Concrete Bridge
Shear Load Rating Synthesis Report

U.S. Department of Transportation
Federal Highway Administration

FOREWORD

Currently, evaluating shear capacity for load rating of concrete bridges is quite challenging due to the evolution of the shear design provisions over approximately 80 years in the AASHTO Standard Specifications for Highway Bridges (Standard Specifications) and the multiple procedures included in past and current editions of the AASHTO LRFD Bridge Design Specifications (LRFD Specifications). There is also a degree of uncertainty and inconsistency in the shear capacity load rating requirements of the AASHTO Manual for Bridge Evaluation (MBE) which adds to the complexity of the task.

The objective of this report is to synthesize the technical aspects of shear load rating for concrete bridges and the challenges and difficulties States experience in doing so. This report summarizes the history of changes in the shear design and rating provisions in the aforementioned AASHTO publications. The report also includes survey results from nine State Departments of Transportation (States) that have large inventories of concrete highway bridges of diverse types. The survey results summarize each State's practices and identifies the issues they face in rating existing concrete bridges using the MBE shear provisions. Finally, the report makes findings and recommendations to improve the practice and understanding of concrete bridge shear load rating.

The subject matter expert technical reviewers for this report included Matt Farrar (Idaho Transportation Department) and Kevin Keady (California Department of Transportation) as well as other engineering professionals and academics from the highway bridge community. Their advice, counsel and contributions during the preparation of this synthesis report are greatly appreciated.

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Cover Photos: **Top Right:** Bixby Creek Bridge, California State Route 1. Photo taken by John Baker, courtesy Caltrans. **Lower Right:** South Fork of the Eel River Bridge, US-101, California. Photo taken by Matt O'Leary, courtesy Caltrans. **Left:** North to West Direct Connector, I-35/US-183 Interchange, Texas. Photo taken by Lisa Powell, courtesy P.E. Structural Consultants, Inc. Cover logo: courtesy of FHWA.

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16. Abstract

This report synthesizes the current state of practice in the shear load rating of concrete bridges. A brief overview of the historical approaches, encompassing the methods from the first to the latest AASHTO Specifications, to shear demand and resistance is presented. The overview includes load combinations, live load distribution, and shear strength in both reinforced and prestressed concrete. A historical overview of the bridge load rating is presented. The use of software in concrete shear load rating is also investigated. The study of current policies, practices, and challenges in concrete bridge shear load rating is conducted through a survey of nine state DOTs. An extensive literature review is performed to highlight recent and relevant research on concrete shear strength, concrete bridge behavior in shear and concrete bridge shear load rating. This synthesis report identifies recommendations on research, modifications to existing specifications and manuals, and the development of clear and concise guidance to practitioners.

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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				
in²	square inches	645.2	square millimeters	**mm²**
ft²	square feet	0.093	square meters	**m²**
yd²	square yard	0.836	square meters	**m²**
ac	acres	0.405	hectares	**ha**
mi²	square miles	2.59	square kilometers	**km²**
Volume				
fl oz	fluid ounces	29.57	milliliters	**mL**
gal	gallons	3.785	liters	**L**
ft³	cubic feet	0.028	cubic meters	**m³**
yd³	cubic yards	0.765	cubic meters	**m³**
NOTE: volumes greater than 1000 L shall be shown in m³				
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 degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	**°C**
Illumination				
fc	foot-candles	10.76	lux	**lx**
fl	foot-Lamberts	3.426	candela/m2	**cd/m²**
Force and Pressure or Stress				
lbf	poundforce	4.45	newtons	**N**
lbf/in²	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				
mm²	square millimeters	0.0016	square inches	**in²**
m²	square meters	10.764	square feet	**ft²**
m²	square meters	1.195	square yard	**yd²**
ha	hectares	2.47	acres	**ac**
km²	square kilometers	0.386	square miles	**mi²**
Volume				
mL	milliliters	0.034	fluid ounces	**fl oz**
L	liters	0.264	gallons	**gal**
m³	cubic meters	35.314	cubic feet	**ft³**
m³	cubic meters	1.307	cubic yards	**yd³**
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 degrees)				
°C	Celsius	1.8C+32	Fahrenheit	**°F**
Illumination				
lx	lux	0.0929	foot-candles	**fc**
cd/m²	candela/m2	0.2919	foot-Lamberts	**fl**
Force and Pressure or Stress				
N	newtons	02.225	poundforce	**lbf**
kPa	kilopascals	0.145	poundforce per square inch	**lbf/in²**

Source: Federal Highway Administration. 2013. "SI* (Modern Metric) Conversion Factors". https://www.fhwa.dot.gov/publications/convtabl.cfm.

Note: SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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Abbreviations

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ADTT	Average Daily Truck Traffic
ASD	Allowable Stress Design
ASR	Allowable Stress Rating
BrR	AASHTOWare Bridge Rating™ software
CIP	Cast-in-Place
FHWA	Federal Highway Administration
HSC	High Strength Concrete
LFD	Load Factor Design
LFR	Load Factor Rating
LLDF	Live Load Distribution Factors
LRFD	Load and Resistance Factor Design
LRFR	Load and Resistance Factor Rating
MBE	AASHTO Manual for Bridge Evaluation
MCFT	Modified Compression Field Theory
NCHRP	National Cooperative Highway Research Program
PCF	Pounds per Cubic Foot
PCI	Precast Concrete Institute
PDF	Portable Document Format
PT	Post-Tensioned
RC	Reinforced Concrete
RCDG	Reinforced Concrete Deck Girder
SHV	Specialized Hauling Vehicles
STM	Strut and Tie Modeling
SU	Single Unit

CHAPTER 1.   INTRODUCTION

As the most prevalent bridge construction material in the United States, concrete has been widely accepted as an effective bridge construction material since the early twentieth century and remains so to the present day. More than 614,000 bridges exist in the National Bridge Inventory with approximately two-thirds of that inventory possessing a concrete superstructure. Concrete has been used in many forms in these bridges—reinforced and prestressed—reflecting concrete's on-going effectiveness, economy, and adaptability.

While concrete use in bridges has been constant, specifications and provisions for its design have not, and particularly so for shear. Shear design for reinforced and prestressed concrete has evolved from the early twentieth century using methods based on limited physical testing and theory to today, with multiple approaches derived from extensive research programs using full-scale physical testing, nationally and internationally. Concrete bridge shear design provisions in the U.S. began with an Allowable Stress Design (ASD) approach to today's *Load and Resistance Factor Design (LRFD) Bridge Design Specifications* that have shear design provisions depending on the following:

- Concrete type, reinforced or prestressed.
- Appropriateness of a sectional design approach versus the strut and tie method.
- Construction method, such as segmentally constructed or by more conventional means.

Provisions for design do not always translate into accurate methods of predicting strength for load rating, because design methods may incorporate conservative assumptions or simplifications. Strength determination for load rating needs accurate strength prediction; engineers need to be cognizant of this characteristic of design versus load rating.

Design live loads, similar to shear design provisions, have also changed over the years. Changes to design live load provisions range from the simple axle loads of the H10 truck in the inaugural AASHO Standard Specifications[1] in 1931 to the sophistication and complexity of the current live load model, HL-93, which envelopes the force effects of a variety of heavy vehicles. How live load is distributed to individual bridge components has also been subject to change from various editions of the AASHTO Standard Specifications to current LRFD Specifications.

With the multiple approaches for load rating and strength determination, it is not surprising that load rating concrete bridges in shear has proved challenging, complicated, and, at sometimes, confusing to practitioners. Some engineers reportedly claim concrete shear controls load ratings but the bridges do not exhibit signs of shear distress.

The purpose of this synthesis report is to identify issues and challenges States face with regard to load rating concrete bridges in shear and to address specific problems that State Departments of Transportation (DOTs) have encountered when applying the provisions in design and load rating specifications. This report is organized by first summarizing past and current shear design provisions, bridge load rating requirements, and load rating software.

Also presented in this report are survey results from nine State DOTs that focused on their practices and policies in concrete bridge load rating. Results from a literature review are presented, which focused on recent research addressing concrete shear strength, shear behavior in concrete bridges and shear load rating for concrete bridges.

Lastly, this report synthesizes findings on concrete bridge shear load rating and makes recommendations to enhance design and load rating specifications, with a goal to provide clear and concise guidance to practitioners.

1.1. Concrete Bridge Background

1.1.1. United States Statistics

The 2017 National Bridge Inventory reported approximately 615,000 bridges in the United States. Approximately 42 percent of bridges are reinforced concrete bridges and 25 percent are prestressed concrete bridges, totaling two-thirds of all bridge construction in the United States. Reinforced concrete bridges have been built since the early 1900s; most prestressed concrete bridges were built after the 1950s. The 1980s saw a transition in predominant bridge construction from reinforced concrete to prestressed concrete.

Types of concrete bridge elements based on where they are cast include:

- Cast-in-place (CIP) bridge elements such as the decks, girders, bents, and foundations that are constructed on site.
- Precast bridge elements such as deck panels, girders, and bent caps that are fabricated in a controlled environment and assembled on site.

Types of concrete bridge elements broken down by type of reinforcing include:

- Reinforced concrete elements that are strengthened with mild steel reinforcing bars.
- Prestressed concrete elements that are strengthened with high-strength tensioned steel strands. This includes pretensioned elements where strands are stressed before concrete placement or post-tensioned elements where strands in ducts are stressed after concrete has hardened.
- Segmental bridge elements composed of precast or CIP concrete segments typically connected by means of post-tensioning.

Based on the 2017 National Bridge Inventory data, a breakdown of concrete bridges in relation to total number of bridges by superstructure types is as follows:

- Slab span (reinforced concrete [RC] or prestressed, voided or solid, simple or continuous spans) = approximately 77,300 bridges or 13 percent.
- RC T-girder (simple or continuous spans) = approximately 25,900 bridges or 4 percent.
- RC box girder (simple or continuous spans) = approximately 7,300 bridges or 1 percent.
- Pretensioned girder (all shapes, simple spans) = approximately 117,000 bridges or 19 percent.
- Pretensioned girder (all shapes, continuous for LL only) = approximately 18,000 bridges or 3 percent.
- Post-tensioned spliced girder (all shapes, simple or continuous spans) = approximately 600 to 800 bridges or less than 1 percent.
- Post-tensioned box girder (non-segmental, simple or continuous spans) = approximately 8,000 bridges or 1 percent.
- Post-tensioned box girder (segmental, simple, or continuous spans) = 330 bridges or less than 1 percent.
- Box culvert (all sides, bridge class) = approximately 122,200 bridges or 20 percent.

- RC stringers (simple or continuous spans) = approximately 17,000 bridges or 3 percent.

1.1.2. Governing Entities

AASHO was formed December 12, 1914, with the purpose of promoting better highway construction, maintenance, and design methods through a uniform system for all States. Many States had established their own highway designs in the early twentieth century. To achieve uniformity in design, AASHO created a technical guide in 1928 to be used as a standard for design by all States. In 1931, AASHO published the *Standard Specifications for Highway Bridges and Incidental Structures*.[1] On November 13, 1973, AASHO became the AASHTO.

1.1.3. Governing Publications

The *Standard Specifications for Highway Bridges and Incidental Structures*,[1] later renamed to *Standard Specifications for Highway Bridges*, and the *LRFD Bridge Design Specifications* have governed bridge design provisions since 1931 when the 1st edition of the Standard Specifications was published.[1] The 17th edition of the Standard Specifications was the last edition, published in 2002.[20] Today, the Standard Specifications are used as reference or design standards for maintenance and rehabilitation of older, existing structures. The *LRFD Bridge Design Specifications* were first published in 1994;[21] the 2017 *LRFD Bridge Design Specifications* (8th Edition) provides the most current bridge design provisions.[28] The *LRFD Bridge Design Specifications* are the reference specifications for all highway bridge design in the United States and provide bridge evaluation and rehabilitation criteria.

The following publications have been used for the evaluation and rating of existing bridges since 1941: the *Standard Specifications for Highway Bridges* (1941 to 1976), the *Manual for Maintenance Inspection of Bridges* (1970 to 1990),[12] the *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges* (1989),[29] the *Manual for Condition Evaluation of Bridges* (1994 to 2003).[33] The current governing publication for the evaluation of existing bridges is the *Manual for Bridge Evaluation* (MBE);[30] the 1st edition was published in 2008 and the current, 3rd edition, was published in 2018.

1.1.4. Shear Design

Shear design of reinforced and prestressed concrete has evolved from allowable stress methods to strut and tie and strength design over the years. Allowable stress methods were used in the early editions of the AASHO Standard Specifications. Later, strength design was introduced to the AASHO Standard Specifications in the 1960s for prestressed concrete design and in the 1970's for non-prestressed concrete design. Modified compression field theory and strut and tie shear design methods were implemented with the publication of the *LRFD Bridge Design Specifications* in the 1990s.

1.1.5. Load Rating

Load rating is defined as the calculation of a bridge's safe live load carrying capacity. Bridges must be load rated for multiple reasons including discrepancies in design loads, deterioration during the course of their service life, and safety. The load rating of bridges is discussed in many AASHO and AASHTO publications. AASHO Standard Specifications provided provisions for the evaluation and inspection of bridges. With the introduction of the *Manual for Maintenance Inspection of Bridges,* the load capacity rating section was eventually removed from the Standard

Specifications. Early forms of the rating factor equation were seen in the Load Factor Rating section of the *Manual for Maintenance Inspection of Bridges*. The rating factor equation took many forms in the *Guide for Strength Evaluation of Existing Steel and Concrete Bridges*, the *Manual for Condition Evaluation of Bridges*, the *Guide Manual for Condition Evaluation* and *Load and Resistance Factor Rating of Highway Bridges,* and the most current 3rd edition of the MBE.

CHAPTER 2. REQUIREMENTS FOR SHEAR DESIGN OF CONCRETE BRIDGES

2.1.   Concrete Bridge Design Requirements, 1931 to Present

Shear design requirements for concrete bridges include design loads, required shear strength, and the design of shear reinforcing.

2.1.1.   Design Loads

This section provides a timeline of the various methods of loading used for the design and analysis of bridge members. Aspects of bridge loading can be subdivided into load combinations, vehicular live load, dynamic load allowance (or impact), and live load distribution as discussed in the following. It is important to discuss loading methodologies because they can strongly influence the bridge rating factors. The following narrative on the historical development of design loads and load combinations focuses on those specifically related to load rating, namely dead loads and live loads.

Load Combinations and Load Factors

Load combinations and load factors are essential in capturing the worst-case loading scenario in design. Early ASD did not list specific loading combinations to be used. As the number of loading sources grew, it was important to understand which combination of loads the structural members would experience. Load factors were used with the introduction of Load Factor Design (LFD) and LRFD.

The 1931 *Standard Specifications for Highway Bridges and Incidental Structures* (1st Edition)[1] listed load types to be used for the design of structures: dead loads, live loads, impact or dynamic effect of the live load, lateral forces, longitudinal forces, centrifugal forces, and thermal forces. The specifications instructed the engineer to use a combination of loads and forces producing maximum total stress. In 1941, the Standard Specifications (3rd Edition)[3] replaced lateral forces with wind forces and expanded the list of forces to include the following: earth pressure, buoyancy, shrinkage stresses, rib shortening, erection stresses, ice and current pressure, and earthquake stresses. Specified loads and forces were given notation in the 1965 Standard Specifications (9th Edition). [9] Table 1 lists the notations.

Table 1. 1965 Standard Specifications Loading and Force Notation

Notation	Definition
D	Dead Load
L	Live Load
I	Live Load Impact
E	Earth Pressure
B	Buoyancy
W	Wind Load on Structure
WL	Wind Load on Live Load
LF	Longitudinal Force from Live Load
CF	Centrifugal Force
F	Longitudinal Force from Friction
R	Rib Shortening
S	Shrinkage
T	Temperature
EQ	Earthquake
SF	Stream Flow Pressure
ICE	Ice Pressure

The 1965 Standard Specifications (9th Edition)[9] also introduced nine possible groups of service loading combinations to be used with an allowable overage of the unit stresses. Each part of the structure was proportioned for all applicable loading combinations, at the percentage of basic unit stress indicated and the resulting required maximum section was used. Table 2 lists the loading combinations pertinent to load rating. The 1974 interim revisions of the Standard Specifications moved the centrifugal force load from Group III to Group I. It is important to note that lateral loads due to wind loads, longitudinal loads, and thermal forces are not considered in the load rating of bridges.

Table 2. 1965 Standard Specifications Service Load Design Combinations

Group	Loading Combination	Percentage of Unit Stress
I	D + L + I + E + B + SF	100
III	Group I + LF + F + 30% W + WL + CF	125
IV	Group I + R + S + T	125
VI	Group III + R + S + T	140
VIII	Group I + ICE	140

Interim Revisions

When AASHO introduced LFD to non-prestressed concrete in the 1972 interim revisions, a new set of load combinations were added to the specifications. With the LFD load combinations, all loads were factored by 130 percent and the live load effects were increased further in a couple of combinations. The LFD load combinations used for bridge load rating in the 1972 interim revisions are shown in Table 3. Group I applied to design live loads. Group IA applied only when design live load is lower than H20 and considers infrequent heavy loads where the live load was assumed to occupy a single lane without concurrent live loads.

Table 3. 1972 Interim Revisions Load Factor Design Combinations

Group	Loading Combination
I	1.30 [D + 5/3 (L + I)]
IA	1.30 [D + 2.2 (L + I)]
III	1.30 [D + (L + I) + CF + 0.3W + WL + F + LF]

The 1974 interim revisions modified the concrete LFD load combinations to be consistent with the load combinations used in the Loads section of the Standard Specifications. The updated load combinations have nine combinations and include additional load factors for dead loads and earth pressure loads. The updated load combinations used for bridge load rating are shown in Table 4. Table 4 is shown as modified by the 1975 interim revisions where the earthquake and ice pressure loads were switched in Groups VII and VIII. The 1974 *Manual for Maintenance Inspection of Bridges* stated that lateral loads due to wind, longitudinal loads, thermal forces, and deflection limits were not required to be used in the load rating of bridges.[13]

Table 4. 1975 Interim Revisions Load Factor Design Combinations

Group	Loading Combination
I	1.30 [β_DD + 5/3 (L + I) + CF + β_EE + B + SF]
IA	1.30 [β_DD + 2.2 (L + I)]
III	1.30 [β_DD + L + I + β_EE + B + SF + LF + F + 30%W + WL + CF]
IV	1.30 [β_DD + L + I + CF + β_EE + B + SF + R + S + T]
VI	1.25 [β_DD + L + I + CF + β_EE + B + SF + LF + F + 30%W + WL + R + S + T]
VIII	1.30 [β_DD + L + I + CF + β_EE + B + SF + ICE]

For column design, the β_D factor is equal to 0.75 when checking the member for minimum axial load and maximum moment or maximum eccentricity; the factor must equal 1.0 when checking the member for maximum axial load and minimum moment. The β_E factor is equal to 1.3 for lateral earth pressures for retaining walls and rigid frames excluding rigid culverts and 1.0 for vertical earth pressures.

Equations

The 1977 Standard Specifications introduced a common equation to be used for both LFD and Service Load Design.[14] The equation is used in conjunction with a table that lists the load factors for each different type of load and group number. The grouping load combination equation is shown in Equation 1.

$$Group\ (N) = \gamma[\beta_D D + \beta_L(L + I) + \beta_C CF + \beta_E E + \beta_B B + \beta_S SF + \beta_W W + \beta_{WL} WL + \beta_L L + \beta_R(R + S + T) + \beta_{EQ} EQ + \beta_{ICE} ICE]$$

(Equation 1)

where:

N = group number

γ = load factor

β = load coefficient

AASHTO provided a table that defined the β and γ factors to be used in Equation 1. β_D and β_E factors depended on the loading scenario. γ and β factors were not applied to loads for foundation or foundation stability checks. Table 5 and Table 6 list the load combinations pertinent to load rating. The load factors listed in the tables are consistent through the 2002 Standard Specifications (17th Edition).[20] Group IA uses a live load plus impact consistent with the overload criteria of the operation agency and Group X was used for culverts.

Table 5. 1977 Standard Specifications Service Load Design Combinations

Group	Loading Combination	Percentage of Unit Stress
I	D + L + I + CF + β_EE + B + SF	100
IA	D + 2 (L+I)	150
IB	D + L + I + CF + β_EE + B + SF	-
IV	Group I + R + S + T	125
VIII	D + L + I + CF + E + B + SF + ICE	140
X	D + L + I + β_EE	140

Table 6. 1977 Standard Specifications Load Factor Design Combinations

Group	Loading Combination
I	1.30 [β_DD + 1.67 (L + I) + CF + β_EE + B + SF]
IA	1.30 [β_DD + 2.2 (L + I)]
IB	1.30 [β_DD + L + I + CF + β_EE + B + SF]
IV	1.30 [β_DD + L + I + CF + β_EE + B + SF + R + S + T]
VIII	1.30 [β_DD + L + I + CF + β_EE + B + SF + ICE]
X	1.30 [β_DD + 1.67 (L + I) + β_EE]

The *LRFD Bridge Design Specifications* included new load combinations with the release of the 1st edition in 1994.[21] Load combinations were categorized as either Strength, Service, Extreme Event, or Fatigue combinations, with each targeting a different limit state. The limit states represent scenarios where certain portions of various loads occur simultaneously to create worst-case force effects on different elements. Service limit states control stress, deformation, and crack widths. Fatigue limit states are used to control stress ranges experienced by members. Strength limit states ensure strength and stability. Lastly, the extreme event limit states consider earthquakes, floods, vessel collisions, and ice flow. Table 7 lists the loads and their notations used by the *LRFD Bridge Design Specifications*.

Table 7. 1994 LRFD Bridge Design Specifications Load Notation

Notation	Definition
DD	Down Drag Force
DC	Dead Load (Structural Components, Nonstructural Attachments)
DW	Dead Load (Wearing Surfaces, Utilities)
EH	Horizontal Earth Pressure Load
EL	Accumulated Locked-in Effects due to Construction Process
ES	Earth Surcharge Load
EV	Vertical Pressure from Dead Load of Earth Fill
BR	Vehicular Braking Force
CE	Vehicular Centrifugal Force
CR	Force Effects due to Creep
CT	Vehicular Collision Force
CV	Vessel Collision Force
EQ	Earthquake Load
FR	Friction Load
IC	Ice Load
IM	Vehicular Dynamic Load Allowance
LL	Vehicular Live Load
LS	Live Load Surcharge
PL	Pedestrian Live Load
SE	Force Effects due to Settlement
SH	Force Effects due to Shrinkage
TG	Temperature Gradient
TU	Uniform Temperature
WA	Water Load, Stream Pressure
WL	Wind on Live Load
WS	Wind Load on Structure

Total Factored Force Effect

The total factored force effect is calculated using a load modifier, load factor, and the specified load. The load modifier is prescribed by the limit state and relates to ductility, redundancy, and operational importance of the member. LRFD uses probability-based load factors that account for uncertainties in variable loads. The LRFD design load is given by Equation 2.

$$\Sigma \eta_i \gamma_i Q_i$$

(Equation 2)

where:

$\eta =$ load modifier

$\gamma =$ load factor

$Q =$ force effect

The 2008 MBE requires only the Strength I, Strength II, Service I, and Service III load combinations to be used in the load rating of reinforced concrete and prestressed concrete bridges.[30] The load factors used in load rating combinations in the 1994 *LRFD Bridge Design Specifications* (1st Edition) are shown in Table 8.[21]

Table 8. 1994 LRFD Bridge Design Specifications Load Combinations and Load Factors

Load Combination Limit State	DC, DD, DW, EH, ES, EV	BR, CE, IM, LL, LS, PL	WA	WS	WL	FR	CR, EL, SH, TU	TG	SE
Strength I	γ_p	1.75	1.00	--	--	1.00	0.50/ 1.20	γ_{TG}	γ_{SE}
Strength II	γ_p	1.35	1.00	--	--	1.00	0.50/ 1.20	γ_{TG}	γ_{SE}
Service I	1.00	1.00	1.00	0.30	0.30	1.00	1.00/ 1.20	γ_{TG}	γ_{SE}
Service III	1.00	0.80	1.00	--	--	1.00	1.00/ 1.20	γ_{TG}	γ_{SE}

The load factors γ_{SE} and γ_{TG} are determined on a project-specific basis. Note that γ_{TG} may be taken as 0.0 at the strength and extreme limit states, 1.0 at the service limit state when live load is not considered, and 0.5 at the service limit state when live load is considered. Force effects due to settlement (SE) are only used when geotechnical conditions require. The permanent loads factor γ_p is specified in a separate table—see Table 9. According to the 2008 MBE, the following forces are not used in the load rating of bridges: earthquake, temperature, creep, shrinkage, stream flow, and wind.[30]

Permanent Load Factors

Table 9. 1994 LRFD Bridge Design Specifications Permanent Load Factors

Load	Max γ_p	Min γ_p
DC	1.25	0.90
DD	1.80	0.45
DW	1.50	0.65
EH	1.50	0.90
EV	1.95	0.90
ES	1.50	0.75

The load combinations and load factors were updated in the 1998 *LRFD Bridge Design Specifications* (2nd Edition).[22] The accumulated locked-in effects due to construction process (EL) load was moved to join the other permanent loads. Its load factor γ_p is taken as 1.00. The wind on live load (WL) factors were increased from 0.4 and 0.3 to 1.00 for both the Service I and Strength V load combinations. Lastly, the Service IV load combination was added in the 2003 *LRFD Bridge Design Specifications* interim revisions.

The 2005 *LRFD Bridge Design Specifications* interim revisions slightly modified the permanent load factors. The dead load (structural components, nonstructural attachments) (DC) factor in the Strength IV load combination had a maximum value of 1.50 and a minimum value of 0.90. Additionally, the load factors for down drag force (DD) were lowered and depend on the method of analysis.

The 2008 *LRFD Bridge Design Specifications* interim revisions to the 4th Edition introduced a new permanent load, (PS), defined as the secondary forces from post-tensioning. The interim revisions also re-categorized force effects due to creep (CR) and shrinkage (SH) as permanent loads. The three load types mentioned were considered permanent loads due to superimposed deformations. Consequently, a new table was added to define the load factors for the load types - see Table 10.

Table 10. 2008 LRFD Bridge Design Specifications Interim Revisions Load Factors for Permanent Loads Due to Superimposed Deformations

Bridge Component	PS	CR, SH
Segmental Superstructures and Concrete Substructures that support them	1.0	γ_p for DC
Non-segmental Concrete Superstructures	1.0	1.0
Substructures supporting non-segmental Superstructures (Using I_g)	0.5	0.5
Substructures supporting non-segmental Superstructures (Using $I_{effective}$)	1.0	1.0
Steel Substructures	1.0	1.0

Notes: PS = Permanent Load; CR = Force Effects due to Creep; SH = Shrinkage.

The 2008 *LRFD Bridge Design Specifications* interim revisions also added a section that discussed blast loading. The section briefly described design considerations such as size and shape of explosive charge, type of explosive, and stand-off distance. No specific provisions were provided until the release of the 2012 *LRFD Bridge Design Specifications* (6th Edition) where blast loading (BL) was added to the Extreme Event II load combination with a load factor of 1.0.[26]

The Fatigue limit state was split into two checks in the 2009 *LRFD Bridge Design Specifications* interim revisions. The Fatigue I limit state was used for the analysis of bridges with an infinite load-induced fatigue life and had an increased load factor of 1.50. The Fatigue II limit state was used for finite load-induced fatigue life and retained the load factor of 0.75. These provisions do not apply to concrete bridges.

The 2016 *LRFD Bridge Design Specifications* interim revisions to the 2014 *LRFD Bridge Design Specifications* (7th Edition) provided changes to the wind load calculations.[27] The load factors for the wind load on structures (WS) were all changed to 1.0. Additionally, the wind speed calculations were changed from fastest-mile wind speeds to 3-second gust wind speeds to match modern wind codes. The interim revisions included a modification to the Service III live load factor. If the prestressed concrete component analyzed is designed using refined estimates of time-dependent losses in conjunction with taking advantage of the elastic gain, the factor is taken as 1.0. Otherwise, the factor is taken as 0.80.

The 2017 *LRFD Bridge Design Specifications* (8th Edition) provided minor changes to the load combinations.[28] The Extreme II load combination dead load factor was changed from γ_P to 1.0. The Fatigue I limit state changed the live load factor from 1.50 to 1.75 and the Fatigue II live load factor was changed from 0.75 to 0.80.

Vehicular Live Load

The traffic lane width was originally defined by the standard truck clearance width taken as 9 feet in accordance with the 1931 Standard Specifications (1st Edition).[1] The lane width was later increased to 10 feet in the 1941 Standard Specifications (3rd Edition).[3] The 1949 Standard Specifications (5th Edition)[5] provided a table to assist the engineer in calculating the design traffic lane depending on the roadway width and the number of design traffic lanes. The traffic lane widths ranged from 10 feet to 15 feet and supported a 10-foot-wide truck or lane load. The 1977 Standard Specifications (12th Edition) established the 12-foot-wide traffic lane.[14] The lane and truck loading width remained at 10 feet. These values remain consistent through the 2017 *LRFD Bridge Design Specifications* (8th Edition).[28]

The 1931 Standard Specifications (1st Edition)[1] introduced standard truck loads and configurations along with equivalent lane loads. The standard two-axle trucks introduced were H20, H15, and H10; the numbers after the H refer to the gross weight of the truck in tons. Eighty percent of the truck weight was assumed to be transferred to the bridge through the back axle and the remaining 20 percent through the front axle. The front and back axles were spaced at 14 feet and wheels were spaced at 6 feet along the width of the lane. Highway loadings consisted of either truck trains or equivalent loadings. Truck train loading involves the design truck followed or preceded by trucks weighing three-fourths the weight of the design truck. Loaded lengths of 60 feet or less required the use of truck train loading. Loaded lengths of 60 feet or more required equivalent loadings. Equivalent loadings refer to a lane loaded with both a uniform load and a concentrated load acting simultaneously. The concentrated load was applied at a location that would cause the largest stresses in the member and its value varied depending on the type of loading and the type of analysis conducted; each standard truck has an assigned equivalent load. Table 11 lists the equivalent lane loading.

Table 11. 1931 Standard Specifications H Equivalent Lane Loading

Loading	Concentrated Load - Moment (lbs)	Concentrated Load – Shear (lbs)	Uniform Load (lbs per LF of Lane)
H20	18,000	26,000	640
H15	13,500	19,500	480
H10	9,000	13,000	320

Notes: lbs = pounds; LF = linear feet.

HS Loading

The 1941 Standard Specifications (3rd Edition)[3] introduced HS loading. The HS loading standard truck consisted of a three-axle truck with 14-foot axle spacing. Two classes of HS loading were defined by the specifications, H20-S16 and H15-S12. Similar to H loading, the number after the H in HS loading refers to the vehicle weight in tons; however, the weight is only the sum of the two axles nearest to the front of the vehicle. The number after the S refers to the individual weight of either of the two axles at the rear of the vehicle in tons. It was assumed that the back two axles each transfer 80 percent of the tractor weight (the front two axle weights) and the front axle transfers 20 percent. For example, the H20-S16 vehicle transfers 8,000 pounds (lbs) through the front axle, 32,000 lbs through the middle axle, and 32,000 lbs through the back axle. Additionally, the specifications required the engineer to choose H truck or equivalent loading depending on which produced the maximum stresses. On the other hand, HS loading required the engineer use truck loading for loaded lengths less than 40 feet and equivalent loading for loaded lengths greater than 40 feet. The HS equivalent loading is shown in Table 12.

Table 12. 1941 Standard Specifications HS Equivalent Lane Loading

Loading	Concentrated Load – Moment (lbs)	Concentrated Load – Shear (lbs)	Uniform Load (lbs per LF of Lane)
H20-S16	32,000	40,000	640
H15-S12	24,000	30,000	480

Notes: lbs = pounds; LF = linear feet.

The 1944 Standard Specifications (4th Edition)[4] provided slight changes to the vehicular live loading. The policy of affixing the year of adoption to the end of loadings (that is, H20-44) was instituted with this edition. Variable spacing between rear axles of the HS truck was introduced with the intention of producing maximum stresses. By varying the spacing of the axles, the engineer could more closely approximate the loading provided by tractor trailers used at the time and achieve more accurate negative moment values in continuous spans. H15-S12 loading was specified as the minimum load for highways that could carry heavy truck traffic. Furthermore, the 1944 Standard Specifications (4th Edition)[4] reduced equivalent HS lane loading to match the equivalent H lane loading provisions. A second concentrated load of equal weight was required to be used for equivalent H lane loading when designing continuous spans. This provision was expanded to apply to HS equivalent lane loading in the 1949 Standard Specifications (5th Edition).[5]

The H10 loading class was removed in the 1977 Standard Specifications (12th Edition).[14] Interstate highway bridges were required to be designed for HS20 loading or an Alternate Military Loading, which consists of two axles each weighing 24,000 lbs, spaced 4 feet apart. Minimum loading for other highways that could carry heavy truck traffic remained as HS15. The 1992 Standard Specifications (15th Edition) consolidated the minimum live load requirement by requiring all bridges supporting highways that could carry heavy truck traffic to be designed for HS20 loading or Alternate Military Loading.[18] Many states used HS25 loading before the introduction of HL-93 loading in the *LRFD Bridge Design Specifications*.

The 1994 *LRFD Bridge Design Specifications* (1st Edition) replaced H and HS loading classes with HL-93 loading.[21] HL-93 loading consists of a design truck or a design tandem with a design lane load intended to represent the force effects of vehicles permitted on highways under grandfather exclusions to weight laws. The HL-93 design truck is identical to the HS20 truck. The design tandem is defined by two, 25,000-lb axles spaced 4 feet apart; it is nearly identical to previous Alternate Military Loading. The design lane load is taken as 640 lbs per linear foot of lane, the same as the distributed load in the HS20 equivalent lane loading. Three cases should be considered in the application of the design vehicular live load: the effect of the design tandem plus the design lane load, the effect of one design truck with variable axle spacing plus the design lane load, or 90 percent of the effect of two design trucks spaced a minimum of 50 feet apart with the rear axles of a single truck spaced at 14 feet plus 90 percent of the design lane load. The last case mentioned is only applicable when calculating the negative moment between points of contraflexure under a uniform load and the reaction at interior piers. HL-93 loading is current with the 2017 *LRFD Bridge Design Specifications* (8th Edition).[28]

Reduction Factors

To account for the probability of simultaneous live load on adjacent lanes the specifications provided reduction factors to be applied to live load effects. The 1931 Standard Specifications (1st Edition)[1] allowed the engineer to reduce loads by 1 percent for each foot of loaded roadway width in excess of 18 feet with a maximum reduction of 25 percent. The 1941 Standard Specifications (3rd Edition)[3] updated the load reduction allowance to a tiered system that provided the percentage of resultant live load stresses required for design as shown in Table 13.

Table 13. 1941 Standard Specifications Reduction in Load Intensity

Number of Loaded Lanes	Percentage
1 or 2	100
3	90
Greater than 3	75

The 1994 *LRFD Bridge Design Specifications* (1st Edition) accounted for the probability of simultaneous live load on adjacent lanes by using the multiple presence factor, *m*.[21] The multiple presence factor is applied to the live load force effect. Table 14 gives the multiple presence factor values used in the *LRFD Bridge Design Specifications*. Note that the factor will increase the live load effect when loading only one lane.

Table 14. 1994 LRFD Bridge Design Specifications Multiple Presence Factor

Number of Loaded Lanes	Multiple Presence Factor, *m*
1	1.20
2	1.00
3	0.85
Greater than 3	0.65

Dynamic Load Allowance and Impact

The dynamic, vibratory, and impacts of vehicular live loads on bridges are included in the analysis by applying an impact factor to the calculated static live load. The 1931 Standard Specifications (1st Edition)[1] used an equation to define the impact factor as a fraction of the live load to be added to the static live load. Sidewalk, centrifugal, tractive, and wind loads were not required to be increased by the impact factor. See Equation 3.

$$I = \frac{50}{L + 125}$$

(Equation 3)

where:

I = impact fraction

L = length of the portion of span which is loaded to produce maximum stress in the member considered

The 1941 Standard Specifications (3rd Edition)[3] expanded on the application of the impact factor. It stated that impact should be applied to H or HS loadings that affect superstructure elements, including supporting columns, steel towers, legs of rigid frames, members that extend down to the main foundation, and the portion of piles that extend above the ground line. The specifications also provided a list of loads and members that should not see the effects of impact including substructures (abutments, retaining walls, piers, piling, and other elements subject to static loads), foundation pressure, and sidewalk loads. An upper limit of 30 percent was implemented for the impact factor, *I*.

The 1994 *LRFD Bridge Design Specifications* (1st Edition) changed the impact factor calculation method and renamed it to Dynamic Load Allowance.[21] The static effects of the design truck or design tandem are increased by a dynamic load allowance labeled *IM*. Note that the *IM* factor does not apply to lane loading. Table 15 lists values of *IM* based on the bridge component and the limit state.

Table 15. 1994 LRFD Bridge Design Specifications Dynamic Load Allowance

Component	Limit State	Dynamic Load Allowance, *IM*
Deck Joints	All Limit States	75%
All Other Components	Fatigue, Fracture	15%
	All Other Limit States	33%

Note: IM = Vehicular Dynamic Load Allowance

Logically, deck joints will experience the largest dynamic effects from vehicular live load. For this reason, they have the largest *IM* value.

The factor accounts for wheel load impact from moving vehicles due to the hammering effect or the dynamic response of the bridge to passing vehicles. The hammering effect can be described as the dynamic response of the wheel assembly to riding surface discontinuities at deck joints, potholes, etc. The specifications state that the dynamic load allowance is not applicable to centrifugal or braking forces, design lane load or pedestrian loads, foundation components entirely below ground, or retaining walls not subject to vertical reactions from the superstructure. The dynamic load allowance provisions described are current with the 2017 *LRFD Bridge Design Specifications* (8th Edition).[28]

Live Load Distribution

Vehicular live load distribution is required to accurately distribute the wheel load to the appropriate girders on a bridge. The calculation is typically carried out by applying distribution factors defined by the specifications to the wheel load. The use of live load distribution factors eliminates the need to carry out a complicated loading model that will take more time and resources.

The 1931 Standard Specifications (1st Edition)[1] required wheel loads to be modeled as point loads and not be further distributed laterally or longitudinally when calculating beam end shears and reactions. Slabs designed for bending moment in accordance with the specifications were considered adequate for shear without special reinforcement.

The 1941 Standard Specifications (3rd Edition)[3] modified the distribution of wheel loads to girders and beams for calculation of end shears for certain locations of applied live load. For wheel or axle load adjacent to the end at which stress is being determined, no lateral or longitudinal distribution of the wheel load was assumed when calculating end shears. For loads in other positions on the span, the distribution for shear was determined by the same method specified for bending moment. The live load distribution factor used for concrete girders when only one traffic lane was considered was equal to the average spacing of the girders divided by six; when two or more traffic lanes were considered, the distribution factor was equal to the average spacing of the girders divided by five. When the girder spacing exceeded 6 feet for one traffic lane and 10.5 feet for two traffic lanes, the girders each received the full reaction of the

wheel load. The exterior girders also received the full reaction of the wheel load as no lateral distribution was assumed. No transverse distribution of bending moments on floor (lateral) beams was assumed unless the bridge did not have longitudinal girders. In this case, the distribution factor was equal to the floor beam spacing divided by six. It is important to note that the flooring between girders or floor beams is assumed to act as a simple span for the wheel load.

Interior Girders

The calculation of end shears and end reactions for interior girders was changed in the 1965 Standard Specifications (9th Edition). The lateral distribution of the wheel load was assumed to be the load transfer resulting from the assumption that the flooring is a simple span between the longitudinal girders or lateral beams. In cases where this assumption cannot be made, the specifications instruct the engineer to use the method specified for bending moment. Table 16 shows the distribution factors to be used for interior girders with concrete deck where S is the average girder spacing.

Table 16. 1965 Standard Specifications Interior Girder Live Load Distribution

Type of Stringer	One Traffic Lane	Two or More Traffic Lanes
Prestressed Concrete Girder	$S \div 7.0$	$S \div 5.5$
Concrete T-Beams	$S \div 6.5$	$S \div 6.0$
Concrete Box Girders	$S \div 8.0$	$S \div 7.0$

Exterior box girders are assumed to have a distribution factor equal to the girder width divided by seven. The 1973 Standard Specifications (11th Edition)[11] updated the wheel load distribution method. A new provision specified that all exterior girders were required to have at least the same carrying capacity as the interior girders. For locations of wheel loads where lateral shear distribution was determined by the moment distribution method, the prestressed concrete box beam distribution factor was defined as shown in Equation 4.

$$D.F. = \frac{2N_L}{N_B} + \frac{kS}{L}$$

(Equation 4)

where:

$D.F.$ = distribution factor

N_L = number of design traffic lanes

N_B = number of beams

S = beam spacing

L = span length

k is given by the expression shown in Equation 5.

$$k = 0.07W - N_L(0.10N_L - 0.26) - 0.20N_B - 0.12$$

where:

W = the roadway width between curbs

The methods previously presented were consistent through the later editions of the Standard Specifications. Alternate distribution factors were introduced in the 1973 Standard Specifications (11th Edition)[11] for prestressed spread box beams and multi-beam precast concrete beams. These alternate distribution factors used more of the bridge's geometry to establish more accurate factors.

The 1994 *LRFD Bridge Design Specifications* (1st Edition) provided a more detailed analysis method for calculating the live load distribution factors.[21] The method was introduced as a solution to the oversimplified and highly conservative equations used in the Standard Specifications. By means of finite element analysis, unique equations were derived for multiple superstructure types that depend on factors such as number of lanes loaded, girder spacing, beam span length, girder geometric properties, and the number of girders used. When evaluating shear, four different live load distribution factors can be calculated and applied to the ultimate shear force for a particular superstructure type: live load distribution factor for interior girder for one lane loaded, live load distribution factor for interior girder for two or more lanes loaded, live load distribution factor for exterior girder for one lane loaded, and live load distribution factor for exterior girder for two or more lanes loaded. A few cases require the lever rule to be used for the calculation of the live load distribution factor. Live loads may be distributed using these live load distribution factor equations given that the bridge satisfies the following requirements: a constant deck width along the bridge length, beams are parallel with approximately the same stiffness, a maximum slab overhang of 3 feet measured from the centerline of the exterior web of the exterior beam to the interior edge of the curb or traffic barrier, and a superstructure cross-section consistent with the tables in *LRFD Bridge Design Specifications*. When evaluating concrete box beams, if the moment of inertia or the St. Venant torsional inertia are not within the limitations defined by the shear live load distribution factor tables, the factor for shear may be taken as that for moment. Alternatively, many box girders are designed and rated using mechanics of materials equations such as thin-walled tube shear flow equations to determine live load force effects. The shear live load distribution factor equations remained consistent through the 2017 *LRFD Bridge Design Specifications* (8th Edition), except for slight modifications to the equations used for concrete box beams.[28]

2.1.2.   Required Shear Strength

This section of the report outlines the general shear requirements of the Standard Specifications and the *LRFD Bridge Design Specifications*. Shear design requirements initiated with limitations on concrete unit stresses caused by diagonal tension in the members. Web reinforcement and bond unit stress limits were later introduced. The specifications eventually implemented shear design equations to assist the engineer with calculating the appropriate stresses in the materials. Design equations were modified as AASHTO transitioned from ASD and LFD methods to LRFD methods.

Concrete Shear Unit Stress

Concrete unit stress limits are compressive, tensile or shear upper stress limits allowed in a concrete member when subject to axial, flexural, or shear forces. Generally, shear unit stresses are defined by a shear force divided by the concrete area between the resisting couple forces in a member. Members can be resized to reduce the unit stresses experienced by the concrete.

AASHO implemented upper shear unit stress limits in 1931.[1] Concrete member shear unit stress limits depended on whether the member provided shear reinforcement. For example, members including shear reinforcement were allowed higher concrete shear unit stress limits. Concrete members without shear reinforcement were subdivided into beams with or without anchored longitudinal bars; beams with anchored longitudinal bars were allowed higher shear unit stress limits.

Table 17 lists the varying concrete shear unit stress limits in previous editions of the Standard Specifications. Shear unit stress limits remained constant for concrete members lacking shear reinforcement. In contrast, concrete members with shear reinforcement were allowed larger shear unit stresses in later editions of the Standard Specifications.

Table 17. Standard Specifications Concrete Shear Unit Stresses

Standard Specifications Edition	Long. Bars Anchored (No Shear Reinforcement)	Long. Bars Not Anchored (No Shear Reinforcement)	Long. Bars Anchored (with Shear Reinforcement)	Long. Bars Not Anchored (with Shear Reinforcement)
1st and 2nd	[90 psi]	[60 psi]	[160 psi]	[160 psi]
3rd	[90 psi]	[60 psi]	[180 psi]	[140 psi]
4th	$0.03\,f'_c$ [90 psi]	$0.02\,f'_c$ [60 psi]	$0.06\,f'_c$ [180 psi]	$0.046\,f'_c$ [138 psi]
5th	$0.03\,f'_c$ [90 psi]	$0.02\,f'_c$ [60 psi]	$0.06\,f'_c$ [180 psi]	$0.046\,f'_c$ [138 psi]
6th	$0.03\,f'_c$ [90 psi]	$0.02\,f'_c$ [60 psi]	$0.075\,f'_c$ [225 psi]	$0.075\,f'_c$ [225 psi]
7th through 11th	$0.03\,f'_c$ max. 90 psi.	$0.02\,f'_c$ max. 75 psi.	$0.075\,f'_c$ [225 psi]	$0.075\,f'_c$ [225 psi]

Note: All bracketed values are based on a concrete ultimate strength of 3,000 pounds per square inch (psi).

Assuming ultimate concrete strength to be 3,000 pounds per square inch (psi), unit stress limits ranged from 160 psi in 1931 to 225 psi in 1973 for concrete members with shear reinforcement.[1 11] In 1944, equations were introduced to consider concrete ultimate strength when calculating unit stress limits.[4] Prior to 1944, the engineer was instructed to proportionately reduce the unit stress limits when using concrete with strengths lower than 3,000 psi.

Web Reinforcement Unit Stresses

Steel reinforcement unit stress limits are upper stress limits for the steel reinforcement in the concrete members when subject to external forces. Web reinforcement unit stresses are calculated using external shear after deducting that carried by the concrete. Note that larger steel areas with tighter spacing will produce smaller unit stresses in the web reinforcement.

Steel unit stress limits were added to the 1935 Standard Specifications (2nd Edition);[2] however, unit stress limits specific to web reinforcement were implemented in 1941 (3rd Edition).[3] The web reinforcement unit stresses are a function of shear force, transverse reinforcement spacing, area of reinforcement, internal force couple lever arm, and beam depth (d). When using bent-up bars, the angle of inclination must also be included in the calculations. Table 18 shows the transverse reinforcement unit stress limits for previous editions of the Standard Specifications.

Table 18. Standard Specifications Reinforcement Unit Stresses

Standard Specifications Edition	Steel Grade	Allowable Reinforcement Stress (psi)
2nd and 3rd	All	16,000
4th	Structural Grade	16,000
	Intermediate Grade	18,000
5th	Structural Grade	16,000
	Intermediate Grade	16,000
6th through 9th	Structural Grade	18,000
	Intermediate/Hard Grade	20,000
10th	All	20,000
11th and 12th	Grade 40	20,000
	Grade 60	24,000

The table shows that the allowable reinforcement unit stresses ranged from 16,000 psi to 24,000 psi during the years 1935 through 1977.[2 14] In 1950, after the publication of the 1949 Standard Specifications (5th Edition),[5] the allowable steel unit stresses were revised to the values that would be specified in the 6th through 9th editions (1953 through 1965).[6 7 9 9] Note that Structural Grade steel has a minimum yield strength of 33,000 psi, Intermediate Grade steel has a minimum yield strength of 40,000 psi, and Hard Grade steel has a minimum yield strength of 50,000 psi. Transverse reinforcement allowable stresses shown in Table 18 are all within 40 to 50 percent of the yield stress.

Bond Unit Stresses

Shear strength may be limited by allowable bond unit stresses. Bond stresses refer to the stresses at the interface of the steel reinforcement and the concrete resulting from externally applied forces. The bond stresses in the transverse reinforcement are dependent on the area of the interface between the two materials and the shear force acting on the member.

Bond allowable stresses were introduced in the 1935 Standard Specifications (2nd Edition).[2] The allowable stresses are dependent on whether the steel reinforcement bars are adequately anchored. Analogous to concrete allowable unit stresses, bond allowable unit stresses become a function of the concrete ultimate strength in 1944 (4th Edition).[4] Table 19 summarizes the allowable bond unit stresses specified in previous editions of the Standard Specifications. Allowable bond stresses increase from 120 psi in 1935 to 300 psi in 1969 assuming an ultimate concrete strength of 3,000 psi.[2 10] The 1970 interim revisions to the Standard Specifications dissolved the transverse reinforcement bond stress requirements.

Table 19. Standard Specifications Bond Unit Stresses

Standard Specifications Edition	Bars Anchored	Bars Not Anchored
2nd	120 psi	80 psi
3rd	150 psi	100 psi
4th	0.05 f'$_c$ max. 150 psi	0.033 f'$_c$ max. 100 psi
5th	0.075 f'$_c$ max. 225 psi	0.05 f'$_c$ max. 150 psi
6th through 10th	0.10 f'$_c$ max. 350 psi	0.10 f'$_c$ max. 350 psi

Shear Design

This section provides a summary of the shear equations presented by AASHO/AASHTO used to derive stresses, strengths, and required transverse reinforcement areas. There are three distinct design methodologies used by AASHO/AASHTO: ASD, LFD, and LRFD. ASD methods require the maximum applied stress on a member not exceed a specified limit. LFD methods account for variability in live loads and dead loads; the loads applied to the system are multiplied by a specified load factor to achieve the ultimate loading scenario. Additionally, the computed theoretical capacity is modified by a capacity modification factor that reduces the capacity of the member. Lastly, LRFD methods use a probability-based approach to determine both loading factors and resistance factors.

Allowable Stress Design for Reinforced Concrete

AASHO introduced allowable concrete stresses in 1931;[1] however, shear design equations were not adopted until the 1941 Standard Specifications (3rd Edition).[3] Design equations define the shearing unit stress, stress in the vertical web reinforcement, bond stress, and required transverse reinforcement area in a reinforced concrete member given an external shear force V.

In 1941, AASHO provided a formula to calculate the shearing unit stress as a function of shear force, beam depth, beam width, and the ratio of the lever arm of the resisting beam force couple to the beam depth (d).[3] The shearing unit stress formula used from 1941 to 1974 is provided in Equation 6.

$$v = \frac{V}{bjd}$$

<div align="right">(Equation 6)</div>

where:

$v =$ shearing unit stress

$V =$ external shear on any section

$b =$ width of the beam

$d =$ effective depth

$j =$ ratio of the lever arm of the resisting force couple to depth, d

Moment Arm Refinement

The 1974 interim revisions removed the moment arm refinement of the shear area due to the lack of clear definition of the actual distribution of shear stress over a given cross section. The updated formula is expressed as an average stress on the full cross section as shown in Equation 7.

$$v = \frac{V}{b_w d}$$

<div align="right">(Equation 7)</div>

where:

$b_w =$ web width or diameter of a circular section

Equations

In 1979, AASHTO began the transition away from the use of unit stress tables and instead defining permissible stresses through equations. The 1979 interim revisions include both allowable stresses and shear stress carrying capacity equations for concrete. This method allowed more refined equations to be used for members experiencing combined force effects. Additionally, explicitly defining the stress carried by the concrete v_c assisted with determining the amount of shear stress required to be resisted by the steel reinforcement. The simplified formula for computing concrete shear stress is shown in Equation 8.

$$v_c = 0.95\sqrt{f'_c}$$

<div align="right">(Equation 8)</div>

where:

v_c =   shear stress carried by concrete

f'_c =   ultimate concrete strength in psi

Concrete member

Equation 8 was to be used for the cases where the concrete member was subject to combined shear and flexure or shear and axial compression. More detailed, less conservative equations that require more information about the member geometry and loading are included in the 1979 interim revisions.

If the concrete member is subject to shear and flexure only, the following equation was provided (Equation 9).

$$v_c = 0.9\sqrt{f'_c} + 1100\rho_w \frac{Vd}{M} \leq 1.6\sqrt{f'_c}$$

(Equation 9)

where:

ρ_w =   reinforcement ratio

V =   external shear

M =   design moment occurring simultaneously with V

The reinforcement ratio used in this equation accounts for the longitudinal steel providing shear resistance. The longitudinal steel assists with resisting shear by providing dowel action along the member cross section and reducing the length of the flexural cracks in the concrete; thus, providing more concrete area to resist shear forces. The Vd/M ratio is an interpretation of the shear span to beam depth ratio. A larger Vd/M ratio will result in a smaller shear span ratio and provide additional concrete shear resistance. Smaller shear span ratios result in local compression induced at the supports and loaded areas. This local compression provides additional shear strength to the concrete.

Lastly, if the concrete member is subject to shear and axial compression or tension, the following equations were provided (Equations 10 and 11).

$$v_c = 0.9\left(1 + 0.0006\frac{N}{A_g}\right)\sqrt{f'_c}$$

(Equation 10)

$$v_c = 0.9\left(1 + 0.004\frac{N}{A_g}\right)\sqrt{f'_c}$$

(Equation 11)

where:

$N =$ design axial load normal to the cross section occurring simultaneously with V

$A_g =$ gross cross-sectional area of the member

Note that the design axial load N is negative for tension and positive for compression. Equation 10 applies to members subject to axial compression and Equation 11 applies to members subject to axial tension. Equations 10 and 11 use the existing axial compression forces to provide more concrete shear capacity and axial tension forces to reduce the concrete shear capacity of the member. If the concrete shear stress in Equation 11 was not calculated, the shear reinforcement was designed to carry the total shear stress.

Equations 10 through 11 are used from 1979 to 2002 in Standard Specifications 12th through 17th editions.[14 20]

Equations used for calculating required shear reinforcement areas were introduced in the 1941 Standard Specifications (3rd Edition).[3] The shear reinforcement area is typically a function of the external shear after deducting shear carried by the concrete, the spacing of the transverse reinforcement, the shear stress in the reinforcement, the distance between the force couple in the member, and the angle of inclination. Because the reinforcement area is a function of the reinforcement stress, assuming maximum permissible stress in the reinforcement will result in the minimum required area of transverse reinforcement. The equation used to calculate transverse area for a series of web bars or bent-up longitudinal bars is shown as Equation 12.

$$A_v = \frac{V' s \sin \alpha}{f_v j d}$$

(Equation 12)

where:

$A_v =$ total area of web reinforcement in tension within a distance, s

$s =$ spacing of the web reinforcement bars

$V' =$ shear on a section after deducting shear carried by the concrete

$f_v =$ tensile unit stress in the web reinforcement

$\alpha =$ angle of inclination of transverse reinforcement to longitudinal axis

The shear area was calculated in this manner up until 1965 with the release of the 1965 Standard Specifications (9th Edition) when the formula was adjusted to optimize reinforcement capacity at an angle of inclination of 45 degrees. Any angle higher or lower than 45 degrees will require larger areas of transverse reinforcement.

$$A_v = \frac{V' s}{f_v j d (\sin \alpha + \cos \alpha)}$$

(Equation 13)

The required transverse reinforcement area for longitudinal bent-up bars in a single plane is shown in Equation 14. Note that the angle of inclination in this equation must be within 20 to 45 degrees; therefore, the area of reinforcement is optimized when bent upward 45 degrees.

$$A_v = \frac{V'}{f_v \sin \alpha}$$

(Equation 14)

Equations 13 and 14 were used until 1979 when they were revised in the 1979 interim revisions. The revised equations more clearly define the shear stress carried by the concrete by using the concrete shear stress calculation. The revised transverse reinforcement area equation is shown as Equation 15.

$$A_v = \frac{(v - v_c)b_w s}{f_s(\sin \alpha + \cos \alpha)}$$

(Equation 15)

where:

$b_w =$ width of the member web

$f_s =$ tensile stress in the transverse reinforcement

$v_c =$ permissible shear stress carried by the concrete

$v =$ design shear stress at the section

The revised shear reinforcement area equation for bent-up bars at the same distance from the support is shown as Equation 16.

$$A_v = \frac{(v - v_c)b_w d}{f_s \sin \alpha}$$

(Equation 16)

Equations 15 and 16 remained in the Standard Specifications up through the 17th edition published in 2002.[20] The shear reinforcement area equations took many forms from 1941 to 2002; however, the fundamental method of calculation remained constant.

The 1974 interim revisions introduced a minimum shear reinforcement required when the design shear stress is greater than one-half the permissible concrete shear stress, v_c. Equation 17 shows the minimum required shear reinforcement area introduced in the interim specifications.

$$A_v = \frac{50b_w s}{f_y}$$

(Equation 17)

where:

$f_y =$ design yield stress of the reinforcement

Equation 17 existed in the Standard Specifications from 1974 through 2002 with the release of the 17th edition.[20]

Load Factor Design for Reinforced Concrete

AASHO introduced LFD to non-prestressed concrete design in the 1972 interim revisions of the Standard Specifications. Recall that LFD uses load and capacity modification factors to account for variability in loads, materials, and construction. The design shear stress in a member was to be calculated as shown in Equation 18.

$$v_u = \frac{V_u}{bd}$$

(Equation 18)

where:

$v_u =$ factored shear stress

$V_u =$ factored shear force

$b =$ width of the beam

$d =$ effective depth

The 1972 interim revisions to the Standard Specifications go on to define the concrete shear stress capacity of a member as a function of f'_c. The concrete shear capacity is defined by the equation shown as Equation 19 at a distance d away from the support.

$$v_{uc} = 2\varphi\sqrt{f'_c}$$

(Equation 19)

where:

$v_{uc} =$ factored concrete shear capacity

$\varphi =$ capacity modification factor taken as 0.85 for shear

When the factored shear stress surpasses the concrete capacity, the interim specifications require shear reinforcement be used to resist the excess stress. The required shear reinforcement area to resist the excess stress is as shown in Equation 20.

$$A_v = \frac{(v_u - v_{uc})bs}{\varphi f_y}$$

(Equation 20)

where:

$A_v =$ required shear reinforcement area

$f_y =$ design yield stress of the reinforcement

In 1977, AASHTO introduced the fundamental LFD shear design equation in the 1977 Standard Specifications (12th Edition).[14] The equation uses factored shear force and capacity to establish design requirements as shown in Equation 21.

$$V_u \leq \phi V_n$$

(Equation 21)

where:

$V_u =$ factored shear force

$V_n =$ nominal shear strength

$\phi =$ strength reduction factor

The nominal shear strength is the sum of the shear strengths provided by the concrete and the shear reinforcement.

Furthermore, the 1977 Standard Specifications (12th Edition) redefined the shear capacity of concrete in the LFD section.[14] The concrete shear capacity calculation methods are analogous to those used in ASD; however, LFD methods require a shear force capacity be calculated instead of a shear stress capacity. The specifications divide concrete shear capacity calculations into three categories: members subject to shear and flexure only, members in compression, and members in tension.

AASHTO uses two equations to define concrete shear strength for concrete members subject to shear and flexure only. The first equation is a simplified equation that uses the member cross sectional area and the concrete ultimate strength. This equation is shown as Equation 22.

$$V_c = 2\sqrt{f'_c}\,b_w d$$

(Equation 22)

The second equation uses the reinforcement ratio and the shear span to depth ratio as discussed in the ASD section. This equation is shown as Equation 23.

$$V_c = \left(1.9\sqrt{f'_c} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d \leq 3.5\sqrt{f'_c}\,b_w d$$

(Equation 23)

Lastly, when the members are subject to shear and axial tension or compression, Equations 24 and 25 were used.

$$V_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\sqrt{f'_c}\,b_w d$$

(Equation 24)

$$V_c = 2\left(1 + \frac{N_u}{500A_g}\right)\sqrt{f'_c}\,b_w d$$

(Equation 25)

Equation 24 applies to members in compression. Equation 25 applies to members in tension.

The LFD equations for concrete shear capacity yield larger capacities than those used in ASD. This is because calculated LFD capacities are then reduced by strength reduction factors and loads are increased by load factors. Equations 22 through 25 remained constant in the Standard Specifications from 1977 to 2002.[14 20]

The 1977 Standard Specifications (12th Edition) introduced the shear strength provided by the web reinforcement.[14] Instead of directly calculating a required shear reinforcement area as was done in the 1972 interim revisions to the Standard Specifications, a shear strength V_s is calculated as a function of shear reinforcement area, steel yield strength, effective depth, and spacing of the reinforcement. The equation for shear strength provided by transverse reinforcement is shown as Equation 26.

$$V_s = \frac{A_v f_y (\sin \alpha + \cos \alpha)\, d}{s}$$

(Equation 26)

When bent-up bars are used, the following equation controls (Equation 27).

$$V_s = A_v f_y \sin \alpha$$

(Equation 27)

Equations 26 and 27 are in the Standard Specifications from 1977 to 2002.[14 20] Overall, general shear design for non-prestressed reinforced concrete in the Standard Specifications saw no change from 1977 to 2002.

The *Guide Specifications for Design and Construction of Segmental Concrete Bridges* (1989 edition) introduced shear provisions for segmental concrete bridges.[34] Concrete shear strength was determined by the equation shown as Equation 28.

$$V_c = 2K\sqrt{f'_c}\,b_w d$$

(Equation 28)

where K is the stress variable given by the equation shown as Equation 29.

$$K = \sqrt{1 + \frac{f_{pc}}{2\sqrt{f'_c}}} \leq 2.0$$

(Equation 29)

where:

f_{pc} = unfactored compressive stress in concrete after prestress losses

The guide specifications restricted the value of $\sqrt{f'c}$ to a maximum value of 100 psi. Shear effects were allowed to be neglected in areas where the factored shear force V_u was less than $\Phi V_c/2$; however, a minimum of two #4 stirrups spaced at 12 inches was required in these areas. Non-prestressed transverse shear reinforcement was required to have a design yield strength of 60 psi or less.

In areas where shear reinforcement was needed, the guide specifications required the minimum tensile capacity be equal to $50b_ws$, where b_w is the minimum web width and s is the bar spacing. The guide also specified an equation for shear strength provided by transverse reinforcement to be added to the concrete shear strength in order to calculate the shear capacity of the member assuming reinforcement perpendicular to the axis of the member. Equation 30 defines the shear strength provided by transverse reinforcement.

$$V_s = A_v f_{sy} d/s$$

(Equation 30)

where:

f_{sy} =   specified yield strength of non-prestressed reinforcement

Prestressed Concrete

AASHO introduced prestressed concrete provisions in the 1961 Standard Specifications (8th Edition).[8] The AASHO prestressed concrete design provisions were the first to use LFD methods. The specifications define a required shear reinforcement area based on the factored shear load V_u and the concrete shear strength V_c. The equation for required shear reinforcement area is shown as Equation 31.

$$A_v = \frac{(V_u - V_c)s}{2f'_y jd}$$

(Equation 31)

where:

V_u = shear due to ultimate load and effect of prestressing

V_c = shear carried by the concrete

s = longitudinal spacing of the web reinforcement

f'_y = yield stress of the shear reinforcement

jd = lever arm of the beam force couple

Note that the concrete shear strength was defined by AASHO in the 1965 Standard Specifications (9th Edition) to be as shown in Equation 32. [9] The shear strength is limited by a maximum concrete compressive strength of 3000 psi. Equation 32 was reintroduced in the AASHTO 1979 interim revisions and was considered an acceptable alternative up through the 2002 Standard Specifications (17th Edition). The equation is conservative and generally results in lower concrete shear capacities when designing with high strength concrete.[49]

$$V_c = 0.06f'_c b' jd \leq 180 \, b'jd$$

(Equation 32)

where:

b' = width of the member web

AASHO also defined a minimum value for the shear reinforcement as shown in Equation 33.

$$A_v = 0.0025b's$$

(Equation 33)

The 1971 interim revisions of the Standard Specifications modified the minimum required shear reinforcement area. The equation was changed to accommodate the use of high strength bars for stirrups. Reinforcement yield stresses were limited to 60,000 psi, considering that the intention of the stirrups was to control cracking and any higher strengths used would not contribute to crack control. This minimum shear reinforcement requirement is double that used in American Concrete Institute (ACI) 318 at the time because it was assumed that building design would require more shear reinforcement area with larger beam width to depth ratios. The modified formula is shown as Equation 34.

$$A_v = 100b's/f_{sy}$$

(Equation 34)

The 1980 interim revisions to the Standard Specifications introduced the general shear design equation for prestressed concrete. The equation establishes the LFD principles of load factors and member capacity modifiers as shown in Equation 35.

$$V_u \leq \phi(V_c + V_s)$$

(Equation 35)

Prestressed concrete shear resistance V_c is provided by the lesser of V_{ci} and V_{cw}. V_{ci} is the shear strength provided by concrete to resist flexure-shear cracks and V_{cw} is the strength required to resist web shear cracks resulting from excessive principal tensile stresses in the web. Equation 36 defines the concrete shear strength used to resist flexure-shear cracks.

$$V_{ci} = 0.6\sqrt{f'_c}b'd + V_d + \frac{V_i M_{cr}}{M_{max}}$$

(Equation 36)

where:

V_d = shear force at the section due to unfactored dead load

V_i = factored shear force at section due to externally applied loads occurring simultaneously

M_{cr} = moment causing flexural cracking at the section due to externally applied loads

M_{max} = maximum factored moment at the section due to externally applied loads

The moment required to produce flexural cracking is defined by Equation 37.

$$M_{cr} = \frac{I}{Y_t}\left(6\sqrt{f'_c} + f_{pe} - f_d\right)$$

(Equation 37)

where:

Y_t = distance from the centroidal axis of the gross section to the extreme fiber in tension

f_{pe} = compressive stress in concrete due to effective prestress forces only at the extreme tensile fiber of the section

f_d = stress due to unfactored dead load at the extreme tensile fiber of the section

The strength provided by concrete to resist web shear cracks is shown in Equation 38.

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3\, f_{pc}\right)b'd + V_p$$

(Equation 38)

where:

f_{pc} = compressive stress in concrete at the centroid of the cross section resisting externally applied loads or at the junction of web and flange when the centroid lies within the flange

V_p = vertical component of effective prestress force at a section

The shear resistance provided by the shear reinforcement is calculated using a similar formula used by non-prestressed concrete design. Equation 39 shows the calculation.

$$V_s = \frac{A_v f_{sy} d}{s} \leq 8\sqrt{f'_c} b' d$$

(Equation 39)

The 1983 Standard Specifications (13th Edition) included a provision that allowed for voided slabs to be cast without shear reinforcement.[17] Shear reinforcement could be omitted if the factored shear force was less than half of the shear strength provided by the concrete. The provision is similar to the previously described minimum shear reinforcement area requirements in the 1974 interim revisions.

Prestressed concrete shear provisions remained unchanged from 1980 to 2002 in the Standard Specifications.[20]

Load and Resistance Factor Design

In the mid 1980s, the AASHTO Subcommittee on Bridges and Structures requested the National Cooperative Highway Research Program (NCHRP) to conduct a study for an updated AASHTO Bridge Specification to correct the flaws in the existing specifications and study the feasibility of basing the updated specifications on LRFD methods. As a result, NCHRP Task 20-07 was created to satisfy the request. The findings of the task force lead to NCHRP Project 12-33 "Development of Comprehensive Specification and Commentary". The objectives of the project included providing a specification that was up to date on current research and technology, combining plain, reinforced, and prestressed concrete design, and introducing modified compression field theory and strut and tie concepts to concrete bridge design. Following the completion of the project in 1993, the 1st Edition of the *LRFD Bridge Design Specifications* was published in 1994.[21]

Strut and Tie Modeling

The 1994 *LRFD Bridge Design Specifications* (1st Edition) introduced STM to reinforced concrete bridge design specifications.[21] STM applies truss analogy to concrete member design by modeling the reinforcement as ties and concrete in compression as struts. This method provides an alternative to conventional design methods by allowing the designer to approximate load paths and combine force effects in the member. STM provides a means of design when the member is subject to a non-linear strain distribution. The Standard Specifications recommend using STM when designing deep footings, pile caps, or in other situations where the applied load is near the supporting reaction. STM provisions in the Standard Specifications are based on the works of Jörg Schlaich, Michael P. Collins, and Denis Mitchell. [38][39]

The axial capacity of the struts and ties is governed by the following equation (Equation 40).

$$P_r = \phi P_n$$

where:

P_r = factored resistance

P_n = nominal capacity

Compressive Struts

Compressive struts are proportioned to provide adequate axial capacity. The compressive strut cross-sectional area is dependent on the available concrete area and the anchorage conditions at the end of the strut. If reinforcement runs along the compressive strut, the compressive strength of the steel may be accounted for.

Tension tie strength is dependent on the area and the stress of the steel reinforcement. Mild steel in the tie is assumed to yield; therefore, the yield strength is used to determine the tensile strength of the tie. When prestressing steel is used, the tie strength is dependent on the stress in the prestressing steel plus the yield stress of the mild steel. It is assumed that the strain, which will cause concrete to crack, will be transferred to the prestressing steel. The estimated increase in stress that the prestressing steel will undergo is approximately equal to the yield stress in the mild steel.

Nodal regions have compressive stress limits based on the type of the node. There are three cases for which the Standard Specifications provide a stress limit: nodes bounded by compressive struts and bearing areas (CCC), nodes anchoring a one-direction tension tie (CCT), and nodes anchoring tension ties in more than one direction (CTT).

STM methods in the *LRFD Bridge Design Specifications* remained unchanged from their introduction in 1994 until the 2016 interim revisions.[21] The 2016 interim revisions provided additional guidance for nodal geometries and modified the procedure in which compressive struts are designed. Compressive capacities are calculated at nodal faces where factors for confinement and crack control are now considered.

The method of calculating minimum transverse reinforcement in LRFD remained constant throughout all editions of the specifications. The minimum transverse reinforcement equation used in the *LRFD Bridge Design Specifications* considers the effects of the concrete strength. The equation used to calculate minimum transverse reinforcement is shown as Equation 41.

$$A_v \geq 0.0316 \sqrt{f'_c} \frac{b_v s}{f_y}$$

Sectional Design Model

A refined sectional design model was introduced in the 1994 *LRFD Bridge Design Specifications* (1st Edition).[21] The sectional design model can be used where it is reasonable to assume that plane sections remain plane. The reinforced concrete member nominal shear strength under the sectional design model is composed of the concrete shear strength, the shear strength provided by the transverse reinforcement, and the shear strength provided by the prestressing force. The nominal shear capacity is determined by the lesser of the two equations given as Equation 42 and Equation 43.

$$V_n = V_c + V_s + V_p$$

(Equation 42)

$$V_n = 0.25f'_c b_v d_v + V_p$$

(Equation 43)

where:

V_p = effective prestressing force component in the direction of the applied shear

Modified compression field theory methods developed by Frank J. Vecchio and Michael P. Collins are used to calculate the concrete and transverse reinforcement shear strength contributions.[52] The concrete shear contribution is as shown in Equation 44.

$$V_c = 0.0316\beta\sqrt{f'_c}b_v d_v$$

(Equation 44)

where:

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear

d_v = effective shear depth

b_v = minimum web width within the depth d_v.

The shear strength provided by the transverse steel is as shown in Equation 45.

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s}$$

(Equation 45)

where:

θ =   angle of inclination of diagonal compressive stresses

α =   angle of inclination of transverse reinforcement to the longitudinal axis

Differences in Units

It is important to note that in the *LRFD Bridge Design Specifications*, the units for the ultimate concrete compressive strength, f'_c, are kips per square inch (ksi). The Standard Specifications use pounds per square inch (psi) units for the ultimate concrete compressive strength. Consequently, constants in equations where the square root operator is used for f'_c will not directly coincide between the two specifications.

Equations 44 and 45 are a function of the variables β and θ. Non-prestressed concrete sections that are not subjected to axial tension and containing the minimum amount of transverse reinforcement can take β as 2.0 and θ as 45 degrees. This simplified method of analysis reduces the reinforcement and concrete shear strength equations to the equations used in the LFD method in the Standard Specifications.

The general procedure for calculating β and θ requires convergence on the angle of inclination of diagonal compressive stresses, θ. The value for θ is derived from a graph that is a function of the strain in the reinforcement on the flexural tension side of the member. The strain in the reinforcement is a function of θ; therefore, it is necessary to first assume a value of θ then iterate to converge onto a solution. Equation 46 gives the formula for the strain in the reinforcement.

$$\epsilon_x = \frac{\left(\frac{M_u}{d_v}\right) + 0.5N_u + 0.5V_u \cot\theta - A_{ps}f_{po}}{E_s A_s + E_p A_{ps}} \leq 0.002$$

(Equation 46)

where:

M_u = factored moment

N_u = factored axial force

V_u = factored shear force

A_{ps} = area of prestressing steel on the flexural tension side

f_{po} = stress in the prestressing steel

E_s = non-prestressed reinforcing steel elastic modulus

A_s = area of non-prestressing steel

E_p = prestressing steel elastic modulus

Factored Shear Force

The factored shear force, V_u, is used for the longitudinal strain calculations because diagonal compressive stresses will result in a longitudinal compressive force equal to $V_u \cot\theta$. Because of the compressive force in the web, tensile forces must exist in the flanges for equilibrium. Therefore, each flange provides *0.5V_u cot θ* to maintain equilibrium. Note that the area of the non-prestressed and prestressed steel must be reduced for any lack of full development.

For members with transverse reinforcement, β and θ are also dependent on the shear stress in the member. In the case where members do not have transverse reinforcement, β and θ are dependent on the crack spacing, s_x. In the 1994 *LRFD Bridge Design Specifications* (1st Edition), there was no equation for crack spacing.[21] Crack spacing was estimated to be approximately 12 inches for members with stirrups, d_v for members with concentrated longitudinal reinforcement and no stirrups, or the vertical spacing of the crack control reinforcement for members with well-distributed longitudinal reinforcement and no stirrups.

The 2000 interim revisions to the *LRFD Bridge Design Specifications* modified the longitudinal reinforcement strain equation to give smaller strain values for members with at least the minimum shear reinforcement and larger values for members with less than the minimum shear reinforcement. Additionally, the interims accounted for prestressing forces when calculating the diagonal compression forces in the web that result in tensile forces in the flanges. Typically, these prestressing forces should reduce the strain in the longitudinal reinforcement. Equation 47 is used to calculate the longitudinal reinforcement strain for members that contain at least the minimum amount of transverse reinforcement.

$$\varepsilon_x = \frac{\left(\dfrac{M_u}{d_v}\right) + 0.5 N_u + 0.5 (V_u - V_p) \cot\theta - A_{ps} f_{po}}{2(E_s A_s + E_p A_{ps})}$$

(Equation 47)

When the member contains less than the minimum amount of transverse reinforcement, Equation 48 should be used.

$$\varepsilon_x = \frac{\left(\dfrac{M_u}{d_v}\right) + 0.5 N_u + 0.5 (V_u - V_p) \cot\theta - A_{ps} f_{po}}{E_s A_s + E_p A_{ps}}$$

(Equation 48)

When the longitudinal reinforcement strain is negative, the concrete compression must be accounted for in the strain calculation as shown in Equation 49.

$$\varepsilon_x = \frac{\left(\dfrac{M_u}{d_v}\right) + 0.5 N_u + 0.5 (V_u - V_p) \cot\theta - A_{ps} f_{po}}{2(E_c A_c + E_s A_s + E_p A_{ps})}$$

(Equation 49)

where:

E_c = concrete elastic modulus

A_c = area of concrete on the flexural tension side of the member

Crack Spacing Parameter

The 2000 interim revisions to the *LRFD Bridge Design Specifications* introduced an equation for crack spacing parameter, s_{xe}. The equation is shown as Equation 50.

$$s_{xe} = s_x \frac{1.38}{a_g + 0.63} \leq 80 \ in$$

(Equation 50)

where:

a_g = maximum aggregate size

s_x = lesser of d_v or the maximum distance between layers of longitudinal crack control reinforcement.

The 2007 *LRFD Bridge Design Specifications* (4th Edition) provided a simplified concrete shear strength calculation procedure for prestressed and non-prestressed sections.[24] The method is similar to the shear design methods for prestressed members provided by the Standard Specifications; however, the method introduced is modified to accommodate non-prestressed sections. The simplified method requires the evaluation of two nominal concrete shear resistances: the shear resistance when inclined cracking results from combined shear and moment, V_{ci}; and the shear resistance when inclined cracking results from excessive principal tensions in the web, V_{cw}. The formulas for the two concrete strengths are shown as Equation 51 and Equation 52.

$$V_{ci} = 0.02\sqrt{f'_c}b_v d_v + V_d + \frac{V_i M_{cre}}{M_{max}} \geq 0.06\sqrt{f'_c}b_v d_v$$

(Equation 51)

$$V_{cw} = \left(0.06\sqrt{f'_c} + 0.30 f_{pc}\right)b_v d_v + V_p$$

(Equation 52)

where:

V_d = shear force at the section due to unfactored dead load

V_i = factored shear force at the section due to externally applied loads occurring simultaneously with M_{max}

M_{cre} = moment causing flexural cracking at the section due to externally applied loads

f_{pc} = compressive stress in the concrete

M_{cre} is defined in Equation 53.

$$M_{cre} = S_c \left(f_r + f_{cpe} - \frac{M_{dnc}}{S_{nc}} \right)$$

(Equation 53)

where:

f_{cpe} = compressive stress in the concrete due to effective prestress forces at the tensile extreme fiber

M_{dnc} = total unfactored dead load moment acting on the non-composite section

S_c = section modulus for the extreme fiber of the composite section

S_{nc} = section modulus for the extreme fiber of the monolithic or non-composite section where tensile stress is caused by externally applied loads

f_r = concrete modulus of rupture

The simplified concrete shear strength calculation procedure for prestressed and non-prestressed sections was removed from the 2017 *LRFD Bridge Design Specifications* (8th Edition).[28]

Revised Sectional Design Model

The 2008 interim revisions to the *LRFD Bridge Design Specifications* revised the sectional design model to provide a non-iterative method for the evaluation of β and θ. The procedure involved providing algebraic equations, derived by Evan C. Bentz, Frank J. Vecchio, and Michael P. Collins in 2006, to determine β and θ.[52] The tables used to generate the two values were removed with the introduction of the β and θ equations. The evaluation of β depended on whether sections contained at least the minimum amount of shear reinforcing. When the section contained less than the required minimum amount of shear reinforcement, the following equation was used (Equation 54).

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)}$$

(Equation 54)

where:

ε_s = strain in non-prestressed longitudinal tension reinforcement

When the section contains at least the minimum amount of shear reinforcement, the following equation was used (Equation 55).

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \frac{51}{39 + s_{xe}}$$

(Equation 55)

where:

s_{xe} =   crack spacing parameter

The equation for the angle of inclination of diagonal compressive stresses is shown in Equation 56.

$$\theta = 29 + 3500\varepsilon_s$$

(Equation 56)

The longitudinal reinforcement strain equation was no longer dependent on θ after release of the 2008 interim revisions to the *LRFD Bridge Design Specifications*. The updated equation is shown as Equation 57.

$$\varepsilon_s = \frac{\dfrac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}}{E_sA_s + E_pA_{ps}}$$

(Equation 57)

Equation 57 replaces the shear force component of *0.5(V$_u$ - V$_p$) cot θ* with *|V$_u$ - V$_p$|*. The term *0.5 cot θ* is approximated to equal 1.0 without significant loss of accuracy. Note that the approximation assumes a value of approximately 27 degrees for *θ*. Larger values of *θ* result in a lower longitudinal reinforcement strain value, which ultimately results in higher concrete shear capacities. The approximation removes the need for iteration when calculating θ; however, the appendices allowed the designer to use the iterative method with tables.

A reinforcement shear strength equation to calculate the contributions of longitudinal bent-up bars was added with the release of the 2010 *LRFD Bridge Design Specifications* (5th Edition).[25] The equation is a function of the transverse steel area, the reinforcement yield stress, and the angle of inclination of the transverse reinforcement. The equation is shown as Equation 58.

$$V_s = A_v f_y \sin\alpha \le 0.095\sqrt{f'_c}b_v d_v$$

(Equation 58)

Concrete Density Modification Factor

The 2016 interim revisions to the *LRFD Bridge Design Specifications* added the concrete density modification factor, λ, to concrete strength equations to reduce the strength for lightweight concrete appropriately. The density modification factor varies from 0.75 to 1.00 depending on the concrete unit weight. A concrete unit weight of 100 pounds per cubic foot (pcf) or less requires a density modification factor of 0.75. A concrete unit weight of 135 pcf or more requires a density modification factor of 1.00. If the concrete unit weight is between 100 pcf and 135 pcf, linear interpolation is used to determine the modification factor.

Principal tensile stress limits were applied to the webs of segmental concrete bridges when analyzed at the Service III limit state to check the longitudinal shear resistance. The principal tensile stress limit at the neutral axis in the web was equal to *0.110√f'$_c$*. Construction principal tensile stress limits are similar when excluding Other Loads. When including Other Loads, the stress limit can be increased to *0.126√f'$_c$*.

Segmental bridge shear capacity equations from the *Guide Specifications for Design and Construction of Segmental Concrete Bridges* were adopted by the 2005 interim revisions to the *LRFD Bridge Design Specifications*. The nominal shear resistance for post-tensioned segmental concrete box girders was given as the lesser of the sum of the steel shear resistance and the concrete shear resistance or the equation shown as Equation 59.

$$V_n = 0.379\sqrt{f'_c}\,b_v d_v$$

(Equation 59)

The steel shear resistance equation is identical to the equation provided for prestressed concrete members. The concrete shear resistance is given by Equation 60. Note that this equation is shown in its incorrect form. An error was made in translating the equation from units of psi to ksi.

$$V_c = 0.0316K\sqrt{f'_c}\,b_v d_v$$

(Equation 60)

$$K = \sqrt{1 + \frac{f_{pc}}{0.125\sqrt{f'_c}}} \leq 2.0$$

(Equation 61)

The concrete shear resistance and shear stress variable were corrected in the 2007 *LRFD Bridge Design Specifications* (4th Edition) as shown in Equation 62 and Equation 63.[24]

$$V_c = 0.0632K\sqrt{f'_c}\,b_v d_v$$

(Equation 62)

$$K = \sqrt{1 + \frac{f_{pc}}{0.0632\sqrt{f'_c}}} \leq 2.0$$

(Equation 63)

It is important to note that the shear component of longitudinal prestress force, V_p, is instead added to the load effect with a load factor of 1.0 when designing post-tensioned segmental concrete box girder bridges. When the effects of torsion are required to be considered, the post-tensioned segmental concrete box girder cross-sectional dimensions are controlled by the equation shown as Equation 64.

$$\frac{V_u}{b_v d_v} + \frac{T_u}{2A_o b_e} \leq 0.474\sqrt{f'_c}$$

(Equation 64)

where:

T_u = applied factored torsional moment

A_o = area enclosed by shear flow path

b_e = effective thickness of the shear flow path of the elements making up the space truss model resisting torsion.

The shear provisions for segmental concrete bridges and post-tensioned box girders are termed as an alternative shear design procedure to MCFT for segmental concrete bridges with the release of the 2017 *LRFD Bridge Design Specifications* (8th Edition).[28]

2.1.3.   Shear Reinforcement Details

Types of Shear Reinforcement

Several types of shear reinforcement are listed in the Standard Specifications. The types have expanded gradually over several editions. The two original types of shear reinforcement introduced in the Standard Specifications are vertical stirrups and longitudinal bars bent up in series or in a single plane (Figure 1). A combination of stirrups and bent-up bars is also permitted.
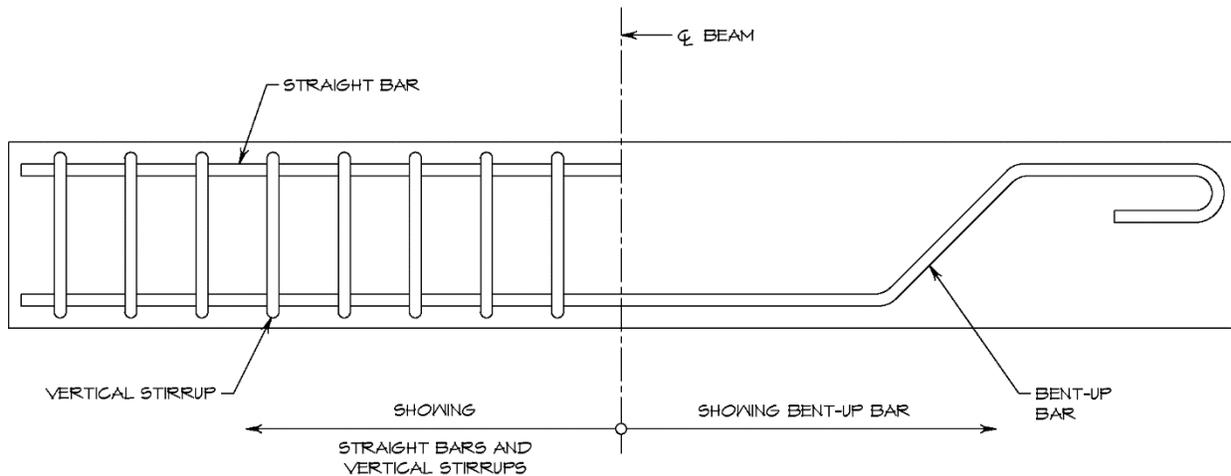

Figure 1. Illustration. Vertical Stirrups and Bent-up Longitudinal Bars.

The AASHTO 1974 interim revisions to the 1973 Standard Specifications (11th Edition)[11] expand the types of shear reinforcement. Welded wire reinforcement with longitudinal anchorage wires, and spiral reinforcement, are listed as new types of shear reinforcement.

Several changes were made in the 1974 interims to improve clarity. General provisions for spacing and anchorage are included, with sections for individual types presented as needed. Existing sections for stirrups are expanded to provide guidance for single leg, single-U, and multiple U-stirrups. Closed stirrups formed by splicing two U-stirrups are also addressed. Torsional reinforcement is addressed in a separate section, requiring the use of longitudinal bars and transverse reinforcement such as closed stirrups, closed ties or spirals.

The *LRFD Bridge Design Specifications* include two more types of shear reinforcement: angled anchored prestressed tendons and, as of the 5th edition (2010), hoops.[25] These two types of reinforcement are also possible choices for torsional reinforcement, as well as welded wire reinforcement cages as of the 3rd edition (2002).

Limits on Spacing

Limits on the spacing of shear reinforcement have changed substantially between the Standard Specifications and the current *LRFD Bridge Design Specifications* (2017).[28] The maximum allowable spacing has been reduced over time. Research in diagonal shear crack opening and propagation led to this gradual reduction in spacing to control these phenomena. As research has progressed, so has the sophistication of the provisions in the specifications.

The AASHO Standard Specifications provide requirements for the spacing of stirrups and bent-up bars, in addition to spacing of the shear reinforcement from the face of the support. Stirrups and bent-up bars are limited to a spacing of three-fourths the effective depth of the beam or less, measured at the neutral axis and in the direction of the beam's longitudinal axis. The U.S. Department of Commerce Bureau of Public Roads' *Criteria for Prestressed Concrete Bridges* (1954) recommended similar stirrup spacing requirements as those presented in the Standard Specifications.

The first stirrup or bent-up bar's maximum spacing from the face of the support was originally limited to one-half the effective depth. This support face spacing changed for vertical stirrups in the 1949 Standard Specifications (5th Edition),[5] where this distance was reduced to one-fourth the effective depth. One-half the effective depth remained an acceptable distance for bent-up bars.

The AASHTO editions of the Standard Specifications address the shear cracking phenomena more directly in addition to providing limits on bar to bar spacing. The 1977 Standard Specifications (12th Edition) required the spacing between bent-up bars or inclined stirrups to result in at least one line of reinforcement crossing every 45-degree line extending toward the reaction from mid-depth of the member to the longitudinal tension reinforcement.[14] The maximum spacing between bars was reduced, limited to the lesser of half the effective depth (d/2) or 24 inches. This is the first instance of a hard limit to the maximum spacing, as opposed to a limit solely based on the effective depth of the member. Finally, the provisions for the maximum distance between the first bar and the face of the support were removed in the 1977 Standard Specifications (12th Edition).[14] Spacing limitations remain unchanged for subsequent editions of the Standard Specifications.

The *LRFD Bridge Design Specifications* see substantial changes to the spacing limitations. Limits on spacing become based on the ultimate loading of the member. The *1994 LRFD Bridge Design Specifications* (1st Edition) limits shear reinforcement spacing based on the following two equations (Equations 65 and 66).[21]

$$V_u < 0.1 f_c' b_v d_v$$

(Equation 65)

$$V_u \geq 0.1 f_c' b_v d_v$$

(Equation 66)

where:

V_u = shear force

b_v = minimum web width within depth d_v, modified for presence of ducts where applicable

d_v = effective shear depth, the distance between resultants of tensile and compressive forces due to flexure, measured perpendicular to the neutral axis and limited to the greater of $0.9d_e$ or $0.72h$

If Equation 65 is true, the maximum spacing is limited to the lesser of $0.8d_v$ or 24 inches. If Equation 66 is true, the maximum spacing is limited to the lesser of $0.4d_v$ or 12 inches. This reduction in maximum spacing is intended to provide crack control to sections subject to high shear stress. It should be noted that the maximum spacing of $0.8d_v$ might not restrict diagonal crack opening in certain situations. NCHRP Report 579 found that diagonal cracking in prestressed girders can be at a sufficiently steep angle that no stirrups would intersect and impede the opening of a diagonal crack.[53] The commentary sections of the 2014 and 2017 *LRFD Bridge Design Specifications* (7th and 8th Editions) state that a limit of $0.6d_v$ may be a viable way to address this issue, amongst other approaches unrelated to shear reinforcement spacing.[27 28] Commentary in the *LRFD Bridge Design Specifications* also directly instructs the engineer to orient inclined stirrups and prestressed tendons to intercept potential diagonal cracks while keeping the reinforcement as close to normal as is practical.

Equations 67 and 68 are revised in the 1998 *LRFD Bridge Design Specifications* (2nd Edition)[22] to be in terms of stress rather than force, simplifying them to:

$$v_u < 0.125 f_c'$$

(Equation 67)

$$v_u \geq 0.125 f_c'$$

(Equation 68)

The coefficient is increased by 25 percent, increasing the shear loading required to reduce maximum spacing of reinforcement.

The 2004 *LRFD Bridge Design Specifications* (3rd Edition) includes additional requirements for segmental post-tensioned box girder bridges for shear and torsion. Equations 69 and 70 are modified as follows:

$$v_u < 0.19\sqrt{f_c'}$$

(Equation 69)

$$v_u \geq 0.19\sqrt{f_c'}$$

(Equation 70)

The maximum spacings associated with these equations have also changed; if Equation 69 is true, the maximum spacing is limited to the lesser of $0.8d$ and 36 inches. If Equation 70 is true, the maximum spacing is limited to the lesser of $0.4d$ and 18 inches. These maximum spacings are further reduced for closed stirrups or ties required to resist shear effects due to torsional moments; their spacing is limited to the lesser of half of the shortest dimension of the member or 12 inches. Finally, all transverse reinforcement is required to extend at least $h/2$ beyond the point it is theoretically required.

Anchorage Requirements

Anchorage requirements for shear reinforcement have become more rigorous over time. Several methods are introduced in the Standard Specifications.

The anchorage provisions of the AASHO Standard Specifications do not change over their 11 editions. Several methods of anchorage are provided for bent-up bars and stirrups. Both types of reinforcement must not experience stress beyond the capacity of the reinforcement's anchorage in the upper or lower half of the member's effective depth.

Bent-up bars may have adequate anchorage due to continuity with the main reinforcement. Bars may be considered completely anchored by embedment of the appropriate length in the upper or lower half of the beam if at least half of this embedment is as close to the upper or lower surface of the beam as is allowed by clear cover requirements.

Stirrups must be anchored at both ends, through use of one or several methods. Rigid attachment of the stirrups to main longitudinal reinforcement (such as by welding) is one acceptable method. Stirrups may be bent around longitudinal bars and kept in close contact with them to form a U-stirrup or hook. Standard hooks should be placed as close to the upper or lower surface of the beam as allowed by clear cover requirements. Standard hooks for stirrup and tie anchorage require a 90- or 135-degree turn in addition to a minimum extension of six bar diameters or 2.5 inches. Anchorage may be provided by simply providing an adequate length of embedment in the upper or lower half of the effective depth of the beam. The code advises against using this type of anchorage alone when shear stress in the web exceeds that recommended for beams without end anchorage.

The 1977 AASHTO Standard Specifications (12th edition) introduced more rigorous anchorage requirements specific to different types of shear reinforcement. In general, longer embedment lengths or larger hook bends are required. Open stirrups are required to have an embedment length of 0.5 l_d in addition to standard hooks. Embedment as an independent method of anchorage for open stirrups requires a full development length l_d or a minimum of 24 bar or wire diameters or 12 inches long. Bending open stirrups around longitudinal reinforcement requires a 180-degree hook at minimum. Pairs of U-stirrups or ties placed together to form a closed unit may be used; a lap length of 1.7 l_d is required for the pairs to be considered properly spliced. All stirrups, open or closed, are required to be at a 45-degree angle or greater to the longitudinal reinforcement. Commentary in later editions of the *LRFD Bridge Design Specifications* addressed this, stating that stirrups at angles shallower than 45 degrees are difficult to anchor effectively against slip. Bent-up bars are also given a more restrictive angle. Originally, longitudinal bars could be bent up at an angle between 20 and 45 degrees; however, the AASHTO Standard Specifications change this to an angle of 30 degrees or more. In addition, bent-up bars are required to be continuous with longitudinal reinforcement when the bent-up bars are extended into a region of tension, as well as being anchored beyond $d/2$ for a development length described elsewhere in the code when extended into a region of compression.

Welded wire fabric has some additional provisions for anchorage beyond those given for stirrups. Two longitudinal wires must be included for each leg of welded smooth wire fabric forming single U-stirrups. They may be placed at a 2-inch spacing along the member at the top of the U. Alternatively, one may be placed at most a distance $d/4$ from the compression face with the second at least 2 inches from the first wire. This second wire may be located on the stirrup

leg beyond a bend or on a bend with an inside bend diameter greater than or equal to eight wire diameters. This addition was made on the recommendations of the Precast Concrete Institute (PCI) Technical Activities Committee's Joint PCI and Wire Reinforcement Institute Ad Hoc Committee on Welded Wire Fabric for Shear Reinforcement in 1980.[41] The committee examined existing code in the ACI Building Code to determine adequate anchorage provisions for welded wire reinforcement that complied with the intent of existing code. The requirements previously described were adopted prior to the 1983 Standard Specifications (13th Edition) in addition to the 1983 edition of the ACI Building Code.[17]

Anchorage for welded wire fabric has been investigated in several experimental studies. Robertson and Durrani examined the impact of longitudinal wires in a study of 13 prestressed T-beams.[40] Four of the beams used one anchorage wire, five beams used two wires, one beam used hooks, and the remaining three beams had no wire reinforcement. The beams using two wires did not experience anchorage failure despite failure of the welds of one of the two wires. One of the four beams that used one anchorage wire experienced a premature anchorage failure because a vertical wire separated from the single anchorage wire just above a shear crack. The shear crack opened and caused the beam to experience an early shear failure. The researchers concluded that the use of two longitudinal wires is desirable to avoid early shear failure due to local weaknesses in the welded wires. Other studies have noted the satisfactory behavior of two longitudinal wires, such as Xuan (1988)[42] and Mitchell (1994)[43].

Some general provisions are retained along with the type-specific requirements. The requirement for the location of stirrups and bent-up bars relative to the compression and tension faces of the member is applied to all forms of shear reinforcement. The section limiting the stress in the reinforcement to the capacity of the anchorage also changes to require the reinforcement to be anchored for its design yield strength.

The *LRFD Bridge Design Specifications* further refine the required embedment length for stirrup legs. Ends of single-leg, simple-U, or multiple U-stirrups have anchorage provisions based on bar size and grade of steel. Number 5 bars and D31 wire and smaller of any grade of steel, and Number 6, 7, and 8 bars of f_y less than 40 ksi require a standard hook around longitudinal reinforcement. Number 6, 7, and 8 bars with f_y greater than 40 ksi require an embedment length, l_e, between mid-depth of the member and the outside end of the hook in addition to the standard hook. This embedment length must satisfy the following equation as shown in Equation 71.

$$l_e \geq \frac{0.44 d_b f_y}{\sqrt{f_c'}}$$

(Equation 71)

2.2. Evaluation of Existing Bridges, 1941 to Present

Regulation on the evaluation and rating of existing bridges began in 1941 when AASHO included the "Rating of Existing Bridges" section in the Standard Specifications (3rd Edition).[3] The section discussed the types of loads to consider, defined maximum unit stresses to be used for different types of members, and briefly discussed field inspection. The Standard Specifications continued to include a load rating section until 1977[14] when the specification began instructing the designer to refer to the *Manual for Maintenance Inspection of Bridges*.[12] The first edition of this manual was published in 1970 and revisions were made up until 1990.

The intent of the manual was to provide uniformity in the determination of the physical conditions and maintenance needs of highway bridges. AASHTO published the *Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges* in 1989,[29] which introduced new methodologies for rating bridges. In 1994, the first edition of the *Manual for Condition Evaluation of Bridges* was published.[33] This was the main reference used for bridge load rating until the release of the *Guide Manual for Condition Evaluation of Load and Resistance Factor Design of Highway Bridges* in 2003.[36] The most current load rating provisions are defined by the MBE, first published in 2008.[30] This section provides an outline of the shear load rating procedures discussed by these documents.

2.2.1. Field Inspection

Field inspection influences bridge load rating, because the observed conditions may affect the rating calculations (for example, observations of spalling and cracking may reduce the concrete section) and can corroborate load ratings that are less than the design loads.

Inspection Requirements

When AASHO introduced field inspection requirements in 1941, it mandated inspections to be conducted by a thoroughly trained and competent engineer familiar with all phases of bridge design and construction.[3] Recorded bridge information included line diagrams showing lengths and positions of all members, detailed dimensions of all members and connections, and information about the condition of materials showing reduced sections due to deterioration, accident, or other causes.

The inspection requirements broadened when AASHO published the *Manual for Maintenance Inspection of Bridges* in 1970.[12] Inspector qualifications became more detailed and specified that the engineer be capable of determining the safe load carrying capacity of the structure. The engineer is required to recognize any structural deficiency and take appropriate action necessary to keep the bridge in safe condition. Additionally, the 1970 manual recommended the use of sketches and photographs to reduce long, wordy descriptions.[12]

The 1970 *Manual for Maintenance Inspection of Bridges* provided a list of inspection items to be examined while at the bridge.[12] The list included approaches, waterway, piers and abutments, bents, girders, bearings, expansion joints, deck, curbs, sidewalks, railings, and observation of the passage of heavy loads. Pier and abutment inspections required investigation of suspected movement or settlement, evidence of scour or undercutting, and examination for cracks or deterioration of the concrete. Bent caps were to be observed for deterioration and excessive deflections. It recommended that girder stems be examined for abnormal cracking or disintegration of the concrete and checked for damage from vehicles passing under the bridge. Prestressed concrete girders were examined for alignment, cracking, and deterioration. It required recording locations and sizes of cracks found on girders to assist with future analysis and inspection of the girder. It also required checking concrete decks for cracking, leaching, scaling, potholing, spalling, deterioration, slipperiness, and drainage.

The 1974 *Manual for Maintenance Inspection of Bridges* (2nd Edition) refined the qualifications of inspection personnel.[13] The manual states that the inspector must be a registered professional engineer or be qualified for registration as a professional engineer and have a minimum of 10 years of experience in bridge inspection assignments. Lastly, the inspector was required to have completed a comprehensive training course based on the *Bridge Inspector's Training Manual*.

The 1994 *Manual for Condition Evaluation of Bridges* defined five types of inspections: initial, routine, damage, in-depth, and special inspections.[33] Initial inspections are normally the first inspection a bridge sees and could be performed after a widening or a change of ownership. The purpose of an initial inspection is to provide structure inventory and appraisal data and to determine the baseline structural conditions of the bridge. Routine inspections are conducted in accordance with the National Bridge Inspection Standards. Observations and measurements are performed to determine the condition of the bridge. Areas that have previously been deemed critical to load-carrying capacity should be closely monitored in these inspections. Damage inspections are unscheduled and are necessary to analyze structural damage to a bridge element. The inspection is conducted to determine the need for emergency load restrictions or bridge closure and to assess the level of effort necessary for repair. The purpose of in-depth inspections is to inspect members for deficiencies that are not readily detectible using routine inspection procedures. In-depth inspections may require special equipment or skilled personnel depending on the situation. A load rating of the bridge may be necessary depending on the extent of the observed damage. Lastly, a special inspection is scheduled to observe a known deficiency in the bridge. Special inspections usually do not meet National Bridge Inspection Standards requirements for biennial inspections.

The 2008 MBE included two other inspection types: fracture-critical inspections and underwater inspections.[30] Fracture-critical inspections require the engineer to identify fracture-critical members and develop a plan for inspecting the members. Underwater inspections are required to locate deterioration or structural deficiencies in members not easily accessible due to inundation. Underwater inspections can be subdivided into routine wading inspections and in-depth underwater inspections. Routine wading inspections should be conducted during all routine inspections to evaluate the structural integrity of foundations. In-depth underwater inspections are required where members cannot be inspected visually or by wading. The 2010 interim revisions to the MBE removed the "In-depth Underwater Inspection" section from the manual.[31]

Inspection Frequency

The 1970 *Manual for Maintenance Inspection of Bridges* (1st Edition) established the first requirements for inspection frequencies of bridges.[12] The manual stated that bridges should be inspected at regular intervals not to exceed 2 years. Bridges with known deficiencies or bridges that are in questionable condition required interim inspections. Bridges with a posted weight limit that was less than the legal limit at the time were also required to have interim inspections. The 1986 interim revisions to the *Manual for Maintenance Inspection of Bridges* listed a few examples for bridges requiring interim inspections, including new structure types, structures incorporating details which have no performance history, structures with potential foundation and scour problems, and non-redundant structures.

The 1994 *Manual for Condition Evaluation of Bridges* provided inspection frequencies based on the type of inspection conducted.[33] Initial inspections were required after initial construction, a retrofit, or a change in ownership. A routine inspection typically requires the regular interval of inspection to not exceed 2 years. Certain bridges with prior Federal Highway Administration (FHWA) approval allowed a regular interval not to exceed 4 years when justified by previous evaluation. Damage inspections are required as necessary to assess structural damage following an incident. In-depth inspections are conducted as needed for specific members not easily

evaluated during a routine inspection. Lastly, special inspections are scheduled as needed to monitor a known or suspected deficiency.

2.2.2. *Concrete Bridge Shear Load Rating*

A bridge load rating is the safe load-carrying capacity of a bridge based on the existing conditions of the bridge. Existing conditions may be determined by bridge design plans and field inspections. The bridge load rating is represented by a rating factor that can be multiplied by the rating vehicle in tons to obtain the weight that the bridge can safely support. Multiple rating factors may be defined for a single bridge depending on the type of rating being conducted.

Rating Categories

Rating categories are defined by the type of live load the bridge is expected to see and the allowable stress limits for the materials. The oldest and most common ratings are the inventory rating and the operating rating, first introduced in the 1941 Standard Specifications (3rd Edition).[3] The 2003 *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges* established three different types of load ratings: legal load rating, design load rating, and permit load rating.[36] This section describes each of the different types of load ratings.

Inventory Rating

The 1941 Standard Specifications (3rd Edition) defined the inventory rating as the "classification of existing bridges in terms of the standard H loadings."[3] Essentially, the inventory rating defines a vehicular live load that the bridge can withstand for an indefinite period. Typically, the live load specified in the most current design specifications is used. If the rating factor is greater than 1.0 after conducting an inventory rating, the bridge requires no restrictive posting. If the rating factor is less than 1.0, an operating rating may be conducted.

Operating Rating

The operating rating is defined as the safe load-carrying capacity of the structure and is associated with higher allowable stresses to allow for higher live loads. This allows for a less conservative level of design, which leads to a shorter bridge design life. The intent is to keep the structure in service with more frequent inspections until repair or replacement can occur.

Design Load Rating

The design load rating method was included in the 2008 MBE under LRFD methods.[30] It is based on HL-93 loading and LRFD standards. Two levels of reliability are used under the design load rating and are comparable to the inventory rating and operating rating. The inventory rating is associated with a higher level of reliability, whereas, the operating rating is conducted under a lower level of reliability. The load and resistance factors are calibrated to achieve evaluation under higher or lower reliability. It is important to verify that legal loads are not significantly larger than HL-93 loading for bridges that only satisfy operating level reliability. Note that the term of design load rating is only applicable when conducting a load rating using LRFD methods.

Legal Load Rating

If the design load rating yields a rating factor less than 1.0, a legal load rating is conducted, which provides a safe load capacity for the bridge. According to the MBE, the live load factors are selected based on the truck traffic conditions at the site. The strength limit state is primarily used for this load rating. The purpose of a legal load rating is to determine the need for load posting and strengthening.

Permit Load Rating

Lastly, permit load rating involves determining the effects of loads exceeding federal or state legal vehicle weight limitations on the structure. The permit usually specifies the vehicle's loading and route. It may be given for a single trip, a limited number of trips, or an unlimited number of trips in a defined duration of the permit. As needed, restrictions may be prescribed on the permit for the vehicle when crossing the bridge, such as controlling speed, lane position, and presence of other vehicles on the bridge.

Rating Methods

Allowable Stress Rating

Allowable stress methods have been used since 1941 to load rate bridges. The 1941 Standard Specifications (3rd Edition)[3] required inventory ratings to use the live loads used for the design of new bridges. Operating ratings required determining live loads based on vehicle size and type using the highway and determining the impact factor based on the local conditions. The load rating section of the specifications directed the engineer to the design sections to determine the safe live load capacity of the bridge. Unit stresses for steel were limited to 0.545 times the yield strength when determining the load carrying capacity of each member. In 1944, the specifications stated that steel unit stress shall not exceed 0.545 of the yield point for inventory ratings and steel unit stress shall not exceed 0.85 of the yield point for operating ratings.[4]

The 1962 interim revisions to the Standard Specifications redefined the load rating section of the specification. This updated section focused only on overload permits. The tensile stresses in steel reinforcement were limited to 75 percent of the yield strength. Generally, allowable stresses to be used for the overload permit were those specified in the design section of the specification. These requirements remained in the specifications until the release of the 1977 Standard Specifications (12th Edition),[14] where the designer was instructed to refer to the *Manual for Maintenance Inspection of Bridges* (2nd Edition).[13]

Manual for Maintenance Inspection of Bridges

The 1st edition of the *Manual for Maintenance Inspection of Bridges* was released in 1970.[12] The manual states that the steel working unit stresses are limited to 0.55 times the yield strength for inventory ratings and 0.75 times the yield point for operating ratings. The manual directs the engineer to the most current Standard Specifications for matters not covered by the manual. It is expected that the investigating engineer increase safety factors when dealing with uncertainties, reduce member sizes or area to account for deterioration, and reduce allowable stresses in materials based on their quality. The manual requires the engineer use one of three typical vehicles defined in the manual or the standard AASHO H loading when determining the inventory rating; HS loading is introduced to the manual in 1974. The effects of impact were to be included in the live load. Lateral loads, longitudinal loads, and thermal forces were not

considered in determining load restrictions. In cases where longitudinal stability is an issue, speed restrictions were set. In general, the maximum unit stresses used to check capacity are expected to be taken from the manual. The allowable unit stress for reinforcing steel is 25,000 psi. The manual does not clarify whether this allowable stress applies to the operating rating or inventory rating. Furthermore, the definition of an allowable unit stress for the reinforcing steel is contradictory to the stress limits set by the manual. The allowable shear stress is defined by the equation shown as Equation 72.

$$v = v_s + v_c = v_s + 0.05f'_c$$

(Equation 72)

where:

v = total unit shear

v_s = shear taken by steel

v_c = shear taken by concrete

f'_c = breaking strength of concrete (max. 3,250 psi)

The 1974 *Manual for Maintenance Inspection of Bridges* (2nd Edition) more clearly defined the allowable unit stresses for reinforcing steel by providing steel grades for both the inventory and operating ratings.[13] Table 20 lists the allowable reinforcing steel stresses used in the manual.

Table 20. 1974 Manual for Maintenance Inspection of Bridges Reinforcement Unit Stresses

Steel Grade	Inventory Rating (psi)	Operating Rating (psi)
Structural or Unknown Grade	18,000	25,000
Grade 40 (Intermediate)	20,000	28,000
Grade 50 (Hard)	20,000	32,500
Grade 60	24,000	36,000

Unit Stresses

The allowable shear unit stresses and reinforcing steel unit stresses defined in the *Manual for Maintenance Inspection of Bridges* remained constant through all later editions of the manual.

In 1989, the *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges* was published with the purpose of establishing a methodology for rating existing bridges.[29] The guide is discussed in the "Load Factor Rating" section of this report because the methodology uses load and strength reduction factors.

The 1994 *Manual for Condition Evaluation of Bridges* makes use of the rating factor defined previously in the *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges*.[33] The rating factor is defined as the ratio of the member capacity minus the dead load effects on the live load. The ratio provides a measure of the excess capacity of a structural member. A bridge has passed the rating when its most critical member has a rating factor calculated as greater than 1.0. The rating of the structure can be calculated by multiplying the

rating factor by the gross weight of the rating vehicle in tons. Equation 73 shows the rating factor calculation.

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

(Equation 73)

where:

RF = rating factor

C = capacity of the member

D = dead load effect on the member

L = live load effect on the member

I = impact factor

A_1 = factor for dead loads

A_2 = factor for live loads

Note that load effects can be defined as axial forces, vertical shear forces, bending moments, axial stress, shear stresses, and bending stresses. The load factors A_1 and A_2 are both equal to 1.0 when using the allowable stress method.

The allowable reinforcing steel unit stresses defined in the 1994 *Manual for Condition Evaluation of Bridges*[33] are identical to those previously introduced in the 1970 *Manual for Maintenance Inspection of Bridges* (1st Edition).[12] The allowable shear unit stress equations are similar until the 1998 interim revisions to the *Manual for Condition Evaluation of Bridges.* The updated shear unit stress equation provides a more detailed calculation for the shear stress carried by the concrete. Alternatively, the shear stress carried by the concrete could be calculated as $1.3\sqrt{f'_c}$. Note that this method is similar to the method introduced in the 1977 Standard Specifications (12th Edition);[14] however, the concrete shear stress equations used in the *Manual for Condition Evaluation of Bridges* produce larger capacities than those used in the Standard Specifications because of larger coefficients used in the equation. The concrete contribution to shear strength is shown as Equation 74.

$$v_c = 1.25 \sqrt{f'_c} + 1600\rho_w \frac{Vd}{M} \leq 2.3 \sqrt{f'_c}$$

(Equation 74)

When severe shear cracks have been observed, the concrete shear stress v_c must be taken as zero and all shear stress should be resisted by the steel reinforcement.

The *Manual for Condition Evaluation of Bridges* instructs the engineer to use the AASHTO Design Specifications when calculating an inventory level rating. Specifications established by the bridge owner may be used if the requirements are more stringent. The manual limits the moments in prestressed concrete members to 75 percent of the ultimate moment capacity. The 1996 interim revisions to the *LFRD Bridge Design Specifications* referred the engineer to the "Load Factor Method" section of the manual for specifications on prestressed concrete.

AASHTO released the 3rd edition of the MBE in 2018.[32] The allowable stress rating method presented in this manual keeps the two levels of evaluation, the inventory and operating rating levels. The procedure used in calculating the rating factor by means of the allowable stress method remains unchanged from the method presented in the *Manual for Condition Evaluation of Bridges*.

Load Factor Rating

The Load Factor Method was introduced in the 1978 *Manual for Maintenance Inspection of Bridges* (3rd Edition).[15] The load factor method includes both load factors and capacity reduction factors in the rating equations. The manual provided two equations for the rating factor depending on the level of rating. The inventory strength analysis equation is given as Equation 75.

$$\phi S_u = 1.3[S_D + \left(\frac{5}{3}\right)(RF)(S_L + I)]$$

(Equation 75)

The operating strength analysis equation is shown as Equation 76.

$$\phi S_u = 1.3[S_D + (RF)(S_L + I)]$$

(Equation 76)

where:

ϕ = capacity reduction factor

S_u = ultimate theoretical strength

S_D = effect of dead load

$S_L + I$ = effect of live load plus impact

RF = rating factor

The operating level rating equation will yield a rating factor larger than that given by the inventory rating equation.

The 1983 *Manual for Maintenance Inspection of Bridges* expands on the load factor rating method for bridges.[16] The manual added a serviceability strength rating factor equation to the inventory level rating. The serviceability strength equation only applied to steel members. The 1985 interim revisions to the *Manual for Maintenance Inspection of Bridges* added the fatigue strength and concrete crack control rating factor equation. It was required that all three conditions be satisfied in an inventory strength analysis. The fatigue strength and concrete crack control equation is shown as Equation 77.

$$\phi S_u = [D + (RF)(L + I)]$$

<div align="right">(Equation 77)</div>

where:

D = effect of dead load

L = effect of live load

The fatigue equation simply removes the load factors. Equation 77 is identical to the serviceability strength equation used in the operating strength analysis. The operating level rating only analyzes the maximum strength and the serviceability limits states. The *Manual for Maintenance Inspection of Bridges* states that the specification only applies to simple and continuous beam and girder bridges with spans of up to 500 feet. Additionally, it requires the engineer to refer to the Standard Specifications for provisions and design requirements that are not covered by the manual.

The 1989 *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges* redefined the rating factor equation by replacing the dead and live load factors with variables.[29] The updated rating equation is shown as Equation 78.

$$R.F. = \frac{\phi R_n - \gamma_D D}{\gamma_L L(1 + I)}$$

<div align="right">(Equation 78)</div>

where:

R_n = the nominal strength

γ_D = dead load factor

γ_L = live load factor

It is important to note that the guide recommends three AASHTO legal vehicles for the vehicular live load used when rating the bridge. The guide discourages the engineer from using the standard AASHTO H or HS design loading. Live load factors are dependent on Average Daily Truck Traffic (ADTT) and the effectiveness of overload enforcement on the bridge. Table 21 provides the load factors used in the rating factor equation in Equation 78. The table shows that the live load factors will increase with larger volumes of truck traffic and lack of overload enforcement.

Table 21. Rating Equation Load Factors from 1989 Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges

Loading	Load Factor
Dead Load	$\gamma_D = 1.20$
Live Load (ADTT < 1000, reasonable enforcement and apparent control of overloads)	$\gamma_L = 1.30$
Live Load (ADTT > 1000, reasonable enforcement and apparent control of overloads)	$\gamma_L = 1.45$
Live Load (ADTT < 1000, significant sources of overloads without effective enforcement)	$\gamma_L = 1.65$
Live Load (ADTT >1000, significant sources of overloads without effective enforcement)	$\gamma_L = 1.80$

Note: ADTT = Average Daily Truck Traffic.

Resistance factors were calculated based on the existing conditions of the bridge. The resistance factors ranged from 0.55 to 0.95 depending on the superstructure condition, redundancy, inspection, and maintenance. For example, a member with heavy deterioration, no redundancy, loosely estimated section losses, and intermittent maintenance activity will have a resistance factor of 0.55. The guide provides a descriptive method and table to assist the engineer in determining the resistance factors. Lastly, the guide allows the engineer to reduce the live load effect on the bridge when multiple lanes are loaded. The reduction factors used are shown in Table 22.

Table 22. Rating Equation Reduction Factors from 1989 Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges

Number of Lanes	Reduction Factor
One or two lanes	1.0
Three lanes	0.8
Four lanes	0.7

The 1994 *Manual for Condition Evaluation of Bridges* uses both the allowable stress and load factor methods in determining the rating factor.[33] Equation 78 is used to calculate the rating factor using the load factor method. Note that this equation is the same equation used in the allowable stress method. The dead load factor, A_1, is taken as 1.3. For the inventory level, the live load factor, A_2, is taken as 2.17. For the operating level, A_2 is taken as 1.3.

The MBE provides minimal guidance for the rating of bridges under the load factor method. The load factor method, like the allowable stress method, has only the two traditional rating levels, inventory, and operating. Furthermore, the rating factor equation remains unchanged in this manual. The manual directs the engineer to the Standard Specifications for determination of the nominal capacity of concrete members.

Load and Resistance Factor Rating

The 2003 *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges* recognizes three load-rating procedures for Load and Resistance Factor Rating (LRFR): design load rating, legal load rating, and permit load rating.[36] Recall that design load rating consists of both the inventory rating level and the operating rating level. It is important to note that it is not required to check concrete members for shear when the member shows no visible signs of shear distress when rating for the design load or legal load. The general rating factor equation used for LRFR is shown as Equation 79.

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)}$$

(Equation 79)

where:

C = capacity

γ_{DC} = LRFD load factor for structural components and attachments

DC = dead load effect due to structural components and attachments

γ_{DW} = LRFD load factor for wearing surfaces and utilities

DW = dead load effect due to wearing surface and utilities

γ_P = LRFD factor for permanent loads other than dead loads

P = load effect due to permanent loads other than dead loads

γ_{LL} = evaluation live load factor

LL = live load effect

IM = dynamic load allowance

The capacity of a given member is calculated with the equation shown as Equation 80 for the strength limit state.

$$C = \varphi_c \varphi_s \varphi R_n$$

(Equation 80)

where:

φ_c = condition factor

φ_s = system factor

φ = LRFD resistance factor

R_n = nominal member resistance.

The service limit state capacity is calculated as shown in Equation 81.

$$C = f_R$$

(Equation 81)

where:

f_R = allowable stress specified in the LRFD code

Load Factors for Dead Loads and Design Load Rating Live Loads

The load factors for dead loads and design load rating live loads are defined in Table 23 for reinforced and prestressed concrete. The legal load and permit load rating load factors require evaluation of existing conditions such traffic volume, loading condition, and permit type. The load factor for permanent loads other than DC or DW is always taken as 1.0.

Table 23. 2003 Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges

Concrete Bridge Type	Limit State	Dead Load γ_{DC}	Dead Load γ_{DW}	Design Load Inventory γ_{LL}	Design Load Operating γ_{LL}
Conventionally Reinforced	Strength I	1.25	1.50	1.75	1.35
Conventionally Reinforced	Strength II	1.25	1.50	-	-
Conventionally Reinforced	Service I	1.00	1.00	-	-
Prestressed	Strength I	1.25	1.50	1.75	1.35
Prestressed	Strength II	1.25	1.50	-	-
Prestressed	Service III	1.00	1.00	0.80	-
Prestressed	Service I	1.00	1.00	-	-

The design load rating requires reinforced concrete be checked under Strength I load combinations and prestressed concrete be checked under Strength I and Service III load combinations. Legal load rating requires both reinforced concrete and prestressed concrete bridges be checked under the Strength I load combination; satisfying the concrete tensile stress limits under the Service III loads is considered optional for the legal load rating of prestressed concrete bridges. Lastly, the Strength II load combination is used for permit load rating; it is considered optional to check the reinforcing steel stresses under the Service I load combination.

Limit states used for the rating of segmental bridges differ from those used for other bridges. Design load ratings require Strength I, Service I, and Service III limit states be checked. Legal and permit load ratings require both Service I and Service III be checked in addition to the requirements listed in the previous paragraph. For all but the inventory load rating, the number of live load lanes may be taken as the number of striped lanes. Segmental concrete bridges are

the only concrete bridge type for which concrete tensile stresses are utilized for operating and permit load ratings.

Average Daily Truck Traffic

The live load factors for the legal load rating are dependent on the ADTT. Note that the live load factors given by the manual are intended for AASHTO legal loads. The factors may also be used for State legal loads that are similar in weight to the AASHTO legal loads. Table 24 lists the live load factors to be used for legal load rating when applying the Strength I limit state.

Table 24. 2003 Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges Live Load Factors for Routine Commercial Traffic

Traffic Volume (One direction)	Load Factor for Type 3, Type 3S2, Type 3-3 and Lane Loads
Unknown	1.80
ADTT ≥ 5000	1.80
ADTT = 1000	1.65
ADTT ≤ 100	1.40

Note: ADTT = Average Daily Truck Traffic.

Live load factors used for routine commercial traffic are reduced in the 2013 interim revisions to the MBE (2nd Edition) as a result of the NCHRP 12-78 project.[3197] A load factor of 1.45 is used for values of ADTT greater than 5,000 and in cases where the ADTT is unknown. When ADTT values are less than 1,000, a load factor of 1.30 is used. If the ADTT lies between 5,000 and 1,000, then linear interpolation is permitted.

The manual provides generalized live load factors for special hauling vehicles to be used for legal load rating. Table 25 lists the live load factors for the Notional Rating Load and single units (SUs) 4 through 7. The numbers refer to the number of axles on the hauling vehicle.

Table 25. 2003 Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges Live Load Factors for Specialized Hauling Vehicles

Traffic Volume (One direction)	Load Factor for Notional Rating Load, SU4, SU5, SU6, and SU7
Unknown	1.60
ADTT ≥ 5000	1.60
ADTT = 1000	1.40
ADTT ≤ 100	1.15

Note: ADTT = Average Daily Truck Traffic; SU = single unit.

Similarly, the findings in the NCHRP 12-78 project resulted in lower live load factors used for specialized hauling vehicles with the release of the 2013 interim revisions to the MBE (2nd Edition).[31] The updated factors are identical to those used for routine commercial traffic.

The permit load factors depend on multiple bridge and load conditions such as the permit type, vehicle crossing frequency, loading condition (i.e., with traffic), ADTT, and the permit weight. The 2008 MBE (1st Edition) live load factors ranged from 1.10 to 1.85.[30] Permit load factors

were reduced to a maximum load factor of 1.4 in the 2013 interim revisions to the MBE (2nd Edition). [31] Permit vehicles requiring multiple trips, mixed with traffic, with an ADTT greater than 5000 will have the highest live load factor.

The condition factor is used to account for the uncertainty in the resistance of deteriorated members and future deterioration and section losses of the member. The values for the condition factor are allowed to be increased by 0.05 if the section properties of deteriorated members are obtained accurately; however, the condition factor is not allowed to exceed 1.00. The condition factor is not meant to be used for damage caused by accidents. Table 26 shows the condition factors for deteriorated members. Condition factors shown in Table 26 are current with the 2018 MBE. [32]

Table 26. 2003 Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges Condition Factor

Structural Condition of Member	φ_c
Good or Satisfactory	1.00
Fair	0.95
Poor	0.85

The system factor accounts for the level of redundancy of the superstructure system. A bridge with a higher level of redundancy is likely to safely find a different load path when a critical member is damaged. Bridges with lower levels of redundancy will have lower load ratings. The manual instructs the engineer to use a system factor equal to 1.0 when checking shear at the strength limit state.

The 2018 MBE instructs the engineer to refer to the *LRFD Bridge Design Specifications* for the shear and torsion capacity of post-tensioned segmental bridges and closed box sections. [32]

Bridge Load Rating Software

AASHTOWare Bridge Rating™

AASHTOWare Bridge Rating™ (BrR), formerly known as Virtis™, is a software tool developed by AASHTO beginning in 1995 as a successor to the Bridge Analysis and Rating Systems. BrR uses an Oracle® or Microsoft SQL Server™ database that can be shared with two other AASHTO tools: Bridge Design™ (BrD) and Bridge Management™ (BrM).

BrR evaluates inventory, operating, permit, and legal level load ratings for superstructures in accordance with the AASHTO specifications as listed in Table 27.

Table 27. AASHTOWare Bridge Rating Specification

LRFD Bridge Design Specifications	Standard Specifications for Highway Bridges	Manual for Bridge Evaluation	Manual for Condition Evaluation of Bridges
4th through 8th editions	16th and 17th editions	1st through 3rd editions	1st and 2nd editions

BrR is capable of rating superstructure components individually as listed in Table 28.

Table 28. AASHTOWare Bridge Rating Rated Member Types

Description	ASR	LFR	LRFR
Prestressed I Girders		X	X
Prestressed Box Girders		X	X
Prestressed Tee Girders		X	X
Prestressed U Girders		X	X
Reinforced Concrete I Girders	X	X	X
Reinforced Concrete Tee Girders	X	X	X
Reinforced Concrete Slab	X	X	X
Post-Tensioned Multi-Cell Box		X	X
Reinforced Concrete Multi-Cell Box		X	X
Reinforced Concrete Box Culverts		X	X

Superstructures

Multiple superstructure types may be defined for a single bridge and independently load rated. Multiple superstructures may be used to represent each phase of a phased construction sequence, future widenings, proposed replacements, or other historical versions of the same bridge. This feature is useful to keep a record of load rating for a bridge whose configuration may be modified over its service life. Once a superstructure is defined, BrR generates a section view schematic for verification of input as shown in Figure 2.
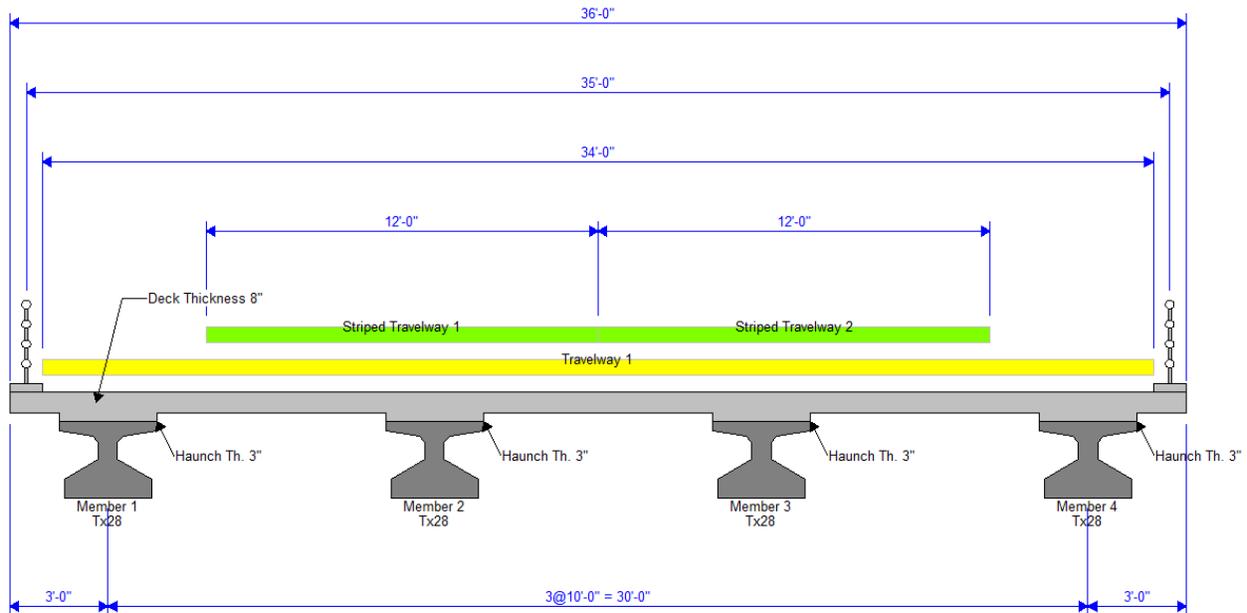
Figure 2. AASHTOWare Bridge Rating Generated Superstructure Section View[44]

BrR calculates dead load (DC, DW), vehicular load (LL, IM, CE), and pedestrian load (PL) (see Table 7 for more detailed definitions of notations). Additional concentrated loads, distributed loads or girder settlements may be input by the user for each applicable load case. Non-composite, composite (short-term), or composite (long-term) stages are assigned to all loads.

Live load distribution factors are typically input manually by the user. Custom vehicles for distribution factor analysis may be of non-standard gauge with any number of axles. When load rating for permit vehicles, bridges on an entire route may be rated for a custom permit vehicle in a batch run.

BrR includes an option to calculate distribution factors automatically using a three-dimensional (3D) finite element model. Reinforced and prestressed concrete sections are represented in BrR's finite element model by beam elements located at the centroid of the beam as shown in Figure 3. The shell elements which model the concrete deck are rigidly linked to beam elements of the girder.
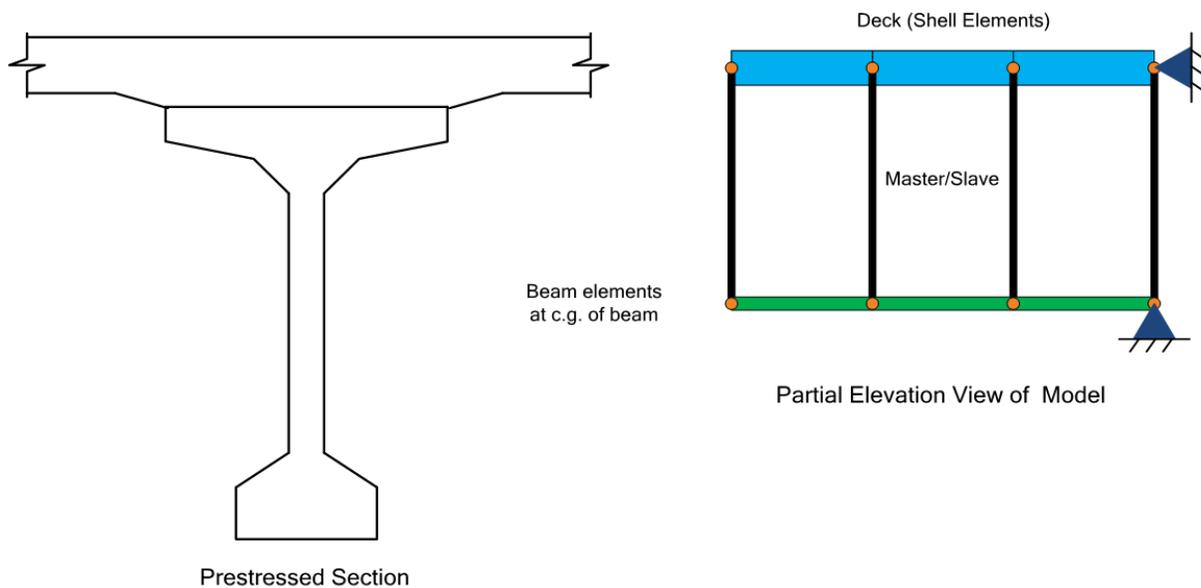

Figure 3. AASHTOWare Bridge Rating Optional Finite Element Model of Prestressed Section[44]

Prestress Losses

BrR calculates prestress losses at the midpoint of each span, and post-tension losses at every analysis location. Either gross or transformed sections may be considered in the calculation of loss. Elastic gains and losses due to dead load application may be included with user input. Table 29 summarizes the methods used for calculating loss for each analysis engine.

Table 29. AASHTOWare Bridge Rating Stress Loss Calculation Methods

Calculation Method	Prestress Loss ASD/LFD	Prestress Loss LRFR/LRFD	Post-Tension Loss ASD/LFD	Post-Tension Loss LRFR/LRFD
AASHTO Approximate	X	X		
AASHTO Refined	X	X	X	X
Lump Sum Loss	X	X	X	X
Pre-2005 AASHTO Refined		X		

BrR makes a few assumptions relating to prestressed and post-tensioned concrete girder analysis. Compression reinforcement is always considered in specification checking, all external axial loads are neglected, and all hooks are assumed to meet AASHTO detailing requirements.

The user may select from four methods for determining shear resistance parameters β and θ:

- The *LRFD Bridge Design Specifications* general procedure

- The *LRFD Bridge Design Specifications* general procedure with the provisions of Appendix B

- The simplified procedure assuming β is 2 and θ is 45 degrees

- The simplified procedure assuming β is 2 and θ calculated using Equation 82 from 2012 *LRFD Bridge Design Specifications* (6th Edition) 5.8.3.4.3-4 when V_{ci} is greater than V_{cw}.[(26)]

$$\cot \theta = 1.0 + 3 \left(\frac{f_{pc}}{\sqrt{f'c}} \right) \le 1.8$$

(Equation 82)

where:

f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when centroid lies within the flange (ksi)

Equations 83 through 85, used by BrR for calculating net longitudinal strain at mid-depth of the member ε_x under the general procedure with provisions of Appendix B, neglect any strands located on the flexural compression side of the member.

$$\varepsilon_x = \frac{\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})} \le 0.001$$

(Equation 83)

$$\varepsilon_x = \frac{\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}}{E_s A_s + E_p A_{ps}} \leq 0.002$$

(Equation 84)

$$\varepsilon_x = \frac{\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}}{2(E_c A_c + E_s A_s + E_p A_{ps})}$$

(Equation 85)

Note that A_{ps} is the area of prestressing steel on the flexural tension side of the member.

The commentary in the *LRFD Bridge Design Specifications* recommends Equation 86 be used for prestressed sections where compression side strands exist. This procedure is not currently available in BrR.

$$\varepsilon_x = \frac{\varepsilon_t + \varepsilon_c}{2}$$

(Equation 86)

where:

ε_t =   extreme tensile reinforcement strain

ε_c =   compressive strain at the center of compression

Tensile strain should be given as positive and compressive strain should be given as negative.

Shear rating in BrR is calculated using the maximum moment and maximum shear for each user selected limit state and vehicle at the section in consideration. BrR does not calculate the moment that is concurrent with the maximum shear. BrR does not iteratively adjust vehicle weight in calculation of forces for shear rating.

Detailed flowcharts of analysis conducted by BrR are included in method of solution portable document format (PDF) for each analysis engine. These flowcharts include direct references to AASHTO specification formulas with changes between editions noted. Analysis results are sorted by specification reference and are available as a formatted list as shown in Figure 4 or as text files with values given for intermediate calculations.

Specification Reference	Limit State	Flex. Sense	Pass/Fail
✓ 5.4.2.1 Compressive Strength	N/A	Passed
📄 5.4.2.5 Poisson's Ratio	N/A	General Comp.
📄 5.4.2.6 Modulus of Rupture	N/A	General Comp.
📄 5.4.2.8 Concrete Density Modification Factor	N/A	General Comp.
NA 5.5.3.2 Reinforcing Bars and Welded Wire Reinforcement	N/A	Not Required
📄 5.5.4.2 PS Strength Limit State - Resistance Factors	N/A	General Comp.
📄 5.6.2.2 Rectangular Stress Distribution	N/A	General Comp.
✓ 5.6.3.2 Flexural Resistance (Prestressed Concrete)	N/A	Passed
✓ 5.6.3.3 Minimum Reinforcement	N/A	Passed

Figure 4. AASHTOWare Bridge Rating Example Specification Check Report[44]

Moment, shear force, axial force, and deflection graphs are generated for each girder, load source, and stage analyzed. These graphs are generated at girder tenth points by default; however, the user may define additional points of interest along the girder length. All values displayed in the graphs are also provided in spreadsheet format for export.

Rated capacities are output in tons for each rating level and vehicle analyzed. Rating reports may be summarized by bridge, span, or individual member. Rating reports for individual members list the controlling analysis location.

BRASS-GIRDER™

BRASS-GIRDER™ is one of the individual programs that forms the Bridge Rating and Analysis of Structural Systems (BRASS) Suite. It has been developed by the Wyoming Department of Transportation and BridgeTech, Inc. since 1987. The program is intended to assist engineers in design and rating of bridge girders. Both LFD and LRFD procedures may be performed. The provisions of the *Standard Specifications,* the *LRFD Bridge Design Specifications*, and the *MBE* are used for these calculations, although some additional methods for certain calculations may be used at the user's discretion. The most recent editions of the AASHTO documents are referenced, which are the 2002 *Standard Specifications* (17th Edition),[20] 2017 *LRFD Bridge Design Specifications* (8th Edition),[28] and 2018 *MBE* (3rd Edition).[32] The equations outlined in these documents are used alongside sectional analysis and a finite element model to perform analyses, designs, and ratings of structures.

BRASS-GIRDER™ allows the user to model steel, reinforced concrete, prestressed concrete, or timber members. An entire girder system, a single girder line, or a floorbeam line can be defined. Vertical or inclined piers may be optionally included. Figure 5 shows a possible frame structure. All members must be of the same material. Some material types do not allow for certain choices, for example, use of prestressed concrete girders does not allow the user to model piers. Supports may be modeled as fixed points or elastic springs with settlements. Hinges may be implemented in the model if the structure remains stable after their inclusion.
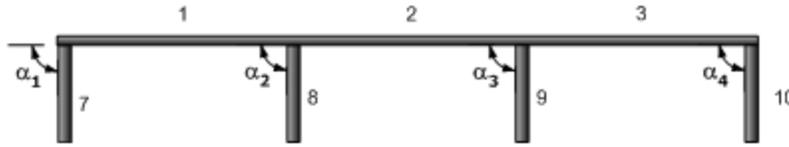
Figure 5. Screenshot of a Possible Frame Configuration in BRASS-GIRDER™ (45)

BRASS-GIRDER™ creates a finite element model for the structure based on user input. Nodes are located at tenth points along spans, cross-section change points and other locations defined by the user. The finite element model is analyzed using a direct stiffness solver. The solver creates a stiffness matrix for the modeled structure in one or two dimensions. Only one member line is analyzed at a time; 3D modeling is not included. The solver will produce actions such as moment, shear and axial force at the ends of elements and displacements for each node.

Loading

BRASS-GIRDER™ will automatically calculate the self-weight of the modeled structure (DC, DW). The user may specify additional dead loads (including DU), vehicular and pedestrian live loads (LL, IM, PL) and settlements (SE). Vehicular live loads may be selected from a library of typical design, legal, and permit loads included with the default installation. The user may also define their own live loads by axle weights and spacings. If the modeled structure is prestressed or post-tensioned concrete, prestressing or post-tensioning loads (PS) will be applied. The user may choose to compute prestress losses in accordance with article 5.9.3 of the *LRFD Bridge Design Specifications* or in accordance with the PCI General Method. Anchorage losses are computed only if post-tensioned girders are used. Friction losses are computed if straight and parabolic post-tensioned strand profiles are used. Once all losses are computed, BRASS-GIRDER™ will perform load-balancing, which applies all prestress loads to the entire structural analysis model as opposed to simply applying them in cross-sectional analyses. The stress in the prestressing strand is converted to internal loads on the structure that result in equilibrium with the external forces.

Analysis at Points of Interest

The elastic properties of the section are determined at each user-defined point of interest, such as the neutral axis, moment of inertia and section moduli for the outer fiber of each flange. If different concrete strengths are input for the girder, girder top flange or slab, a modular ratio is used to calculate a modified width for section properties. BRASS-GIRDER™ uses an iterative method to determine the neutral axis during flexural resistance calculations. A trial depth is selected by the program, with which the internal axial force is computed. If the calculated internal axial force is not equal to the applied axial force, a new neutral axis depth is selected, and the process is repeated. The flexural resistance is used to compute the effective shear depth with Equation 87. The result of Equation 87 has a lower bound corresponding to the greater of $0.9d_e$ and $0.72h$.

$$d_v = \frac{M_n}{A_s f_s + A_{ps} f_{ps}}$$

(Equation 87)

where:

M_n = nominal moment resistance

f_{ps} = tensile stress in prestressing strands

Maximum and minimum values for each action at the point of interest are calculated along with concurrent actions. These calculations are dependent on the location of live loads; the critical locations of live load are determined with influence lines. Analyses are performed for all combinations of limit state, construction stage, and design vehicle (when live loads are applicable for the construction stage).

An exception to the location of shear computation may be input by the user. A shear distance may be specified on an individual basis for points of interest located at supports, which will result in BRASS-GIRDER™ calculating the shear for the point of interest at the shear distance away from the support. This shear value is used with shear resistances computed at the true location of the point of interest when computing rating factors or design ratios.

Shear Resistance Calculations for LRFR Rating Analyses

BRASS-GIRDER™ calculates shear resistance for reinforced concrete and prestressed concrete at every point of interest identified by the user. Several methods are available. The user may choose to use the equations of 2017 *LRFD Bridge Design Specifications* (8th Edition) section 5.7.3.3 – Nominal Shear Resistance (Equations 42-45), or the Simplified (V_{ci}, V_{cw}) Procedure (Equations 51-53) found in the 2014 *LRFD Bridge Design Specifications* (7th Edition).[27][28] If the equations of section 5.7.3.3 are used, the user may select one of several methods for the calculation of β and θ:

- The simplified procedure, where $\beta = 2$ and $\theta = 45°$.
- The general procedure in section 5.7.3.4.2.
- The general procedure in Appendix B5.

Similar to AASHTOWare Bride Rating™, BRASS-GIRDER™ does not use Equation 85 when prestressing strands are placed in the compressive region. Should the user select the Simplified (V_{ci}, V_{cw}) Procedure, they may limit the M_{cr}/M_{max} ratio in the calculation of V_{ci} to 1.0. When the selected equations use effects such as M_u or V_u as input, BRASS-GIRDER™ will use the maximum value at the point of interest. These equations are calculated for positive and negative flexural sense. Concurrent actions are not used for some calculations, such as in Equations 46-49. Longitudinal reinforcement checks per AASHTO LRFD section 5.7.3.5 are the exceptions; these checks are conducted with maximum and minimum moment with concurrent shears and with maximum and minimum shear with concurrent moments.

The user may select different methods for shear resistance calculation for separate regions of a girder. For example, the user may select the general procedure in Appendix B5 for the first 10' of the girder, then use the Simplified (V_{ci}, V_{cw}) Procedure for the rest. Regions may not overlap. Shear resistance computations may be modified in a few ways beyond the method of

computation. A percentage of the cross-section used to resist shear may be specified to model a cracked section; this percentage applied to the calculated value of V_c. The use may also opt to ignore axial force.

Load Rating

BRASS-GIRDER™ calculates rating factors with Equation 88:

$$Rating\ Factor = \frac{A - B}{C}$$

<div align="right">(Equation 88)</div>

where:

A = Total factor load resistance of the structure

B = Factored dead load effect

C = Factored live load effect

Effects due to loads which are not categorized as dead or live loads by AASHTO publications, such as prestress loads, are grouped with dead load effects. Rating factors may be calculated for several code checks such as shear, shear friction, and longitudinal reinforcement.

Rating factors are calculated at every point of interest for every combination of limit state, live load and construction stage that the user has selected (except for stages prior to casting of the deck, where live loads are not applied). The rating factors are then summarized for every analysis location, with the critical rating factor for each code check provided. Load ratings in tons are given for the critical rating factor for each design vehicle. Load rating values are calculated by multiplying the critical rating factor by the total weight of the design vehicle.

2.3.   Summary – Evolution of Shear Design Requirements

The following timeline provides a summary of the significant changes to concrete bridge shear design as governed by AASHO and then AASHTO from 1931 to present.

1931

> Traffic lane width, standard truck loads, equivalent lane loads, and the impact factor were defined in the AASHO Standard Specifications (1st Edition);

> Upper shear unit stress limits implemented in AASHO Standard Specifications (1st Edition);

1935

> Reinforcing steel and bond unit stress limits were added to the AASHO Standard Specifications (2nd Edition);

1941

> HS loading and reduction factors for multiple lanes loaded were introduced in the AASHO Standard Specifications (3rd Edition);

> Shearing unit stress formula and required shear reinforcement formula introduced in AASHO Standard Specifications (3rd Edition);

Field inspection requirements introduced in the AASHO Standard Specifications (3rd Edition);

AASHO Standard Specifications (3rd Edition) required inventory ratings to use live loads used for the design of new bridges;

1944

Allowable concrete and bond unit stresses were modified to be a function of the concrete ultimate strength in the AASHO Standard Specifications (4th Edition);

1961

Prestressed concrete design provisions were introduced using LFD methods in the AASHO Standard Specifications (8th Edition);

1965

Nine service loading combinations were introduced in the AASHO Standard Specifications (9th Edition);

Bending moment LLDF equations were allowed to be used for shear LLDF calculations in the AASHO Standard Specifications (9th Edition);

The required shear reinforcement formula was adjusted to optimize reinforcement capacity at an angle of inclination of 45 degrees in the AASHO Standard Specifications (9th Edition);

Prestressed concrete LFD shear strength defined by AASHO Standard Specifications (9th Edition);

1970

Manual for Maintenance Inspection of Bridges was released;

Manual for Maintenance Inspection of Bridges (1st Edition) field inspection requirements included frequency, qualified inspector requirements and inspection item list;

1971

Minimum shear reinforcement area was modified to accommodate high strength reinforcement in the interim revisions to AASHO Standard Specifications (10th Edition);

1972

Load Factor Design was introduced to non-prestressed concrete with new set of load combinations in the interim revisions to the AASHO Standard Specifications (10th Edition);

1974

Bridge inspector requirements became more stringent with the release of the Manual for Maintenance inspection of Bridges (2nd Edition);

1977

A common design equation for loading combinations was introduced to be used for both LFD and Service Load Design in the AASHTO Standard Specifications (12th Edition);

Fundamental LFD shear design equation was introduced in the AASHTO Standard Specifications (12th Edition);

1978

Manual for Maintenance Inspection of Bridges (3rd Edition) introduced the Load Factor Method for rating bridges along with the rating equation;

1979

Refined equations were introduced in the interims to the AASHTO Standard Specifications (12th Edition) to more accurately calculate shear stress in concrete members;

1980

The interims to the AASHTO Standard Specifications (12th Edition) introduced an LFD shear design equation for prestressed concrete;

1985

The interims to the Manual for Maintenance Inspection of Bridges added a fatigue strength and concrete crack control rating factor equation;

1989

Shear provisions for segmental bridges were provided by the Guide Specifications for Design and Construction of Segmental Bridges (1st Edition);

Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges updated rating factor equation by making the live load factor a function of ADTT;

1994

LRFD Bridge Design Specifications (1st Edition) introduced limit states, multiple presence factors, dynamic load allowance factors, live load distribution factor equations, and a refined sectional design model based on modified compression field theory. In addition, strut and tie methods were introduced to concrete bridge design specifications;

Manual for Condition Evaluation of Bridges was released with ASD and LFR methods for bridge rating. The manual requires inspection frequency be based on type of inspection conducted;

1998

The interims to Manual for Condition Evaluation of provided an updated concrete shear strength equation;

2003

Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges recognizes three load-rating procedures (design, legal, and permit);

2005

Interim revisions to the LRFD Bridge Specifications (3rd Edition) adopted segmental concrete bridge shear provisions;

2007

LRFD Bridge Specifications (4th Edition) provided simplified shear calculation procedure for prestressed and non-prestressed sections;

2008

Interim revisions to the LRFD Bridge Design Specifications (4th Edition) revised the sectional design model to provide a non-iterative method for the evaluation of the β and θ factors;

2016

Interim revisions to the LRFD Bridge Design Specifications (7th Edition) modified the strut and tie procedure in which compressive struts are designed, including factors for confinement and crack control;

2018

The Manual for Bridge Evaluation (3rd Edition) was released and is the most current bridge rating specification.

CHAPTER 3.   SURVEY OF STATE DOTS

3.1.   Introduction

To better understand current concrete shear load rating practices, a survey was sent to nine State DOTs. The selection of survey recipients was based on the number of concrete bridges and the diversity of bridge type in each State's inventory. States that had funded research in concrete bridge shear and load rating were also prioritized. All State DOTs completed the survey for a 100 percent return rate.
The nine States surveyed included the following:

- California
- Florida
- Illinois
- Minnesota
- New York
- Oregon
- Pennsylvania
- Tennessee
- Texas

The survey had four topic areas: agency load rating policy and procedures; shear load rating for concrete bridges; software used for load rating; and research involvement of DOTs in concrete shear. A copy of the survey each State DOT received can be found in Appendix A.

3.2.   Survey Results Summary

3.2.1.   *Topic 1: Agency Load Rating Policy and Procedures*

Survey responses clearly indicate that LRFR and LFR are the most used load rating methods. Three States reported using assigned load ratings, with two States using Allowable Stress Rating (ASR). The specific shear strength methods used by States had more variability, with all current methods allowed by *LRFD Bridge Design Specifications* used. A clear majority of States used the general modified compression field theory (MCFT) method, while several States reported using STM; simplified MCFT; V_{ci}/V_{cw} method; and the segmental provisions. Only one-third of States surveyed used the V_{ci}/V_{cw} method from the Standard Specifications.

States were questioned if they had policies for shear above and beyond the LRFD, Standard Specifications, or MBE. Responses included the following:

- One State requires a minimum rating factor (RF) of 1.2 using LRFR for new concrete shear designs.
- One State listed cutoff locations of longitudinal reinforcement as critical sections for load rating shear.
- One State has slightly higher skew adjustment factors for LL distribution for shear.
- One State determines shear strength using Standard Specifications or the 1979 interim revisions to the Standard Specifications with LFR.

- One State requires new designs have a principal stress check in the webs of spliced precast girders (now adopted in 2017 *LRFD Bridge Design Specifications* [8th Edition]) and requires shear serviceability checks with select substructure.[28]
- One State requires shear design for slab span bridges.

The last question in this survey asked State DOTs about their requirements on design load rating for concrete shear as part of the bridge design process. Most States indicated they do require a concrete shear load rating, but for the superstructure only.

3.2.2.   *Topic 2: Shear Load Rating For Concrete Bridges*

A majority of the States indicated they evaluate shear for design loads, legal loads, and permit loads, with most indicating they always rate superstructures for shear, but only load rate substructure for shear when displaying shear distress.

When asked if shear load ratings controlled the overall bridge rating, a majority of States responded yes for design, legal, and permit ratings. Where this most commonly occurs is with bridges beginning in the 1950s, the interstate era, up to the adoption of LRFD. One-third of States responded that shear controlled load ratings for bridges built prior to 1950 and those bridges designed with LRFD. No State responded that load ratings governed by shear had decreased over time, with most States responding that they have increased. Reasons offered for the increase in shear-controlled load ratings include the following:

- Change in rating method.
- Change in truck size/configuration.
- Change in agency or federal policy.
- Change in shear strength equations.

When asked which types of superstructures in their inventories are most likely to have a low shear load rating (RF ≤ 1.0), a majority indicated both simple span and continuous for LL pretensioned girders (of any shape), which coincides with the interstate era. Another type frequently identified is reinforced concrete T girders, with either simple or continuous spans. When asked the same question, but in terms of substructure, a slim majority listed reinforced concrete pier or bent caps.

Only one State responded that they have bridge elements common to their inventory that routinely display shear distress. The reason for this distress was attributed to past inadequate design specifications.

In terms of shear strength equations, there was no clear method identified as responsible for lower than expected shear load ratings, which could be attributable to the variability in the shear strength equations. When confronted with a low shear rating, most states re-evaluated the rating using another strength equation or specification and when doing so, found a significant difference in the rating. When confronted with two different shear strengths based on equations in the *LRFD Bridge Design Specifications*, most States responded that they used the highest calculated strength, while only one State responded that they used the lowest calculated strength.

The approximate live load distribution factors for shear, including the lever rule, and skew correction factors were identified as being the source of low shear load ratings by five of the responding States; four States responded that these factors were not responsible for low shear load ratings. It is clear that using the approximate live load distribution factors can be problematic in obtaining favorable shear rating factors.

When asked if a shear load rating clearly contrasted with a bridge's observed condition, a majority of States responded with yes, most responding that the bridge had a low rating but had no shear distress. One-third of the responses indicated favorable load rating, but the bridge was exhibiting shear distress. Without knowing the specifics of each case where a low shear load rating contrasts with a good structural condition, it is hard to draw conclusions on the reasons why this occurs.

Only one-third of respondents indicated they always verify the adequacy of the reinforcing details to develop their calculated strength, with four States responding that inadequate detailing has been the source of low shear load ratings.

3.2.3.   Topic 3: Load Rating Software

The first question in this section was about load rating software used by the respondents. Most respondents indicated they use in-house rating software tools and an equal amount identified BrR as their primary load rating software. Most States indicated they also use general purpose structural analysis/design software, such as SAP2000™, RISA-3D™, STAAD.Pro™, or midas Civil™, to assist in load rating for concrete bridges.

Most States responded that their software rating tools routinely indicated deficient load ratings at common span locations, such as quarter points or contraflexure points. This could indicate needed refinement in the software used or it could be a result of changes in shear design requirements over the years for these specific span locations. Most States indicated they further investigate low shear load ratings with other software or other tools.

When asked if they used the software Response-2000™ as a load rating tool, only three States responded yes and then only for unusual situations or for further investigation with a low load rating from another method or tool. One State reported not using Response-2000™ because they did not have access to this software, which is noteworthy because this software is available for free.

3.2.4.   Topic 4: Research

Five States responded that they have funded research or performed studies on concrete shear evaluation, including load distribution for shear.

Surprisingly, only one State responded that it was aware of shear-related research funded by a sister agency. Not being aware of current research in shear, especially when confronted with shear issues, is very noteworthy. The survey did not include questions on why a State might not know of relevant shear research/studies by other States.

CHAPTER 4.   LITERATURE REVIEW

4.1.   Concrete Shear Behavior and Design

4.1.1.   *History of US Shear Design Methods*

Ramirez and Breen [46] provided a comprehensive review of the development of design recommendations for shear in the United States. The review showed AASHTO and ACI methods were similar in their approach to calculating shear capacity. Shear methods were said to be difficult to use only in unusual or unfamiliar design cases. The authors noted that the shear computation methods of the time tended to be empirical and did not have a rational basis as flexural and axial methods. The authors examined a space truss model with a variable inclination of diagonals as a rational approach to shear design. This model provided a useful and understandable basis for the interaction of moment, shear, and torsion.

4.1.2.   *Cost Comparison between Shear Design Methods*

Ma et al. [47] provided a review of computation methods for concrete contribution to shear strength (V_c) in "Shear Design of Stemmed Bridge Members: How Complex Should It Be?" The research focused on the level of complexity in the computational method and its effect on the eventual cost of prestressed concrete beams. The factor for cost comparison was the amount of steel required to meet the nominal shear strength (V_s). The computational methods for shear strength of concrete considered in this research were as specified in the Standard Specifications, 1994 *LRFD Bridge Design Specifications,*[21] 1979 interim revisions to the Standard Specifications, and 1989 *Guide Specifications for Design and Construction of Segmental Concrete Bridges.*[34] The ease of computing V_c is discussed and the LRFD method is shown to be most complicated due to the iterative nature of the calculation. It is to be noted that the more recent versions of *LRFD Bridge Design Specifications* provide a simplified non-iterative design procedure for calculating concrete shear strength. The methods are compared with two example cases, a single span adjacent box beam bridge and a two-span I-girder bridge. A typical interior beam is selected in both cases and the shear capacity is computed using the methods prescribed in the specifications previously mentioned. The conclusion from this research was that even with a significant variance in the cost of steel reinforcement when compared with the steel requirements from the Standard Specifications method, the effect of steel cost on the overall cost was minimal. While the *LRFD Bridge Design Specifications* method produced a 61 percent increase in cost of steel, the overall impact on the cost of the beam was 1 percent. The use of 1979 interim revisions to the Standard Specifications or another simplified computation method was recommended in this research based on costs.

4.1.3. Simplified Shear Design

Hawkins et al. [48] researched simplified methods for computing shear capacity published in *Simplified Shear Design of Structural Concrete Members*. The computation methods that were reviewed are as follows:

- ACI 318R-02
- AASHTO Standard Specifications for Highway Bridges 16th Edition (1996)[19]
- AASHTO 1979 provisions
- Canadian Standards Association: Design of Concrete Structures, 1994 (CSA A23.3-94)
- AASHTO *LRFD Bridge Design Specifications* 2nd Edition with 2003 interim revisions
- CSA A23.3-04
- Eurocode EC2
- German Code (DIN 2001)
- AASHTO *Guide Specifications for Design and Construction of Segmental Concrete Bridges*
- The Japanese Code (JSCE Standards 1986)
- Shear design procedure developed by Tureyen and Frosch [49]

The accuracy of these methods was investigated using a database of experimental results. Additionally, a survey of State DOTs and Federal Lands bridge design agencies was conducted on the use of Standard Specifications shear method and the *LRFD Bridge Design Specifications* sectional design model. A significant disagreement was observed within these methods about the relative magnitude of concrete and steel contributions to shear resistance, the factors influencing these contributions, and their significance for different design cases. The diagonal cracking strength is not an adequate measure for concrete contribution to ultimate strength for members with shear reinforcement. The relationship between V_c and diagonal cracking strength for members with shear reinforcement is purely empirical and a comprehensive database is required for validation of this relationship. The rules for calculating the angle of the diagonal compression must be accounted for in calculation of V_c for members with shear reinforcement. Because research tests used to validate these shear methods do not necessarily reflect as-built conditions, there remains some uncertainty about these shear methods in terms of actual performance metrics. A particular example of this disconnect is stated as that of the location of point of contraflexure in a continuous beam. The researchers noted a wide spread in shear requirements for this region of a continuous beam for different provisions. The overall safety of a code is difficult to assess from observed field conditions due to redundant load paths, additional load resisting elements not considered in analysis and design, and the low probability of the member being subjected to ultimate load. In terms of methods available, there was a wide variance in concrete strength predictions. For the same section and forces, the amount of steel required by one code varied by two to three times over another. The maximum allowable magnitude of shear stress has a large variation between different methods. The factor of this difference was found to be twice for Standard Specifications and one-half for *LRFD Bridge Design Specifications* Sectional Design Model.

4.1.4.   Simplified Method for MCFT

Bentz et al. [50] presented a simplified method for the MCFT approach in "Simplified Modified Compression Field Theory for Calculating Shear Strength of Reinforced Concrete Elements." The method proposed in this research was to simplify the MCFT method by introducing simple calculations for the factor for tensile stresses in cracked concrete (β) and the inclination of diagonal compressive stresses in the web (θ) thereby precluding the need for iterating these values. The comparison of the ACI method, MCFT, and the proposed simplified MCFT approach was presented in this research. The effectiveness of these methods was compared using a database of over 100 tests on reinforced concrete elements subjected to pure shear. The ratio of the experimental-over-predicted shear strength was 1.40 for ACI with a variance of 46.7 percent, 1.01 for MCFT with a variance of 12.2 percent, and 1.11 with a variance of 13.0 percent for simplified MCFT. The simplified approach was eventually adopted in *LRFD Bridge Design Specifications*.

4.1.5.   Shear with High Strength Concrete

Hawkins and Kuchma [51] researched the application of *LRFD Bridge Design Specifications* Sectional Design Model to high strength structural concrete (greater than 10 ksi) in *Application of LRFD Bridge Design Specifications to High-Strength Structural Concrete: Shear Provisions*. Five potential issues were noted to extend the usage of the LRFD method to high strength concrete (HSC). In LRFD, a part of the concrete contribution depends on the assumption of 12-inch spacing of diagonal cracks. The potential concern with using the LRFD method for HSC is that diagonal cracks can be spaced greater than this assumption, leading to a reduction in concrete contribution to shear strength. In terms of the steel contribution, the LRFD method produces three times more capacity than the Standard Specifications method because the angle of the compression diagonal can be as low as 18.1 degrees compared to 45 degrees for the Standard Specifications. The accuracy of calculating θ becomes more critical for HSC because the LRFD method allows higher shear forces requiring a greater contribution from steel stirrups. The LRFD requirements for minimum shear reinforcement are evaluated for use in HSC. The maximum shear stress limit of LRFD is evaluated for HSC. In general, the assumptions made for the *LRFD Bridge Design Specifications* Sectional Design Model are evaluated for use in HSC. To study these factors, a database of shear tests was assembled and a prestressed beam type and span were selected on the basis of most economical application of HSC beams. Beam specimens were created and tested in a laboratory. The following conclusions were made from this research. The limit on concrete compressive strength in 1996 *LRFD Bridge Design Specifications* and its interim revisions through 2006 can be raised to 18 ksi for normal weight concrete. The *LRFD Bridge Design Specifications* Sectional Design Model was reasonably accurate and conservative with a compressive strength limit of 18 ksi, except in cases where the shear stress on concrete was greater than $0.18f'_c$, or when a staggered shear design concept was used away from the end regions. The shear design provisions in the Standard Specifications, 2004 Canadian Standards Association specifications, and the recommended simplified shear specifications in *Simplified Shear Design of Structural Concrete Members* NCHRP Report 549 [48] provided similar conservative estimates of shear capacity of test beams as subjected to the same limits prescribed above. Designing members for shear stress in excess of $0.15f'_c$ could lead to shear cracking and localized steel yielding under service level loads. The *LRFD Bridge Design Specifications* allow

for the use of the sectional design model at the beam ends, but the use of strut and tie approach is preferred.

4.1.6.   Deep Beam Shear Strength and STM

Birrcher et al. [52] conducted a study on STM for predicting strength of shear in deep beams published in *Strength and Serviceability Design of Reinforced Concrete Deep Beams*. An experimental study was performed on 37 deep beam specimens. The results from the experimental program and test results of 179 deep beam tests available in the literature were used for the development of strength, serviceability, and performance requirements of deep beams. A new STM approach was proposed in this study. Eight distinct tasks were developed to study their influence on strength and serviceability of deep beams. These tasks and respective findings were as follows:

- Influence of stirrup distribution along beam width: Distribution of shear reinforcement horizontally reduced crack widths of beams with 0.2 percent web reinforcement. There was no significant effect on beam with 0.3 percent web reinforcement.
- Influence of triaxially confined nodes: The triaxial confinement of load and support plates (CCC and CCT nodes) was investigated with 0.2 percent and 0.3 percent web reinforcement in each direction. It was found that specimens with 0.2 percent web reinforcement were more sensitive to bearing plate configuration with wider and erratic cracks.
- Influence of web reinforcement: This task was devised to recommend an appropriate amount of web reinforcement (stirrups and side face) to ensure adequate serviceability performance of deep beams. It was found that the specimens with shear span-to-depth (a/d) ratio less than 2.0 (with 0.2 percent or 0.3 percent web reinforcement) failed in a manner consistent with a direct strut mechanism assumed in STM. 0.3% web reinforcement provided better crack control than 0.2% crack control reinforcement. Hence, the provision of reinforcement greater than that required for maintaining equilibrium at the joint has no influence on the capacity. For beams with a/d ratio greater than 2.5, the behavior was consistent with the behavior assumed in the sectional model. Hence, addition of reinforcement increased the shear capacity of the beams.
- Influence of member depth: This task was used to study the effect of depth on strength and serviceability of deep beams. For deep beams with a/d less than 2.0, the stress conditions in the node determined the capacity instead of the beam depth, assuming that the bottle-shaped strut was adequately reinforced and the tension tie was not the governing aspect of the node. In terms of serviceability, it was found that the crack widths were dependent on the depth for specimens with depths less than 42 inches. For specimens with depths between 42 inches and 75 inches, the effect of depth on diagonal crack widths was mitigated.
- Proposal of simple STM strength design methodology: A new STM method was proposed on the basis of *fib* (1999)[35] while remaining consistent with the provisions of ACI 318-08 and 2008 *LRFD Bridge Design Specifications*. The truss geometry was required to be explicitly defined in the new method as the proposed efficiency factors are intrinsically linked to nodal geometry. The proposed method was derived from comprehensive stress checks that are a part of STM design. Stress checks on all faces of CCC, CCT nodes, and tension reinforcement were incorporated in the method. The

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splitting of the strut was indirectly included by providing a minimum amount of transverse reinforcement.

- Recommendation to reduce discrepancy between STM and sectional shear provisions for beams with shear a/d ratio of 2.0: The behavior of beams gradually varies from the deep beam behavior to sectional beam behavior as the a/d ratio approaches and exceeds 2.0. This gradual transition causes inaccuracy in results because the incorrect model may be applied for the prediction of shear strength for beams with a/d of 2.0. The proposed method accounted for a reduction in shear strength for beams with a/d of 2.0 and removed the inherent conservatism of the STM method prescribed in 2008 *LRFD Bridge Design Specifications*. Limiting the ratio of steel shear capacity to concrete shear capacity to 2.0 for sectional shear approach reduced the difference in capacity of the two design models. The use of a single-panel truss model was recommended for beams with a/d less than 2.0 based on the failure modes observed in the experimental study. The use of the new STM efficiency factors of this study reduced the significant discrepancy of calculated strength as the behavior transitioned from sectional shear to STM shear.
- Recommendation on the feasibility of limiting diagonal cracking under service loads: A minimum shear check was proposed along with a minimum amount of reinforcement to mitigate diagonal cracks. A simple equation was developed as a function of the shear area, the square root of concrete, and a/d ratio to predict diagonal cracking in deep beams.
- Recommendation on the methodology for relating the maximum diagonal crack width of a deep beam to its residual capacity: A simple chart was developed to aid field engineers in assessing the residual capacity of a member with diagonal cracks. The chart provided correlation between maximum crack width in a deep beam to the load (as a percentage of ultimate capacity) acting on the member.

This research by Birrcher et al. [52] in *Strength and Serviceability Design of Reinforced Concrete Deep Beams* forms the basis for the STM procedure introduced in the 2016 interim revisions to the *LRFD Bridge Design Specifications* (7th Edition).

4.1.7.   *Comparisons of Shear Design Methods*

Avendaño et al. [53] conducted an experimental program to study the shear behavior of a new family of Texas I-shaped girders in *Shear Strength and Behavior of Prestressed Concrete Beams*. The strength of the girders was predicted using ACI 318-08 Simplified method, ACI 318-08 Detailed method (V_{ci} and V_{cw}), 2007 *LRFD Bridge Design Specifications* General Procedure (MCFT),[24] 2007 *LRFD Bridge Design Specifications* Simplified Procedure (as detailed in NCHRP 549),[24] 1999 *Guide Specifications for Design and Construction of Segmental Concrete Bridges* (2nd Edition),[35] and Rational Shear Provisions for *LRFD Bridge Design Specifications*, and TxDOT Report 4759 [54]. Four tests were conducted on two Tx28 type beams. The loads were applied to a single end and the shear span was varied for both beams. The typical failure modes observed in testing were localized web crushing, horizontal shear failure (sliding shear) at the web to bottom flange interface, and in some cases, strand slip. The significant findings of this study were as follows. All methods previously mentioned provided conservative predictions of shear strength for the beam specimens. The most conservative results were obtained from the ACI 318-08 Simplified method. The shear expression (V_{cw}) of ACI 318-08 Detailed method provided the best performance in terms of predicting shear cracking. A further comparison of the prediction methods was performed on predicting the shear capacity of 506 test beams. On this basis, the following conclusions were made. ACI 318-08 (both methods) and AASHTO (both

LRFD and Segmental methods) provided conservative estimates of shear capacity for a wide range of span-to-depth ratios, concrete strengths, web reinforcement quantities, and overall member sizes. For shorter span-to-depth ratios (a/d < 2.0), ACI shear strength equations were more conservative than the *LRFD Bridge Design Specifications* methods. For specimens with a/d > 2.0, the AASHTO Simplified MCFT Procedure provided more conservative results than either ACI method. Another finding from this research was that the AASHTO methods over-predicted the shear strength on a significant number of beams leading the authors to recommend the reduction of resistance factor from 0.9 to 0.75 for shear design.

Nakamura et al. [55] compared eight shear computation methods using a database of 1,696 beams in *Shear Database for Prestressed Concrete Members*. The database comprised beam shear tests performed in North America, Japan, and Europe from 1954 to 2010. The researchers compared the following shear computation methods:

- ACI 318-11, Simplified Method.
- ACI 318-11, Detailed Method.
- AASHTO-LRFD 2010, General MCFT Procedure.[25]
- AASHTO-LRFD 2010, Simplified MCFT Procedure.[25]
- AASHTO-LRFD 2010, Segmental Procedure.[25]
- CSA, A23.3-04.
- JSCE 2007, empirical equation with a 45 degree truss model.
- *fib* MC 2010.

The MCFT based design equations produced the most accurate estimation of shear strength for prestressed members with sufficient shear reinforcement. The ACI 318-11 Detailed Method provided slightly less conservative estimations than the MCFT-based methods. The empirical methods were not as accurate as the MCFT-based calculations. It was noted that the MCFT-based methods provided unconservative estimations for specimens that exhibited signs of horizontal shear damage or anchorage zone distress prior to shear failure. It should be noted that MCFT was not developed to predict the behavior, or to estimate the strength, of specimens failing in atypical shear failure modes.

4.1.8.   *Shear Reinforcement Anchorage*

Mathys et al. [56] researched the anchorage of shear reinforcement with straight legs in *Anchorage of Shear Reinforcement in Prestressed Concrete Bridge Girders*. Minnesota DOT used epoxy coated U-bars with straight legs in place of the bent legs as recommended by the Standard Specifications. The ability of the bars with straight legs to achieve yield was investigated. It was found that the pre-compression in the bottom flange of the girder in regions of web-shear cracking improved the anchorage on the straight bars. The straight legs of the bars are embedded within the bottom flange, which contains strands outside the width of the shear reinforcement. This allowed for an improved resistance to vertical splitting cracks. An experimental program was conducted to verify that the straight bars reached yield. Pullout tests were performed on 13 beam sub assemblages. The strains measured in all tests indicated that the reinforcing was well into the strain-hardening region prior to failure. At least 50 percent of the specimens exhibited failure through reinforcing bar rupture, indicating sufficient anchorage. The specimens with deeper embedment, higher concrete strengths, and greater pre-compression led to higher strains in the reinforcement prior to failure. Additional full-scale tests were performed on

two beams with different depths. One test was performed to achieve flexural-compression failure and three tests were performed to achieve web shear failure. The girder shear capacities were computed using the Standard Specification with 1991 interim revisions. The results of the testing indicated that the shear computation method provided conservative estimate of shear strength. The strains measured in all tests indicated that the shear reinforcing had yielded and had reached the strain-hardening phase when tests were stopped (prior to failure). The effect of shear stirrup spacing was investigated as well and it was found to have no effect on the anchorage of shear reinforcement. It was noted that in specimens with close stirrup spacing the diagonal cracks were not conspicuous after the load was removed. In terms of an in-service bridge, it is possible not to see diagonal cracking (shear distress) after an overload has passed.

4.2. Concrete Shear Load Rating

4.2.1. Shear Capacity with Corrosion Damaged Reinforcement

The shear capacity of reinforced concrete bridge beams with corroded reinforcement is discussed in *Shear Capacity Assessment Of Corrosion-Damaged Reinforced Concrete Beams* by Higgins et al. [57]. The study focused on conventional reinforced concrete bridges in service in the United States. The study mentions that a large number of conventional reinforced concrete bridges have been deemed to be structurally deficient. Methods are said to be lacking in correlating the deterioration observed on the bridge and the rating category for the bridge. The focus of the study was on the presence of shear cracks in the conventional reinforced concrete bridges near the Oregon coast. The study described the visual changes observed in the cracking maps as well as the change in the shear capacity of these beams with the progression of corrosion. Four stages of corrosion were defined to describe deterioration observed in beams. The study concluded that additional damage states were required to describe the deterioration of the conventional reinforced concrete bridges affected by chloride induced corrosion and diagonal cracking. Shear stirrups with localized section loss exhibited localized yielding and a reduction in beam ductility. Shear stirrups with corrosion damage displayed a reduced ability to constrain diagonal cracks. Reduced stirrup sections within a span equal to the beam depth should be identified during inspections and a detailed assessment is to be performed. The study states that the bridge girders with significant reinforcing corrosion and localized section loss have a higher propensity to fail abruptly after diagonal cracking at the concrete core. Inspections of bridges that have been rehabilitated and/or zinc sprayed should focus on evidence of diagonal cracking.

4.2.2. Remaining Life Evaluation of Reinforced Concrete Girders with Shear Distress

The remaining life of beams with diagonal cracks were studied in *Remaining Life of Reinforced Concrete Beams with Diagonal-Tension Cracks* by Higgins et al. [58]. CIP reinforced concrete deck-girder (RCDG) bridges in Oregon were studied. The study identified more than 500 bridges in the Oregon DOT inventory that exhibited diagonal tension cracking. Most of these bridges have been load posted. Field testing, laboratory testing, and computational analysis were performed as a part of this study. In phase 1, a database was created to inventory the bridges most prone to diagonal-tension cracks. In phase 2, a bridge with observed instances of diagonal tension cracks was instrumented and field tested. The crack characteristics were documented and eight cracks were instrumented for observing crack movements and measuring strain in stirrups. This data was collected for ambient traffic as well as controlled truck loading. Factors such as dynamic and impact loading, load distribution between girders, deck thickness, diaphragm

stiffness, creep, shrinkage, and temperature effects were included in the analysis. A linear finite element model was created to predict cracking. Design values for one of the bridge girders was compared with the shear requirements in 1953 Standard Specifications and 2002 Standard Specifications.[6][20] A significant finding of phase 1 was that bridges with advanced stages of cracking typically consisted of larger girders and longer spans. This was attributed to the design practices of the time, which favored increasing beam size to increase the concrete contribution, to mitigate constructability constraints on stirrup spacing. Significant findings of phase 2 were that the bridge girders did not meet modern design requirements due to the overestimation of concrete contribution to shear capacity allowed by the prevalent design codes. Addition of overlays to these bridges should be carefully considered to avoid significant increase in stirrup stress under permanent loads. Support settlements had an effect on diagonal tension stress with a 10 percent decrease in stress at an interior support and 6 percent increment at an exterior support.

4.2.3. Assessing Reinforced Concrete Girders with Shear Distress

Further research on RCDG beams with diagonal cracks was performed by Higgins et al. [59] in *Assessment Methodology for Diagonally Cracked Reinforced Concrete Deck Girders.* Three bridges were instrumented and monitored for ambient traffic conditions as well as controlled truck loading. The load distribution factors in the Standard Specifications were found to be conservative when compared with distribution factors calculated from steel stirrup stresses. The impact coefficients were found to be below that recommended by *LRFD Bridge Design Specifications* for strength design. Other findings from the field testing were that the exterior girders had a widespread occurrence of diagonal cracks as compared to interior girders. The diagonal cracks tended to be concentrated in the quarter span locations adjacent to the supports; the potential cause of this cracking was identified as the increases in allowable concrete shear stress at the time of design.

Forty-four RCDG beam specimens were constructed with 1950s construction practices and tested in the laboratory. The major findings of the laboratory testing were that the flexural reinforcement terminated before the girder end provided lesser constraint to diagonal crack propagation and led to an overall reduction in ultimate load capacity. Crack widths were not adequate indicators of the previous damage experienced by a beam. Crack widths at failure were smaller for stirrups with tighter spacing and larger for stirrups with wider spacing. Other findings of this research were that bond deterioration initiated due to high cycle fatigue might cause a reduction in capacity based on MCFT and produce wide cracks. High cycle fatigue did not produce a significant reduction in capacity in the specimens for the loading conditions considered.

A comparison of five analytical methods was performed. The capacity of the laboratory specimens was predicted using the ACI method, Response-2000[TM] software, MCFT, STM, and Finite Element Method. MCFT and Response-2000[TM] (using MCFT approach) methods provided reliable estimates of shear capacity for beams with previous damage and wide diagonal cracks. Best correlation with the experimental results was provided by Response-2000[TM] while MCFT approach provided slightly conservative estimates of shear capacity. MCFT and Response-2000[TM] provided low estimates of capacity for bridge elements with low span to depth ratio, such as bent caps. The study indicated that more refined models are required for bent caps because the estimated shear capacity was limited by treatment of steel capacity and anchorage of flexural steel. The analytical models for estimating RCDG capacities were less accurate for

beams with span/depth ratios closer to a deep beam definition, especially bent caps. MCFT and Response-2000™ did not provide accurate estimation of shear strength for such members. The STM method was recommended for bent caps. Diagonal cracks were observed to propagate to the deck-stem interface in in-service bridges as well as laboratory specimens. Some of these cracks were observed to have propagated along the interface. The reinforcement crossing the interface may not have sufficient development length. The composite action can be compromised in such instances, especially if the deck is in flexural tension. A method was developed to assess reliability of bridges. A Reliability Index (β) is calculated at critical sections within a girder by comparison of the maximum forces with the estimated capacity and the variability in the prediction of capacity.

4.2.4.　　Deep Beam Shear

Higgins et al. [60, 61] conducted a two-part study on the evaluation of shear distress in bent caps in *Evaluation of Bent Caps in Reinforced Concrete Deck Girder Bridges, Part 1* and *Part 2*. The first part of the study focused on the influence of various parameters on the strength of flexural bar anchorages terminating in columns. The second part of the study focused on the structural performance of bent cap systems and their analytical evaluation. The remaining capacity and life of six in-service RCDG bridge bent caps was investigated in this part. Six full-scale bent cap specimens were constructed and tested in a laboratory. A comparison was made between the experimental results and various analytical predictions. Significant findings from this research were as follows. Initial diagonal tension cracking was observed at an average concrete shear stress equal to 1.8 times the square root of the specified compressive strength of concrete. The ACI 318-05 shear calculation method produced variable results, including some results that were unconservative. ACI 318-99 deep beam shear method produced conservative results. MCFT sectional analysis performed with Response-2000™ produced conservative results for bent caps with stress shear span/effective depth (a/d) of 1.38. The same method displayed better accuracy for bent cap specimens with a/d of 2.1.

4.2.5.　　Study of Shear Performance in Existing Bridge Girders

Llanos et al. [62] conducted an experimental study in *Shear Performance of Existing Concrete Bridge Girders* to determine the shear capacity and behavior of beams available in the Florida DOT inventory. These were AASHTO Type III, AASHTO Type IV, and post-tensioned (PT) girders that were used in the 1950s. The shear span to depth ratio was varied in this testing. The capacity of beams was predicted using the MCFT from 2007 *LRFD Bridge Design Specifications* (4th Edition),[24] STM from 2007 *LRFD Bridge Design Specifications* (4th Edition),[24] and ACI 318 Detailed method (2008). The beams were tested in three-point bearing with the point of application offset to achieve high shear-to-moment ratios. The Type IV girders were constructed for the testing and contained a debonding pattern that did not meet LRFD requirements. The Type III girders were recovered from an existing bridge and contained a combination of harped and straight strands. The PT girders were constructed for testing and contained a bar post-tensioning system that was in use more than 40 years ago. The PT girders also had no mild steel shear reinforcement beyond the end blocks, which has led to low shear rating for existing bridges with these beams. The tests produced the following results. The Type IV girders failed due to the separation of the bottom flange. This was attributed to grouping the debonded strands in the web and not providing sufficient confinement to the strands near supports. The shear strength of the Type IV girder was less than that predicted by all three methods. The Type III girders with small

shear span to depth ratios failed by strand slip. The slip was attributed to the formation of flexure-shear crack that interrupted strand development length. The capacities predicted by all three methods were still conservative for almost all test specimens with respect to the experimental data. The capacity prediction for the PT girders was conservative for all three methods.

4.2.6.　　　　　*Effects of Bar Terminations on Shear and Diagonal Cracking*

Higgins et al. [63] evaluated the effects of curtailing flexural reinforcement on the shear capacity of beams in RCDG bridges in *Flexural Anchorage Performance at Diagonal Crack Locations: Final Report*. The controlling load rating was governed by the anchorage of the flexural reinforcement. The focus of the research was to study the bond stresses on large diameter bars in presence of diagonal cracks. Eight large-scale specimens were constructed and tested in the laboratory. Some of the specimens were constructed with pre-formed crack at appropriate locations from the flexural steel cut-off locations. The tests on these specimens displayed that the length of the cutoff bars did not affect eventual beam specimen failure. The bond stresses were higher due to the pre-formed crack at service level loads. The eventual failure location was not at the pre-formed crack location and was dependent on the reinforcement detailing and load patterns. The experimental data showed that the *LRFD Bridge Design Specifications* equation for determining demand on longitudinal reinforcement provided a reasonable estimate of tensile demand. For an accurate prediction of additional demand on longitudinal steel, the input of coincident moment and shear values was deemed appropriate as opposed to the worst-case values. The bond stress values for anchorage failure were about 175 percent higher than that predicted by the less conservative of ACI and *LRFD Bridge Design Specifications* development length calculations. The authors suggested the use of ACI method for development length calculations where the *LRFD Bridge Design Specifications* method produced more conservative results. Other major findings of this research were that the presence of a diagonal crack at service level loads is not a definitive indicator of the failure location. The critical location and the critical angle of the diagonal crack at failure need to be calculated. A procedure for doing this is described by the authors. This involved calculating the critical angle by *LRFD Bridge Design Specifications* method and using MCFT to determine the shear that would produce sufficient forces in flexural steel to precipitate anchorage failure. The authors described the cracking patterns observed in testing that were indicative of anchorage failure.

4.2.7.　　　　　*Discrepancies in Shear Rating Methods*

Dereli et al. [64] researched the discrepancies and inconsistencies in the shear rating methods for prestressed beams in *Discrepancies in Shear Strength of Prestressed Beams with Different Specifications*. It was noted that some prestressed beams with no apparent shear distress are rated lower than the design capacities at inventory level. The researchers suggest that the lower ratings can be attributed to inherent conservatism in LFR method. Other possibilities for low ratings were attributed to possible flaws in rating tools (BrR software) and the exclusion of additional shear capacity parameters such as end blocks for example. An analytical research program was conducted to identify the sources of these discrepancies in shear capacities. The researchers selected 54 girders from the Minnesota DOT inventory that were deemed susceptible to under-prediction of capacity. These girders were rated in accordance with 2002 Standard Specifications (17th Edition) to determine the adequacy of designs on the LFR inventory and operating rating methods.[20] The BrR tool was validated using example bridges from the Minnesota DOT

inventory. The researchers found one significant error in computation of concrete shear capacity. Compression used in concrete contribution to web shear was evaluated at the incorrect location when the centroid of the composite section was above the web-flange intersection. This led to a capacity reduction of 25 percent at h/2 away from the support, that is, the critical section in accordance with Standard Specifications. Based on the findings of NCHRP Project 12-61, the authors present the conclusion that the 1979 interim revisions of the Standard Specifications did not provide reliable results for predicting shear capacity. The 2002 Standard Specifications (17th Edition) provided reasonable predictions of shear capacity with a low coefficient of variation in test to predicted shear capacity ratios.[20] The researchers studied the effect of concrete material properties on shear strength. A 20 percent difference was found between the nominal and the 28-day measured concrete strengths. The additional strength of concrete provided only a 6 percent increase of concrete contribution to shear capacity. The authors focused on the effect of shear live load distribution factors. The shear distribution factors calculated by the 2002 Standard Specifications (17th Edition) were found to be less conservative compared to other methods.[20]

Mylanarski et al. [65] compared the available methods for bridge load rating in *A Comparison of AASHTO Bridge Load Rating Methods*. A database of 1,500 bridges was created. The researchers surveyed states to determine the appropriate live loads to use for the analysis. The bridge girders were rated for moment and shear by both LFR and LRFR methods. The significant findings from this research were as follows. A significant percentage of girders that were rated greater than 1.0 by LFR produced ratings less than 1.0 by the LRFR method. This was attributed to LFR method not covering certain criteria that are covered by LRFR. Some of these criteria have been known to be problematic. The research proposed that the criteria not covered by LFR be ignored if there were no visible signs of distress similar to the exceptions allowed by the MBE.

Rogers and Jáuregui [69] compared LRFR to LFR ratings for 5 prestressed girder bridges in New Mexico using BRASS-GIRDER™. For the bridges in this study, LRFR ratings were found to be generally lower than LFR ratings, which was attributed to both differences in live load effects and the shear resistance equations used. They also found for the study bridges that shear ratings governed over flexural ratings and a number of these had inadequate shear ratings while flexural ratings were satisfactory. At the time of this study (2005), discrepancies were found between hand calculations and the results from BRASS-GIRDER™.

4.2.8. *Load Rating with Superloads*

Farrar et al. [66] conducted a study on the superload permitting in 18 States in *Advances in State DOT Superload Permit Processes and Practices*. The objective of this research was to identify best practices and improvement in the implementation of uniform permitting practices in superload permitting. The States were surveyed on the load rating methods used for bridge analysis. The LFR method was found to be the most commonly used. In many instances, ASR, LFR, or LRFR were used depending on the live loads used in the design of bridges.

4.2.9. *Shear Live Load Distribution and Load Rating*

Dymond et al. [67] researched the conservatism in shear live load distribution factors used to determine the shear on individual girder in *Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges*. The literature review performed by the authors revealed a lack of research into the effect of end diaphragms on shear. An experimental and analytical study was performed to study the effect of end diaphragms on shear. A full-scale bridge span with four prestressed girders was constructed. The bridge was loaded within its elastic range and the results were used to validate a finite element model. The bridge was then loaded until failure. The results from the finite element analysis of the bridge indicated that end diaphragms affected the shear distribution in the elastic range. The shear on a loaded interior girder increased by 4 to 6 percent due to end diaphragms. The increase in shear was seen near the point of load application. The end diaphragms reduced the shear at the end of the spans. The inelastic testing of the bridge indicated that the load is redistributed to adjacent girders and the loaded girder starts losing stiffness due to the occurrence of flexural cracks and web shear cracks. The use of linear elastic distribution factors for ultimate load was found to be conservative. The validated FE model was then modified for a parametric study. Girder spacing, deck thickness, and span lengths had the greatest effect on shear distribution. More shear distribution was observed for cases with higher ratio of transverse bending stiffness to longitudinal bending stiffness per girder. The authors created criteria for structures based on the ratio of longitudinal stiffness to transverse stiffness, referred to as the screening tool. For structures with a ratio less than 1.5 the shear demand calculated by finite element method (FEM) was less than that calculated from the AASHTO distribution factors. For bridges with a ratio ranging from 1.5 to 5.0, the FEM results were similar to the AASHTO results. Finally, for bridges with a ratio greater than 5.0, the live load distribution factor calculated from FEM was lower than that calculated by AASHTO equations. The researchers compared the effect of using a simple two-dimensional (2D) grillage analysis and found that the results did not significantly vary from the FEM results. The authors recommended that the transverse stiffness to longitudinal stiffness ratio be considered prior to selecting the method for calculating shear distribution factors.

Snyder and Beisswenger [68] conducted a follow-up study in *Implementation of a Refined Shear Rating Methodology for Prestressed Concrete Girder Bridges* for the methods for calculating shear distribution factors proposed in *Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges* by Dymond et al. [67]. The researchers selected 50 bridges from a database of 522 bridges identified with shear rating deficiencies in the Minnesota DOT inventory. For the bridges selected in this study, the refined methods of computing the shear live load distribution factors improved shear ratings by 16 percent.

CHAPTER 5.   SUMMARY OF FINDINGS AND RECOMMENDATIONS

These views are those of the authors, not necessarily those of FHWA, and that inclusion of the findings and recommendations does not mean FHWA necessarily agrees or will advance them.

5.1.   Introduction

The challenge of synthesizing concrete bridge shear load rating is well captured in this quote from *Review of Design Procedures for Shear and Torsion in Reinforced and Prestressed Concrete* by Ramirez and Breen:[46]

> A comprehensive review, dealing with all of the factors influencing behavior and strength of reinforced and prestressed concrete beams failing in shear and/or torsion, and all of the ways researchers and designers have attempted to mold these factors into code or specification formats would be a monumental task. Not only are these factors numerous and complex, but the individual contributions of researchers are difficult to integrate into an orderly and comprehensive body of knowledge.

Regarding load rating of bridges, concrete shear provisions outlined in *LRFD Bridge Design Specifications* and the MBE present challenges and difficulties to bridge owners and bridge engineers in their intended application.

Key shear load rating challenges include the following:

- When is load rating in shear needed?
- How to apply current MCFT provisions to continuous PT girders near the inflection points, where tendon(s) may be at the neutral axis or even on the flexural compression side of the girder.
- Interpreting the role of tension reinforcement anchorage in shear load rating.
- Lack of direction for handling load rating cases where existing shear reinforcement comes close to, but not meeting, minimum shear reinforcement requirements in MCFT.
- The interaction of moment and shear in determination of shear resistance with MCFT and how this can result in fluctuating load ratings with a series of heavy vehicles.
- ASR/LFR shear evaluation approaches do not accurately predict shear capacities of existing bridges.

This report briefly outlines the historical treatment of shear demand and resistance from the 1st edition of 1931 Standard Specifications[1] to the 2017 LRFD Bridge Design Specifications (8th Edition),[28] including loads, load combinations, live load distribution, and shear strength in both reinforced and prestressed concrete. It presents a historical overview of load rating bridges and reports on features of commonly used load rating software. A survey of nine state DOTs, to capture their policies, practices, and challenges in concrete bridge shear load rating is also presented. Lastly, a literature review of recent and relevant research and studies on concrete shear strength, concrete bridge behavior in shear, and concrete bridge shear load rating is performed.

These views are those of the authors, not necessarily those of FHWA, and that inclusion of the findings and recommendations does not mean FHWA necessarily agrees or will advance them.

5.2.   Findings and Recommendations

Findings and recommendations based on the information gathered for this synthesis are summarized in the following.

Finding 1: Some, but not all, State DOTs are requiring shear load ratings during the design process. They routinely load rate concrete bridges in shear beyond MBE requirements and shear frequently controls the load ratings.

From the survey, it is clear that some DOTs are routinely load rating concrete bridge superstructures, and to a lesser extent, substructures, for shear for design, legal, and permit loads, regardless of whether the structure is exhibiting shear distress (cracks) or not. These same DOTs have found that shear governs select bridge load ratings, so DOTs that do not load rate for shear during design, or do not load rate bridges for shear when no shear distress is evident may be unaware of their concrete bridge inventory's status relative to shear.

Finding 2: Load rating methods in the MBE are inconsistent in addressing the need for load rating concrete bridges in shear.

The surveyed DOTs routinely use both LFR and LRFR for load rating concrete bridges. LFR and LRFR differ in when shear load ratings are required. For example, LFR does not exempt reinforced and prestressed concrete components from inventory and operating shear load ratings; whereas, LRFR recommends shear load ratings for permit loads with both reinforced and prestressed concrete, but allows an exemption for design and legal load ratings when no shear distress is exhibited. Addressing post-tensioned segmental bridges separately from other concrete bridges, LRFR requires design, legal, and permit load ratings with this specific bridge type; this category of bridges is not specifically singled out in LFR, so operating and inventory load ratings are assumed to be required.

Recommendation

Require concrete shear load rating for all superstructures, which removes the inconsistencies in LRFR. In accordance with the survey results, most DOTs are already doing this.

Finding 3: The MBE should identify bridge construction years where shear should be more closely evaluated. [46]

Structural failures usually lead to improvements in design specifications. The Air Force warehouse failure at Wilkins Air Force Depot in 1955 exposed flaws in the ACI and AASHTO shear design provisions, which were subsequently addressed by requiring a minimum amount of shear reinforcement and better detailing of longitudinal reinforcement, recognizing the load demand on it from shear. Designs with the older Standard Specifications could have inadequate margins of safety against sudden shear failure and should be critically scrutinized by load raters. The MBE identifies material properties to use in load rating based on year of construction. Something similar could be used as a flag to load rating engineers for eras where potentially unconservative design specifications may have been used.

Recommendation

Identify construction years in the MBE commentary that highlights where potentially unconservative shear designs and detailing could be encountered.

Finding 4: Multiple load rating methods can make load rating bridges potentially confusing to practitioners.

There are four load rating methods—LRFR, LFR, ASR, and by assignment. These methods are coupled individually or in combination with three governing AASHTO publications—MBE, *LRFD Bridge Design Specifications*, and the Standard Specifications—in the load rating process.

Live loads for load rating are exceptionally numerous, with H- and HS-trucks, HL-93 design loads, AASHTO legal loads, permit loads, etc.

LRFR has two limit states required in load rating concrete bridges—strength and service, each represented by multiple load combinations. Distribution of live load in LRFD has multiple selections based on bridge type and whether the girders are exterior or interior; hosts of variables are needed to compute live load distribution factors with LRFD. Beyond limit states and load distribution, LRFD has three methods for determining shear strength with MCFT, an optional method specific for post-tensioned segmental bridges, STM for D-regions, and principal stress checks in webs of select post-tensioned girders. The Standard Specifications is somewhat simpler with H- and HS- truck and lane loads, less cumbersome load distribution, and has just two methods for reinforced concrete shear strength and one for prestressed concrete (which is really an evaluation of two separate equations, with the lower value controlling).

Beyond these permutations in load rating methods and governing specifications, concrete bridges are discretized into reinforced concrete, prestressed concrete, and post-tensioned segmental concrete in the MBE. Load rating methods in the MBE are not immune from complexity, with LRFR having design, legal, and permit load ratings; whereas, LFR and ASR have inventory and operating load ratings.

The above excludes any discussion of individual DOT policy or practice related to load rating or any direction on load rating from FHWA. To expect a load-rating engineer to be proficient at navigating through these specifications, loads, limit states, etc., increases risk in making mistakes in load rating. This starkly contrasts with the burden imposed on bridge designers. The surveyed states acknowledge use of all four load-rating methods, so the burden on load rating engineers to handle this complexity is real.

Recommendation

Move to one load rating method, LRFR, which corresponds to one design method, LRFD. It is acknowledged that doing so could potentially result in low load ratings for select bridges and that LRFR is not fully mature. Both issues can be addressed by focused research and an implementation plan that allowed States adequate time to assess and address their inventory, including past satisfactory ratings. An example of this could be bridges not currently load posted, but that show a deficiency with LRFR. Such a bridge could remain non-load posted until deterioration that reduced capacity was noted on subsequent inspections.

A low load rating with LRFR when contrasted with a more favorable LFR rating for the same bridge element does not mean the bridge has a safety problem, it just reveals a difference in calculation methods. Limiting load rating to LRFR alone could provide the impetus to help it mature in the coming years through research, code language refinement, or load rating policy refinement.

Finding 5: With the evolution of Standard and LRFD Specifications in loads, load distribution, design method—ASD versus LFD versus LRFD—and shear strength methods, it is not surprising to experience low shear load ratings for existing bridges.

Over the years, shear in concrete has been affected by increasing live loads, changes in material strengths, changes in load distribution and combinations of load, changes in design methodology, and introduction of better and more sophisticated means of determining shear strength. To be specific, it is now known that D-regions and deep beams are much better addressed with STM,

that longitudinal reinforcement has demands placed on it by shear both in terms of design and detailing, and that MCFT is the most accurate method for determining shear resistance (in B-regions).

The surveyed DOTs indicate the era of bridge construction most likely to result in low load ratings is during the interstate era up until introduction of the LRFD, approximately 1950 through 2010. This is a surprising finding, considering the improvements in shear design made since the early twentieth century. The introduction of LFD method could contribute to more observed in-service cracks, but this is speculation and more in-depth research into this issue is needed to find the reasons why this era of bridges is more troublesome in obtaining adequate load ratings.

One-third of survey respondents indicated low shear load ratings in the LRFD era, which initially may be more surprising. As shown earlier, shear design provisions in LRFD have changed substantially in its short life, with stability appearing to be achieved in the 2017 *LRFD Bridge Design Specifications* (8th Edition).[28] However, slightly low load rating factors at inventory level should not be unexpected with these modern designs when considering different software being for design and load rating and the subtle differences in input variables used by designers vs load raters. . One State DOT in the survey requires a minimum shear design load rating of 1.2 during the design process. Compared to the specter of a low load rating, this appears to be an economical and commendable best practice for new bridge designs, assuming reinforcement congestion is not created.

Finding 6: MCFT and STM should be used to load rate concrete bridges in shear. [48][52][55]

MCFT and STM in the 2017 *LRFD Bridge Design Specifications* (8th Edition) are the best methods for determining shear strength for B- and D-regions, respectively.[28] MCFT is load dependent, requiring the factored shear and moment on the section to determine its strength, because the strain on the section from flexure, shear, and axial loading is a critical component in determining the concrete strength and the angle of compressive struts. Flatter angles engage more shear reinforcement. Benefits in strength are gained as tensile strain decreases, with resulting flatter compressive strut angles, making prestressing beneficial on girders for shear capacity. With this importance on strain, it is very important to obtain a good estimate of the strain for the remaining calculations.

Load raters should refer to the *LRFD Bridge Design Specifications* for guidance on the appropriateness of which method—MCFT or STM—to use for specific elements being load rated.

Finding 7: Estimation of the strain—correctly—for use in MCFT-based calculations is critical.

The first version of MCFT in the first LRFD used an iterative procedure, where the designer estimated the strain and obtained β and θ values from a table and then verified if the assumed tensile strain exceeded the calculated strain; if so, no more iterations were needed for a conservative design solution. LRFD Appendix B5 retains this as the General Procedure for Shear Design with Tables.

MCFT, as outlined in *LRFD Bridge Design Specifications* 5.7.3, has a direct solution for shear strength without iterations, making it more appealing to designers and load raters. However, using a reasonably accurate strain is still critical to obtain the best assessment of shear strength.

LRFD is inconsistent with the location of the strain calculation. Appendix B5 uses the strain at mid-depth, approximating the mid-depth strain by taking one-half the strain at the level of the flexural tension reinforcement, as does the Canadian Standards Association method from which *LRFD Bridge Design Specifications* 5.7.3 is based on. *LRFD Bridge Design Specifications* 5.7.3 uses a strain value taken at the level of the tension reinforcement, and is later manipulated in the subsequent β and θ calculations to be compatible with a mid-depth strain. This is potentially misleading to load raters and designers on which location in the height of the girder is important for strain in MCFT—the mid-depth strain is the important value. Commentary CB5.2 recognizes this issue and instructs the user to consider averaging the strains at top and bottom of the girder to capture the beneficial effects of prestressing, specifically identifying cases where prestressing is located on the flexural compression side, such as tendons near contraflexure points and ends of pretensioned girders made continuous for LL. This commentary language is missing in *LRFD Bridge Design Specifications* 5.7.3; it should be included directly or referenced.

Recommendation

Repeat Appendix B5 commentary discussion on strain for girders with prestressing on the flexural compression side or refer to it in *LRFD Bridge Design Specifications* 5.7.3. Alternatively, provide a more general treatment on strain determination in the specifications.

Finding 8: Using MCFT and the strain equations in LRFD when prestressing is on the compressive flexural side of a girder can provide incorrect and overly conservative shear strengths. [70]

One problem identified by a State DOT with MCFT is the unique case presented by continuous girders at contraflexure locations where PT tendon(s) are above the girder's mid-depth, on the flexural compressive side. In accordance with the strain equations in *LRFD Bridge Design Specifications* for MCFT, these tendons are neglected in the strain calculations and the beneficial effects of prestressing are ignored. In one case, using the equations verbatim led to a significant drop in shear load rating near the contraflexure point. Subsequent analysis with Response-2000™ software showed no deficiency in the shear capacity. However, using two software programs to load rate a continuous girder is both cumbersome and inefficient.

The strain calculations in MCFT, both in Appendix B5 and *LRFD Bridge Design Specifications* 5.7.3, assume the section is cracked and neglects the axial stiffness of the concrete on the flexural tension half of the girder. For the example presented in the T-18 Committee Document *Low Shear Strength near Inflection Points of Post-Tensioned Multi-Cell Box Girders,[70]* further analysis by the authors of this report shows the girder to remain elastic under factored loads, which is unsurprising because moments are quite low at contraflexure points. The girder is shown to be completely in compression under factored loads, again demonstrating the beneficial effects of prestressing. The proposed solution to this unique case is to first verify the girder is cracked before moving forward with the strain equations. If not cracked, the mid-depth strain can conservatively be taken as zero, and subsequent calculations will provide a much more reasonable and accurate shear strength. Incorporating this cracking check into load rating software should be easy to accomplish. Alternatively, the equation CB5.2-1 in *LRFD Bridge Design Specifications* can be used in this scenario, with the axial stiffness of the concrete included on the flexural tension side when determining ε_t.

Recommendation

Add commentary to *LRFD Bridge Design Specifications* 5.7.3 that highlights the need to confirm cracking prior to using the strain equations. If not cracked, designers and load raters can conservatively assume strain equal to zero.

Finding 9: MCFT-based shear strength calculations are load dependent.

Factored force effects are needed to establish shear strength with MCFT. Different live load vehicles, if fully or partially loaded, will produce different envelopes of factored force effects, and subsequently differences in shear strength.

A typical MCFT-based shear-moment interaction diagram for a generic section is provided in Figure CB5.2-6 of the current *LRFD Bridge Design Specifications*, which shows non-linearity over the range of applied moments. This figure should remind load rating engineers that simple linear ratios in capacity, and hence rating factors, may give incorrect results.

For flexural shear cracking, the Standard Specifications directed the engineer to use the factored shear corresponding to the live load position producing the maximum moment at the point of interest. With MCFT, no similar, explicit direction is provided, where some engineers use the maximum factored shear with the maximum factored moment, regardless of potential differences in live load positioning to obtain these envelope values. MCFT could potentially provide more favorable load ratings if the load rater evaluated the load case with live load positioned for maximum moment at the section in question with the concurrent shear and the load case with live load positioned for maximum shear at the same section with the concurrent moment.

For continuous structures, at the section of interest, two additional combinations also need to be considered: the minimum (or maximum negative) moment with the concurrent shear and the minimum (or maximum negative) shear with the concurrent moment.

For design, it is generally acceptable and conservative to apply the maximum/minimum force effect envelope from a design live loading in determining the shear resistance. For load rating, accurate shear resistance may significantly impact operation and management decision. With MCFT, it is important to use the strain consistent with the applied loads when computing the shear resistance. To achieve the consistency between the applied loads, the strain, and the shear resistance, numerical analysis requires convergence.

Recommendation

Add commentary to the MBE C6A5.8 that reminds load raters to consider both maximum positive and negative moment and concurrent shear along with maximum positive and negative shear and concurrent moment at a section of interest when load rating the particular section.

Add to the MBE 6A.5.8 language that requires load raters to use the strain consistent with the applied loads when determining a section's shear resistance with MCFT.

Add commentary to the MBE C6A.5.8 that numerical analysis requires convergence in order to achieve consistency between the applied loads, the strain, and the shear resistance.

Finding 10: An existing girder not meeting minimum shear reinforcement requirements with MCFT may be have its shear strength unduly penalized.

MCFT follows two paths, one for girders having at least the minimum amount of shear reinforcement and those that do not. This is easily handled in a design situation, but can present

problems in a load-rating situation. When not meeting the minimum amount of shear reinforcement, the concrete contribution to shear strength, V_c, can be lowered significantly. The equations do not discriminate between girders coming quite close to meeting minimum reinforcement requirements and those that may have no shear reinforcement. A methodology that can address this unique situation, with a lowering of shear strength that takes into consideration the amount of shear reinforcement from none to just meeting minimum requirements would be beneficial to shear load rating.

There is an analogous situation with STM in regard to meeting the crack control reinforcement limit. There is an abrupt drop in the concrete efficiency factor from sections meeting the minimum crack control reinforcement to those sections without any crack control reinforcement. A similar methodology to address this situation could likewise prove beneficial for load rating when using STM.

Recommendation

Provide guidance in *LRFD Bridge Design Specifications* 5.7.3 on how to transition from no shear reinforcement to meeting minimum reinforcement without the abrupt drop in strength currently indicated in the current specifications. Likewise, guidance in *LRFD Bridge Design Specifications* 5.8.2 on how to transition the concrete efficiency factor for elements with no crack control reinforcement to those that possess the required crack control reinforcement. Research is needed to fully address both of these enhancements to shear capacity, which would be useful for both design and load rating.

Finding 11: The MBE should offer a method to designers to determine residual shear strength after cracking. [52][57]

In addition to MCFT, researchers have developed methods to evaluate shear-distressed elements by linking diagonal crack width to percent ultimate capacity of reinforced concrete deep beams [52] and suggestions on evaluating member shear strength for members experiencing reinforcement corrosion [57]. Referencing these methods in the MBE, has value to load raters and DOTs in evaluating the safe load carrying capacity of bridges.

Recommendation

Add references to the MBE that point to relevant research that provide methods to ascertain remaining shear strength in the presence of observed shear cracking.

Finding 12: Reinforcement detailing should be verified for adequacy when load-rating concrete bridges in shear. [46][56]

The Air Force warehouse shear failure previously noted points to the importance of adequate detailing and anchorage of both shear reinforcement and longitudinal reinforcement. MCFT requires a verification of the longitudinal reinforcement and this should be included as part of the shear load rating. Recent research has been directed toward the effects of flexural reinforcement termination vs shear cracks and shear capacity. Shear reinforcement not meeting current specifications requirements for anchorage have been researched [56] and are shown to be adequate still—for the specific case investigated.

Recommendation

Load raters should verify that shear reinforcement details and anchorage, as shown in as-built construction documents, are consistent with using the full strength of shear reinforcement when establishing a member's overall shear capacity.

Finding 13: Expand the application of approximate live load distribution factors in LRFD.

Current LRFD directs designers and load raters to use lever rule load distribution for a number of situations common in bridge design. The survey results showed problems were encountered in obtaining favorable load ratings when using the approximate shear live load distribution factors in LRFD. Conservatism is more acceptable for design purposes than it is for load rating. Research should be undertaken to eliminate cases where the lever rule is the default.

When the lever rule is the provided approximate live load distribution method and shear is governing the load rating, there is potential value in using a refined analysis of the live load distribution, but that comes with a cost in level of effort and will likely require more than one software.

Because shear load ratings are performed for permit load rating, it is common for permit vehicles to be substantially wider than a 6-foot gauge vehicle implied with the LRFD live load distribution factors. Load raters need the MBE to provide a method to account for live load distribution of wider permit vehicles.

Recommendation

Research is needed to eliminate the lever rule for live load distribution for shear and to provide direction on distribution of live loads with gauge widths wider than 6 feet, such as commonly encountered super loads and other permit load types.

Finding 14: Requiring a modest over-strength during design can prove beneficial for subsequent load rating. [47]

Shear reinforcement is inexpensive; requiring designers to provide a modest over-strength in shear design, as one of the surveyed DOTs does, can provide advantages in load rating later on during a bridge's life. Reinforcement congestion that can compromise concrete consolidation and quality should always be avoided, but in most cases, a modest tightening of stirrup spacing can be an inexpensive way to obtain conservative levels of safety.

Recommendation

A potential best practice for designers to adopt is to design bridges to have a modest over-strength in shear or to have a minimum inventory rating factor with modest over-strength. This could be done with minimal to no cost increase.

Finding 15: A source of previous AASHTO design specifications and historic State DOT construction specifications would be useful to practitioners.

Older editions of AASHTO design specifications appear to only be available in select DOT libraries. A comprehensive set of all previous AASHTO design specifications in electronic format or internet-based, would be useful to the load rating community and to bridge owners.

Likewise, State DOTs should make available to load raters their past construction specifications, to point load rating engineers to the minimum material properties that would have been required

in the original construction. Depending on the conservative material properties in the MBE based on year of construction can lead to overly conservative load ratings.

Recommendation

State DOTs should engage AASHTO to explore the feasibility of assimilating the historical AASHTO bridge design specifications from 1931 on and making them available to practitioners. State DOTs should likewise determine the feasibility of making their historical construction specifications available to practitioners as well.

CHAPTER 6.   REFERENCES

1. AASHO. (1931). *Standard Specifications for Highway Bridges and Incidental Structures*, 1st Edition, American Association of State Highway Officials, Washington, DC.

2. AASHO. (1935). *Standard Specifications for Highway Bridges*, 2nd Edition, American Association of State Highway Officials, Washington, DC.

3. AASHO. (1941). *Standard Specifications for Highway Bridges*, 3rd Edition, American Association of State Highway Officials, Washington, DC.

4. AASHO. (1944). *Standard Specifications for Highway Bridges*, 4th Edition, American Association of State Highway Officials, Washington, D.C. Including interim revisions for 1945 and 1946.

5. AASHO. (1949). *Standard Specifications for Highway Bridges*, 5th Edition, American Association of State Highway Officials, Washington, D.C. Including interim revisions for 1950.

6. AASHO. (1953). *Standard Specifications for Highway Bridges*, 6th Edition, American Association of State Highway Officials, Washington, D.C.

7. AASHO. (1957). *Standard Specifications for Highway Bridges*, 7th Edition, American Association of State Highway Officials, Washington, D.C.

8. AASHO. (1961). *Standard Specifications for Highway Bridges*, 8th Edition, American Association of State Highway Officials, Washington, D.C.

9. AASHO. (1965). *Standard Specifications for Highway Bridges*, 9th Edition, American Association of State Highway Officials, Washington, D.C. Including interim revisions for 1966 and 1967.

10. AASHO. (1969). *Standard Specifications for Highway Bridges*, 10th Edition, American Association of State Highway Officials, Washington, D.C. Including interim revisions for 1970, 1971, and 1972.

11. AASHO. (1973). *Standard Specifications for Highway Bridges*, 11th Edition, American Association of State Highway Officials, Washington, D.C. Including interim revisions for 1974, 1975, and 1976.

12. AASHO. (1970). *Manual for Maintenance Inspection of Bridges*, 1st Edition, American Association of State Highway Officials, Washington, D.C.

13. AASHTO. (1974). *Manual for Maintenance Inspection of Bridges*, 2nd Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

14. AASHTO. (1977). *Standard Specifications for Highway Bridges*, 12th Edition, American Association of State Highway and Transportation Officials, Washington, D.C. Including interim revisions for 1977, 1978, 1979, 1980, 1981, 1982, and 1983.

15. AASHTO. (1978). *Manual for Maintenance Inspection of Bridges*, 3rd Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

16. AASHTO. (1983). *Manual for Maintenance Inspection of Bridges* American Association of State Highway and Transportation Officials, Washington, D.C. Including interim revisions for 1984, 1985, 1986, 1987, 1988, 1989, and 1990.

17. AASHTO. (1983). *Standard Specifications for Highway Bridges*, 13th Edition, American Association of State Highway and Transportation Officials, Washington, D.C. Including interim revisions for 1984, 1985, 1986, 1987 and 1988.

18. AASHTO. (1992). *Standard Specifications for Highway Bridges*, 15th Edition, American Association of State Highway and Transportation Officials, Washington, D.C. Including interim revisions for 1993, 1994, and 1995.

19. AASHTO. (1996). *Standard Specifications for Highway Bridges*, 16th Edition, American Association of State Highway and Transportation Officials, Washington, D.C. Including interim revisions for 1997, 1998, 1999, 2000, and 2002.

20. AASHTO. (2002). *Standard Specifications for Highway Bridges*, 17th Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

21. AASHTO. (1994). *AASHTO LRFD Bridge Design Specifications*, 1st Edition, American Association of State Highway and Transportation Officials, Washington, D.C. Including interim revisions for 1996 and 1997.

22. AASHTO. (1998). *AASHTO LRFD Bridge Design Specifications*, 2nd Edition, American Association of State Highway and Transportation Officials, Washington, D.C. Including interim revisions through 2003.

23. AASHTO. (2004). *AASHTO LRFD Bridge Design Specifications*, 3rd Edition, American Association of State Highway and Transportation Officials, Washington, D.C. Including interim revisions for 2005.

24. AASHTO. (2007). *AASHTO LRFD Bridge Design Specifications*, 4th Edition, American Association of State Highway and Transportation Officials, Washington, D.C. Including interim revisions for 2008 and 2009.

25. AASHTO. (2010). *AASHTO LRFD Bridge Design Specifications*, 5th Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

26. AASHTO. (2012). *AASHTO LRFD Bridge Design Specifications*, 6th Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

27. AASHTO. (2014). *AASHTO LRFD Bridge Design Specifications*, 7th Edition, American Association of State Highway and Transportation Officials, Washington, D.C. Including interim revisions for 2015 and 2016.

28. AASHTO. (2017). *AASHTO LRFD Bridge Design Specifications*, 8th Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

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

30. AASHTO. (2008). *Manual for Bridge Evaluation*, 1st Edition, American Association of State Highway and Transportation Officials, Washington, D.C. Including interim revisions for 2010.

31. AASHTO. (2010). *Manual for Bridge Evaluation*, 2nd Edition, American Association of State Highway and Transportation Officials, Washington, D.C. Including interim revisions for 2011 and 2013.

32. AASHTO. (2018). *Manual for Bridge Evaluation*, 3nd Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

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

34. AASHTO. (1989). *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, American Association of State Highway and Transportation Officials, Washington, D.C.

35. AASHTO. (1999). *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, American Association of State Highway and Transportation Officials, Washington, D.C.

36. AASHTO (2003). *Guide Manual for Condition Evaluation and Load Resistance Factor Rating of Highway Bridges*, American Association of State Highway and Transportation Officials, Washington, D.C.

37. Lemcoe, M.M. (1954). *Criteria for Prestressed Concrete Bridges*, U. S. Department of Commerce Bureau of Public Roads, Washington, D.C.

38. Schlaich, J., K. Schäfer, M. Jennewein (1987) "Towards a Consistent Design of Structural Concrete*" PCI Journal*. 32. 74-151

39. Collins, M. P., and Mitchell, D. (1991) *Prestressed Concrete Structures*, Prentice Hall, Englewood Cliffs, New Jersey

40. Durrani, A. J., and Robertson, I. N. (1987) "Shear Strength of Prestressed Concrete T Beams with Welded Wire Fabric as Shear Reinforcement" *PCI Journal*. 32. 46-61

41. PCI and WRI Ad Hoc Committee on Welded Wire Fabric for Shear Reinforcement (1980) "Welded Wire Fabric for Shear Reinforcement" *Journal of Prestressed Concrete Institute*. 25(4).

42. Xuan, X., Rizkalla, S., and Maruyama, K. (1988) "Effectiveness of Welded Wire Fabric as Shear Reinforcement in Pretensioned Prestressed Concrete T-Beams" *ACI Structural Journal*. 85. 429-436

43. Griezic, A., Cook, W.D., and Mitchell, D. (1994) "Tests to Determine Performance of Deformed Welded Wire Fabric Stirrups" *Structural Journal. 91. 211-220*

44. AASHTO. (2017). Images from AASHTOWare Bridge Rating™. American Association of State Highway and Transportation Officials, Washington, D.C.

45. WYDOT. (2018). Image from BRASS-GIRDER™. Wyoming Department of Transportation.

46. Ramirez, J.A. and Breen, J.E. (1983). *Review of Design Procedures for Shear and Torsion in Reinforced and Prestressed Concrete.* Research Report 248-2, Center for Transportation Research, University of Texas at Austin.

47. Ma, Z., Saleh, M.A., and Tadros, M.K. (1997). "Shear Design of Stemmed Bridge Members: How Complex Should It Be?" *PCI Journal*. 42. 88-93.

48. Hawkins, N.M., Kuchma, D.A., Mast, R.F., Marsh, L.M., and Reineck, K-H. (2005). *Simplified Shear Design of Structural Concrete Members.* National Cooperative Highway Research Program (NCHRP) Report Number 549. Transportation Research Board, Washington, DC.

49. Tureyen, A.K., and Frosch, R.J. (2003). "Concrete Shear Strength: Another Perspective." *ACI Structural Journal,* Vol. 100, No. 5, pp 609-615.

50. Bentz, E.C., Vecchio, F.J., and Collins, M.P. (2006). "Simplified Modified Compression Field Theory for Calculating Shear Strength of Reinforced Concrete Elements." *ACI Structural Journal*, v.103, n.4, pp.614-624.

51. Hawkins, N.M. and Kuchma, D.A. (2007). *Application of LRFD Bridge Design Specifications to High-Strength Structural Concrete: Shear Provisions.* National Cooperative Highway Research Program (NCHRP) Report Number 579. Transportation Research Board, Washington, DC.

52. Birrcher, D., Tuchscherer, R., Huizinga, M., Bayrak, O., Wood, S., and Jirsa, J. (2008). *Strength and Serviceability Design of Reinforced Concrete Deep Beams.* Research Report Number FHWA/TX-09/0-5253-1, Center for Transportation Research, University of Texas at Austin.

53. Avendaño V, A.R. and Bayrak, O. (2008). *Shear Strength and Behavior of Prestressed Concrete Beams.* Research Report Number IAC-88-5DD1A003-3, University of Texas at Austin.

54. Laskar, A., Wang, J., Hsu, T.T.C., Mo, Y.L. (2007). "Rational Shear Provisions for AASHTO LRFD Specifications." Technical Report 0-4759-1, Texas Department of Transportation, Houston, Texas.

55. Nakamura, E., Avendaño, A.R., and Bayrak, O. (2013). *Shear Database for Prestressed Concrete Members.* ACI Structural Journal. 110. 909-918.

56. Mathys, B., French, C., and Shield, C. (2014). *Anchorage of Shear Reinforcement in Prestressed Concrete Bridge Girders.* Research Report Number MN/RC 2014-36, Department of Civil, Environmental, and Geo-Engineering, University of Minnesota.

57. Higgins, C., Farrow III, W.C., Potisuk,T., Miller, T.H., Yim, S.C., Holcomb, G.R., Cramer, S.D., Covino Jr, B.S., Bullard, S.J., Ziomek-Moroz, M., and Matthes, S.A. (2003). *Shear Capacity Assessment Of Corrosion-Damaged Reinforced Concrete Beams.* Research Report FHWA-OR-RD-04-06, Oregon Department of Transportation.

58. Higgins, C., Yim, S.C., Miller, T.H., Yim, Robelo, M.J. and Potisuk, T. (2004). *Remaining Life of Reinforced Concrete Beams with Diagonal-Tension Cracks.* Research Report FHWA-OR-RD-04-12, Oregon Department of Transportation.

59. Higgins, C., Miller, T.H., Rosowsky, D.V., Yim, S.C., Potisuk, T., Daniels, T.K., Nicholas, B.S., Robelo, M.J., Lee, A-Y., and Forrest, R W (2004). *Assessment Methodology for Diagonally Cracked Reinforced Concrete Deck Girders.* Research Report FHWA-OR-RD-05-04, Oregon Department of Transportation.

60. Higgins, C., Senturk, A.E., and Koester, C.C., (2008). *Evaluation of Bent Caps in Reinforced Concrete Deck Girder Bridges, Part 1.* Research Report Number OR-RD-07, Oregon Department of Transportation.

61. Higgins, C., Senturk, A.E., and Koester, C.C., (2008). *Evaluation of Bent Caps in Reinforced Concrete Deck Girder Bridges, Part 2.* Research Report Number OR-RD-08, Oregon Department of Transportation.

62. Llanos, G., Ross, B.E., and Hamilton III, H.R. (2009). *Shear Performance of Existing Concrete Bridge Girders.* Research Report Number BD 545-56, University of Florida.

63. Higgins, C.C., Triska, M.A., and Goodall, J. (2010). *Flexural Anchorage Performance at Diagonal Crack Locations: Final Report.* Research Report Number FHWA-OR-RD-11-06, Oregon State University.

64. Dereli, O., Shield, S., and French, C. (2010). *Discrepancies in Shear Strength of Prestressed Beams with Different Specifications.* Research Report Number MN/RC 2010-03, Department of Civil Engineering, University of Minnesota.

65. Mlynarski, M., Wassef, W.G., and Nowak, A.S. (2011) *A Comparison of AASHTO Bridge Load Rating Methods.* National Cooperative Highway Research Program (NCHRP) Report Number 700. Transportation Research Board, Washington, D.C.

66. Farrar, M., Becker, S., Braden, R., Gao, L., Honefanger, J., Keady, K., Mallard, J., Wight, M., and Nassif, H. (2014) *Advances in State DOT Superload Permit Processes and Practices.* National Cooperative Highway Research Program (NCHRP) Project 20-68A. Transportation Research Board, Washington, DC.

67. Dymond, B.Z., French, C.E.W., and Shield, C.K. (2016). *Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges.* Research Report Number MN/RC 2016-32, Department of Civil Engineering, University of Minnesota.

68. Snyder, R. and Beisswenger, J. (2017). *Implementation of a Refined Shear Rating Methodology for Prestressed Concrete Girder Bridges.* Research Report Number MN/RC 2017-48, SRF Consulting Group, Inc., Minneapolis.

69. Rogers, B.J., Jáuregui, D.V. (2005). *Load Rating of Prestressed Concrete Girder Bridges: A Comparative Analysis of LFR and LRFR.* Research Report Number NM02STR-01, Department of Civil Engineering, New Mexico State University.

70. AASHTO T-18 Committee Document. (2016) *Low Shear Strength near Inflection Points of Post-Tensioned Multi-Cell Box Girders.*

Survey for Development of Synthesis Report on Concrete Bridge Shear Load Rating

BACKGROUND

The FHWA has engaged HDR, Inc. to develop a synthesis report on concrete bridge shear load rating. This survey is designed to capture current State DOT policy, practices and issues encountered with load rating concrete bridges in shear. Survey responses will be a key component of the findings in the synthesis report.

PLEASE RETURN completed survey to: John Holt **BY FRIDAY, 11-17-2017.**

RESPONDENT INFO:

State	
Respondent	
Title	
Phone	
Email	

TOPIC 1: AGENCY LOAD RATING POLICY AND PROCEDURES

1. Which load rating method(s) does your agency use for rating concrete bridges?

    ☐ Load and Resistance Factor Rating, LRFR
    ☐ Load Factor Rating, LFR
    ☐ Allowable Stress Rating, ASR
    ☐ Assigned load ratings
    ☐ Use more than one method. Please describe:

2. Does your agency have a policy or requirement for concrete shear load rating above and beyond the requirements in the AASHTO MBE?

    ☐ Yes. Please describe:
    ☐ No

3. Does your agency have a policy or requirement for designing concrete shear above and beyond the requirements in the AASHTO LRFD BDS?

    ☐ Yes. Please describe:
    ☐ No

4. AASHTO LRFD BDS has multiple methods for determining shear strength. Please select the shear strength methods your agency allows/uses/requires for design and load rating. Select all that apply.

    ☐ Strut and Tie Method (LRFD Art. 5.6.3)
    ☐ MCFT with tabular values (LRFD Art. 5.8.3.4.2 with Appendix B5)
    ☐ MCFT with equations (LRFD Art. 5.8.3.4.2)

☐ MCFT with $\beta = 2/\theta = 45°$ for non-prestressed sections (LRFD Art. 5.8.3.4.1)
☐ V_{cw}/V_{ci} from LRFD (LRFD Art. 5.8.3.4.3)
☐ Segmental provisions (LRFD Art. 5.8.6)
☐ V_{cw}/V_{ci} for prestressed concrete only (Standard Specifications Art. 9.20)
☐ Other. Please list:

5. Did your agency have a policy or requirement for designing concrete shear above and beyond the requirements in the AASHTO Standard Specifications, current or past edition?

☐ Yes. Please describe:
☐ No

6. Does your agency require a design load rating for concrete shear as part of the bridge design process?

☐ Yes, superstructure only
☐ Yes, superstructure and substructure. Please list which substructure elements:
☐ No

TOPIC 2: SHEAR LOAD RATING FOR CONCRETE BRIDGES

7. Which concrete bridge elements do you evaluate in shear for design load ratings?

☐ Superstructure, only when displaying shear distress
☐ Superstructure, always
☐ Substructure, only when displaying shear distress
☐ Substructure, always. Please list which substructure elements:
☐ None

8. Which concrete bridge elements do you evaluate in shear for legal load ratings, including SHVs?

☐ Superstructure, only when displaying shear distress
☐ Superstructure, always
☐ Substructure, only when displaying shear distress
☐ Substructure, always
☐ None

9. Which concrete bridge elements do you evaluate in shear for permit load ratings?

☐ Superstructure, only when displaying shear distress
☐ Superstructure, always
☐ Substructure, only when displaying shear distress
☐ Substructure, always
☐ None

10. Do shear load ratings control the overall bridge load rating? Provide approximate/estimated percentage of your concrete bridge inventory controlled by a shear load rating, if available. Select all that apply.

    ☐ Yes, for design load rating   %
    ☐ Yes, for legal load rating (including SHV)   %
    ☐ Yes, for permit load rating   %
    ☐ No

11. For bridges with their overall load rating controlled by shear, in what time period were they originally built? Select all that apply.

    ☐ Prior to 1950 (Prior to prestressing in US)
    ☐ 1950-1980 (ASD era)
    ☐ 1980-2010 (LFD era)
    ☐ 2010-Present (LRFD era)
    ☐ Other specific time. Please describe:

12. Has the percentage of bridge load ratings controlled by shear increased, decreased, or held steady in the past 10 years?

    ☐ Increased
    ☐ Decreased
    ☐ Held steady
    ☐ Not applicable

13. If the percentage of bridge load ratings governed by shear has increased, what are the sources of change? Select all that apply:

    ☐ Change in calculating shear strength. Please explain:
    ☐ Change in rating method. Please explain:
    ☐ Change in software. Please explain:
    ☐ Change in truck sizes/configurations
    ☐ Change in agency or federal policy regarding load rating for shear. Please explain:
    ☐ General deterioration of inventory thru environment, heavy trucks, limitations on maintenance, etc.
    ☐ Other. Please describe:

14. If the percentage of bridge design load ratings governed by shear has decreased, what are the sources of change? Select all that apply.

    ☐ Change in calculating shear strength. Please explain:
    ☐ Change in rating method. Please explain:
    ☐ Change in software. Please explain:
    ☐ Change in agency policy regarding load rating for shear. Please explain:
    ☐ General improvement of inventory thru replacement, repair or rehabilitation
    ☐ Other. Please describe:

15. What types of superstructures in your inventory are most likely to have a low shear load rating (RF < 1.0)? Select all that apply. Provide approximate/estimated percentage of each type, if available.

☐ Slab span, RC or prestressed, voided or solid, simple or continuous spans. %
☐ RC T-girder, simple or continuous spans.   %
☐ RC box girder, simple or continuous spans.   %
☐ Pretensioned girder (all shapes), simple spans.   %
☐ Pretensioned girder (all shapes), continuous for LL spans only.   %
☐ Post-tensioned, spliced girders (all shapes), simple or continuous spans.   %
☐ Post-tensioned box girder, non-segmental, simple or continuous spans.   %
☐ Post-tensioned box girder, segmental, simple or continuous spans. %
☐ Box Culvert (all sides).   %
☐ Other. Please list:

16. What types of substructures in your inventory are most likely to have a low shear load rating (RF < 1.0)? Select all that apply. Provide approximate/estimated percentage of each type, if available.

☐ RC rectangular caps for piers/bents.   %
☐ RC inverted T caps for piers/bents.   %
☐ Prestressed caps for piers/bents.   %
☐ RC or prestressed caps for straddle-type piers/bents.   %
☐ RC columns.   %
☐ Concrete piles for trestle-type bents. %
☐ Spread footings.   %
☐ Pile or drilled shaft supported footings.   %
☐ Other. Please list:

17. Does your agency have concrete bridge elements common to its inventory that routinely exhibit shear distress, e.g., shear cracking?

☐ Yes
☐ No

18. If answered "No" to question 17, proceed to question 19. If answered "Yes" above, to what is the shear distress attributed? Select all that apply.

☐ Inadequate design specifications for original design. Please list specification, if available:
☐ Lower design live load, e.g. H10
☐ Past local design policy or practice. Please describe:
☐ Heavy trucks
☐ Concrete deterioration
☐ Inadequate reinforcement anchorage or details
☐ Other. Please describe:

19. Have any of the following shear strength methods listed below provided an unexpected low design load rating (RF < 1.0)? Select all that apply.

☐ Strut and Tie Method (LRFD Art. 5.6.3)
☐ MCFT with tabular values (LRFD Art. 5.8.3.4.2 with Appendix B5)
☐ MCFT with equations (LRFT Art. 5.8.3.4.2)
☐ MCFT with $\beta = 2/\theta = 45°$ for non-prestressed sections (LRFD Art. 5.8.3.4.1)
☐ V_{cw}/V_{ci} from LRFD (LRFD Art. 5.8.3.4.3)
☐ Segmental provisions (LRFD Art. 5.8.6)
☐ V_{cw}/V_{ci} for prestressed concrete only (Standard Specifications Art. 9.20)
☐ Other. Please describe:

20. Have you re-evaluated a low shear load rating using a different shear strength method/equation in either the AASHTO LRFD or Standard Specifications and found a significant difference?

☐ Yes. Please explain:
☐ No

21. If you calculate a shear strength using more than one of the methods in the AASHTO LRFD BDS for load rating, do you use the highest (more liberal) or lowest (more conservative) calculated shear strength in the rating?

☐ Highest shear strength
☐ Lowest shear strength
☐ Depends on the situation. Please explain:

22. Have any of the approximate shear live load distribution factors or skew correction factors in LRFD Article 4.6.2 been the source of a low shear load rating?

☐ Yes, shear LLDF for interior beams
☐ Yes, shear LLDF for exterior beams
☐ Yes, lever rule LLDF
☐ Yes, skew correction factors for exterior beams
☐ No

23. Has the principal stress check for web cracking at Service Limit State controlled a load rating of a segmental box girder?

☐ Yes. Please explain circumstances, e.g. principal stress check not required at time of original design, no vertical PT in web, etc.:
☐ No
☐ No segmental box girders in inventory

24. Has a shear load rating clearly contrasted with a bridge's observed condition? Select all that apply.

☐ Yes, low load rating but with no shear distress
☐ Yes, favorable load rating but with shear distress
☐ No

25. Since concrete compressive strength plays an important role in shear capacity analysis, select the method for determining concrete compressive strength in your shear capacity calculations. Select all that apply.

- ☐ Use original design values
- ☐ Increased design values. Please describe:
- ☐ Core bridges and use in-situ strengths
- ☐ Other. Please describe:

26. Do you verify the adequacy of the shear reinforcement details in the as-builts, e.g. stirrup end anchorage, development, lap lengths, etc., when determining shear strength?

- ☐ Yes, always
- ☐ Yes, only when displaying shear distress
- ☐ No
- ☐ Other. Please describe:

27. Has inadequate shear reinforcement detailing been the source of a low shear load rating?

- ☐ Yes. Provide approximate/estimated percentage of total concrete bridge inventory, if available: %

- ☐ No

TOPIC 3: LOAD RATING SOFTWARE

28. Which primary load rating software does your agency use for load rating concrete bridges? Select all that apply.

- ☐ BrR
- ☐ BRASS
- ☐ LARS Bridge
- ☐ In-house developed software
- ☐ Other. Please list:

29. Do you use a multi-purpose analysis software (SAP2000, RISA-3D, STAAD.Pro, midas Civil, etc.) to load rate concrete bridges?

- ☐ Yes. Please list:
- ☐ No

30. What analysis points does your agency commonly use for load rating? Select all that apply.

- ☐ All 10th points
- ☐ All section change points
- ☐ Other. Please describe:

31. Does your primary load rating software routinely indicate deficient load ratings at a common location, e.g. span quarter point, contraflexure point, critical section, reinforcement spacing change, etc., for select structure types/components?

☐ Yes. Please describe:
☐ No

32. Has your agency needed to further investigate a low concrete shear load rating from your primary or secondary load rating software?

☐ Yes
☐ No

33. If answered "No" to question 32, proceed to question 34. If answered "Yes", is this need routinely identified with a particular bridge type/component or to a particular location in the bridge element?

☐ Yes. Please describe:
☐ No

34. Have you used the software Response-2000 to determine concrete shear strength for a load rating?

☐ Yes, routinely
☐ Yes, for complex or unusual situations
☐ Yes, to further investigate a low load rating
☐ No

TOPIC 4: RESEARCH

35. Has your agency funded any research or done any studies on concrete component shear evaluation, including load distribution for shear?

☐ Yes. Please provide report name(s) and/or link(s):
☐ No

36. Is your agency aware of any research on concrete component shear evaluation by others, including load distribution for shear?

☐ Yes, have implemented recommendations. Please describe:
☐ Yes, but have not implemented recommendations. Please describe:
☐ No

ADDITIONAL INFORMATION

Please provide any additional information regarding issues your agency is experiencing with concrete bridge shear load rating:

APPENDIX B. STATE DOT SURVEY RESPONSES

Q1. Which load rating method(s) does your agency use for rating concrete bridges?
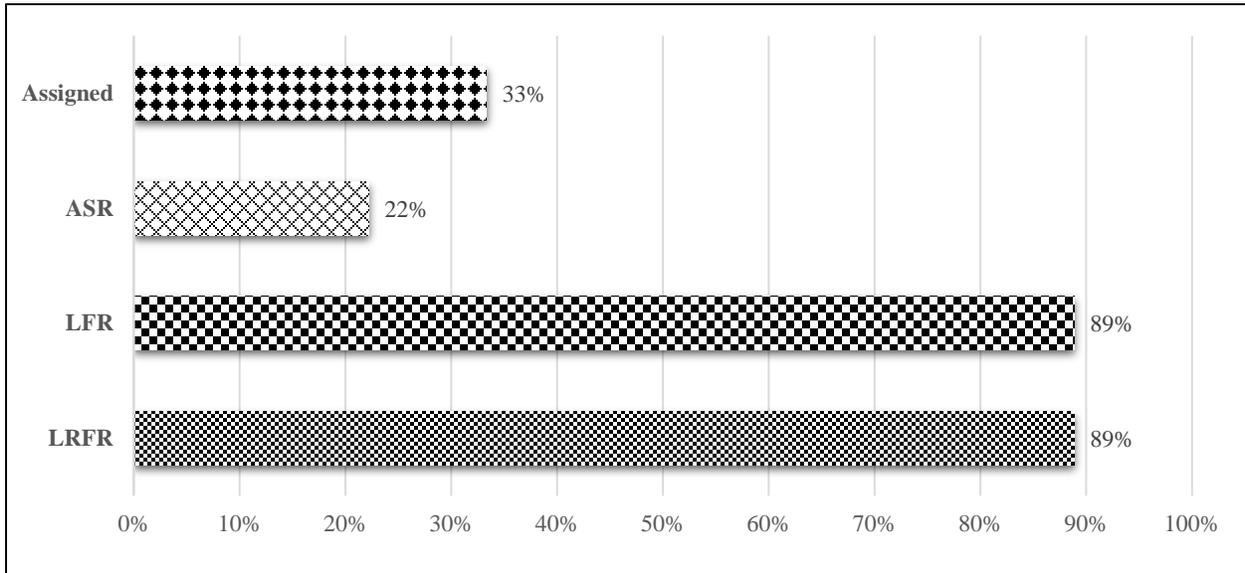
Figure 6. Response to Q1

Q2. Does your agency have a policy or requirement for concrete shear load rating above and beyond the requirements in the AASHTO MBE?
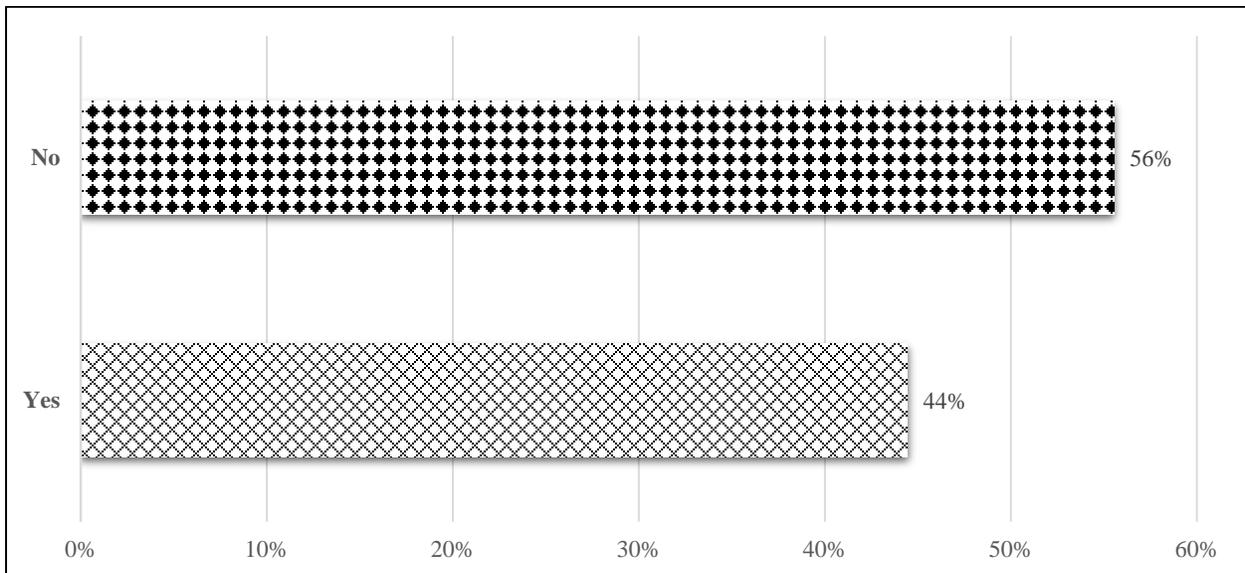
Figure 7. Response to Q2

Q3. Does your agency have a policy or requirement for designing concrete shear above and beyond the requirements in the AASHTO LRFD BDS?
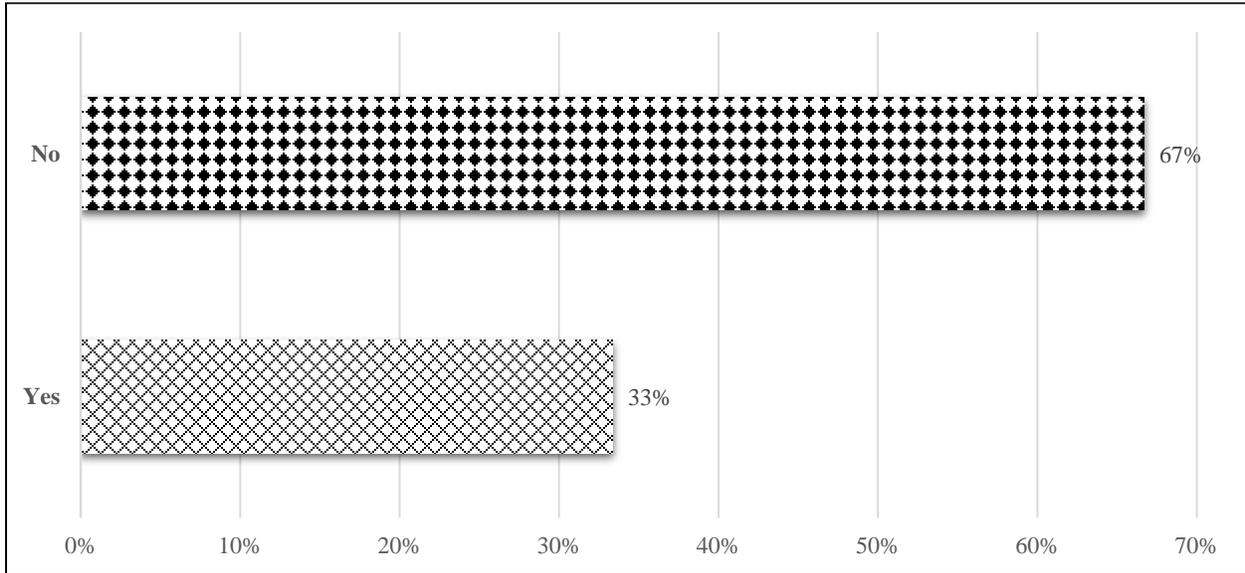
Figure 8. Response to Q3

Q4. AASHTO LRFD BDS has multiple methods for determining shear strength. Please select the shear strength methods your agency allows/uses/requires for design and load rating. Select all that apply.
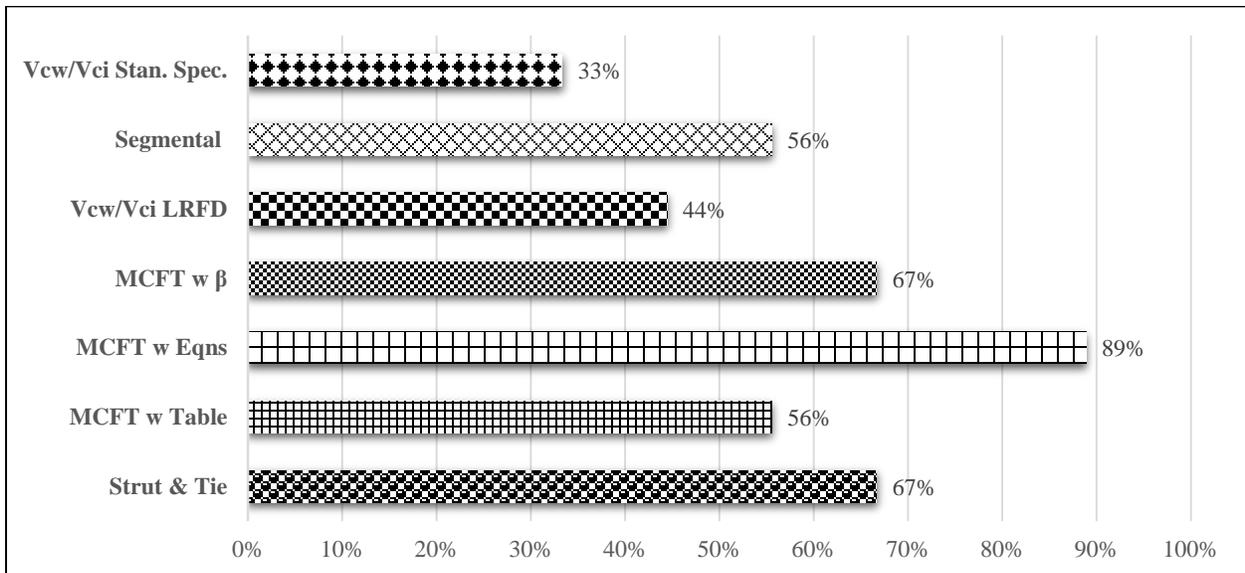
Figure 9. Response to Q4

Q5. Did your agency have a policy or requirement for designing concrete shear above and beyond the requirements in the AASHTO Standard Specifications, current or past edition?
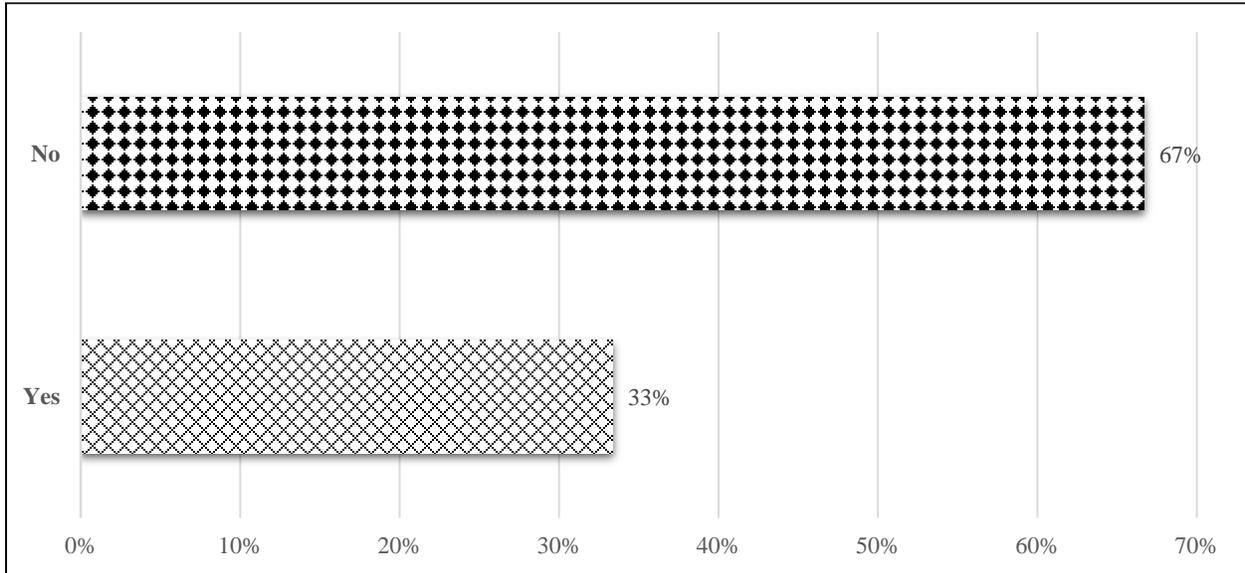
Figure 10. Response to Q5

Q6. Does your agency require a design load rating for concrete shear as part of the bridge design process?
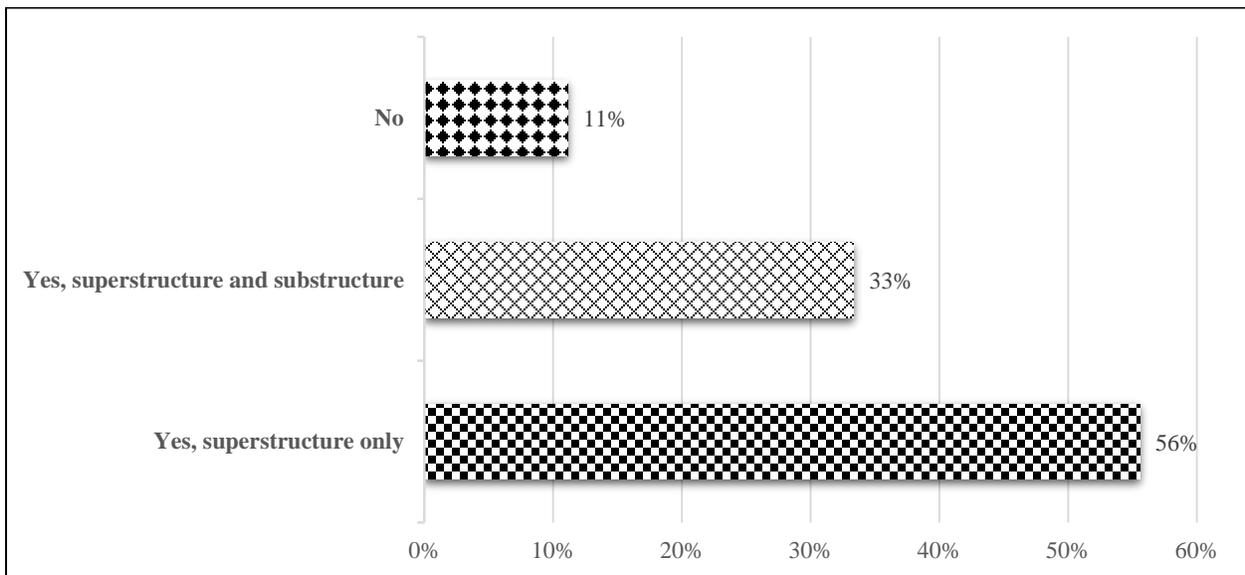
Figure 11. Response to Q6

Q7. Which concrete bridge elements do you evaluate in shear for design load ratings?
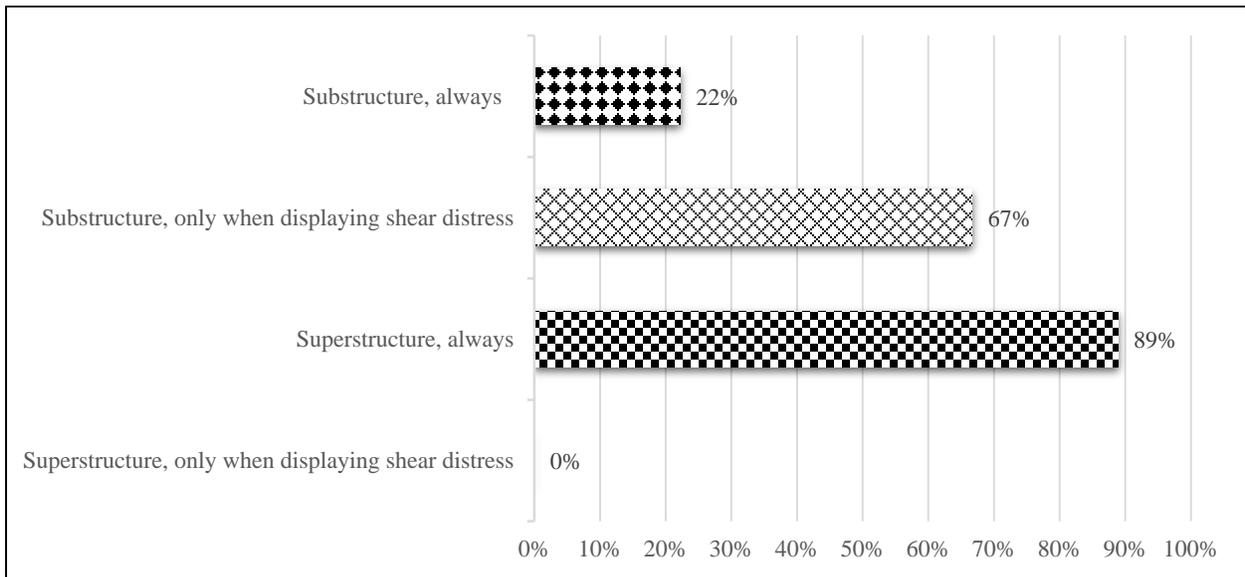
Figure 12. Response to Q7

Q8. Which concrete bridge elements do you evaluate in shear for legal load ratings, including SHVs?
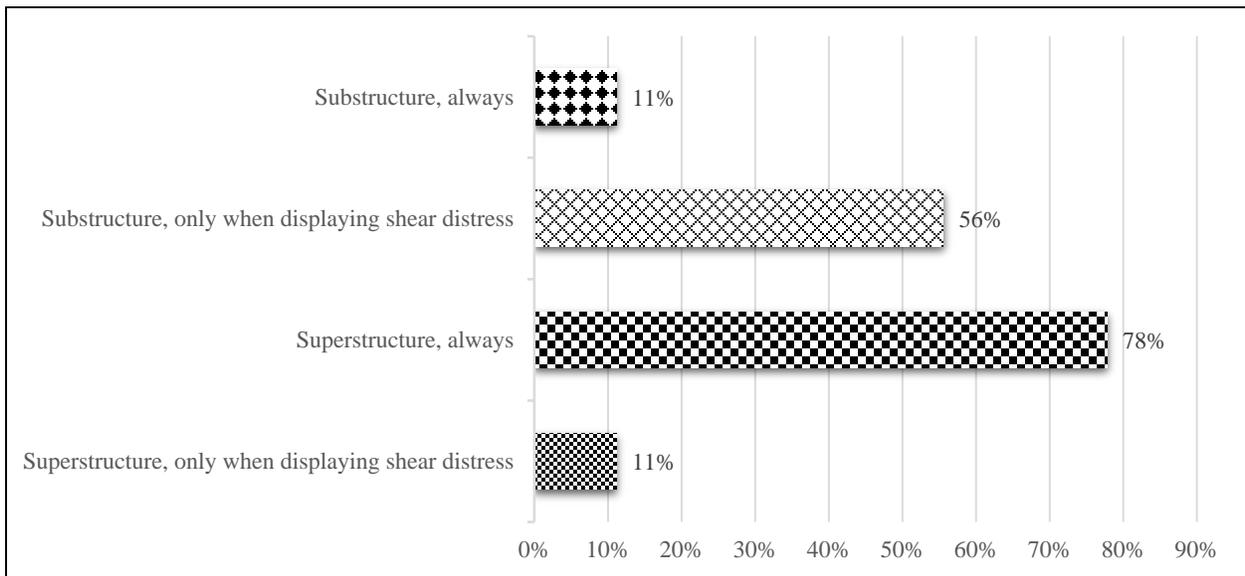
Figure 13. Response to Q8

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Q9. Which concrete bridge elements do you evaluate in shear for permit load ratings?
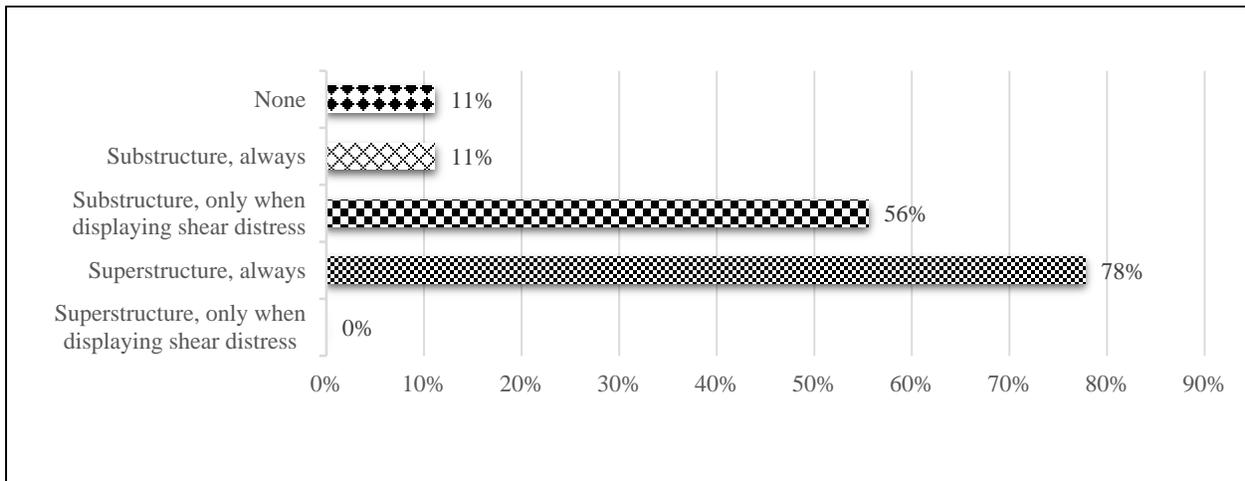
Figure 14. Response to Q9

Q10. Do shear load ratings control the overall bridge load rating? Provide approximate/estimated percentage of your concrete bridge inventory controlled by a shear load rating, if available. Select all that apply.
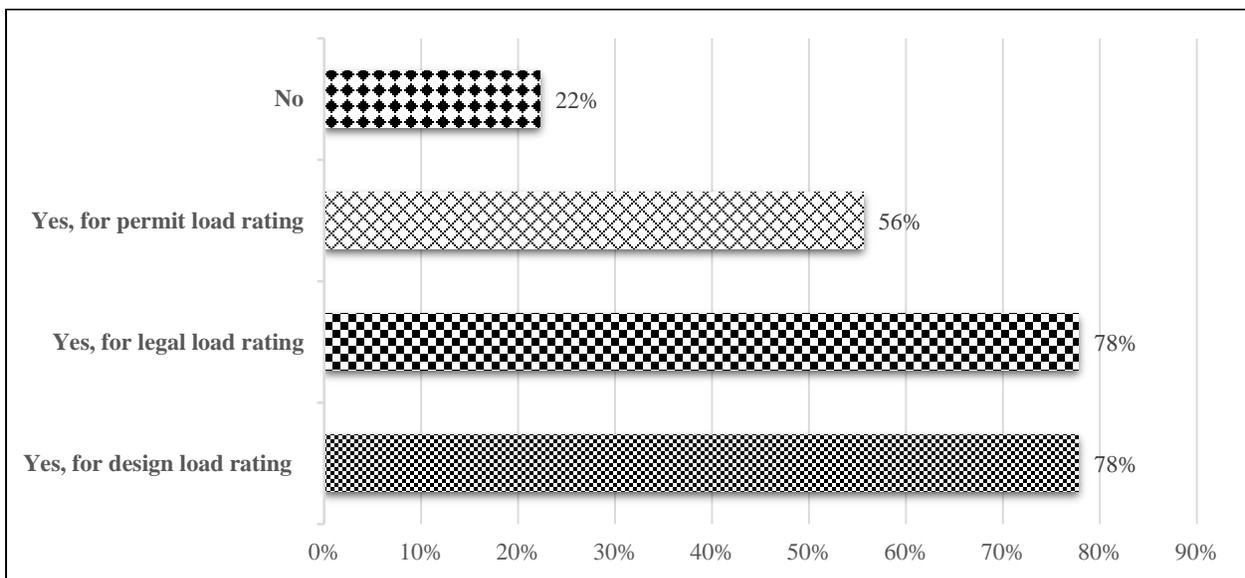
Figure 15. Response to Q10

Q11. For bridges with their overall load rating controlled by shear, in what time period were they originally built? Select all that apply.
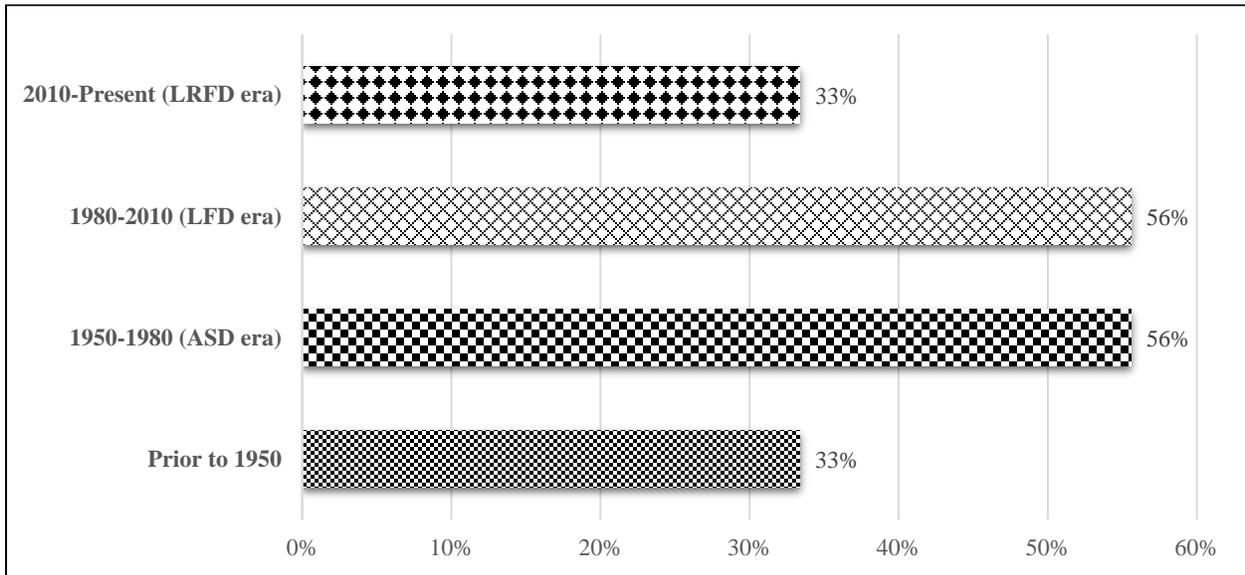
Figure 16. Response to Q11

Q12. Has the percentage of bridge load ratings controlled by shear increased, decreased, or held steady in the past 10 years?
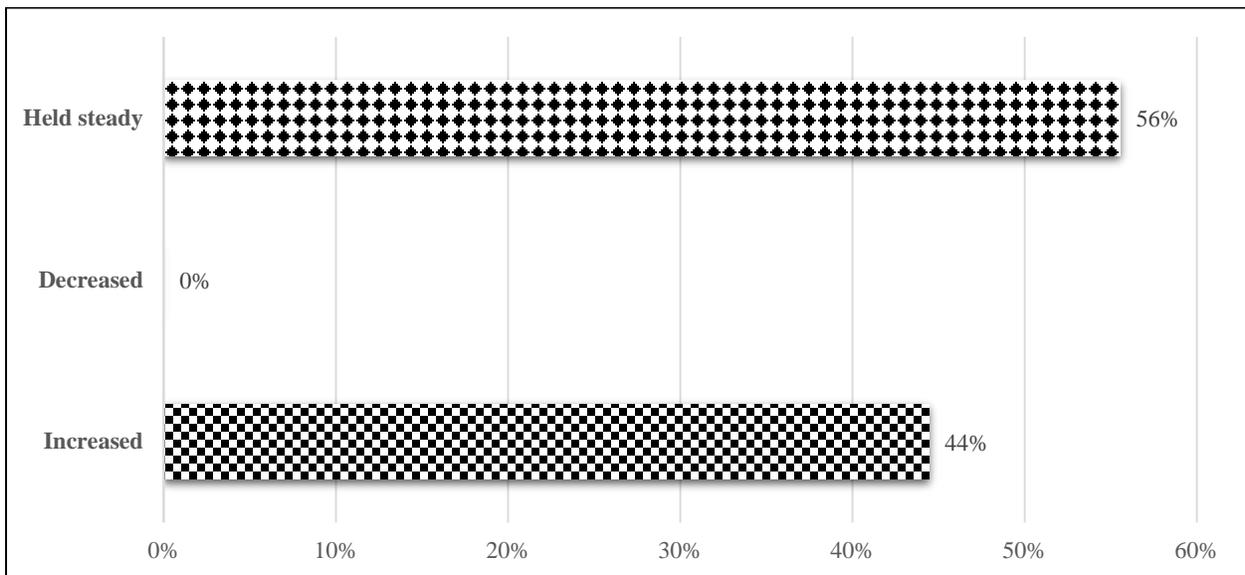
Figure 17. Response to Q12

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Q13. If the percentage of bridge load ratings governed by shear has <u>increased</u>, what are the sources of change? Select all that apply:
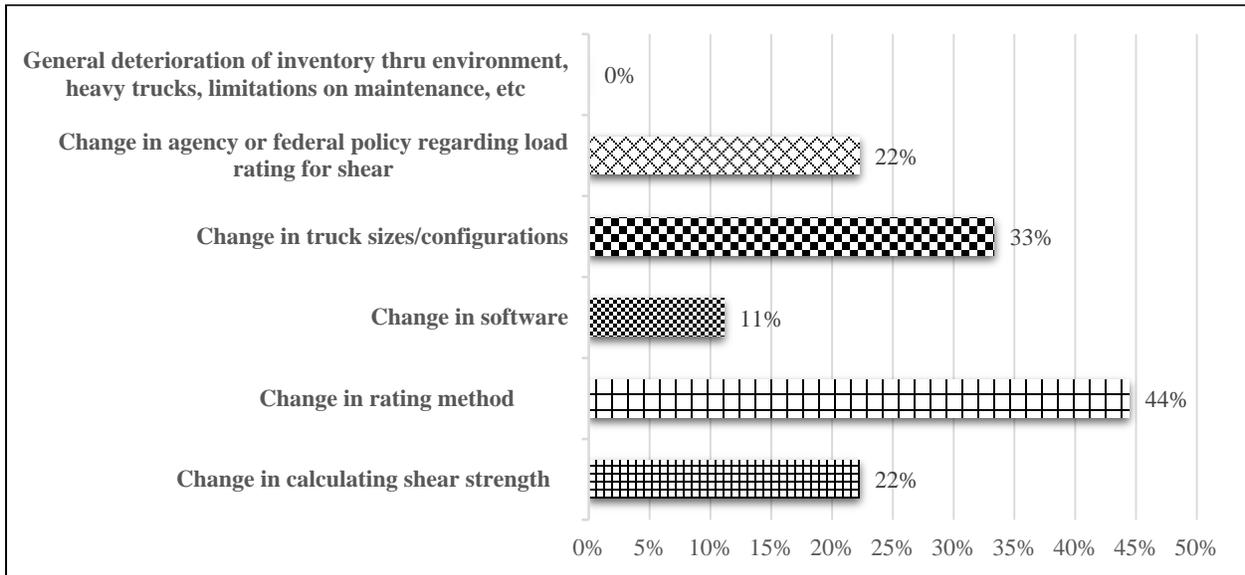
Figure 18. Response to Q13

Q14. If the percentage of bridge design load ratings governed by shear has <u>decreased</u>, what are the sources of change? Select all that apply.

No Responses.

Q15. What types of superstructures in your inventory are most likely to have a low shear load rating (RF < 1.0)? Select all that apply. Provide approximate/estimated percentage of each type, if available.
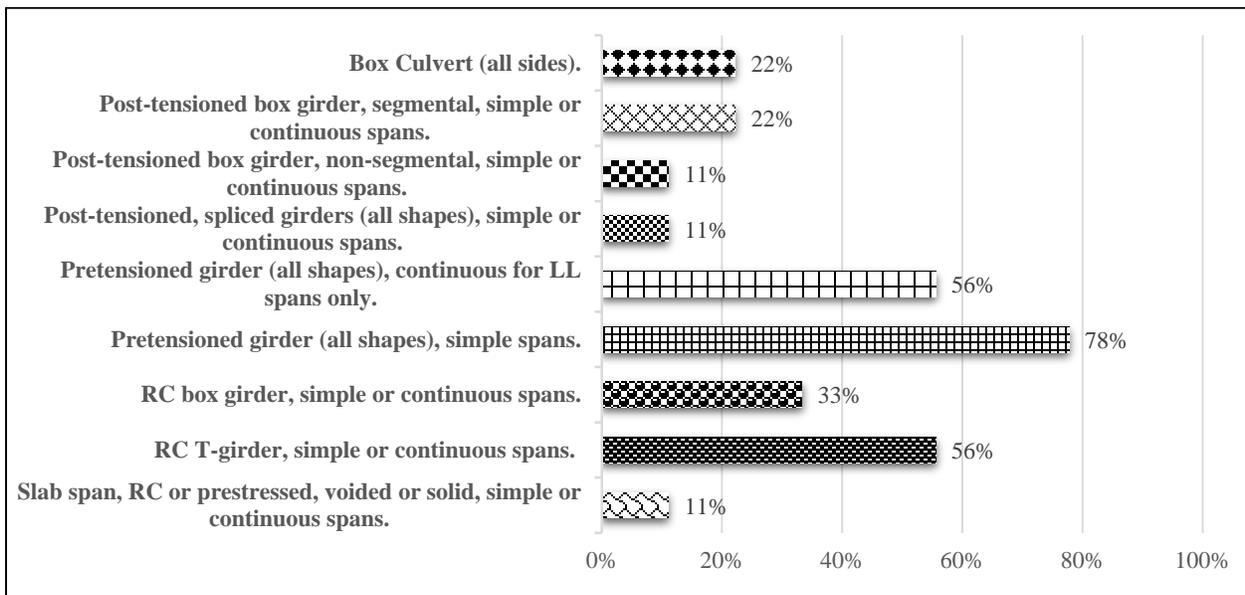
Figure 19. Response to Q15

Q16. What types of substructures in your inventory are most likely to have a low shear load rating (RF < 1.0)? Select all that apply. Provide approximate/estimated percentage of each type, if available.
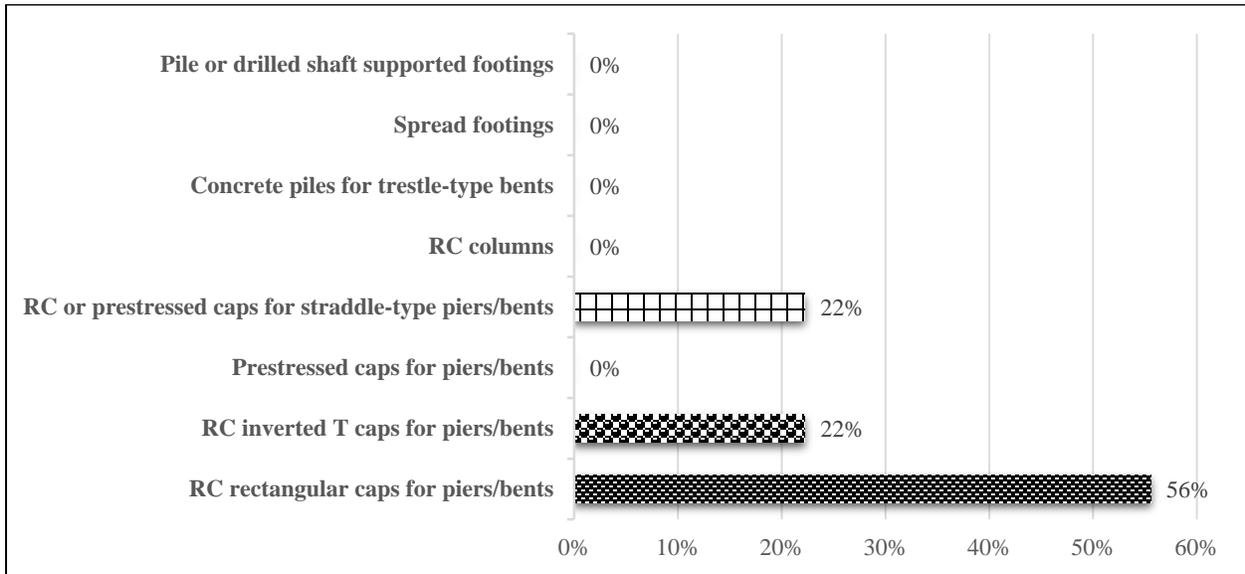
Figure 20. Response to Q16

Q17. Does your agency have concrete bridge elements common to its inventory that routinely exhibit shear distress, e.g., shear cracking?
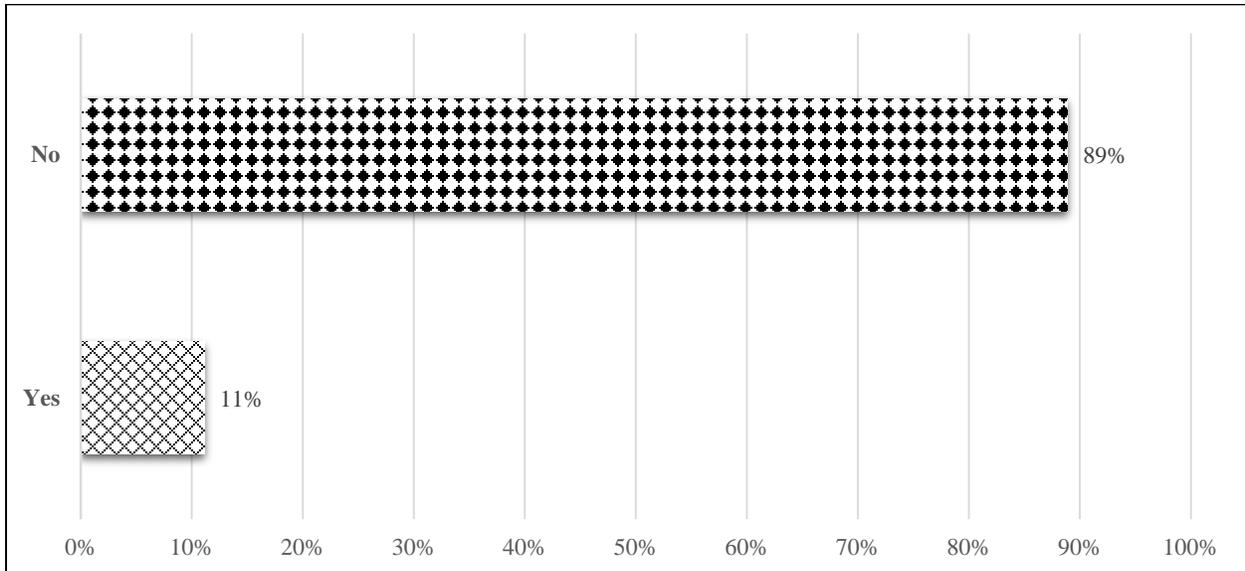
Figure 21. Response to Q17

Q18. If answered "No" to question 17, proceed to question 19. If answered "Yes" above, to what is the shear distress attributed? Select all that apply.

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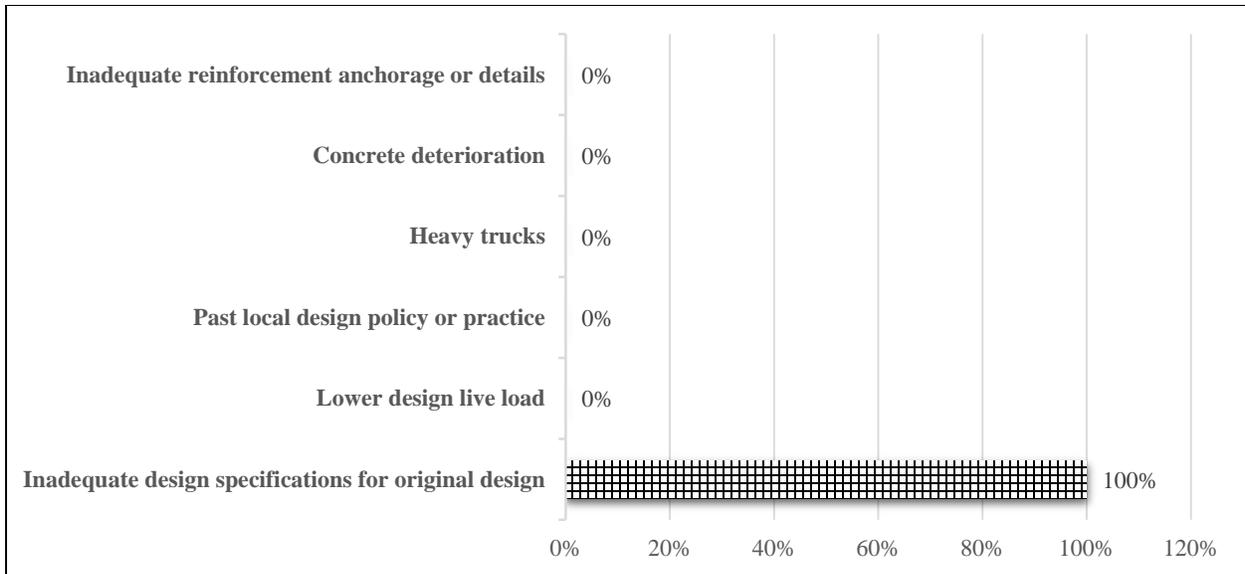
Figure 22. Response to Q18

Q19. Have any of the following shear strength methods listed below provided an unexpected low design load rating (RF < 1.0)? Select all that apply.
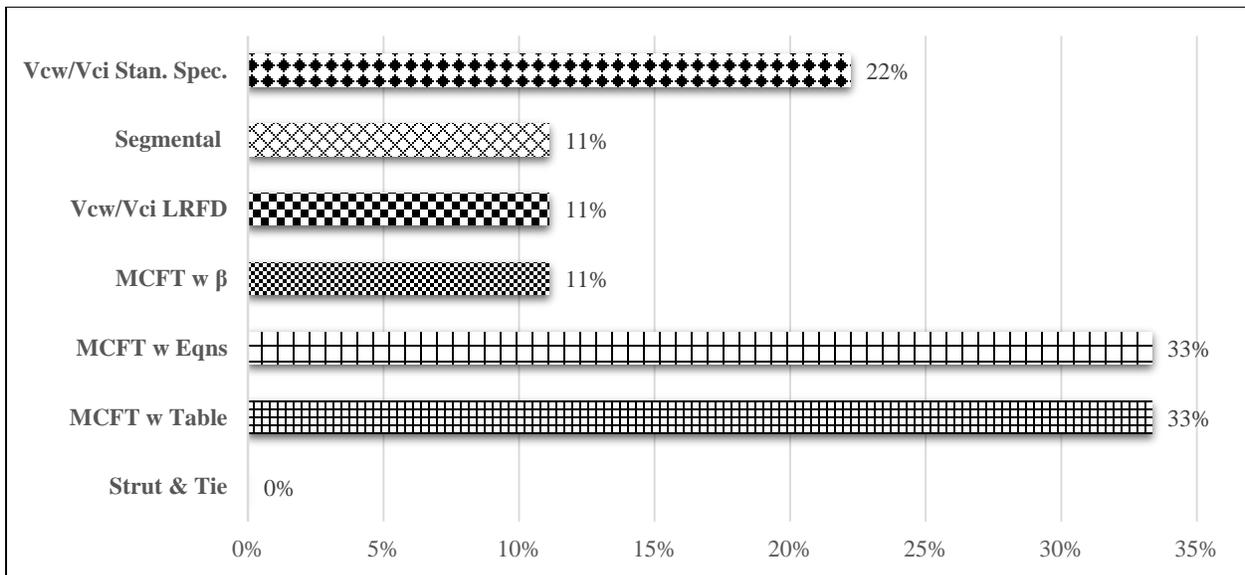
Figure 23. Response to Q19

Q20. Have you re-evaluated a low shear load rating using a different shear strength method/equation in either the AASHTO LRFD or Standard Specifications and found a significant difference?
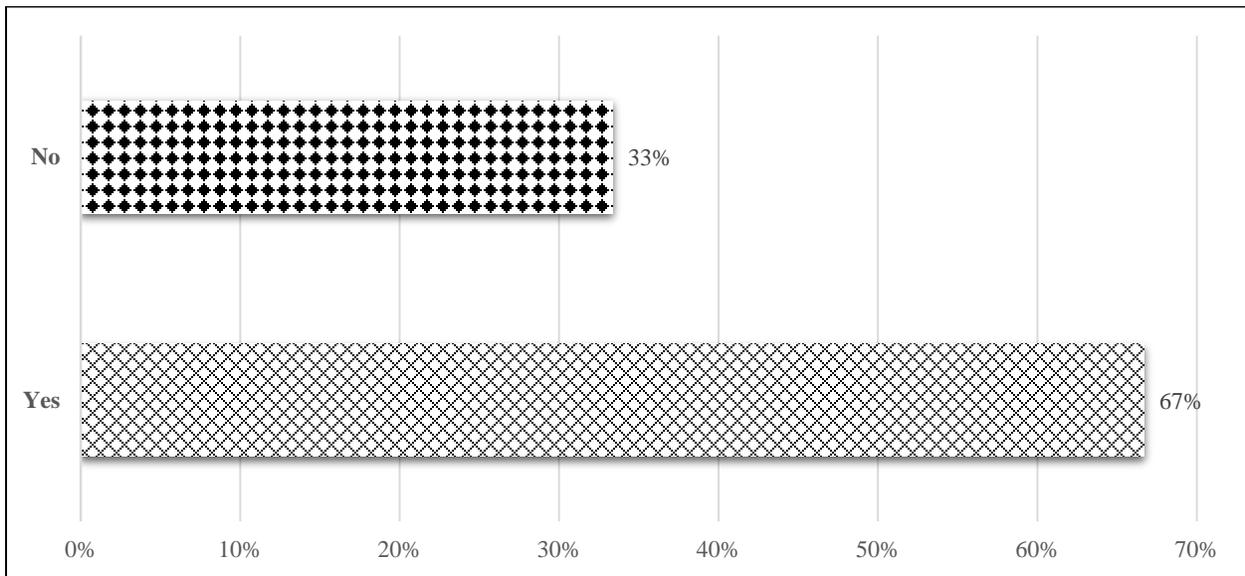
Figure 24. Response to Q20

Q21. If you calculate a shear strength using more than one of the methods in the AASHTO LRFD BDS for load rating, do you use the highest (more liberal) or lowest (more conservative) calculated shear strength in the rating?
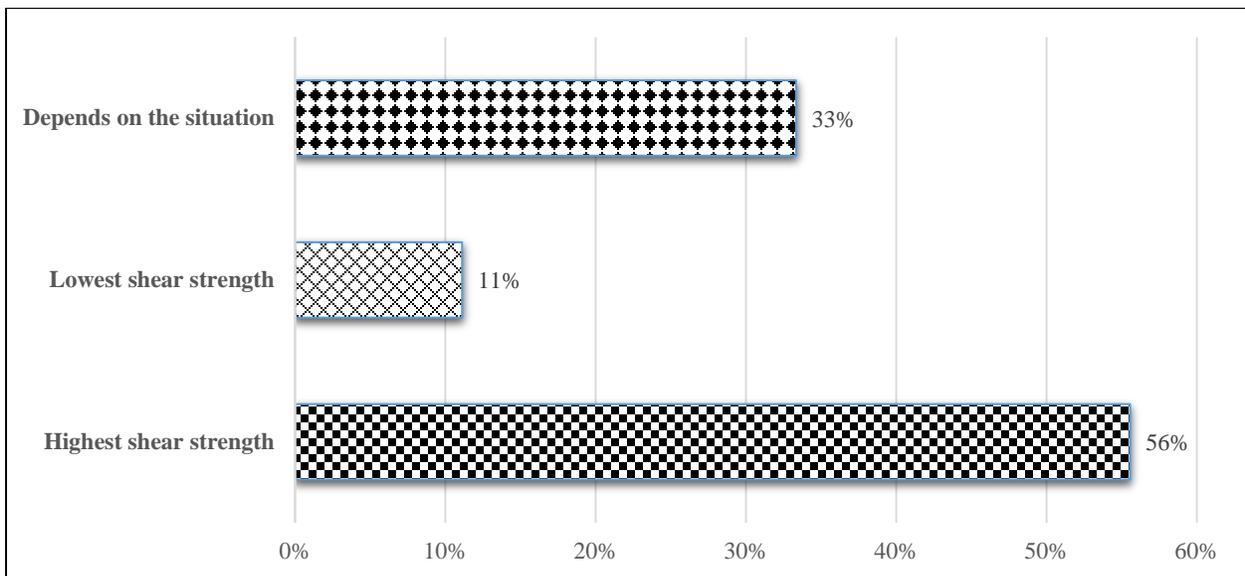
Figure 25. Response to Q21

Q22. Have any of the approximate shear live load distribution factors or skew correction factors in LRFD Article 4.6.2 been the source of a low shear load rating?
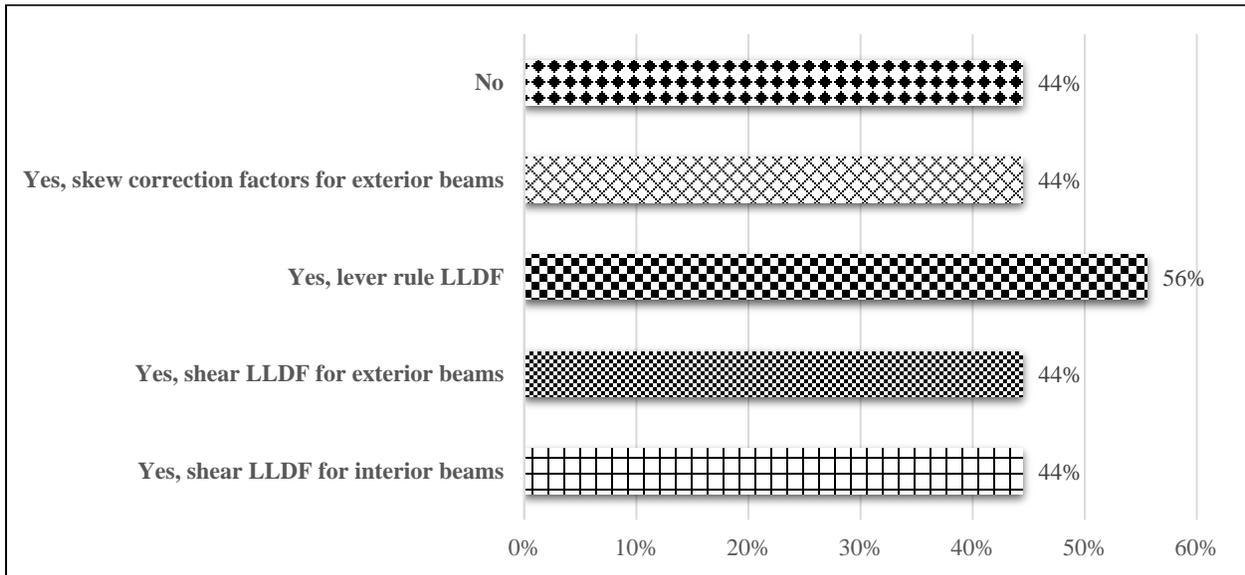
Figure 26. Response to Q22

Q23. Has the principal stress check for web cracking at Service Limit State controlled a load rating of a segmental box girder?
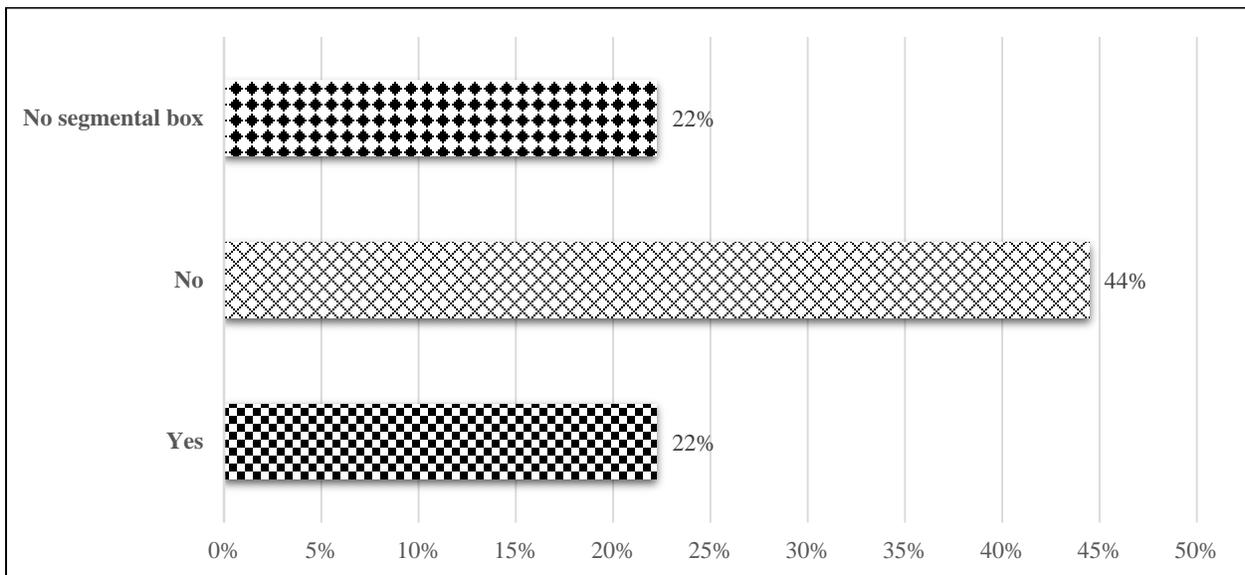
Figure 27. Response to Q23

Q24. Has a shear load rating clearly contrasted with a bridge's observed condition? Select all that apply.
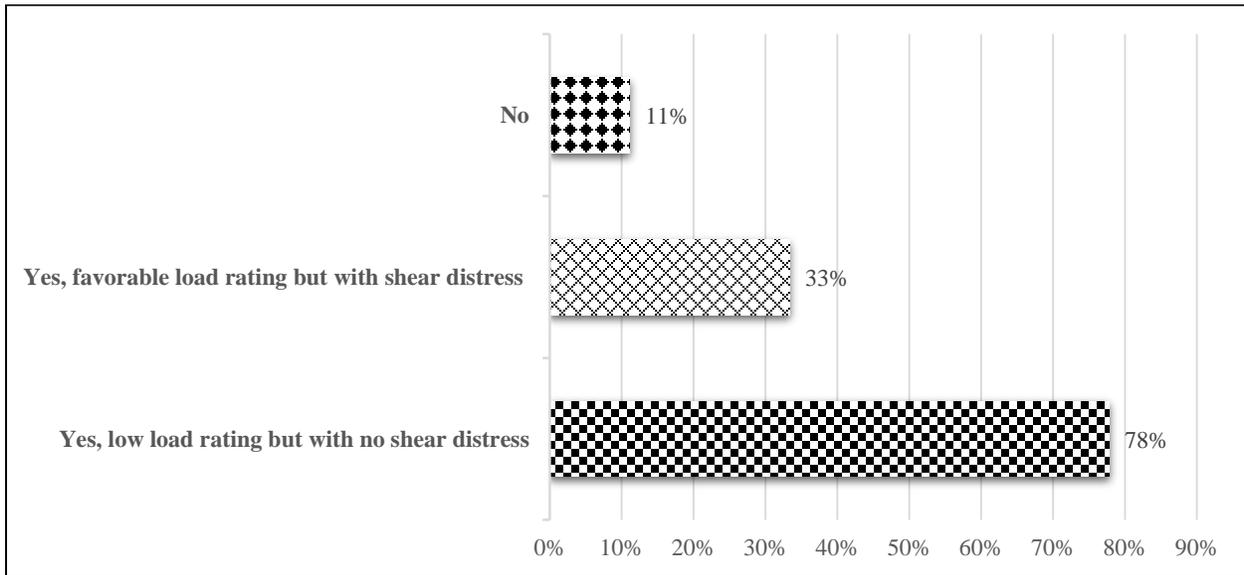
Figure 28. Response to Q24

Q25. Since concrete compressive strength plays an important role in shear capacity analysis, select the method for determining concrete compressive strength in your shear capacity calculations. Select all that apply.
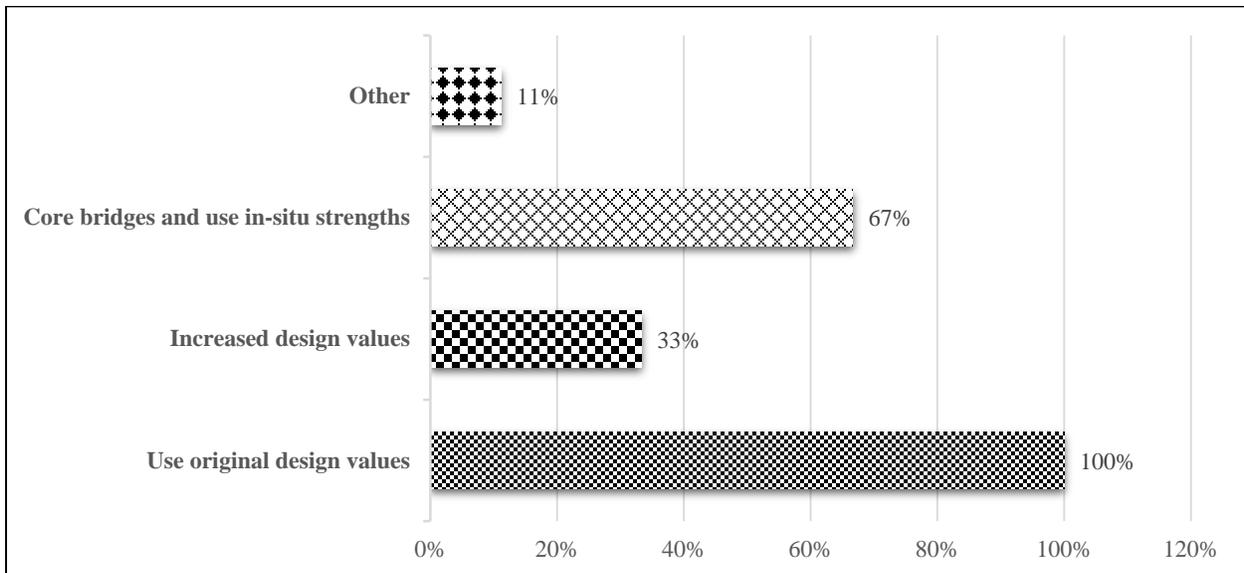
Figure 29. Response to Q25

Q26. Do you verify the adequacy of the shear reinforcement details in the as-builts, e.g. stirrup end anchorage, development, lap lengths, etc., when determining shear strength?
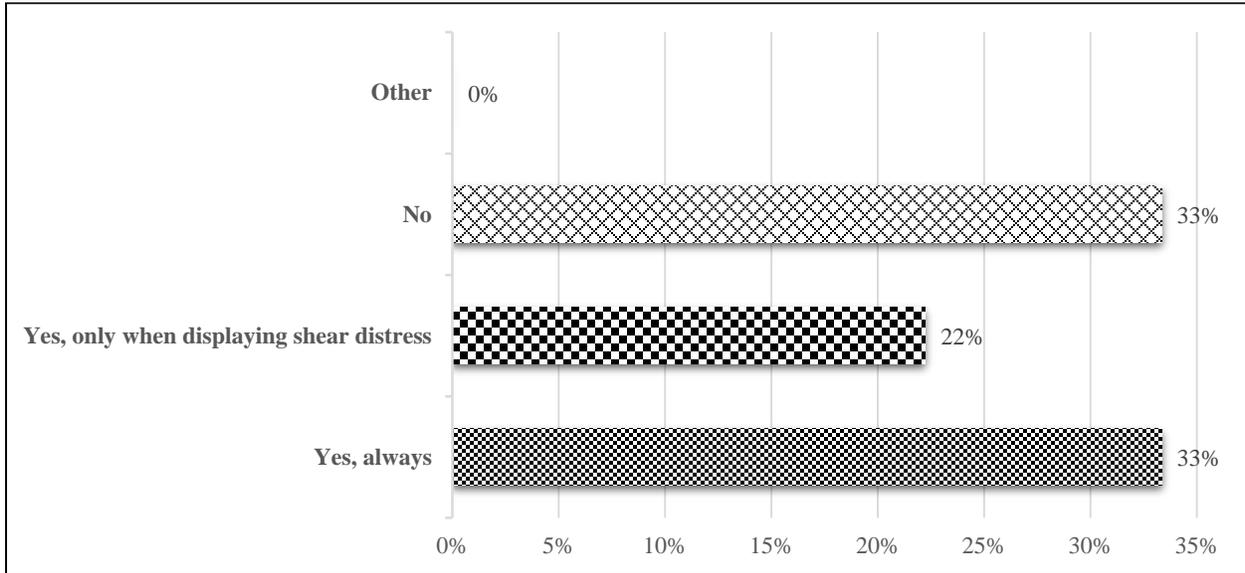
Figure 30. Response to Q26

Q27. Has inadequate shear reinforcement detailing been the source of a low shear load rating?
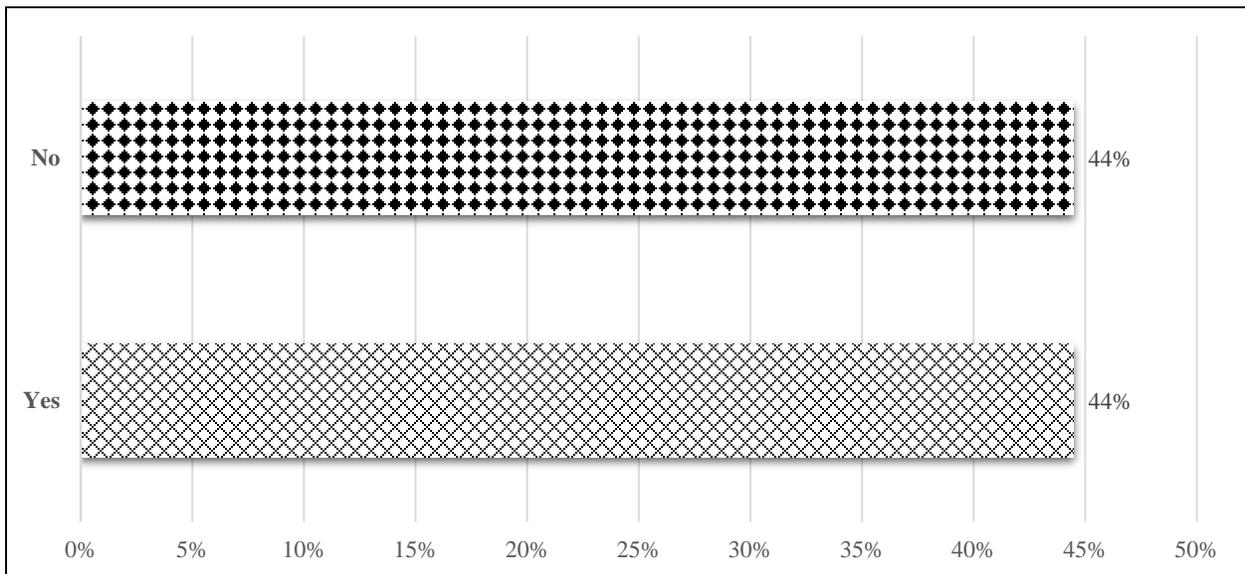
Figure 31. Response to Q27

Q28. Which primary load rating software does your agency use for load rating concrete bridges?
Select all that apply.
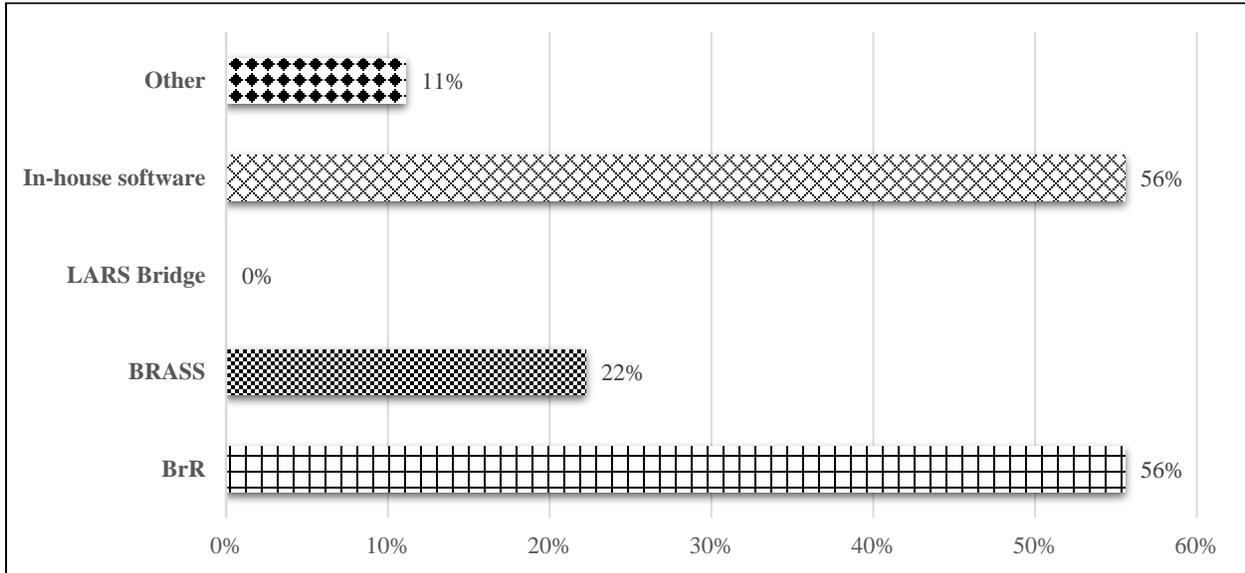
Figure 32. Response to Q28

Q29. Do you use a multi-purpose analysis software (SAP2000™, RISA-3D™, STAAD.Pro™,
midas Civil™, etc.) to load rate concrete bridges?
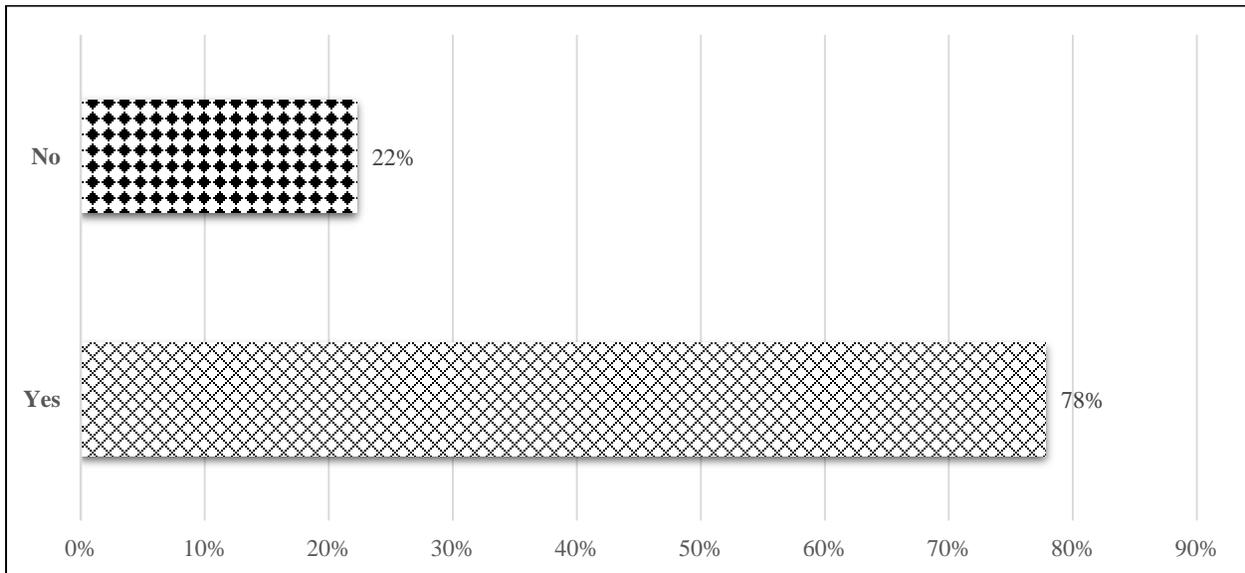
Figure 33. Response to Q29

Q30. What analysis points does your agency commonly use for load rating? Select all that apply.
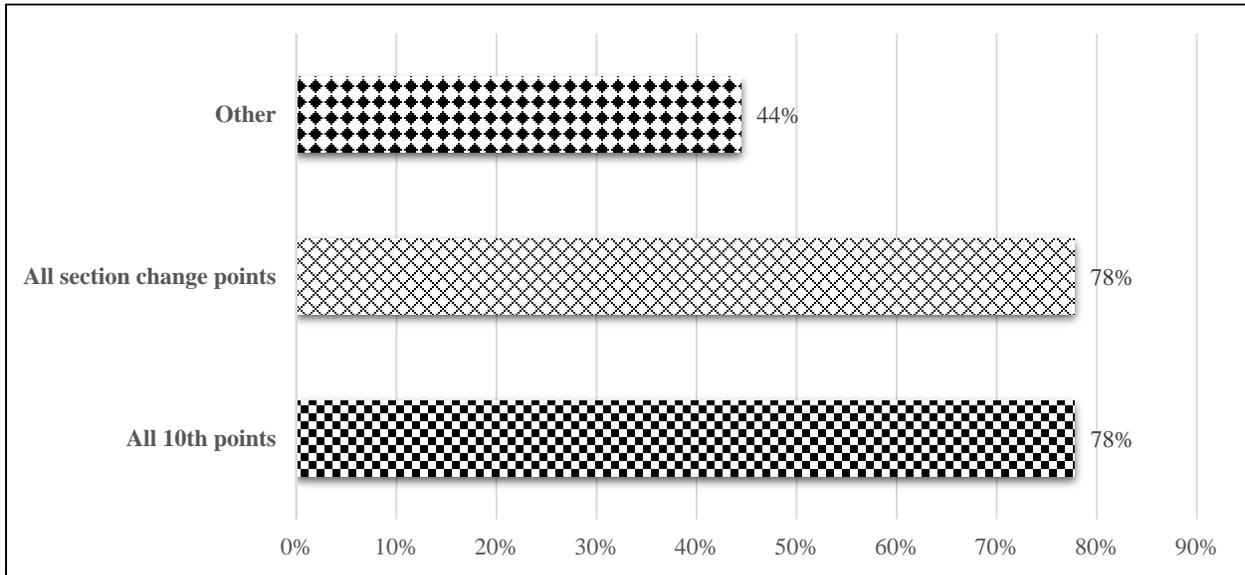
Figure 34. Response to Q30

Q31. Does your primary load rating software routinely indicate deficient load ratings at a common location, e.g. span quarter point, contraflexure point, critical section, reinforcement spacing change, etc., for select structure types/components?
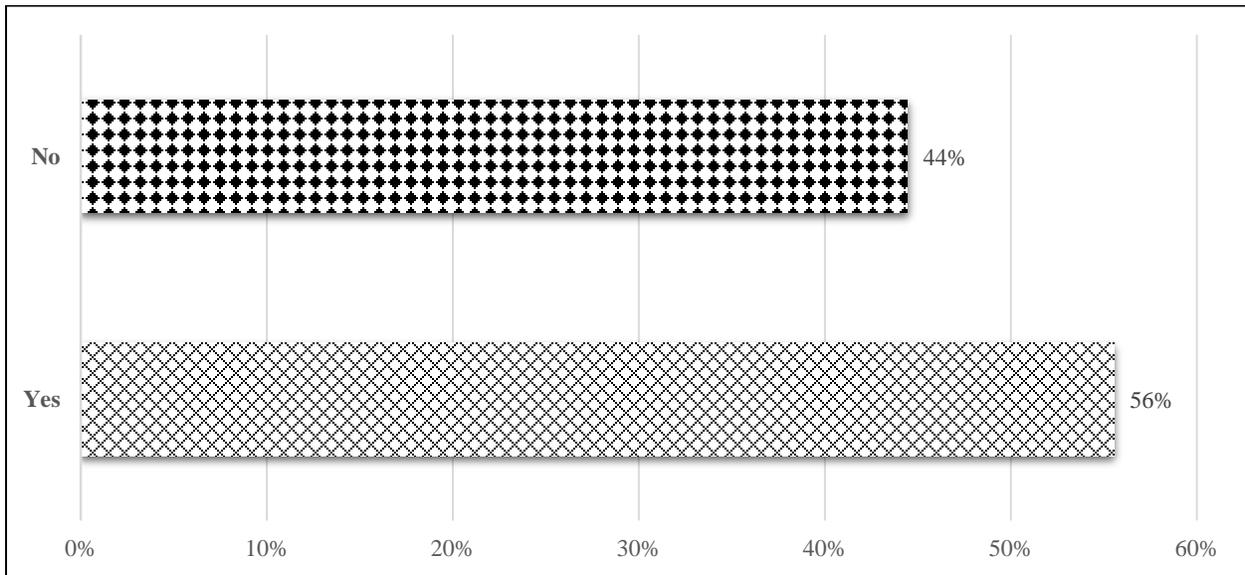

Figure 35. Response to Q31

Q32. Has your agency needed to further investigate a low concrete shear load rating from your primary or secondary load rating software?
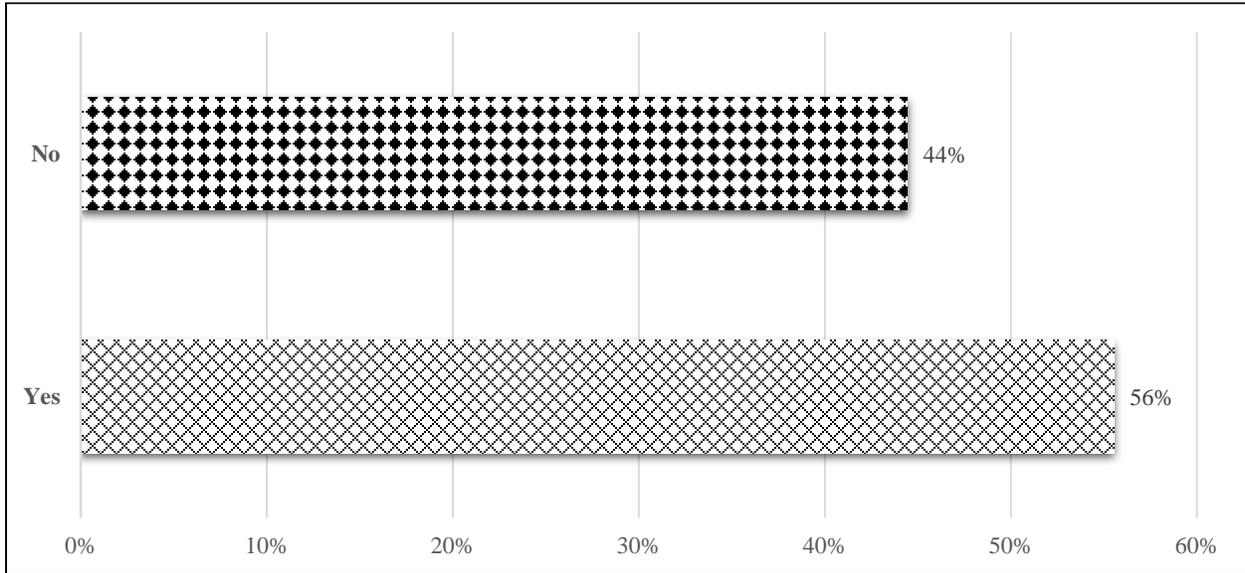

Figure 36. Response to Q32

Q33. If answered "No" to question 32, proceed to question 34. If answered "Yes", is this need routinely identified with a particular bridge type/component or to a particular location in the bridge element?
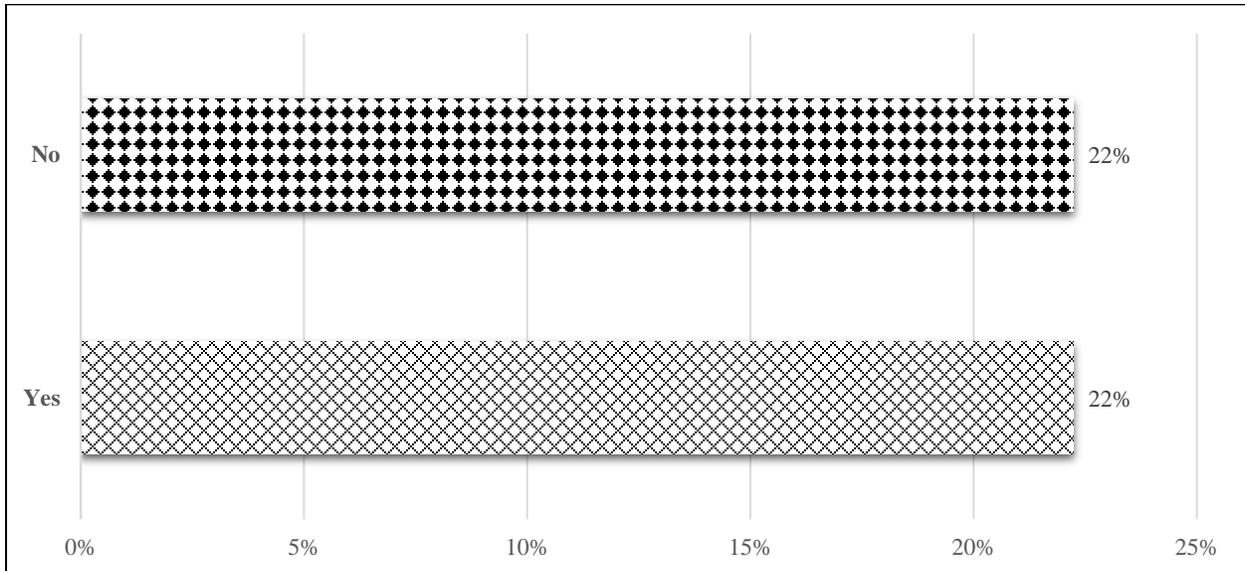

Figure 37. Response to Q33

Q34. Have you used the software Response-2000™ to determine concrete shear strength for a load rating?
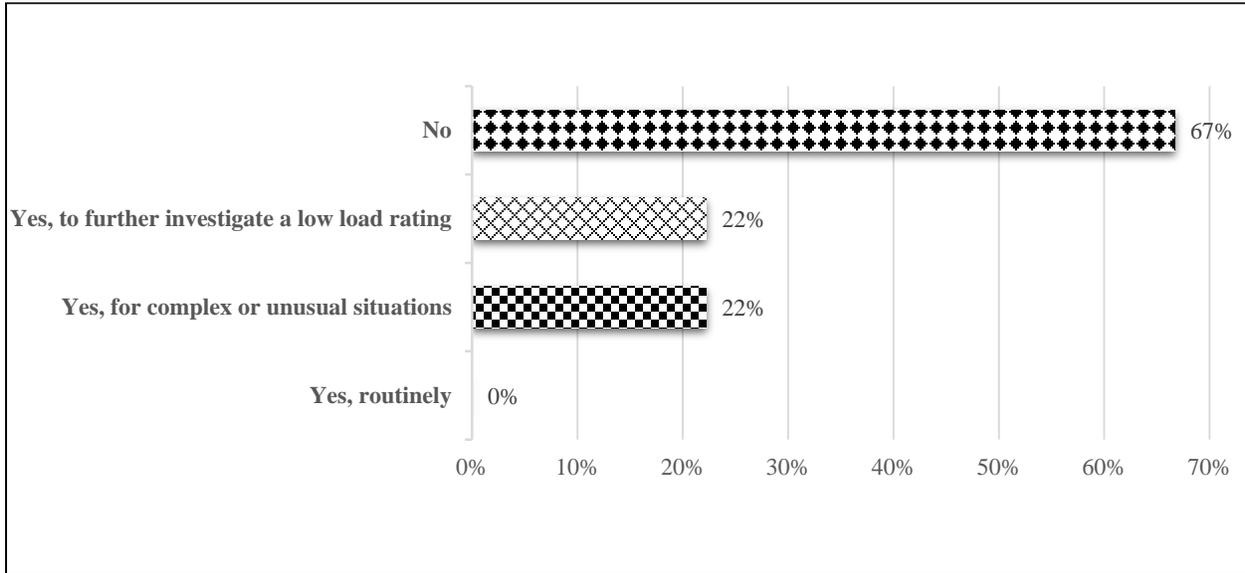
Figure 38. Response to Q34

Q35. Has your agency funded any research or done any studies on concrete component shear evaluation, including load distribution for shear?
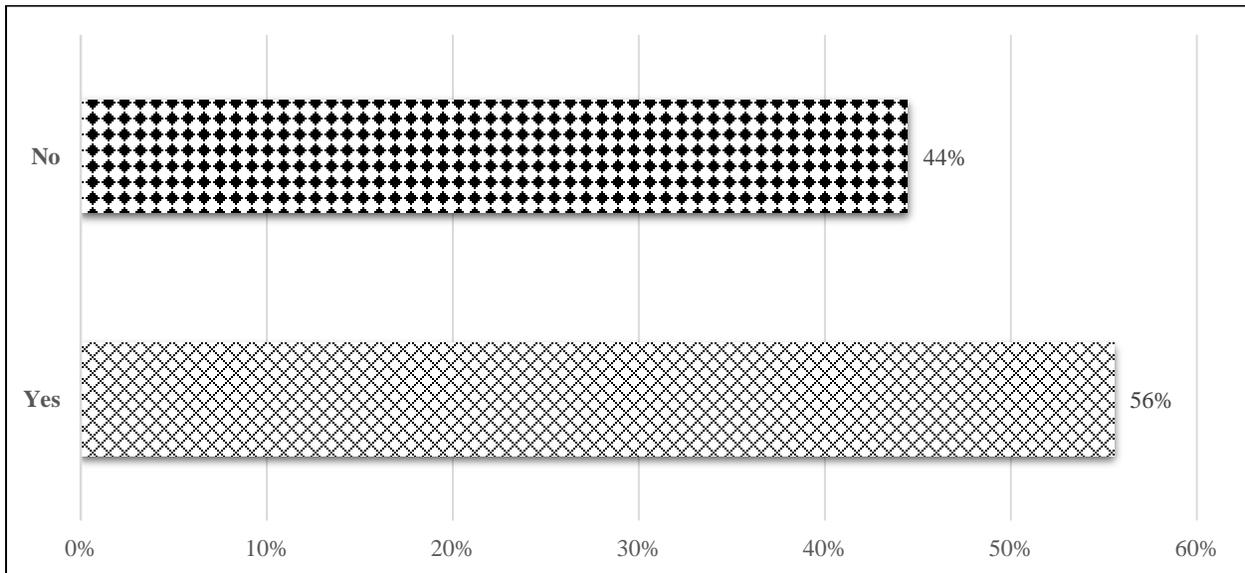

Figure 39. Response to Q35

Q36. Is your agency aware of any research on concrete component shear evaluation by others, including load distribution for shear?
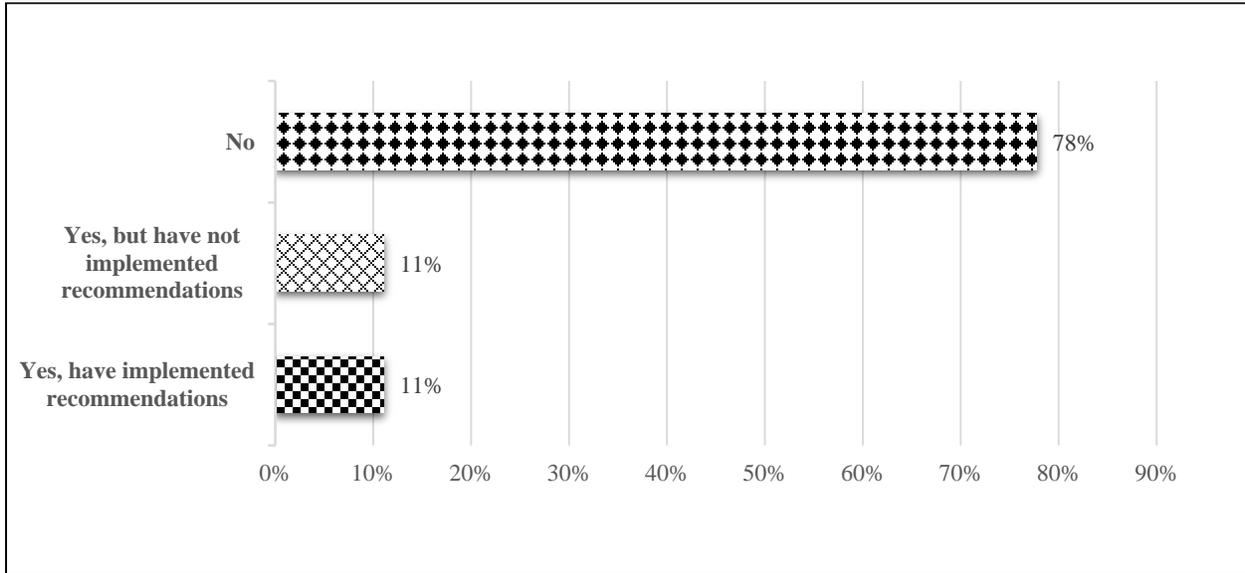
Figure 40. Response to Q36

Please provide any additional information regarding issues your agency is experiencing with concrete bridge shear load rating.

State A

1. Our Agency has been routing larger permit trucks (up to 1.2 million pounds) over our bridges based on moment only analysis until 2010. We have not seen very little shear distress on the girders. As a result of the change in requirements incorporated into the MBE in 2006, FHWA requires our agency to load rate the bridges for shear under permit loading.
When we started to evaluate our bridges for shear, significant drops in ratings were noted under LFR method based on shear capacity. When the rating method was switched to LRFR, rating factors improved; as a result we have been primarily utilizing the LRFR method to load rate our bridges. The LRFR method is still producing lower shear based rating factors for about 40 to 45% of our concrete bridges. When we increase the concrete strength by a factor of 1.2, the percent of bridges controlled by shear drops to 35%. Based on the lack of observed distress in our bridges, this is still an unacceptable percentage, resulting in potential disruption to our trucking industry.
2. There are many other minor factors that affect the overall shear capacity of the members. Per LRFR specification, whenever minimum shear reinforcement requirement is not met (in most bridges, the existing shear reinforcement is about 95% of the requirement), shear capacity is established using Table B5.2-2 which produces almost 50% of the capacity established using Table B5.2-1.1; The effect of this minimum shear reinforcement requirement is as follows: (a) an increase in concrete strength (using in situ strength) results in lower shear capacity (b) increase in stem width, i.e. girder flares, results in lower shear capacity (c) increase in rebar strength (using in situ strength) results in lower shear capacity. We believe any reduction in capacity should be prorated based on what percent of the requirement is met.
3. In general, exterior girders control the overall bridge ratings due to the fact that the LLDFs are established based on the Lever Rule. Our understanding is that the shear LLDF for exterior girders was not fully developed due to lack of funding and as a result, simplified and overly conservative Lever Rule based LLDF was adopted into the Specification. This approach may be acceptable for new bridge design, however, for the rating of in service bridges, this conservative approach does not produce a reasonable RF for shear.
4. We believe that the Response-2000 software should be made available to the AASHTOWare BrR developers for inclusion in the shear capacity modules.
5. Considering the fact that in general, the nation's reinforced concrete bridges do not exhibit significant signs of shear distress, and that our material and design specifications are consistently based on large factors of safety and conservative statistical probabilities, we, as an engineering community should consider incorporating some level of a performance factor into bridge load rating analysis. For example, if a bridge shows no signs of distress, has been in service for decades, and has a known average daily truck traffic (ADTT) history, the results of a low rating analysis should be reviewed using engineering judgment and if applicable, a performance factor similar to the Condition Factor listed in the MBE Table 6A.4.2.3-1, should be considered where the factors range from 1.0 and higher.

State B

Agency still seeks a methodology that reliably approximates true capacity.

State C

One of the early issues we encountered with LRFR has to do with disconnect between design and load rating philosophies. We dubbed the issue as the Longitudinal Tension Check. Article 6A.5.8 in the MBE states that when using the Modified Compression Field Theory (MCFT) for the evaluation of concrete shear resistance, the longitudinal reinforcement should be checked for the increased tension caused by shear, in accordance with LRFD Design Article 5.8.3.5.

LRFD 5.8.3.5 states that at each section the tensile capacity of the longitudinal reinforcement on the flexural tension side of the member shall be proportioned to satisfy the equation shown within the article. For a bridge designer, this is not a problem. The designer will design the flexural reinforcement at a given section to resist the moment demand. They will then move on and design the shear reinforcement for the given shear demand. Since moment and shear are interactive, the next step is to perform this longitudinal reinforcement check to see if the additional tension created by the shear doesn't exceed the limits of this equation. If this check fails, the designer simply goes back and adds additional flexural reinforcement area then recalculates their moment and shear capacities. Then they perform this check again, and keep repeating these steps until the check passes.

In load rating, we do not have the luxury of being able to add more reinforcement to the existing bridge. We can only account for the reinforcement that is already built into the structure. The Manual for Bridge Evaluation does not give any guidance on what to do if this check fails, it simply says to perform the check.

We had a lengthy discussion with several bridge engineers within [our DOT] and some of our consultants, and we never could come to any consensus as to what should be done. Some felt that since the check is checking the tension in the longitudinal reinforcement, that the moment capacity should be reduced by the amount that the tension check was failing. Others felt that since shear was the cause that we reduce the shear capacity, and some felt that both the moment and shear should be adjusted. But they had no idea by what amount. We really didn't like any of the ideas because we didn't have any real data to base any capacity adjustments on, plus research tests have already proven that ultimate moment and shear capacities are being predicted fairly accurately by the analysis methods. So imposing some sort of random reduction to the capacity didn't make a lot of sense.

[Our DOT] uses BRASS-GIRDER for the load rating of most of our slab and beam bridges. Within the general brass output, which is usually a few hundred pages long, there was no reference to the longitudinal reinforcement check or anything referencing LRFD 5.8.3.5. BRASS gives the user the option to request detailed intermediate output at a specific analysis point for the AASHTO Specification checks in a separate text file. This file in itself is a few hundred pages long because it performs all of the capacity and specification checks for every limit state and for every live load vehicle being analyzed. The longitudinal tension check was buried in several places in the output since it was being done for each vehicle and each limit state. BRASS was following the MBE to the letter… it performed the check and then moved on regardless of what the result was.

Due to the extensive amount of output that was being generated by requesting the program to create intermediate output at every analysis point, and then having to search through each output file to find the correct check that corresponded to each vehicle and limit state, we were having a difficult time trying to determine how common this issue was going to be. So we ended up contacting WYDOT to report our issue and to see if they knew of a way to have the program give some sort of summary within the general output showing the results of this check for every analysis point and every vehicle. They were receptive of our issue and agreed that the program should be modified to show some sort of result for this check.

They then took it a step further and modified the software to produce a rating factor based on the longitudinal tension check. So now within the general output, rating factors for the longitudinal tension check are being reported for every vehicle at every analysis point. For each vehicle, the software is calculating a rating factors for four different force combinations: Maximum moment with concurrent shear; Minimum moment with concurrent shear; Maximum shear with concurrent moment; and Minimum shear with concurrent moment.

After WYDOT implemented the longitudinal tension check rating factors in BRASS, we have found that several of our older bridges that were not designed by LRFD have low rating factors for this check. Many of the bridges have good rating factors for shear and moment. [Our DOT] was reluctant to spend the money to strengthen a bridge that doesn't show much distress for a potential failure mode that we don't have any historical examples of bridges failing to. From a management standpoint, [our DOT] decided to only allow the tension check to control the load rating if the NBI superstructure condition was less than 5. So unless the bridge is showing distress, we are not going to let it control the load rating.

State D

Question #7: None of the multiple choice answers fit our condition. For the large majority of our designs, the super/substructure get an "assigned load rating" that matches the design code (HL93). However, we will investigate other structural elements on a case by case basis depending on the situation. This is especially true for existing structures that are being replaced in phases, potential widening, etc.

Question #8 and #9: None of the available answers match agency's approach. Bridge elements that show distress are always looked at, but some others (but not all) are looked at as well. Here again, the type of structures that are rated are dependent upon the unique case. Typically, legal loads are evaluated by comparing their force effects to that of an HS20 or HL93 load. Based upon this comparison, we can identify the critical structure types/span lengths that require a more refined load rating. A similar approach is used regarding permit (super heavy) loads.