges with any of the following features may not be recommended for a proof load test:

6. Deficient and severely deteriorated materials that may not be able to carry the starting proof load as defined in 3.3.2;

7. Potential for brittle failure making the bridge unable to carry the starting proof load as defined in 3.3.2.

8. Low probability of improving the available rating; and/or

9. Severely dêteriorated substructure components.

3.3 Load Magnitude Determination

3.3.1 Target Proof Load

The level of target proof load effect shall be determined using the following equation:

$$\phi\ Yp = \alpha_L\ Ln\ (1 + In\ ) + \alpha_D\ D_a \qquad (A3.1)$$

where ϕ = 0.95 for steel and prestressed concrete,
0.90 for reinforced concrete, or
0.80 for nonredundant structures
α_D = 1.25 for additional dead load

If the existing analytical rating factor of the bridge equals or exceeds 0.7, then

α_L = 1.35 for low volume roadways, with reasonable enforcement and apparent control of overloads
= 1.45 for high volume roadways, with reasonable enforcement and apparent control of overloads
= 1.80 for low volume roadways, and significant sources of overloads without effective enforcement
= 1.90 for high volume roadways, and significant sources of overloads without effective enforcement

or

α_L = 1.45 for low volume roadways, with reasonable enforcement and apparent control of overloads
= 1.55 for high volume roadways, with reasonable enforcement and apparent control of overloads
= 1.90 for low volume roadways, and significant sources of overloads without effective enforcement
= 2.00 for high volume roadways, and significant sources of overloads without effective enforcement

3.3.2 Starting Proof Load

The level of starting proof load effect shall be induced by:

1. The posted load if the bridge structure is load-posted,
2. HS20, or
3. A load determined by the test engineer, if the bridge's capacity to support loads in 3.3.2.1 and 3.3.2.2 is in question.

3.4 Load Positioning

The proof load shall be positioned to induce maximum stresses in primary and/or critical members. Thus, several positions in both longitudinal and transverse directions may be considered.

3.5 Selection of Response Measurement and Instrumentation

3.5.1 Displacement, Rotation, Joint Movement, and Strain

Characteristic displacements and/or strains of primary and/or critical members shall be the major responses monitored in the load test. Measurement instrumentation shall be installed at points where maximum responses are expected. If deemed applicable and reliable, LVDT, inclinometers, strain gages, displacement or strain transducers, and other devices may be selected for response measurement.

3.5.2 Support Movement

Critical movement and/or settlement of supports under loads shall be measured at characteristic locations. Level surveys may be selected for this purpose.

3.5.3 Cracking

Cracking-prone members shall be identified. Crack monitoring procedure and means shall be determined.

3.6 Safety Requirements

Safety requirements to protect test personnel and the public shall be identified to accommodate field conditions and requirements for loading, instrumentation, and measurements, in accordance with Department safety policies. They shall include, but not limited to, traffic control and scaffolding.

3.7 Planning Report

All information collected and decisions made during planning shall be documented in a planning report. The planning report shall include, but not be limited to:

1. Results of the preliminary investigation;

2. Magnitude of starting load and target load, their positions, and the loading sequence of test;

3. Instrumentation selected, their positions, and their monitoring procedure;

4. Safety measures; and

5. An estimation of the probability of successful testing, expected test results compared to the existing rating or the approximate rating obtained in 3.2.5, and recommendations for action to be taken based on

these results (possibly not to proof load test the bridge due to high probability of bridge damage by the test load.)

4. TESTING

4.1 General

Care must be exercised in executing the proof load test to ensure safety of test personnel. Efforts must be made to maximize reliability of the test data to be obtained and minimize the possibility of bridge damage. The test shall be monitored carefully to detect unusual structural behaviors that may warrant changes in the planned test procedure.

4.2 Traffic Control and Safety Measures

Before commencing the test, the traffic affected must be carefully controlled. Precautions must be taken during the test for safety of test personnel and the public, in accordance with Department safety policies. The test engineer shall be responsible for all aspects of safety.

4.3 Instrumentation

The instrumentation shall be placed at predetermined positions, unless changes are found necessary to accommodate field conditions. Wherever applicable, they shall be connected to an automatic data acquisition system in a mobile lab. All instrumentation shall be confirmed for proper working condition before loading. Immediately before loading, the instrumentation shall be set to zero output or zero readings shall be taken.

4.4 Loading, Monitoring, and Unloading

4.4.1 Loading

The load shall be applied slowly to eliminate dynamic effects, and increased no less than four times from the starting proof load given in 3.3.2 until one of the criteria for loading termination in 4.4.4 is met. The increments shall be determined by the test engineer, with consideration to minimizing the probability of bridge damage due to the test load.

4.4.2 Unloading

When Loading Termination Criteria 1 or 2 in 4.4.4 are met, the load shall be decreased by increments of no more than the loading increments. When Loading Termination Criteria 3 or 4 in 4.4.4 are met, the load shall be removed immediately in one step.

4.4.3 Monitoring

Immediately before, immediately after, and 5 minutes after each load increment the following response readings shall be taken:

1. Displacement, rotation, joint movement, and/or strain at the instrumented locations;

2. Movement or settlement of supports; and

3. Crack width.

To maximize the probability of successful testing, critical characteristics of the bridge response shall be examined during the test, and the test shall be stopped by the test engineer whenever validity of the data is in question. Relevant changes in the environment shall be also recorded, such as sudden noise and changes in temperature and weather.

4.4.4 Criteria for Loading Termination

Loading shall be terminated, unloading commenced, and deflection recovery recorded if

1. The target proof load level has been reached;

2. 10 percent or more nonlinearity has been observed in responses;

3. Significant substructure movements or settlements have been observed; or

4. Signs of distress have appeared, such as excessive crack widening, or significant development of cracking.

4.4.5 Repeat of Loading

Critical loading cases may be repeated to reduce or eliminate secondary effects, such as adjustments at connections.

4.5 Inspection

After unloading, the bridge shall be inspected to identify damage, residual movement, or distress due to loading.

4.6 Testing Report

All data and information collected during the test shall be documented in a testing report, including but not limited to:

a. Temperature and weather condition during the test;

b. Instrumentation;

c. Loading sequence (with respect to positions) and time, starting load, load increments, final load;

d. Structural responses to each load level; and

e. Relevant observations during the test.

5. TEST RESULT ANALYSIS AND FINAL REPORT

5.1 Test Result Analysis

Test results shall be analyzed carefully after execution of the test. Load response relationships of the primary and/or critical members shall be examined. A structure shall be considered to possess a strength $Rp \leq Yp$ if at the load level R_p,

a. No visible evidence of failure is observed,

b. The responses recover at least 90 percent of those under the load, within 5 minutes after load removal,

c. Linear behavior of the primary and/or critical members is observed,

d. No significant support movement or settlement is observed, and

e. No excessive crack widening is observed.

5.2 Rating

The bridge shall be rated as follows by using Rp in Equation A5.1:

$$RF = \frac{\phi \, Rp - \alpha_D \, D_a}{\alpha_L \, Ln \, (1 + In)} \tag{A5.1}$$

5.3 Posting

If the rating is found to be inadequate, load posting of the bridge shall be considered in accordance with Department policy, using RF Ln as the safe load carrying capacity.

5.4 Final Report

The final report shall cover the planning and testing phase. It shall also include conclusions from the test, the resulting rating of the structure, and recommendations for action, such as strengthening, posting removal, posting, closing, etc. Other relevant information shall also be included in the final report.

APPENDIX B
COST CALCULATIONS

1. EQUIPMENT COSTS:

Loading Trucks
A = 2 * $400,000 * (A/P,4%,30) = $46,240

Concrete Blocks
A = $20,000 * (A/P,4%,30) = $1,156

Transport Truck
A = 2 * $50,000 * (A/P,4%,30) = $5,780

Transducers
A = 2 * $20,000 * (A/P,4%,20) = $2,944

Data Processing System
A = $25,000 * (A/P,4%,20) = $1,840

Van
A = 2 * $30,000 * (A/P,4%,30) = $3,468

Traffic Control Trucks
A = 2 * 3 * $75,000 * (A/P,4%,30) = $26,010

Total Annual Cost of Equipment
$(46,240+1,156+2,944+5,780+1,840+3,468+26,010) = $87,438

Miscellaneous
0.05*$87,438 = $4,372

Total Annual Cost of Equipment $87,438+4,372 = $91,810

2. PERSONAL SERVICE COSTS:

Cash Flow for the nth Year =
A_n = {(Start Salary + Job Salary)/2}*1.14*1.04^n [1]

Proof-Load Test Crew:

1 Grade 24 $(42,003+51,576)/2*1.14=$53,340
A_1 = $ 53,340 A_{20} = $112,379

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2 Grade 20 2*$(34,030+42,169)/2*1.14=$86,866
A_1 = $ 86,866 A_{20} = $183,014

3 Grade 8 3*$(17,425+22,549)/2*1.14=$68,355
A_1 = $ 68,355 A_{20} = $144,014

Total Cost of Proof Load Test Crew A_1 = $208,561 A_{20} = $439,407

Traffic Control Crew

8 Grade 6 8*$(15,582+20,349)/2*9/12*1.14 = $122,884
A_1 = $122,884 A_{20} = $258,898

Total personal service cost A_1 = $331,445 A_{20} = $698,305

3. TOTAL COSTS OF PROOF LOAD TEST PROGRAM

A_1 = $91,810 + $331,445 = $423,255
A_{20} = $91,810 + $698,305 = $790,115

4. COSTS OF PROOF LOAD TEST PER BRIDGE

Year 1	$423,255/100 = $4,232
Year 20	$790,115/100 = $7,901

APPENDIX C

BENEFIT CALCULATIONS

1. BENEFITS TO THE DEPARTMENT

Cost of rating by consultants per bridge	$6,000
Annual savings of Year 1 $6,000/bridge * 87 bridges	$522,000
Annual savings of Year n	$\$522{,}00*1.04^{n-1}$
Annual savings of Year 20	$1,099,775

2. BENEFITS TO HIGHWAY USERS

Annual reduced mileage by restricted trucks:	
Annual total travel mileage	103 billion miles
Ratio of truck traffic to all traffic	10%
Ratio of restricted truck traffic to total truck traffic	12%
Detour length per restricted bridge	8 miles
Annual number of bridges having restriction removed	11
Annual length of detour to be reduced	88 miles
Total length of road	100,000 miles
Ratio of detour length to total road length	0.00088
Annual reduced mileage by restricted trucks	1,087,680 miles
Saving in reduced travel time:	
Cost of travel time per hour	$7/hr
Average speed	57 mph
Cost of travel time per mile	$7/hr/57mph = $0.1228/mile
Travel time saving (Year 1)	$0.1228*1,087,680 = $133,574
Saving in reduced fuel consumption:	
Fuel consumption	0.109 gal/mile
Gasoline cost	$1/gal
Fuel saving (Year 1)	$1/gal*0.109gal*1,087,680 = $118,557
Saving in lubricant consumption:	
Engine oil consumption	3.33 qt/1,000 miles
Engine oil cost	$1/qt
Lubricant saving (Year 1)	$1/qt*3.33qt*1,087.680 = $3,622
Total annual user savings in Year 1	133,574 + 118,557 + 3,622 = $255,753
Total annual user savings in Year n	$n*\$255{,}753*1.04^{n-1}$
Total annual user savings in Year 20	$10,776,660

3. SAFETY BENEFITS

Annual reduced accidents:	
Ratio of annual reduced accidents	.00088*.12 = 0.0001056
Annual accidents involving trucks	60,000
Annual reduced accidents	60,000*0.0001056 = 6.336
Saving in reduced fatal accidents:	
Cost per fatal accident	$1,086,147
Ratio of fatal accidents to total accidents	1%
Reduced fatal accidents	6.336*.01 = 0.06336
Saving in reduced fatal accidents	1,086,147*.06336 = $68,818
Saving in reduced personal injuries:	
Cost of personal injuries per accident	$13,090
Ratio of personal injury accidents to total accidents	60%
Reduced personal injury accidents	6.336*0.60 = 3.8
Saving in reduced personal injuries	3.8*13,090 = $49,742
Total annual safety savings of Year 1	68,818+49,742 = $118,560
Total annual safety savings of Year n	n*$118,560*1.04^{n-1}
Total annual safety savings of Year 20	$4,995,761