Modified Compression Field Theory (MCFT) for Shear Load Rating – Pretensioned Example

Publication No. FHWA-RC-24-0005 August 2024

U.S. Department of Transportation
Federal Highway Administration

Federal Highway Administration
RESOURCE CENTER
Office of Innovation Implementation

FOREWORD

The modified compression field theory (MCFT) has been used for the past few decades to provide safe and consistent shear design of reinforced and prestressed concrete bridge members. While it has been used for new design, its use for shear load rating can be more challenging. Recommendations provided in recent reports have created a framework for using MCFT for shear load rating, but more guidance and rating examples are needed to help in its widespread adoption.

The information provided in this report provide background, context, and foundational knowledge to bridge owners, designers, and load raters interested in using MCFT for shear load rating. This report will be of interest to bridge owners, designers, and load raters looking for a more consistent and accurate way of estimating the shear resistance of prestressed concrete members.

Shay K. Burrows P.E.
Resource Center Director
Federal Highway Administration
Office of Innovation Implementation

TECHNICAL REPORT DOCUMENTATION PAGE

1. Report No. FHWA-RC-24-0005	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Modified Compression Field Theory (MCFT) for Shear Load Rating – Pretensioned Example		5. Report Date August 2024 6. Performing Organization Code
7. Author(s) David Garber (ORCID: 0000-0001-9561-0538)		8. Performing Organization Report No.
9. Performing Organization Name and Address Office of Innovation and Workforce Solutions Federal Highway Administration 1200 New Jersey Avenue, SE Washington, D.C. 20590		10. Work Unit No. 11. Contract or Grant No.
12. Sponsoring Agency Name and Address Office of Innovation and Workforce Solutions Federal Highway Administration 1200 New Jersey Avenue, SE Washington, D.C. 20590		13. Type of Report and Period Covered 14. Sponsoring Agency Code HIT
15. Supplementary Notes		
16. Abstract This report explains in detail how to use the modified compression field theory (MCFT) for shear load rating of pretensioned concrete members. The report includes a summary of MCFT theory and application, possible shear failure mechanisms that may occur in pretensioned members, and a recently recommended methodology for an iterative process for using MCFT for shear load rating. Additional details and explanations are provided for application of this methodology for shear load rating utilizing MCFT for pretensioned concrete members. Details for a complete shear load rating example are provided for a pretensioned box beam at the critical section, the effective shear depth away from the face of the support, and along the length of the beam.		
17. Key Words Modified compression field theory, MCFT, shear load rating, pretensioned concrete	18. Distribution Statement	
19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages 70
		22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed pages authorized

CONTENTS

LIST OF FIGURES

LIST OF TABLES

NOTATION

A_{ct} area of concrete on the flexural tension side of the member (inch^2)

A_{cv} area of concrete considered to be engaged in interface shear transfer (inch^2)

A_g gross area for section (inch^2)

A_p total area of prestressing strands (inch^2)

$A_{p,0.5in}$ area of 0.5-inch diameter prestressing strand (inch^2)

$A_{p,harped}$ total area of harped prestressing strands (inch^2)

A_{ps} total area of prestressing strands on flexural tension side of member (inch^2)

A_s total area of non-prestressed tension reinforcement (inch^2)

A_{st} total area of non-prestressed reinforcement on flexural tension side of member (inch^2)

A_v area of transverse reinforcement within distance s (inch^2)

A_{vf} area of interface reinforcement crossing the shear plane within the area A_{cv} (inch^2)

a depth of compression block (inch)

a_g maximum aggregate size (inch)

b_{bf} width of bottom flange (inch)

b_e effective flange width (inch)

b_{tf} width of top flange (inch)

b_v width of web for use in shear design or load rating (inch)

C compression resultant force used in shear design or load rating (kips)

c cohesion factor for interface shear transfer (ksi)

DFM distribution factor for moment

DFV distribution factor for shear

d distance from extreme compression face to centroid of non-prestressed reinforcement (inch)

d_b diameter of reinforcement or prestressing strand (inch)

d_e depth of center of gravity of steel; effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (inch)

d_p distance from extreme compression face to centroid of prestressing strands (inch)

d_v effective shear depth, distance between compressive and tensile force resultants (inch)

E	modulus of elasticity (ksi)
E_c	modulus of elasticity of concrete (ksi)
E_{ci}	modulus of elasticity of concrete at transfer (ksi)
E_s	modulus of elasticity of steel reinforcement (ksi)
E_p	modulus of elasticity of prestressing strands (ksi)
e_p	strand eccentricity (inch)
f_1	principal tensile stress in concrete (ksi)
f'_c	compressive strength of concrete for use in design or load rating (ksi)
f'_{ci}	compressive strength of concrete at release for use in design or load rating (ksi)
f_{pbt}	stress in prestressing strands immediately before transfer (ksi)
f_{pc}	axial precompression provided by prestressing at transfer (ksi)
f_{pe}	effective stress in prestressing (ksi)
f_{pj}	stress in prestressing strands at jacking (ksi)
f_{po}	a parameter taken as modulus of elasticity of prestressing steel multiplied by the locked-in difference between the prestressing steel and the surrounding concrete (ksi). For the usual levels of prestressing, a value of $0.7f_{pu}$ will be appropriate for both pretensioned and post-tensioned members.
f_{ps}	stress in prestressing strands (ksi)
f_{pu}	ultimate strength of prestressing strands (ksi)
f_{py}	yield strength of prestressing strands (ksi)
f_s	stress in steel (ksi)
f_x	concrete stress in the longitudinal direction (ksi)
f_y	yield strength of transverse reinforcement (ksi); concrete stress in the vertical direction for MCFT derivation (ksi)
h	section height (inch)
IM	impact factor for design truck
I_g	moment of inertia for gross section (inch^4)
i	iteration number

K factor accounting for precompression in the calculation of nominal shear resistance provided by the concrete using the Alternative Shear Design Procedure in AASHTO LRFD BDS Article 5.12.5.3.8

K_1 correction factor for modulus of elasticity to be taken as 1.0 unless determined by a physical test, and as approved by the owner; interface shear factor associated with $K_1 f'_c A_{cv}$ limit

K_2 interface shear factor associated with $K_2 A_{cv}$ limit

k_d beam shape factor for horizontal shear resistance, 1.0 for I-beam, box-beams, and U-beams with typical reinforcement details and 0.8 for U-beams with detail described in in Hovell et al. (2013)

L span length (ft)

L_{beam} beam length (ft)

LHS left hand side of AASHTO LRFD BDS Eqn. 5.7.3.5-1 or Eqn. 5.7.3.5-2 (kips)

ℓ_{UEP} distance from end of beam to ultimate evaluation point for horizontal shear (inch)

ℓ_b bearing length (inch)

ℓ_{crit} distance from center of support to ultimate evaluation point for horizontal shear (inch)

ℓ_d development length for reinforcement (inch)

$\ell_{d,avail}$ available development length for reinforcement at section of interest (inch)

$\ell_{\ell p}$ length of the load point (inch)

ℓ_{oh} overhang length, distance from centroid of bearing to end of member (inch)

ℓ_t transfer length for reinforcement (inch)

M moment in member (kip-ft)

M_{LL} lane live load moment, without distribution factor (kip-ft)

M_{LT} moment due to truck load, without dynamic allowance or distribution factors (kip-ft)

M_{cr} cracking moment (kip-inch)

M_n nominal flexural resistance (kip-ft)

M_u factored flexural demand (kip-ft)

$M_{u,LL}$ factored flexural demand due to live loads (kip-ft)

$M_{u(DC,DW)}$ factored flexural demand due to dead loads (kip-ft)

$M_{u(LL+IM)}$ factored flexural demand due to live loads (kip-ft)

$M_{u(LL+IM)inv}$	factored flexural demand due to inventory live loads for load rating (kip-ft)
N	axial force in member (kips)
N_b	number of beams
N_{lanes}	number of lanes
N_u	factored axial force demand in member (kips)
P_1	rear axle load for HL-93 loading with 32-kip load (kips)
P_2	middle axle load for HL-93 loading with 32-kip load (kips)
P_3	front axle load for HL-93 loading with 8-kip load (kips)
P_{PS}	total force from all prestressing strands immediately before transfer (kips)
RF	load rating factor
RHS	right hand side of AASHTO LRFD BDS Eqn. 5.7.3.5-1 or Eqn. 5.7.3.5-2 (kips)
S	beam spacing (ft)
S_b	section modulus for gross section for extreme bottom fiber (inch^3)
S_t	section modulus for gross section for extreme top fiber (inch^3)
s	spacing of transverse reinforcement (inch)
s_θ	crack spacing used in MCFT derivation (inch)
s_{max}	maximum permitted spacing of transverse reinforcement (inch)
s_x	crack spacing parameter, taken as the lesser of either d_v or the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than $0.003b_vs_x$ (inch)
s_{xe}	crack spacing parameter as influenced by aggregate size (inch)
T	tension resultant force used in shear design or load rating (kips)
t_{bf}	thickness of bottom flange (inch)
t_{tf}	thickness of top flange (inch)
t_{ws}	thickness of wearing surface (inch)
V_{LL}	shear due to lane live load, without distribution factor (kips)
V_{LT}	shear due to truck load, without dynamic allowance or distribution factors (kips)
V_c	nominal shear resistance of the concrete (kips)
V_n	nominal shear resistance (kips)

V_{n1} nominal shear resistance related to sectional shear resistance (kips)

V_{n2} nominal shear resistance related to longitudinal reinforcement check (kips)

V_{n3} nominal shear resistance related to horizontal shear resistance (kips)

V_{ni} nominal horizontal shear resistance (kips)

V_p component of prestressing force in the direction of the shear force (kips)

V_s nominal shear resistance provided by transverse reinforcement (kips)

V_u factored shear demand (kips)

$V_{u,hs}$ factored horizontal shear demand (kips)

$V_{u(DC,DW)}$ factored shear demand due to dead loads (kips)

$V_{u(LL+IM)}$ factored shear demand due to live loads (kips)

$V_{u(LL+IM)inv}$ factored shear demand due to inventory live load for load rating (kips)

v shear stress (ksi)

v_{avg} average shear stress across section depth (ksi)

v_i shear stress in element *i* (ksi)

v_u shear stress due to factored shear force (ksi)

W_R clear roadway width (ft)

w_a unit weight of asphalt wearing surface (kcf)

w_{bps} barrier weight per side (kip/ft)

w_b barrier weight per beam (kip/ft)

w_c unit weight of concrete or UHPC (kcf)

w_g beam self-weight (kip/ft)

w_{lane} lane live load (kip/ft)

w_{ws} wearing surface weight (kip/ft)

x distance from centroid of left support (ft)

x_{cr} critical section for shear (ft)

y_b distance between bottom and center of gravity of section (inch)

y_{bc} distance between bottom and center of gravity of composite section (inch)

y_{crit} distance from bottom of section to critical horizontal shear plane (inch)

y_p distance between bottom of section and centroid of prestressing strands (inch)

$y_{p,tens}$ distance between bottom of section and centroid of prestressing strands on flexural tension side of member (inch)

y_t distance between top and center of gravity of section (inch)

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

Δf_{pES} prestress losses due to elastic shortening (ksi)

Δf_{pLT} long-term prestress losses due to time dependent effects (ksi)

Δf_{pT} total prestress losses (ksi)

ε general strain (inch/inch)

ε_1 principal tensile strain (inch/inch)

ε_2 principal compressive strain (inch/inch)

ε_C longitudinal strain at centroid of compressive force resultant (inch/inch)

ε_T longitudinal strain at centroid of tensile force resultant (inch/inch)

ε_s net longitudinal tensile strain in the section at the centroid of the tension reinforcement (inch/inch)

ε_x longitudinal concrete strain (inch/inch); longitudinal concrete strain at mid-depth of section (inch/inch)

$\varepsilon_{x,i}$ longitudinal concrete strain in element *i* (inch/inch)

ε_y vertical concrete strain (inch/inch)

η_{LL} live load moment to live load shear ratio, $M_{u(LL+IM)inv}$ / $V_{u(LL+IM)inv}$ (inch)

θ angle of inclination of diagonal compressive stresses (degrees)

κ development length factor, 1.0 for pretensioned panels, piling and other pretensioned members with a depth of less than or equal to 24 inch, 1.6 for pretensioned members with a depth greater than or equal to 24 inch.

λ concrete density modification factor

λ_{duct} shear strength reduction factor accounting for the presence of a post-tensioning duct

μ friction factor for interface shear transfer

ϕ resistance factor

ϕ_c resistance factor for axial resistance

ϕ_f resistance factor for flexure

ϕ_v resistance factor for shear

τ shear stress (ksi)

ψ angle from the horizontal for harped strands (rad/inch)

CHAPTER 1. INTRODUCTION

INTRODUCTION

Holt et al. (2018) conducted a review of shear load rating practices and the history of shear design procedures (referred to hereafter as FHWA-HIF-18-061). Holt et al. (2022) continued this work and developed recommendations for using the modified compression field theory (MCFT) for shear load rating of concrete bridges (referred to hereafter as FHWA-HIF-22-025). FHWA-HIF-22-025 provide details on how the recommended procedures were developed and some details on their use for prestressed concrete members. The objective of this report is to provide additional clarification on the use of MCFT for shear load rating of pretensioned concrete members. The shear load rating of a pretensioned concrete box beam is provided as an example, with details provided for the shear load rating along the length of the member.

FHWA-HIF-22-025 also provide details and recommendations related to shear load rating of post-tensioned members and deep beam members using the strut-and-tie method (STM). These topics are not addressed in this report.

REVISIONS TO THE MBE

The evaluation for shear in load rating is specified in the AASHTO Manual for Bridge Evaluation (hereafter referred to as AASHTO MBE) Article 6A.5.8 and referencing the AASHTO LRFD Bridge Design Specifications (hereafter referred to as AASHTO LRFD BDS) Article 5.7.3.6.3. Revisions were approved to the AASHTO MBE based on FHWA-HIF-22-025. The primary updates to the AASHTO MBE shear load rating are the following:

- The iterative procedure needed for shear load rating using MCFT is described in AASHTO MBE Article C6A.4.2.1.
- Additional details are provided in AASHTO MBE Article 6A.5.8 and C6A.5.8 on how to shear load rate considering the longitudinal reinforcement requirement of AASHTO LRFD BDS Article 5.7.3.5. An equation is provided in the commentary to calculate the rating factor based on this check.
- Two modifications are allowed to AASHTO LRFD BDS Article 5.7.3.4.2: (1) ε_s may be taken as zero if $M_u \leq M_{cr}$ and (2) the β factor for prestressed concrete members (where $f_{pc} / f'_c \geq 0.02$) may be calculated using AASHTO LRFD BDS Eqn. 5.7.3.4.2-1 regardless of if there is minimum transverse reinforcement provided.
- A provision was added specifying that concurrent load effects should be used in shear load rating analyses.

These changes will be used in the example problem and explained in greater depth in this report.

REPORT OVERVIEW

This report provides a brief background on MCFT, possible shear failure mechanisms for beams, and the shear load rating procedures recommended by FHWA-HIF-22-025. A detailed shear load rating example is provided for a pretensioned concrete box beam, where the shear load rating is determined at multiple points along the length of the member.

CHAPTER 2. PROCEDURE FOR USING MCFT FOR SHEAR LOAD RATING

INTRODUCTION

A framework for the basic procedure for using MCFT for shear load rating is provided in FHWA-HIF-22-025. The proposed procedure is summarized in this chapter with additional explanations for use in pretensioned bridge applications.

APPLICATION OF MODIFIED COMPRESSION FIELD THEORY (MCFT)

MCFT was developed by Vecchio et al. (1986) and implemented in the 1st Edition of the AASHTO LRFD BDS (1994). MCFT determines the shear resistance based on a compression stress field in the member between the load and support point. Details about the stress field including the angle of principal compressive stresses and principal strains and stresses can be determined based on the applied loads, applied precompression from prestressing, and the material properties. The concrete material properties to use with this method were measured using reinforced concrete panels with different applied axial and shear stresses.

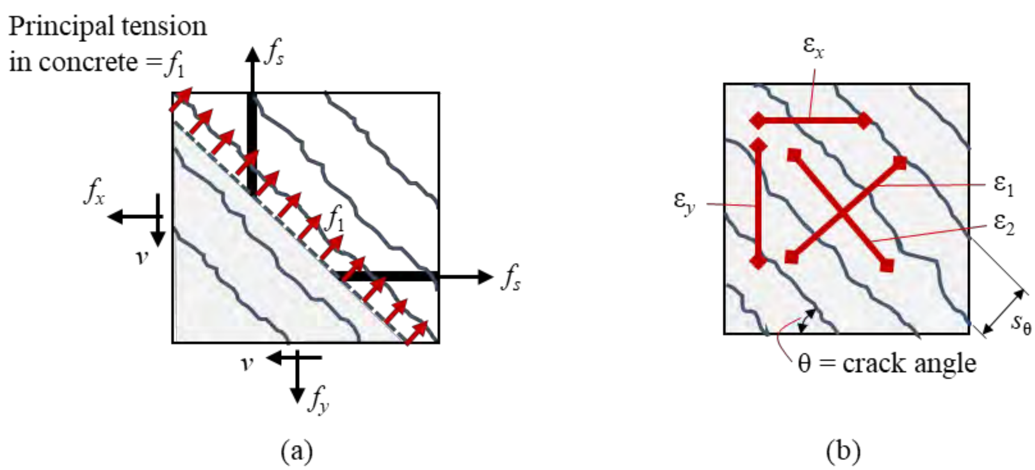

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Figure 1. Illustration. Assumed crack distribution for MCFT with key (a) stresses and (b) strains labeled.

Bentz et al. (2006) explains three different levels of complexity when applying MCFT to the design of concrete elements.

- **Array of biaxial elements**: The most accurate way to apply MCFT is by discretizing the element into a collection of prismatic elements, as shown in Figure 2. The MCFT theory and associated concrete materials properties can be applied to each of these elements. The researchers at The University of Toronto, where MCFT was developed, created a non-linear finite element analysis software for this purpose (VecTor2).

Source: FHWA

Figure 2. Illustration. Most complex application of MCFT by discretizing member into prismatic elements and applying MCFT principals to each element.

- **Vertical stack of biaxial elements**: MCFT can also be applied at individual sections along the length of a member if plane sections are assumed to remain plane, as shown in Figure 3. The demand and other properties (e.g., prestressing) at a section along the length can be used to calculate the axial and shear stresses and apply MCFT to a vertical stack of biaxial elements. The researchers at The University of Toronto developed a computer software for this purpose (RESPONSE-2000).

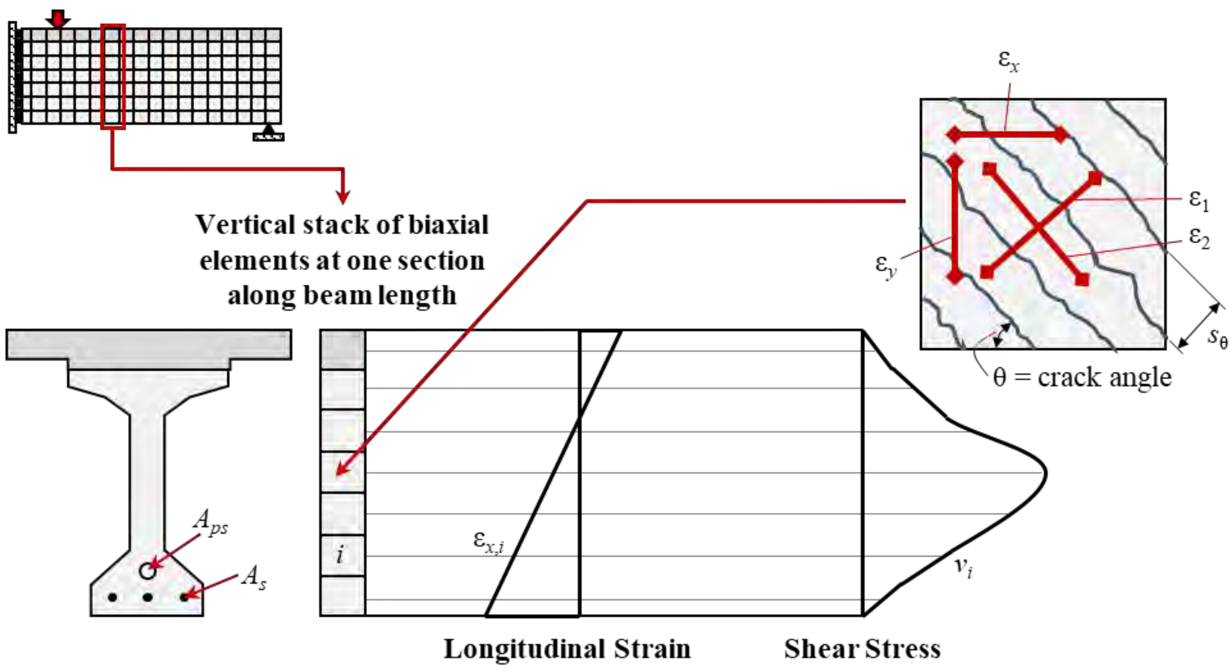

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Figure 3. Illustration. Application of MCFT using vertical stack of biaxial elements.

- **Average response of one biaxial element at mid-height of the section**: The next simplification that can reasonably be made to apply MCFT to typical member design is to assume the shear strength of a section can be represented by one biaxial element at mid-height of the section assuming a constant shear stress over the depth of the section, as shown in Figure 4. This is the simplification used in the equations developed for AASHTO LRFD BDS.

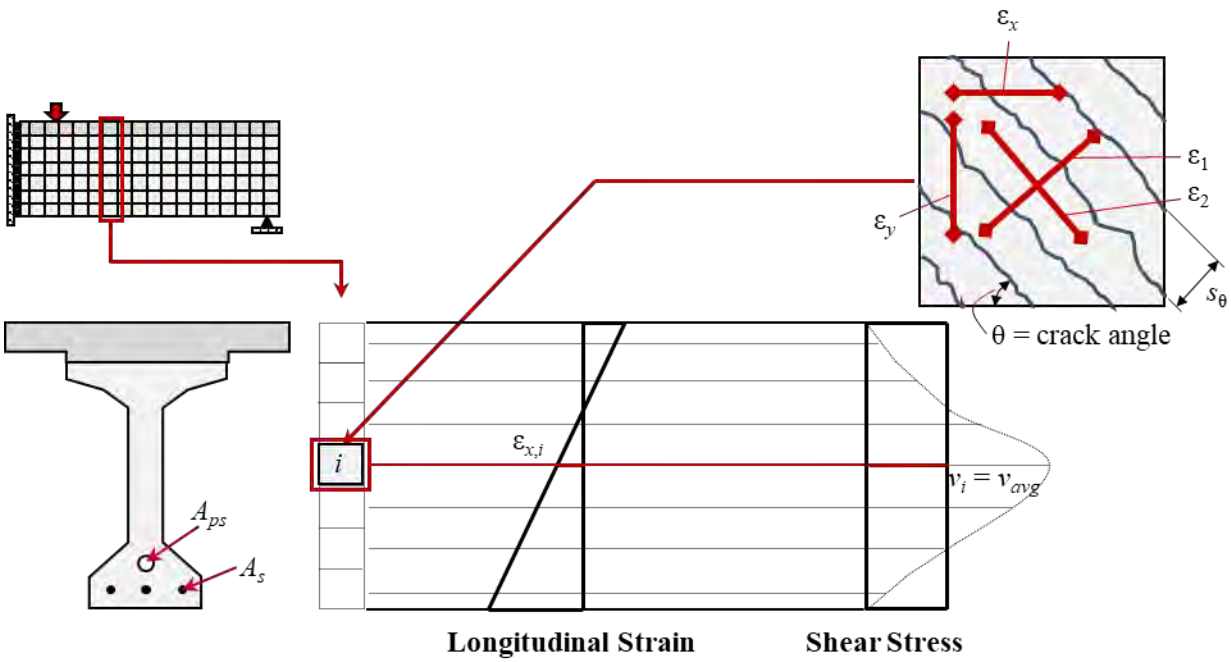

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Figure 4. Illustration. Application of MCFT using average response of one biaxial element at mid-height of the section.

The original shear design procedure specified by AASHTO LRFD BDS (1st Edition, 1994) was iterative, because of the interdependency of the crack angle, longitudinal strain, and shear resistance. Bentz et al. (2006) proposed simplifications to allow for direct calculation of the shear resistance during design (i.e., eliminating the iterative procedure), which were adopted in the 2008 Interim Revisions to the 4th Edition of the AASHTO LRFD BDS. The iterative procedure is now in AASHTO LRFD BDS Appendix B5 and is still an acceptable alternative to the direct calculation simplifications.

More details on the history of the shear provisions in the AASHTO LRFD BDS and background on MCFT can be found in FHWA-HIF-22-025.

POSSIBLE CONTROLLING SHEAR MECHANISMS

There are three possible shear failure mechanisms that can control the resistance for pretensioned members:

- Sectional shear resistance,
- Anchorage distress influencing shear resistance (checked by the longitudinal reinforcement resistance), and
- Horizontal shear resistance.

Illustrations of these three different failure mechanisms are shown in Figure 5.

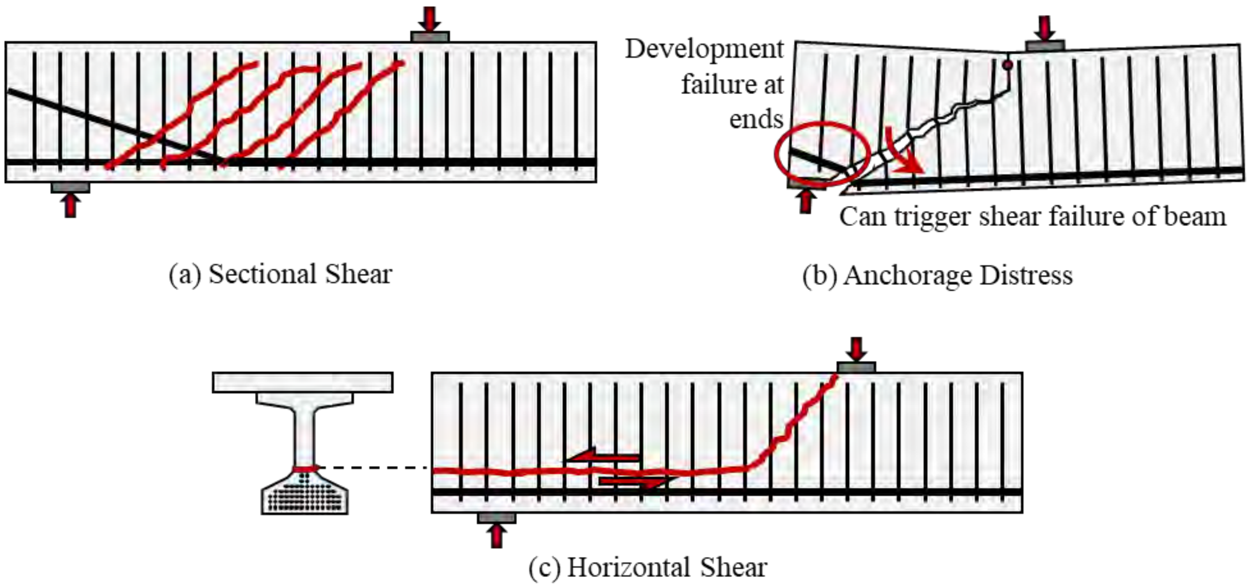

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Figure 5. Illustration. Three possible failure shear failure mechanisms related to MCFT: (a) section shear, (b) anchorage distress leading to shear failure, and (c) horizontal shear.

Sectional shear resistance is associated with diagonal cracking in the web of the member typically associated with shear, shown in Figure 5 (a). The angle of the shear cracking is typically assumed to coincide with the angle of principal compressive stresses, θ. The diagonal shear cracking associated with sectional shear behavior will lead to either a web crushing or flexure-shear failure, shown in Figure 6.

Source: FHWA. Photographs taken by David Garber.

Figure 6. Photographs. Failure crack patterns for (a) web crushing and (b) flexure-shear sectional shear failures.

Anchorage distress and strand development failure can lead to a shear failure, as shown in Figure 5 (b). This type of failure mechanism may control if there is some combination of the following factors.

- Short available development length,
- No shear reinforcement or smaller amount of shear reinforcement,
- No non-prestressed tension reinforcement, and
- Large proportion of strands debonded in the end region.

Anchorage distress leading to a shear failure will typically be preceded by extensive cracking in the bottom flange at the ends of a beam. Diagonal shear cracking will also be present for this failure mechanism. More details on the mechanism for anchorage distress leading to a shear failure can be found in Garber et al. (2016) with a summary of previous research on the topic in Naji et al. (2017).

Horizontal shear is always checked at the construction joint between the top of a precast member and bottom of the cast-in-place composite deck. A horizontal shear failure mechanism may also occur at the interface between the bottom flange and web for modern bulb-T or U-beam sections, as shown in Figure 5 (c). This type of failure mechanism may control if there is some combination of the following factors.

- Thin web and large bottom flange with high prestressing ratios (e.g., bulb-T sections),
- Bearing at end of beam (i.e., no overhang),
- No shear reinforcement or smaller amount of shear reinforcement, and
- Less debonded strands.

Horizontal shear failures will typically be preceded by diagonal shear cracks and horizontal cracking at the bottom flange to web interface at the ends of the beam above the support. More details on horizontal shear failures can be found in Hovell et al. (2013).

Additional differences between these failure mechanisms are discussed in more detail in Garber et al. (2016).

MCFT FOR SHEAR LOAD RATING PROCESS

General Procedure

The procedure used for shear load rating using MCFT is based on the recommended procedure by FHWA-HIF-22-025. The basic procedure is outlined in Figure 7. The shear resistance of a member is based on the minimum of the (1) sectional shear resistance, (2) resistance related to the longitudinal reinforcement check, and (3) horizontal shear resistance.

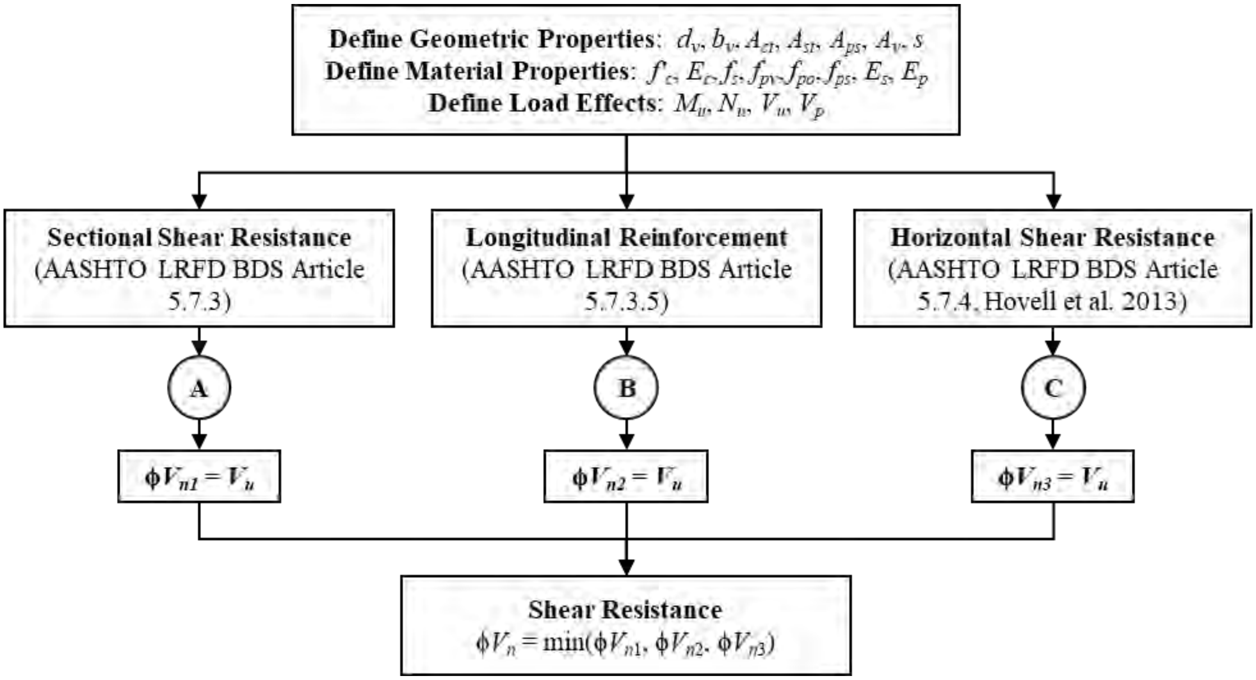

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Figure 7. Flowchart. Analysis process for using MCFT for shear load rating, based on FHWA-HIF-22-025.

There are several general comments related to the procedure shown in Figure 7.

- This procedure can be used for sections with or without minimum transverse reinforcement (based on AASHTO LRFD BDS Article 5.7.2.5).
- Longitudinal tie anchorage should be checked per AASHTO LRFD BDS in all cases.
- Horizontal shear resistance is not currently required by AASHTO LRFD BDS or AASHTO MBE but may control for modern bulb-T sections with narrow webs and heavily prestressed bottom flanges.
- Concurrent V_u, M_u, and N_u should be used for calculating the shear resistance.
- Calculating the sectional shear resistance and resistance based on the longitudinal reinforcement check are iterative procedures. If the section is uncracked (i.e., where $M_u < M_{cr}$), the longitudinal tensile strain can be assumed to be equal to 0, $\varepsilon_s = 0$, which eliminates the iterations. Performing iterations will likely increase the resistance in cases where the *RF* is less than 1.0 and decrease the resistance where the *RF* is greater than 1.0.

These are discussed in more detail in the following sections.

Loads for Iterative Procedures

The AASHTO MBE (Article C6A.5.8) acknowledges that "prestressed concrete shear capacities are load dependent, which means computing the shear resistance involves an iterative process when using the current AASHTO MCFT." This iterative process is further highlighted by the approved revisions to the AASHTO MBE. The shear and moment demand on the structure is caused by applied dead loads and live loads. The dead loads will remain constant on the structure, while the live loads would generally need to be modified during the load rating process to determine the load rating factor.

An iterative process is required for calculating the sectional shear resistance and the resistance based on the longitudinal reinforcement check when the section is cracked. The shear demand due to the live load, $V_{u(LL+IM)}$, should be modified until the total factored resistance, ϕV_n, is equal to the total demand, V_u. This can be done by increasing the live load until $\phi V_n = V_u$. A simplified approach is to calculate the ratio between $M_{u(LL+IM)inv}$ and $V_{u(LL+IM)inv}$ for the live load and then assume this ratio remains the same as the live load is increased.

Moment to shear ratio for live load (recommended in FHWA-HIF-22-025):

$$\eta_{LL} = M_{u(LL+IM)inv} / V_{u(LL+IM)inv}$$

Total moment: $M_u = M_{u(DC,DW)} + M_{u(LL+IM)} = M_{u(DC,DW)} + V_{u(LL+IM)}\ \eta_{LL}$

This will lead to the following simplification for calculating the longitudinal tensile strain.

Simplification for moment term of AASHTO LRFD Eqn. 5.7.3.4.2-4:

$$| M_u / d_v | = | (M_{u(DC,DW)} + V_{u(LL+IM)}\ \eta_{LL}) / d_v |$$

Where $V_{u(DC,DW)}$ and $M_{u(DC,DW)}$ will remain constant and $V_{u(LL+IM)}$ will be increased as needed during the iterative process.

Additional details on calculating the sectional shear resistance, resistance based on the longitudinal reinforcement check, and horizontal shear resistance are provided in the following sections.

Concurrent Loads

The placement of a live load to cause the maximum shear is typically different than the placement of the live load for maximum moment. The placement of HL-93 loading to cause maximum shear in a simply-supported beam at location *x* is shown in Figure 8 (a), while the placement of HL-93 loading to cause the maximum moment is shown in Figure 8 (b).

A provision was added specifying that concurrent load effects should be used in shear load rating analyses. This means that the moment and shear used in the shear load rating should be from the same placement of the live load.

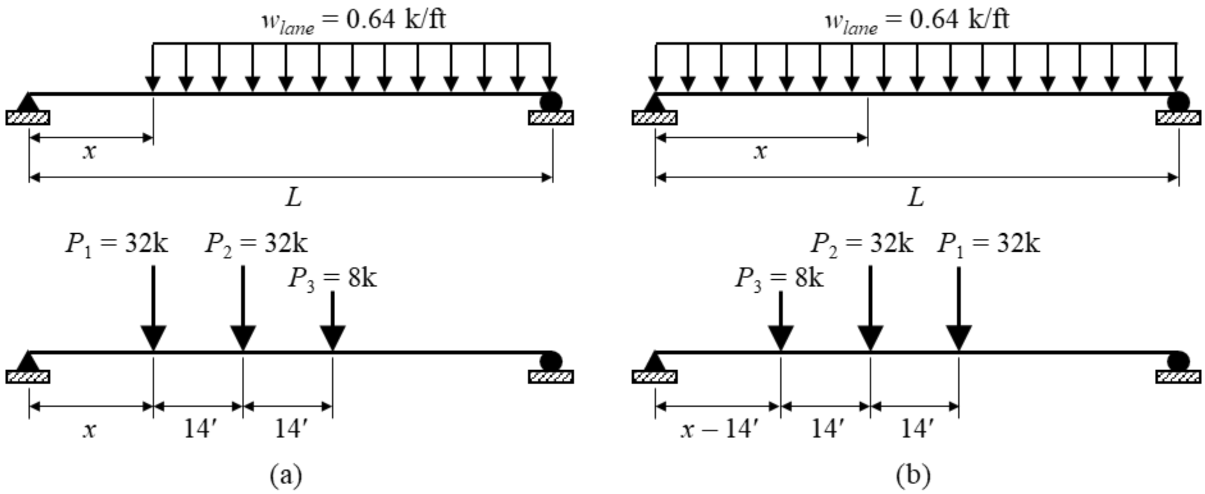

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Figure 8. Illustration. Placement of HL-93 loading to cause (a) maximum shear and (b) maximum moment on a simply-supported beam.

The shear and moment associated with the lane live load placed to cause maximum shear, shown in Figure 8 (a), are as follows.

Shear due to lane live load placed for maximum shear: $V_{LL} = w_{lane}\,(L - x)^2 / (2L)$

Moment due to lane live load placed for maximum shear: $M_{LL} = w_{lane}\, x\,(L - x)^2 / (2L)$

The shear and moment associated with the design truck placed to cause maximum shear, shown in Figure 8 (a), are as follows.

Shear due to design truck placed for maximum shear:

$$V_{LT} = \frac{P_1(L - x)}{L} + \frac{P_2(L - x - 14')}{L} + \frac{P_3(L - x - 28')}{L}$$

Moment due to design truck placed for maximum shear:

$$M_{LT} = \frac{P_1(x)(L - x)}{L} + \frac{P_2(x)(L - x - 14')}{L} + \frac{P_3(x)(L - x - 28')}{L}$$

The shear and moment associated with the lane live load placed to cause maximum moment, shown in Figure 8 (b), are as follows.

Shear due to lane live load placed for maximum moment: $V_{LL} = w_{lane}\,(0.5L - x)$

Moment due to lane live load placed for maximum moment: $M_{LL} = 0.5 w_{lane} x(L - x)$

The shear and moment associated with the design truck placed to cause maximum moment, shown in Figure 8 (b), are as follows.

Shear due to design truck placed for maximum shear:

$$V_{LT} = -\frac{P_3(x - 14')}{L} + \frac{P_2(x)}{L} + \frac{P_1(L - x - 14')}{L}$$

Moment due to design truck placed for maximum shear:

$$M_{LT} = \frac{P_3(x - 14')(L - x)}{L} + \frac{P_2(L - x)(x)}{L} + \frac{P_1(L - x - 14')(x)}{L}$$

The term associated with P_3 should be neglected if (x – 14 ft) is less than 0, which would occur when the front axle of the truck is off the span.

For design, it is common to use the maximum moment with the maximum shear. The PCI Bridge Design Manual (BDM) provides simplified equations in Article 8.11.1 for calculating the shear and moment along the length of a simply supported beam due to HL-93 loading.

Lane live load: $V_{LL} = w_{lane} \times (L - x)^2 / (2L)$ if $0 \le x \le 0.5L$

$V_{LL} = -w_{lane} \times (L - (L - x))^2 / (2L)$ if $0.5L < x \le L$

$M_{LL} = 0.5w_{lane} \times x \times (L - x)$

Truck load without impact:

V_{LT} = (72 kips) × $((L - x) - 9.33$ ft$) / L$ if $0 \le x \le 0.5L$

V_{LT} = – (72 kips) × $((L - (L - x)) - 9.33$ ft$) / L$ if $0.5L < x \le L$

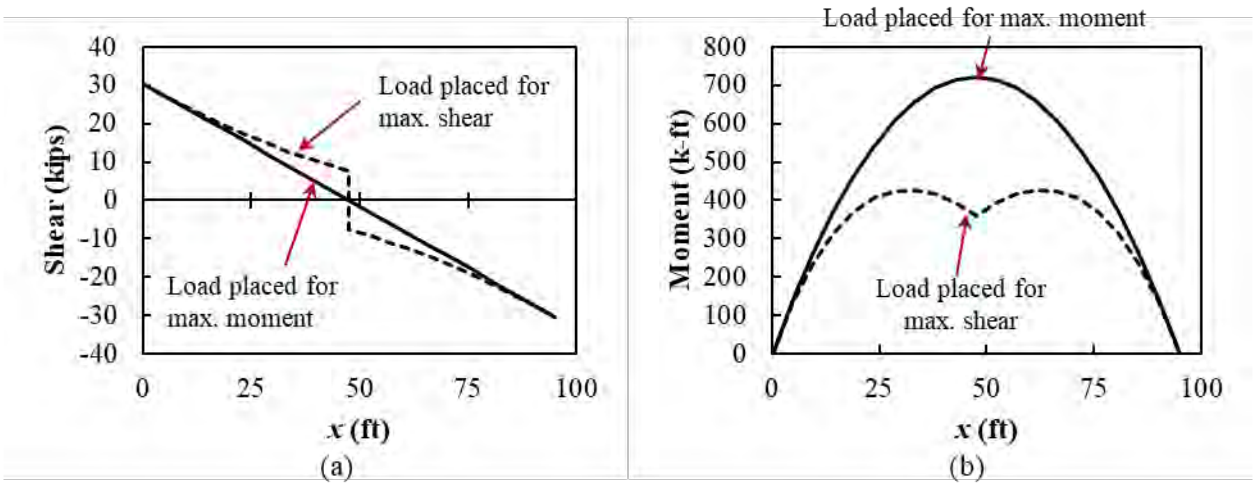
M_{LT} = (72 kips) × $x \times [(L - x) - 9.33$ ft$] / L$ if $0 \le x < 0.333L$

M_{LT} = (72 kips) × $x \times [(L - x) - 4.67$ ft$] / L$ – 112 kip-ft if $0.333L \le x \le 0.5L$

M_{LT} = (72 kips) × $(L - x) \times [(L - (L - x)) - 4.67$ ft$] / L$ – 112 kip-ft if $0.5L < x \le 0.667L$

M_{LT} = (72 kips) × $(L - x) \times [(L - (L - x)) - 9.33$ ft$] / L$ if $0.667L < x \le L$

The shear and moment diagrams corresponding to the load placed to cause maximum shear and maximum moment for a simply-supported beam with a 95-foot span length are shown in Figure 9 for lane load and Figure 10 for the design truck. The largest difference between the shear and moment diagrams with different assumed loading locations are for the moment for the lane load, see Figure 9 (b), and shear for design truck, see Figure 10 (a).



Source: FHWA

Figure 9. Graphs. (a) Shear and (b) moment diagrams corresponding to lane load placed to cause maximum shear and maximum moment.

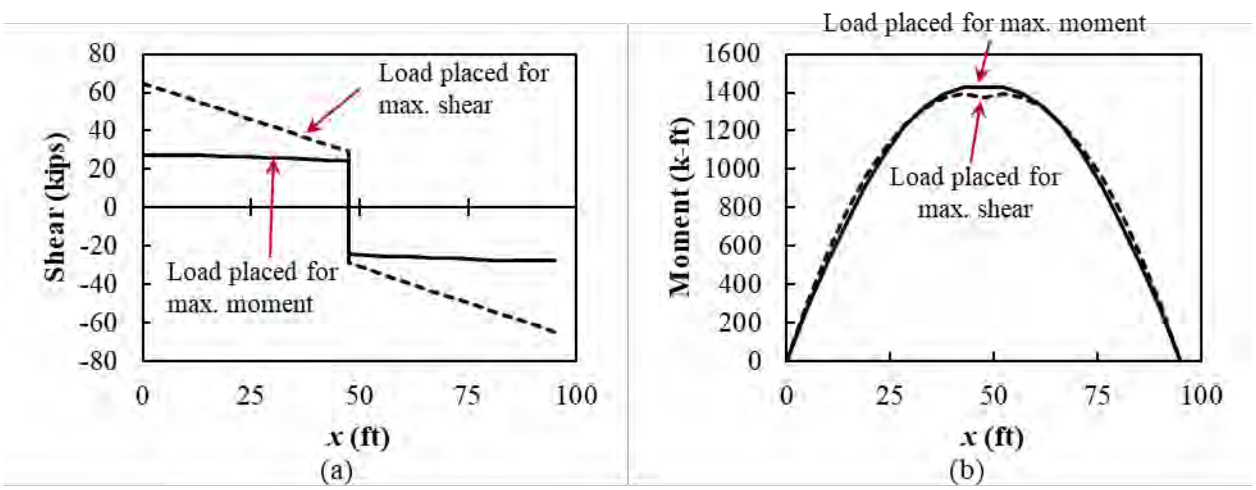


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Figure 10. Graphs. (a) Shear and (b) moment diagrams corresponding to design truck placed to cause maximum shear and maximum moment.

More details on the effect of different assumptions for load placement are provided in the load rating example in Chapter 3.

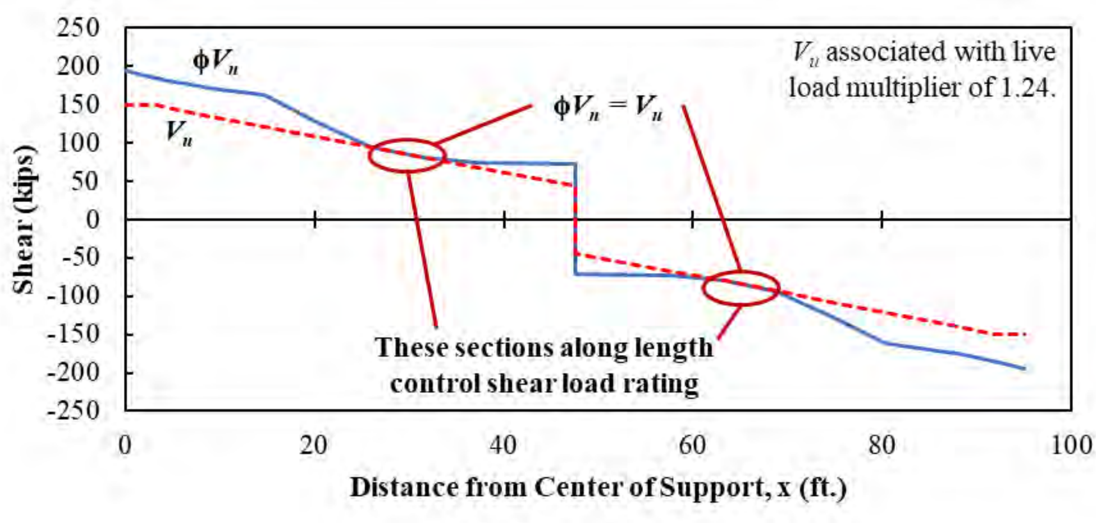
Resistance and Demand along Beam Length

The shear, moment, and axial demand (V_u, M_u, and N_u) at a specific section are used to determine the factored shear resistance, ϕV_n, at that specific section. This means that shear demand and resistance will change along the length of the member. The amount of transverse reinforcement may also change along the length of the member, which will also affect the resistance. The critical section (d_v from the face of the support) may not control the shear load rating.

Sample calculations in examples are often provided at the critical section (d_v from the face of the support). The shear resistance of a beam must be checked along the length of the beam. AASHTO MBE Article C6A.5.8 states that:

> *Multiple locations, preferably at 0.05L points, need to be checked for shear. Locations where shear is highest may not be critical because the corresponding moment may be quite low. Typically, locations near the 0.25L point could be critical because of relatively high levels of both shear and moment.*

The shear load rating will be associated with the smallest live load leading to the factored resistance equal to the demand at any point along the length of the beam. An example of the sectional shear resistance and shear demand at the load associated with the sectional shear load rating for the beam is shown in Figure 11. The live load was increased for this example until the demand equaled the resistance at any point along the length of the beam. The shear load rating for sectional shear in this example is controlled by the resistance and demand at approximately 30 feet from the beam ends, which corresponds to about 0.31*L*.



Source: FHWA

Figure 11. Graph. Example shear resistance and shear demand along length of beam under live load controlling shear load rating (for sectional shear resistance only).

A spreadsheet was developed for this example where the factored demand and factored resistance were calculated at several points along the length of the beam. The difference between the initial resistance and demand, $\phi V_n - V_u$, was calculated at each point. A solver was used to modify the live load multiplier until the minimum difference between resistance and demand was equal to zero.

Checking the shear resistance along the length of the beam for the three different failure mechanisms will add an additional level of complexity but can be done fairly easily in a spreadsheet.

Sectional Shear Resistance

General Procedure for Calculating Sectional Shear Resistance

The sectional shear resistance, V_n, includes components for the nominal shear resistance provided by the concrete, V_c, the transverse reinforcement, V_s, and the vertical component of the prestressing strands, V_p, shown in Figure 12. The nominal shear resistance is defined by AASHTO LRFD BDS Eqn. 5.7.3.3-1 and Eqn. 5.7.3.3-2.

Nominal shear resistance: $V_n = V_c + V_s + V_p \leq 0.25f'_c b_v d_v + V_p$

The sectional shear resistance is dependent on the longitudinal strain in the concrete at mid-depth, ε_x, which is directly related to the longitudinal strain at the centroid of the tension tie, ε_s. The longitudinal strain at the centroid of the tension tie depends on the demand at the section of interest, M_u, V_u, and N_u, and the precompression provided by the prestressing, $A_{ps}f_{po}$. The rating section where it is assumed the demand calculated is located at mid-width of the diagonal shear crack, as shown in Figure 12. The longitudinal tensile strain in the section at the centroid of the reinforcement is defined in AASHTO LRFD BDS Eqn. 5.7.3.4.2-4.

Longitudinal tensile strain in the section at centroid of tension reinforcement (if $\varepsilon_s \geq 0$), AASHTO LRFD BDS Eqn. 5.7.3.4.2-4:

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

Longitudinal tensile strain in the section at centroid of tension reinforcement (if $\varepsilon_s < 0$), from bullet point in AASHTO LRFD BDS Article 5.7.3.4.2:

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

where $|M_u| \geq |V_u - V_p|\, d_v$.

The angle of inclination of the diagonal compressive stresses, θ, is assumed to be the same as the shear crack angle. The θ and the effectiveness of the concrete to resist shear, specified by β, are both dependent on the longitudinal tensile strain in the section at centroid of tension reinforcement. These are defined by AASHTO LRFD BDS Eqn. 5.7.3.4.2-1, Eqn. 5.7.3.4.2-2, and Eqn. 5.7.3.4.2-3.

Concrete shear factor (w/min. transverse reinforcement): $\beta = 4.8 / (1 + 750\varepsilon_s)$

Concrete shear factor (w/o min. transverse reinforcement):

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

Angle of inclination of the diagonal compressive stresses: $\theta = 29 + 3500\varepsilon_s$

The nominal shear resistance provided by the concrete is assumed to be primarily dependent on the aggregate interlock and roughness along the length of the crack. The nominal shear resistance provided by concrete is defined by AASHTO LRFD BDS Eqn. 5.7.3.3-3.

Nominal shear resistance provided by concrete: $V_c = 0.0316\beta\lambda\sqrt{(f'_c)}\, b_v d_v$

The nominal shear resistance provided by the transverse reinforcement only includes the reinforcement crossing the shear crack, which has a horizontal distance of (d_v cot θ). The nominal shear resistance provided by transverse reinforcement perpendicular to horizontal (α = 90°) is defined by AASHTO LRFD BDS Eqn. C5.7.3.3-1.

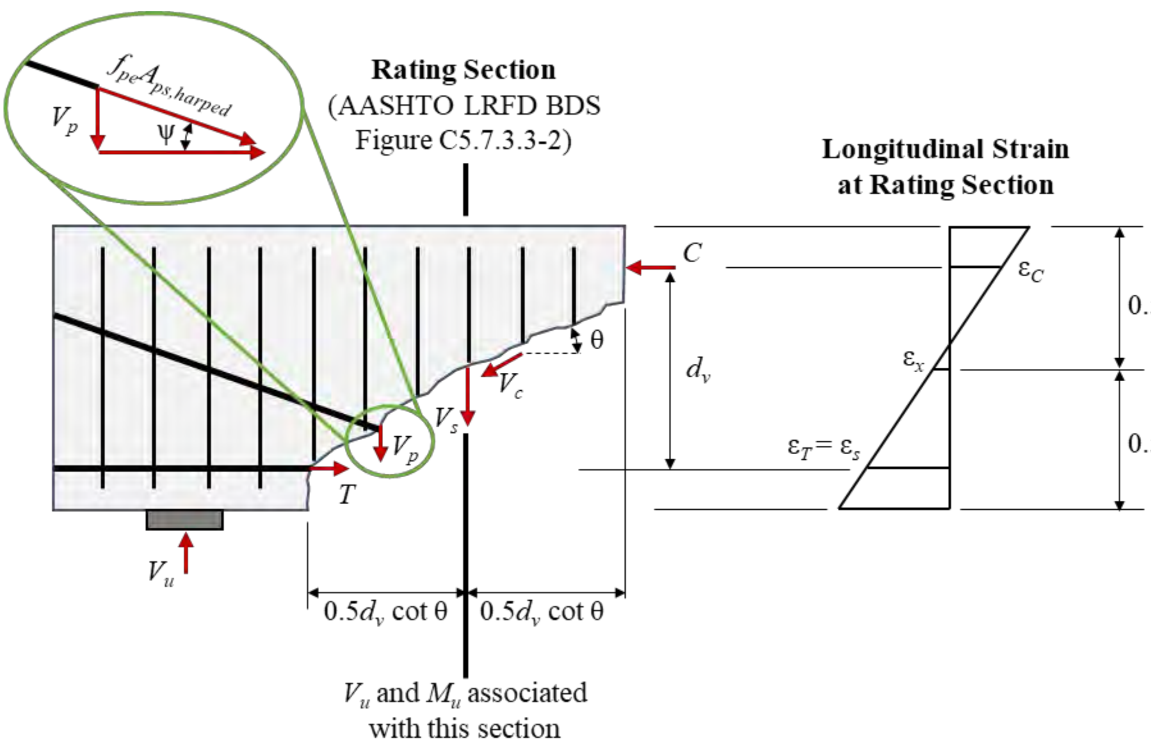
Nominal shear resistance provided by transverse reinforcement (α = 90°):

$$V_s = [(A_v f_y d_v \cot\theta) / s]\, \lambda_{duct}$$

The vertical component of the prestressing strands, V_p, will provide additional shear resistance, as shown in Figure 12. Harped strands with area of $A_{p,harped}$ at an angle ψ from the horizontal with a stress after all losses (not including transient gains) of f_{pe} will provide the following shear resistance.

Nominal shear resistance provided by harped strands: $V_p = f_{pe} A_{p,harped} \sin\psi$

V_p will be equal to 0 if there are no harped strands, $A_{p,harped} = 0$ inch^2.

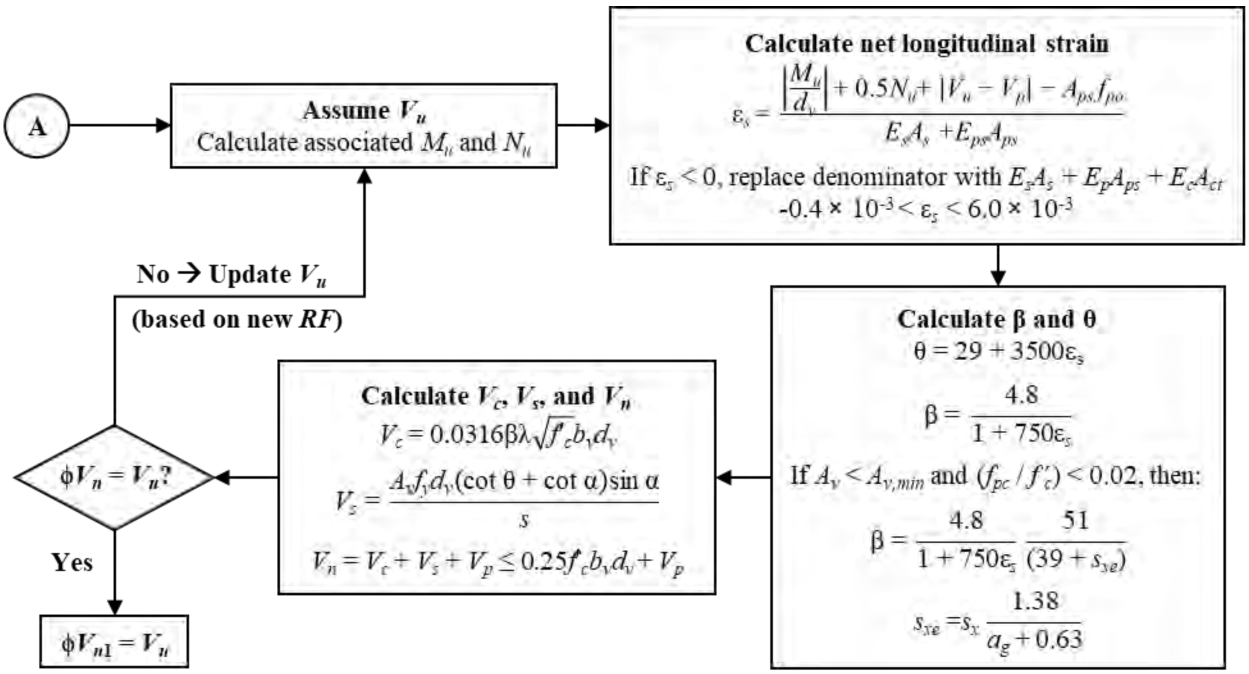


Source: FHWA

Figure 12. Illustration. Rating assumptions for calculating sectional shear resistance using MCFT.

The sectional shear behavior is often depicted as a diagonal shear crack with the force and shear components shown in Figure 12. The sectional shear procedure is really an approximation of a biaxial element at mid-height of the section, see Figure 4, which is a simplification of a stack of biaxial elements, see Figure 3. This means that all components of demand and resistance (including the available development length) should be calculated at the same section, the "Rating Section" highlighted in Figure 12.

The basic procedure for calculating the sectional shear resistance for shear load rating is summarized in Figure 13. This procedure is iterative as the shear, moment, and axial demand, V_u, M_u, and N_u, are used to calculate the resistance. As previously mentioned, the demand caused by the dead load will not change during the iterations; the live load component of the demand should be modified for each iteration.



Source: FHWA

Figure 13. Flowchart. Analysis process for using MCFT for sectional shear load rating, based on FHWA-HIF-22-025 considering updates to MBE.

The iterative procedure for calculating the sectional shear resistance at a single section along the length of the beam includes the following steps.

- **Step 1**: Assume a live load and the associated V_u, M_u, and N_u. The first assumed live load should be equal to HL93 inventory loading or the appropriate rating loading. Future iterations can be equal to this live load times a multiplier that would change during each iteration; the dead load remains constant.
- **Step 2**: Calculate the associated net longitudinal strain in the section at the centroid of the tension reinforcement, ε_s, using AASHTO LRFD BDS Eqn. 5.7.3.4.2-4. ε_s may be taken as zero if $M_u \leq M_{cr}$, which eliminates the iterative procedure.

- **Step 3**: Calculate the associated β and θ using AASHTO LRFD BDS Eqn. 5.7.3.4.2-1, Eqn. 5.7.3.4.2-2, and Eqn. 5.7.3.4.2-3. Use the appropriate equation for finding β depending on if the section has minimum shear reinforcement or a minimal amount of precompression. AASHTO LRFD BDS Eqn. 5.7.3.4.2-1 should be used where minimum shear reinforcement is provided or where (f_{pc} / f'_c) > 0.02. AASHTO LRFD BDS Eqn. 5.7.3.4.2-2 should be used where minimum shear reinforcement is not provided and (f_{pc} / f'_c) < 0.02.
- **Step 4**: Calculate the nominal shear resistance, V_n, using AASHTO LRFD BDS Eqn. 5.7.3.3-1 through Eqn. 5.7.3.3-5 with ϕ from AASHTO LRFD BDS Article 5.5.4.2. For shear and torsion in monolithic prestressed concrete sections, $\phi = 0.9$.
- **Step 5**: Check to see if $\phi V_n = V_u$. If $\phi V_n = V_u$, then proceed to Step 6. Otherwise, return to Step 1 and assume a new live load.
- **Step 6**: $\phi V_n = V_u$ for sectional shear. Go to the longitudinal reinforcement check.

The live load to assume for future iterations can be calculated based on the load rating factors from the previous step and a relaxation factor, R_f. The relaxation factor helps the shear to converge more rapidly. The relaxation factors can be varied to change the rate of convergence but should be less than 1.0. The live load shear for future iterations is as follows.

Live load shear for future iterations: $V_{u,(LL+IM),(i+1)} = ((1 - R_f) \cdot RF_{(i-1)} + R_f \cdot RF_{(i)}) \cdot V_{u,(LL+IM)}$

Rating factor for iteration i: $RF_{(i)} = (\phi V_{n(i)} - V_{u(DC,DW)}) / V_{u,(LL+IM)}$

Rating factor for iteration 0: $RF_{(0)} = 1.0$

This procedure should be completed for multiple points along the length of the beam, as described above.

Possible Expedients for Sectional Shear Resistance

There are several different expedients that were proposed in FHWA-HIF-22-025, one of which was adopted in the revisions to the AASHTO MBE. These are summarized in this section.

- **Expedient #1**: Use the simplified procedure for non-prestressed sections from AASHTO LRFD BDS Article 5.7.3.4.1. This article specifies β = 2.0 and θ = 45°. Load raters can use this for a quick estimate of the strength as it will generally provide a conservative strength estimate for prestressed concrete members.
- **Expedient #2**: Use the alternate shear design approach provided in AASHTO LRFD BDS Article 5.12.5.3.8 as a possible expedient for prestressed concrete members. This is a non-iterative procedure that will generally provide conservative estimates compared to the general shear procedure using MCFT.

  This alternate shear design method is under AASHTO LRFD BDS Article 5.12.5 for Segmental Concrete Bridges. The concrete contribution to shear resistance is calculated using AASHTO LRFD BDS Eqn. 5.12.5.3.8c-3 as follows.

  Concrete contribution to shear resistance: $V_c = 0.0632K\lambda\sqrt{(f'_c)}\, b_v d$

The precompression provided by the prestressing is considered based on AASHTO LRFD BDS Eqn. 5.12.5.3.8c-5 as follows.

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

The precompression stress, f_{pc}, has the same definition as in ACI 318, which is calculated as follows.

Precompression stress for non-composite sections: $f_{pc} = (f_{pbt} - \Delta f_{pT}) / A_g$

Precompression stress for composite sections:

$$f_{pc} = \frac{(f_{pbt} - \Delta f_{pT})A_p}{A_g} - \frac{(f_{pbt} - \Delta f_{pT})A_p e_p (y_{bc} - y_b)}{I_g} + \frac{M_d (y_{bc} - y_b)}{I_g}$$

The steel contribution to the shear strength is calculated using AASHTO LRFD BDS Eqn. 5.12.5.3.8c-4, which assumes a $\theta = 45°$, as follows.

Steel contribution to shear resistance: $V_s = (A_v f_y d) / s$

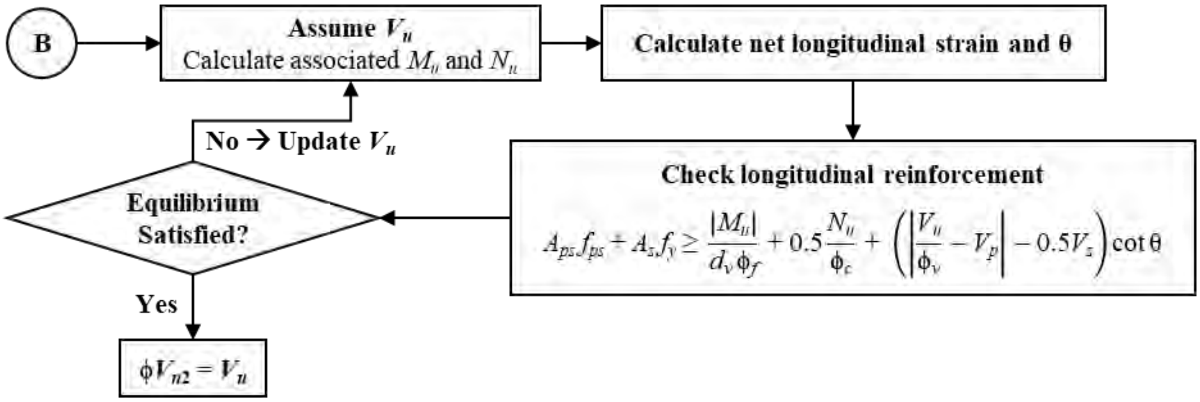
These equations are equivalent to $\beta = 2.0$ and $\theta = 45°$ if $f_{pc} = 0$ ksi.

- **Expedient #3**: Use AASHTO LRFD Article 5.7.3.4.2 (MCFT General Procedure) and treat the load rating problem like a design problem. If the provided (A_v / s) for member in question satisfies design requirements, then the member provides adequate strength. This expedient will show if the member can safely carry the load but does not provide the peak member shear strength, which would be used for determining the shear load rating.
- **Expedient #4**: Use $\varepsilon_s = 0$ if $\varepsilon_s < 0$, which is true if $M_u < M_{cr}$. This expedient is included in the revised AASHTO MBE. This simplification will eliminate the iterative procedure. The load rater must make sure that $M_u < M_{cr}$ for the increased load to get $\phi V_n = V_u$.

The longitudinal reinforcement and horizontal shear requirements should still be checked if using an expedient for sectional shear.

Longitudinal Reinforcement Check

The basic procedure for calculating the shear resistance controlled by the longitudinal reinforcement check is summarized in Figure 14.



Source: FHWA

Figure 14. Flowchart. Analysis process for using MCFT for sectional shear load rating, based on FHWA-HIF-22-025.

The general check for the longitudinal reinforcement is calculated using AASHTO LRFD Eqn. 5.7.3.5-1.

General check for longitudinal reinforcement:

$$A_{ps}f_{ps} + A_sf_y \geq \frac{|M_u|}{d_v\,\phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta$$

The shear resistance for this failure mechanism is calculated based on the demand, V_u, required for the left side of this equation to equal the right side of the equation.

The demand will also affect the longitudinal tensile strain, ε_s, which will affect the principal angle direction, θ. The principal angle direction will directly impact the right side of the equation and the available development length, discussed in the follow section, which will affect f_{ps} on the left side of the equation.

The simplified AASHTO LRFD BDS Eqn. 5.7.3.5-2 may be used at the inside edge of the bearing area of a simple end supports.

Longitudinal reinforcement check at inside edge of bearing:

$$A_{ps}f_{ps} + A_sf_y \geq \left(\frac{V_u}{\phi_v} - 0.5V_s - V_p\right)\cot\theta$$

Additionally, per AASHTO LRFD BDS Article C5.7.3.5, the "values of V_u, V_s, V_p, and θ, calculated for the design d_v from the face of the support may be used" when completing the longitudinal reinforcement check at the inside edge of the bearing.

The balloted revisions to the AASHTO MBE include an equation for calculating the rating factor in Article C6A.5.8.

$$RF = \frac{\left(A_{ps}f_{ps} + A_s f_y\right) - \left[\frac{|M_{DL}|}{d_v \phi_f} + \frac{0.5N_{DL}}{\phi_c} + \left(\left|\frac{V_{DL}}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta\right]}{\left(\left(\frac{|M_{LL+IM}|}{d_v \phi_f}\right) + \frac{0.5N_{LL+IM}}{\phi_c} + \left(\frac{V_{LL+IM}}{\phi_v}\right)\cot\theta\right)}$$

This rating factor can be used to determine the live load shear for the next iteration using the same procedure as for sectional shear resistance as follows.

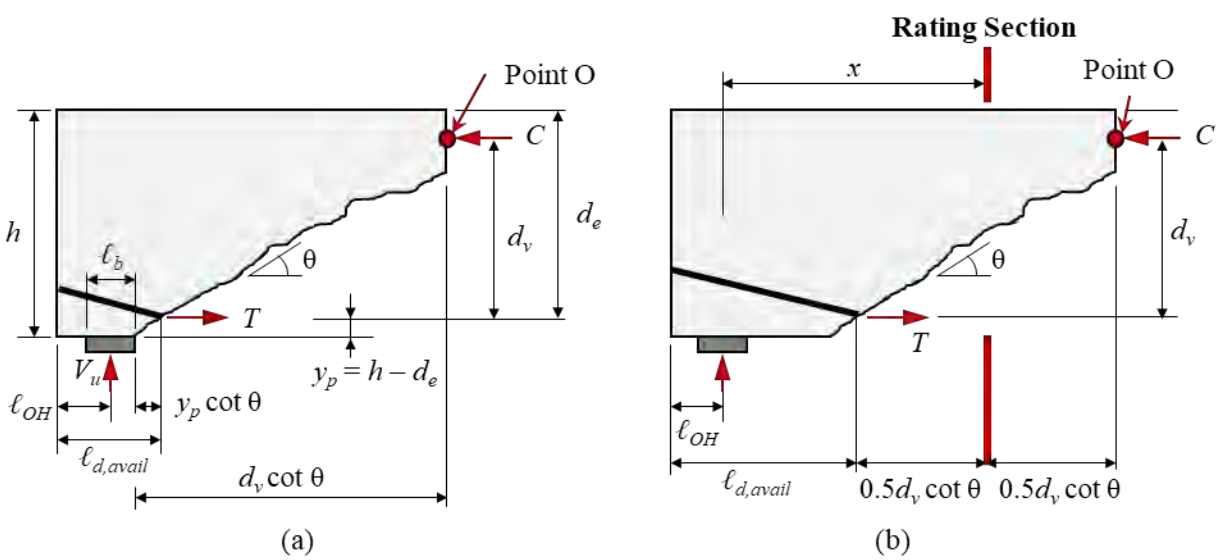
Live load shear for future iterations: $V_{u,(LL+IM),(i+1)} = ((1 - R_f) \cdot RF_{(i-1)} + R_f \cdot RF_{(i)}) \cdot V_{u,(LL+IM)}$

Rating factor for iteration 0: $RF_{(0)} = 1.0$

The values that update for each iteration are those related to MCFT, including f_{ps}, V_s, and θ. The other variables in the rating factor calculation remain constant for each iteration.

Available Development Length at Failure Crack

The available development length of the tension tie at the point the tie crosses the assumed failure crack, $\ell_{d,avail}$, needs to be calculated for the prestressing strands or non-prestressed reinforcement in the tension tie, T. The available development length for bonded prestressing strands is illustrated in Figure 15.



Source: FHWA

Figure 15. Illustration. Details for calculating available development length for bonded prestressing strands at (a) point where crack extends from inside bearing edge and (b) further into span.

For bonded prestressing strands, the available development length when the diagonal crack extends from the inside edge of the bearing is calculated assuming the free body diagram shown in Figure 15 (a). This will result in the following available development length.

Available development length (when crack extends from inside edge of bearing):

$$\ell_{d,avail} = \ell_{OH} + 0.5\ell_b + y_p \cot\theta$$

The free-body diagram shown in Figure 15 (b) is used to calculate the available development length further into the span, where x is the distance from the centroid of the bearing to the rating section.

Available development length (at x): $\ell_{d,avail} = \ell_{OH} + x - 0.5d_v \cot\theta$

The required transfer length and development lengths are calculated using AASHTO LRFD BDS Article 5.9.4.3.

Required transfer length: $\ell_t = 60d_b$

Required development length: $\ell_d = \kappa(f_{ps} - 2/3\, f_{pe})d_b$

The available development length for pretensioned members will often be less than the development length required to develop f_{ps}. AASHTO LRFD BDS Eqn. 5.9.4.3.2-2 should be used where $\ell_{d,avail} < \ell_t$, and AASHTO LRFD BDS Eqn. 5.9.4.3.2-3 should be used where $\ell_t \leq \ell_{d,avail} < \ell_d$.

Strand stress if $\ell_{d,avail} < \ell_t$: $f_{px} = (f_{pe} \times l_{d,avail}) / (60d_b)$

Strand stress if $\ell_t \leq \ell_{d,avail} < \ell_d$: $f_{px} = f_{pe} + (l_{d,avail} - 60d_b) / (l_d - 60d_b) \times (f_{ps} - f_{pe})$

Location for Calculating Demand

The location for calculating demand for the longitudinal reinforcement check is clear for the inside bearing edge simple end supports. The free-body diagram used for this point is shown in Figure 16. The moment M_u is ignored, where $M_u = 0$ kip-ft at the center of the simple end support. As previously mentioned, per AASHTO LRFD BDS Article C5.7.3.5, the "values of V_u, V_s, V_p, and θ, calculated for the design d_v from the face of the support may be used" when completing the longitudinal reinforcement check at the inside edge of the bearing. This simplification only applies to simple end supports, not interior supports of continuous spans.

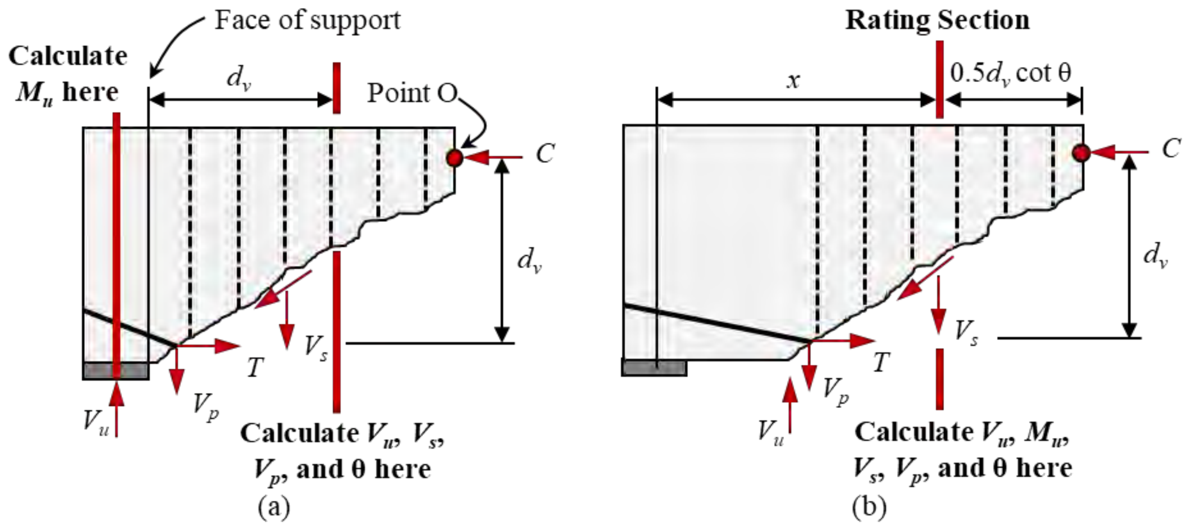


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Figure 16. Illustration. Assumed free-body diagrams for longitudinal reinforcement check for (a) inside edge of bearing and (b) further out into the span.

The free-body diagram for sections other than the inside face of the support is shown in Figure 16 (b). A few notes on this free-body diagram:

- AASHTO LRFD BDS Eqn. 5.7.3.5-1 assumes that V_u and V_p are at the same location, (d_v cot θ) from Point O in the derivation (see C5.7.3.5).
- The "Rating Section" for calculating the V_u and M_u for θ is at the mid-height of the section, i.e., 0.5(d_v cot θ) from Point O, see Figure 12.
- Assuming V_u at the Rating Section would lead to a lower calculated shear. However, it would also decrease the lever arm in the summation of moments about Point O from (d_v cot θ) to 0.5(d_v cot θ), which would suggest the $|V_u / \phi_v - V_p|$ term could be replaced by $|0.5V_u / \phi_v - V_p|$.

These observations on the free-body diagram and equations and commentary from AASHTO LRFD BDS suggest that it is reasonable to assume V_u and M_u be calculated at the mid-height of the crack for calculating θ and for use in AASHTO LRFD BDS Eqn. 5.7.3.5-1, as shown in Figure 16 (b). These same assumptions can be used for interior supports for continuous girders, where there may be significant negative moments.

Horizontal Shear Resistance

Shear load rating related to the horizontal shear resistance between the bottom flange and web of a component is not explicitly required by AASHTO MBE Article 6A.5.8, however, it is implied. AASHTO MBE Article 6A.5.8 requires the shear resistance be evaluated for rating loads and for in-service bridges showing visible signs of shear distress. Evaluation of the shear resistance should consider all types of shear distress that may control the resistance, which would include interface shear transfer. AASHTO LRFD BDS Article 5.7.4 specifies that interface shear resistance be considered across given planes at:

- An existing or potential crack;
- An interface between dissimilar materials;
- An interface between two concretes cast at different times; or
- The interface between different elements of the cross section.

The horizontal shear plane between the bottom flange and web would fall under the last category of planes to be considered. The recommendations in this section clarify how this requirement can be properly applied to shear load rating.

The non-iterative procedure for calculating the horizontal shear resistance of a member is based on research and recommendations provided by Hovell et al. (2013). Illustrations of the assumed failure plane for a horizontal shear failure and some of the key parameters required in demand and resistance calculations are shown in Figure 17.

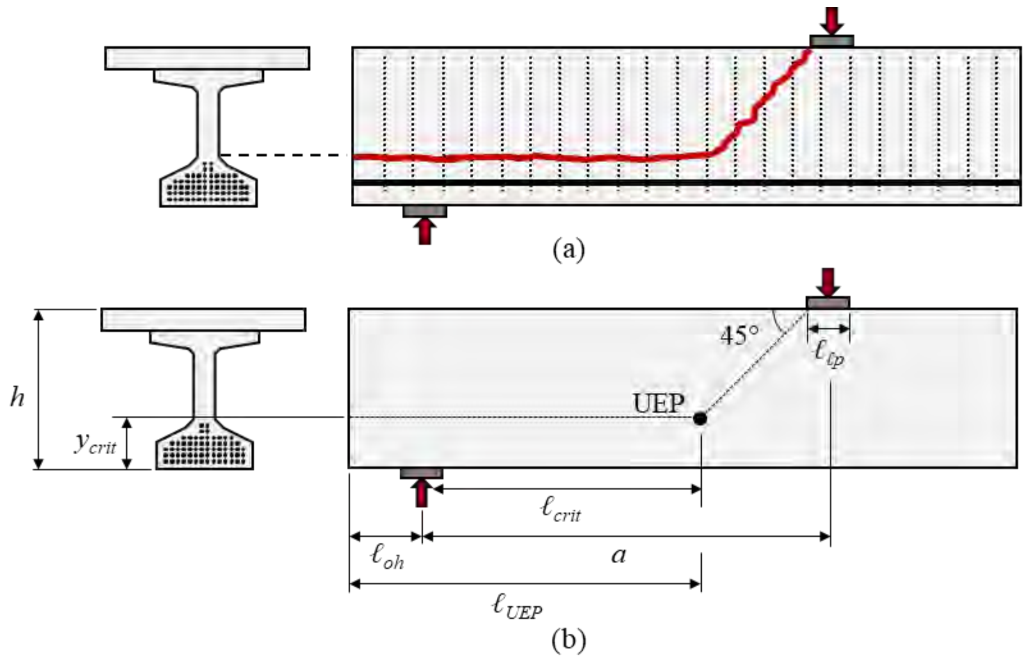

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Figure 17. Illustration. Details for horizontal shear failure calculations: (a) assumed failure plane and (b) key parameters required for calculations.

The diagonal failure crack is assumed to extend from the edge of the load plate toward the support at a 45-degree angle until it intersects with the horizontal failure plane. The point where the diagonal failure crack intersects with the horizontal failure plane is called the ultimate evaluation point (UEP). The distance between the end of the beam and the UEP is calculated as follows.

Distance from support centerline to UEP: $\ell_{UEP} = a + \ell_{oh} - 0.5\ell_{\ell p} - h + y_{crit}$

Horizontal Shear Demand, $V_{u,hs}$

The demand is based on the maximum vertical shear, V_u, when the closest axle of the truck used for the HL93 inventory loading or the appropriate rating loading is located a distance a from the center of the support (in addition to the lane load portion of the live load). The Design Truck used in HL-93 loading per AASHTO LRFD BDS Article 3.6.1.2, shown in Figure 18, is used as an example to show how the location and details of the design truck relate to the horizontal shear demand. The Design Truck has rear axle patch dimensions of 20 inch by 10 inch, shown in Figure 18 (b). The length of the load plate, $\ell_{\ell p}$, can be assumed to be equal to the longitudinal dimension of the axle patch, 10 inches.

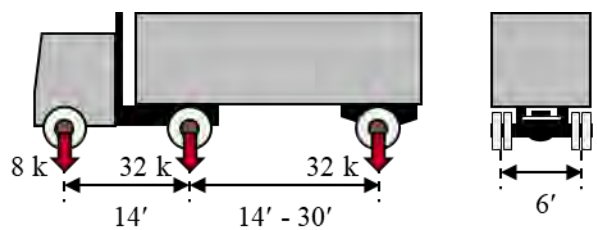

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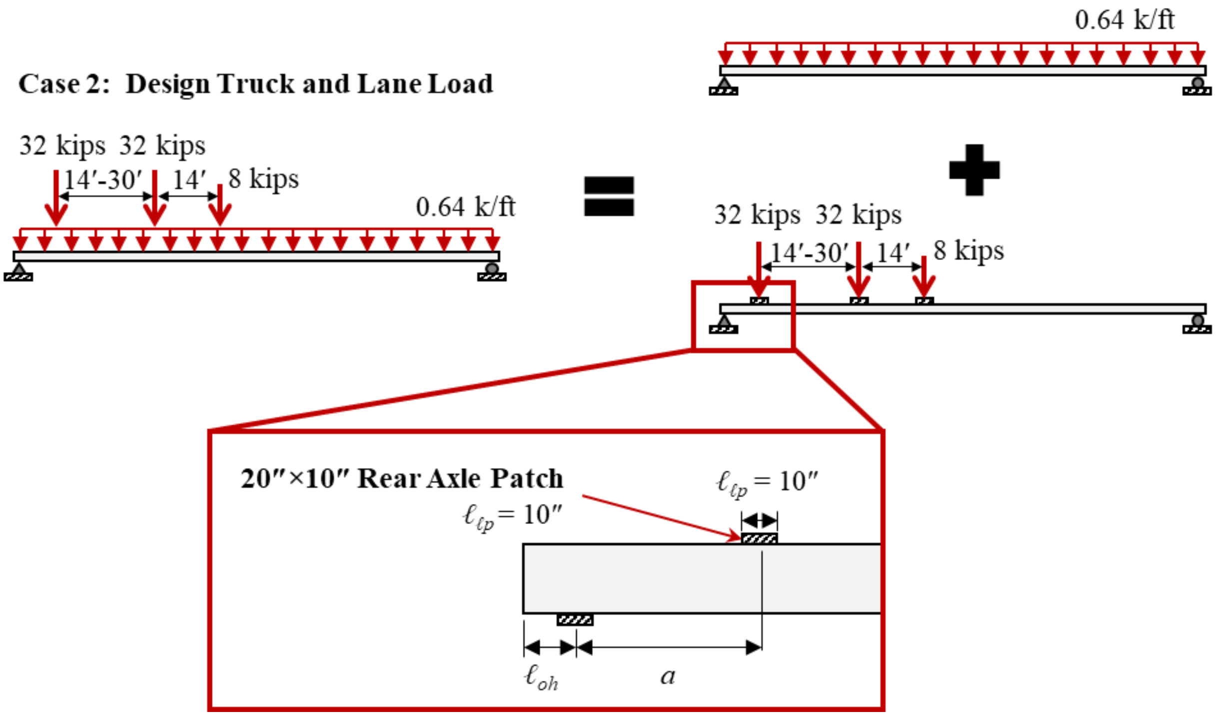
Figure 18. Illustration. Assumed details for Design Truck used in HL-93 loading based on AASHTO LRFD Figure 3.6.1.2.2-1.

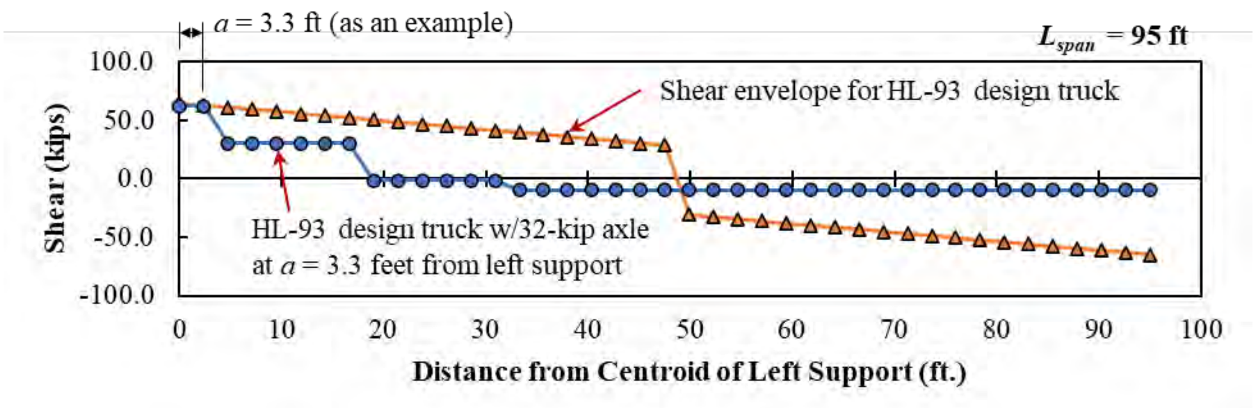
HL-93 loading consists of the loading from the Design Truck or Tandem and design lane load. The distance between the point load and the support, a, can be assumed to be based solely on the Design Truck, as shown in Figure 19.



Source: FHWA

Figure 19. Illustration. Location of HL-93 loading related to horizontal shear failure mechanism.

A sample shear diagram and shear envelope for a 95-foot span length with the Design Truck for HL-93 loading located 3.3 feet from the left support is shown in Figure 20. The shear demand for this point would be associated with the ultimate shear, V_u, at the critical section, d_v from the face of the support, when the Design Truck is in this position.

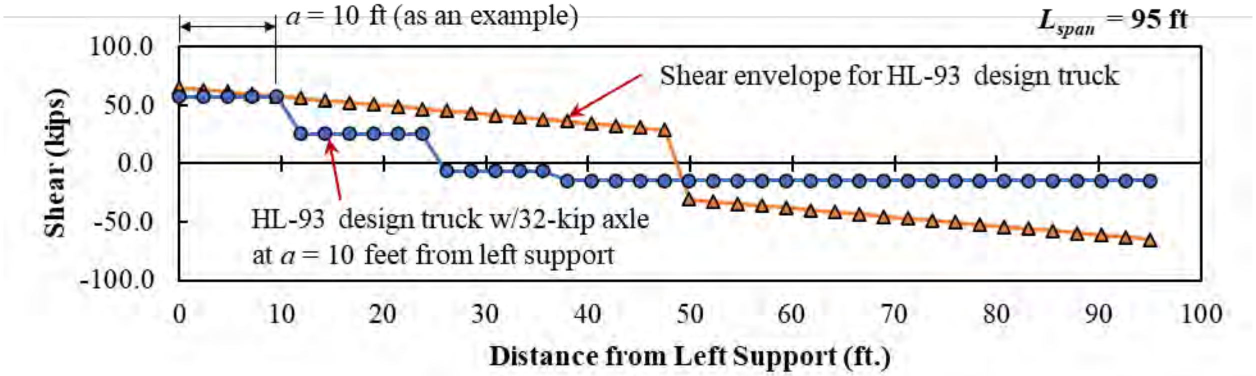


Source: FHWA

Figure 20. Graph. Sample shear demand along the length for Design Truck used in HL-93 loading for a 95-foot span length with *a* = 3.3 feet.

The position of the Design Truck would need to be modified by changing *a* and the associated shear demand used to check the horizontal shear resistance at multiple points along the length.

An example shear diagram for the Design Truck located 10 feet from the left support is shown in Figure 21.



Source: FHWA

Figure 21. Graph. Sample shear demand along the length for Design Truck used in HL-93 loading for a 95-foot span length with *a* = 10 feet.

As shown in Figure 20 and Figure 21, a typical shear envelope for the HL-93 Design Truck will reasonably capture the shear demand caused by the truck positioned with its rear axle at a distance a away from the center of the support. This suggests it is reasonable to assume $x = a$ for the demand and resistance calculations and use the vertical shear demand from the shear envelope at this point. The shear demand from the Design Truck should be combined with the shear demand from the lane loading.

The vertical shear demand, V_u, at the desired location $x = a$ from the center of the support is used to calculate the average vertical shear stress and horizontal shear stress as follows.

Average vertical shear stress: $v_{avg} = V_u / (b_v d_e)$

Horizontal shear force: $V_{u,hs} = v_{avg} b_v \ell_{crit}$

Distance from support centerline to UEP: $\ell_{crit} = \ell_{UEP} - \ell_{oh} = a - 0.5\ell_{\ell p} - h + y_{crit}$

Horizontal Shear Resistance, V_{ni}

The horizontal shear resistance is calculated using an equation modified slightly from AASHTO LRFD BDS Eqn. 5.7.4.3-3.

Horizontal shear resistance: $V_{ni} = k_d[cA_{cv} + \mu(A_{vf}f_y - 0.04P_{PS})]$

Horizontal shear resistance limits: $V_{ni} \leq$ minimum of $K_1 f'_c A_{cv}$ and $K_2 A_{cv}$

The beam shape factor, k_d, accounts for difference in behavior between U-beams and other types of beams observed by Hovell et al. (2013). The beam shape factor is 1.0 for I-beam, box-beams, and U-beams with typical reinforcement details and 0.8 for U-beams with detail described in in Hovell et al. (2013).

The cohesion factor c, friction factor μ, and K_1 and K_2 limit factors are based on the type of interface using the factors specified in AASHTO LRFD BDS Article 5.7.4.4. The horizontal shear friction plane will typically be monolithically placed normal weight concrete: $c = 0.40$ ksi, $\mu = 1.4$, $K_1 = 0.25$, and $K_2 = 1.5$ ksi.

The reinforcement crossing the interface within the transfer length will also resist splitting and bursting stresses from the release of the prestressing strands. AASHTO LRFD BDS Article 5.9.4.4 specifies that this reinforcement resists 4 percent of the prestressing force, P_{PS}. It is assumed that this will reduce the ability of these bars to resist horizontal sliding, so the $0.04P_{PS}$ is subtracted from the reinforcement component of the resistance within the transfer length.

The resistance should be calculated for different regions of interest from the beam end to the UEP. Some typical regions of interest for a pretensioned member are as follows.

- Beam end (resistance equal to zero).
- Transfer length region – distance equal to the larger of the transfer length or 36 inches from any point of prestress application, typically the beam end.
- Points of reinforcing bar spacing change.
- Points of web width change (e.g., end blocks).

The total resistance for the interface will be the summation of resistance in each region. This resistance is checked against the horizontal shear demand.

Load rating check for horizontal shear: $\phi V_{ni} \geq V_{u,hs}$

Shear Load Rating Factor Associated with Horizontal Shear

The shear load rating factor associated with horizontal shear can be calculated directly as follows. The horizontal shear demand, $V_{u,hs}$, and vertical shear demand, V_u, are related to each other through the average vertical shear stress, v_{avg}.

Horizontal shear demand: $V_{u,hs} = v_{avg}\, b_v\, \ell_{crit} = (V_u\, b_v\, \ell_{crit}) / (b_v\, d_e) = \phi V_{ni}$

The web width, b_v, cancels from this equation assuming $b_v \neq 0$ inches. This equation can be solved for the vertical shear demand as follows.

Vertical shear demand and horizontal shear resistance: $V_u = (\phi V_{ni}\, d_e) / \ell_{crit}$

The vertical shear resistance, ϕV_n, can be assumed equal to this vertical shear demand, $\phi V_n = V_u$, and used to calculate the associated shear load rating factor.

Load rating factor: $RF = (\phi V_n - V_{u(DC,DW)}) / V_{u(LL+IM)inv}$

This procedure can be used to calculate the horizontal shear demand, horizontal shear resistance, and associate load rating factor at multiple points along the length of the member.

MINIMUM TRANSVERSE REINFORCEMENT

Minimum transverse reinforcement is required per AASHTO LRFD BDS Article 5.7.2.3 where $V_u > 0.5\phi(V_c + V_p)$ or where consideration of torsion is required by Eqn. 5.7.2.1-3. The minimum transverse reinforcement is specified in AASHTO LRFD BDS Article 5.7.2.5 by Eqn. 5.7.2.5-1.

Min. transverse reinforcement: $A_v \geq 0.0316\lambda\sqrt{(f'_c)}\,(b_v s) / f_y$

The maximum spacing of transverse reinforcement is specified in AASHTO LRFD BDS Article 5.7.2.6, as summarized in Table 1.

Table 1. Maximum spacing of transverse reinforcement per AASHTO LRFD BDS Article 5.7.2.6.

Condition	Maximum spacing requirement
$v_u < 0.125f'_c$	$s_{max} = 0.8d_v \leq 24.0$ inch
$v_u \geq 0.125f'_c$	$s_{max} = 0.4d_v \leq 12.0$ inch

Whether or not a member has the minimum transverse reinforcement will dictate the equation to use for calculating the β factor for the concrete shear contribution for design.

Concrete shear factor w/min. transverse reinforcement (for design): $\beta = 4.8 / (1 + 750\varepsilon_s)$

Concrete shear factor w/o min. transverse reinforcement (for design):

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

Crack spacing parameter: $s_{xe} = s_x\,(1.38 / (a_g + 0.63))$; 12 inch $\leq s_{xe} \leq$ 80 inch

FHWA-HIF-22-025 referencing Choi et al. (2021) recommend that minimum transverse reinforcement is not required for prestressed concrete members where $(f_{pc} / f'_c) \geq 0.02$, where f_{pc} is the axial stress in the concrete $f_{pc} = (A_{ps} f_{pj}) / A_g$. Choi et al. (2021) found that these levels of prestressing provide a longitudinal clamping force across the shear crack that mitigates the size effect typical for members without transverse reinforcement. These recommendations were approved and adopted into the AASHTO MBE.

In summary, FHWA-HIF-22-025 and the approved revisions to the AASHTO MBE specify the following.

Concrete shear factor w/min. transverse reinforcement and/or where $(f_{pc} / f'_c) \geq 0.02$ (for shear load rating):

$$\beta = 4.8 / (1 + 750\varepsilon_s)$$

Concrete shear factor w/o min. transverse reinforcement and where (f_{pc} / f'_c) < 0.02
(for shear load rating):

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

These revisions will increase the concrete contribution to the shear resistance, V_c. A sample of the percentage increase of V_c if the minimum transverse reinforcement requirement is ignored is provided in Table 2. The revisions will have a larger effect on deeper members, as size effect would typically assume a larger reduction in shear strength for deeper members.

Table 2. Sample of size effect reduction for members without transverse reinforcement, based on AASHTO LRFD BDS Eqn. 5.7.3.4.2-2.

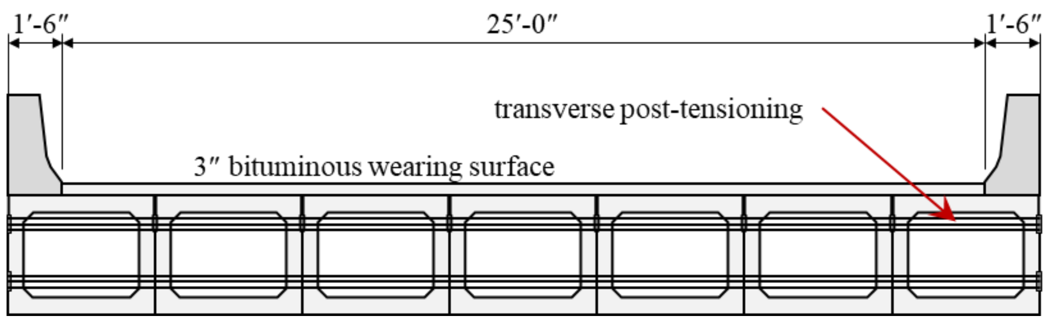
Crack spacing parameter, s_x (inch)	Max. aggregate size, a_g (inch)	Crack spacing parameter, s_{xe} (inch)	[51 / (39 + s_{xe})]	% increase for V_c if size effect ignored
20	0.75	20.0	0.864	13.6%
50	0.75	50.0	0.573	42.7%
80	0.75	80.0	0.429	57.1%
20	1.50	13.0	0.982	1.8%
50	1.50	32.4	0.714	28.6%
80	1.50	51.8	0.561	43.9%

These revisions are used in the following shear load rating example.

CHAPTER 3. SHEAR LOAD RATING EXAMPLE

INTRODUCTION

This shear load rating example is based on PCI *Bridge Design Manual* Example 9.4. This bridge is assumed to have seven BIII-48 box beams in an adjacent beam configuration, as shown in Figure 22. The adjacent box beams have transverse post-tensioning for force transfer between beams, but no composite cast-in-place deck.



Source: FHWA

Figure 22. Illustration. Bridge cross section for adjacent box beam bridge to be load rated for shear, based on Example 9.4 from the PCI *Bridge Design Manual*.

The span length and other properties for the bridge are summarized below.

Span length: L = 95 ft

Beam length: L_{beam} = 96 ft

Width of support: ℓ_b = 0.5 ft

Overhang length (center of support to end of beam): ℓ_{oh} = 0.5 ft

Clear roadway width: W_R = 25 ft

The beam spacing is equal to the beam width for an adjacent box beam configuration.

Beam spacing: S = 4 ft

Number of girders: N_b = 7

The number of lanes is calculated from AASHTO LRFD BDS Article 3.6.1.1.1.

Number of lanes: N_{lanes} = (W_R / 12 ft) rounded down to nearest integer = 2

The thickness of the current wearing surface at the time of the load rating.

Thickness of bituminous wearing surface: t_{ws} = 3 inch

Density of wearing surface (asphalt): w_a = 0.145 kcf

All details for this example can be found in the PCI *Bridge Design Manual* (2014). Only the variable definitions and calculations associated with the shear load rating are provided in this chapter.

DEFINITIONS

Material Definitions

The conventional concrete material properties for this example are as follows:

- Compressive strength at transfer: f'_{ci} = 4.0 ksi
- Compressive strength for use in load rating: f'_c = 5.0 ksi
- Correction factor for modulus of elasticity: K_1 = 1.0
- Concrete unit weight: w_c = 0.150 kcf
- Lightweight concrete factor: λ = 1.0

The modulus of elasticity of the concrete at transfer and at service are found using AASHTO LRFD Eqn. 5.4.2.4-1.

Modulus of elasticity at transfer: $E_{ci} = 120{,}000K_1\, w_c^{2.0} f'^{0.33}_{ci}$ = 4,266 ksi

Modulus of elasticity for use in load rating: $E_c = 120{,}000K_1\, w_c^{2.0} f'^{0.33}_c$ = 4,592 ksi

The material properties for the conventional steel reinforcement (Grade 60) are as follows:

- Modulus of elasticity: E_s = 29,000 ksi
- Yield strength: f_y = 60 ksi

The material properties for the prestressing strands are as follows:

- Low-relaxation
- Modulus of elasticity: E_p = 28,500 ksi
- Ultimate strength: f_{pu} = 270 ksi
- Yield strength: f_{py} = 243 ksi

The area and diameter of the prestressing strands used in this example are as follows.

- Diameter of prestressing strands: d_b = 0.5 inch
- Area of one strand: $A_{p,0.5in}$ = 0.153 inch^2

Section Definition

This bridge consists of BIII-48 box beam sections with the following section properties.

- Height of non-composite section: h = 39 inch
- Gross area: A_g = 813 inch^2
- Gross moment of inertia: I_g = 168,367 inch^4
- Distance from centroid to extreme bottom fiber: y_b = 19.29 inch
- Distance from centroid to extreme top fiber: y_t = 19.71 inch
- Section modulus for extreme bottom fiber: $S_b = I_g / y_b$ = 8,728 inch^3
- Section modulus for extreme top fiber: $S_t = I_g / y_t$ = 8,542 inch^3

Some additional material properties include the following.

- Effective flange width: b_e = 48 inch
- Width of top flange: $b_{tf} = b_e$ = 48 inch
- Thickness of top flange: t_{tf} = 5.5 inch
- Width of girder web: b_v = 10 inch (includes 5 inches for each web of the box section)
- Width of bottom flange: b_{bf} = 48 inch
- Thickness of bottom flange: t_{bf} = 5.5 inch
- Area on flexural tension side of beam: $A_{ct} = 0.5A_g$ (estimate) = 406.5 inch^2

The distance between the bottom and the possible horizontal shear plane will be assumed to be at the top of the chamfer above the bottom flange. The chamfer is 3 inches, so the critical distance is as follows.

Distance from bottom to horizontal shear plane: y_{crit} = 5.5 inch + 3 inch = 8.5 inch

Strand Profile

The strand profile includes (29) 0.5-inch diameter prestressing strands on the flexural tension side and (2) fully-stressed 0.5-inch diameter top strands. Details on the strand profile are shown in Table 3 and Figure 23.

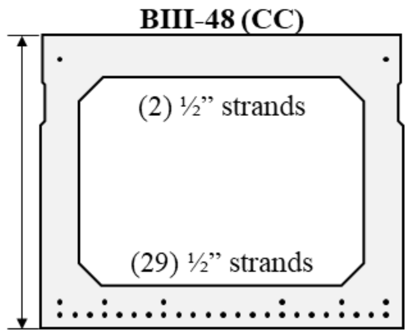
Table 3. Strand profile for box beam load rating example.

Layer #	Number of strands	Distance from bottom to centroid of strands
1	23	2 inches
2	6	4 inches
3	2	36 inches

All strands were fully stressed to a jacking stress, f_{pj}, of 202.5 ksi. The centroid of the strands and centroid of the strands on the flexural tension side of the beam are as follows.

- Total strand area: A_p = (31)(0.153 inch^2) = 4.743 inch^2
- Centroid of all strands: y_p = 4.58 inch
- Area of strands on flexural tension side: A_{ps} = (29)(0.153 inch^2) = 4.437 inch^2

- Centroid of strands on flexural tension side: $y_{p,tens}$ = 2.41 inch



Source: FHWA

Figure 23. Illustration. Cross section and strand layout for BIII-48 beams in example bridge.

Prestress losses were calculated using the AASHTO LRFD BDS Article 5.9.3 and the Approximate Estimate of Time-Dependent Losses in AASHTO LRFD BDS Article 5.9.3.3. The estimated prestress losses were as follows.

- Elastic shortening loss: Δf_{pES} = 8.6 ksi
- Total long-term loss: Δf_{pLT} = 26.2 ksi
- Total loss: Δf_{pT} = 34.8 ksi

Gross section properties were used in this example. Elastic gains were not included in the loss estimates.

The effective shear depth is the distance between the compression and tension force resultants. The effective shear depth was calculated using AASHTO LRFD BDS Article 5.7.2.8 and Eqn. C5.7.2.8-1 as follows.

Depth of prestressing strands on flexural tension side: $d_e = h - y_{p,tens}$ = 36.59 inch

Strand stress at nominal flexural resistance: f_{ps} = 255.2 ksi

Compression block depth: a = 5.74 inch

Nominal flexural resistance: M_n = 38,280 kip-inch

Effective shear depth: $d_v = M_n / (A_{ps} f_{ps})$ = 38,280 kip-inch / [(4.437 inch^2)(255.2 ksi)]
d_v = 33.81 inch

Lower limits for effective shear depth: $d_v \geq$ greater of ($0.9d_e$ = 32.9 inch) and ($0.72h$ = 28.1 inch)

The precompression ratio, f_{pc} / f'_c, for this member is as follows.

Precompression ratio: $f_{pc} / f'_c = (A_{ps} f_{pj}) / (A_g f'_c)$

f_{pc} / f'_c = (4.743 inch^2)(202.5 ksi) / ((813 inch^2)(5.0 ksi)) = 0.236

For this example, (f_{pc} / f'_c) = 0.236 ≥ 0.02, so the approved revisions to the AASHTO MBE based on FHWA-HIF-22-025 would specify that AASHTO LRFD BDS Eqn. 5.7.3.4.2-1 be used for the shear load rating regardless of the presence of minimum transverse reinforcement.

Shear Forces and Bending Moments

The distributed loads applied to the superstructure in this example include the following components.

- Beam self-weight: w_g = 0.847 kip/ft
- Barrier weight (per side): w_{bps} = 0.3 kip/ft
- Barrier weight (per beam): $w_b = (2w_{bps}) / N_b$ = 0.0857 kip/ft
- Wearing surface weight (per beam): $w_{ws} = (t_{ws}\, w_a\, W_R) / N_b$ = 0.125 kip/ft
- Diaphragms (assumed as distributed loads): w_{dia} = 0.030 kip/ft

The shear and moment for each type of load were calculated at each section of interest along the length of the beam. The procedure for calculating the maximum shear and concurrent moment described in Chapter 2 were used for calculating the shear and moment due to the HL-93 loading. Results from using the maximum shear with the maximum moment, calculated using the simplified equations provided in the PCI *Bridge Design Manual* Article 8.11.1, are also provided as a comparison.

Per AASHTO MBE Article 6A.3.2, the approximate methods for distribution described in AASHTO LRFD BDS Article 4.6.2.2 were used to calculate the distribution factors for moment and shear. The distributions for this example were taken from Example 9.4 of the PCI *Bridge Design Manual* for interior girders.

Live load distribution factor for moment: DFM = 0.287 (two or more lanes loaded controls)

Live load distribution factor for shear: DFV = 0.443 (two or more lanes loaded controls)

More details on the distribution factor calculations can be found in the PCI *Bridge Design Manual*.

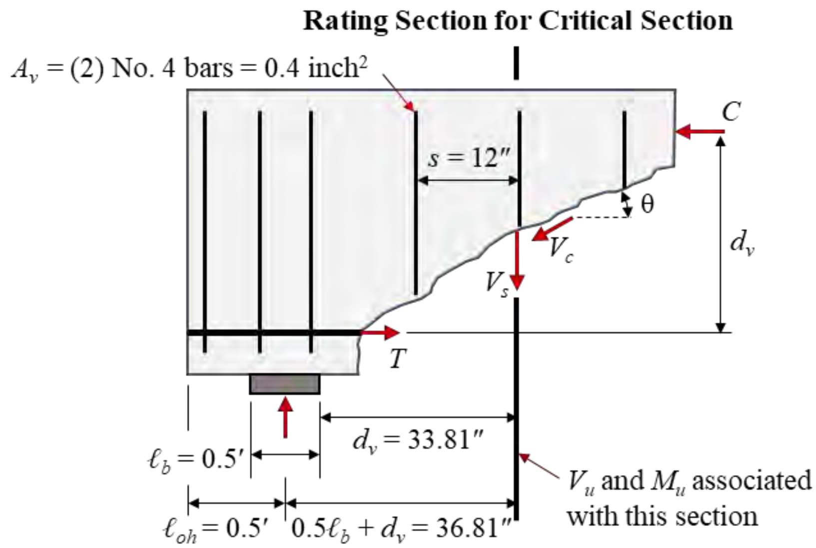
The dynamic load allowance was found using AASHTO LRFD BDS Article 3.6.2. A dynamic load allowance of 33 percent (IM = 0.33) was used for this example.

SHEAR LOAD RATING AT CRITICAL SECTION

Sample calculations are provided in this section for shear load rating at the critical section. The critical section for this example is as follows.

Critical section: $x_{cr} = 0.5\ell_b + d_v$ = 0.5(6 inch) + 33.81 inch = 36.81 inch

Details related to the critical section and end region of the beam are shown in Figure 24.



Source: FHWA

Figure 24. Illustration. Details related to the critical section for load rating example.

The shear and moments at the critical section are summarized in Table 4. The maximum shear and concurrent moment are summarized and shown alongside the maximum moment. The dead load components have the same concurrent moment and maximum moment. The lane load placement to cause maximum shear and maximum moment are different, so there is a small difference between the concurrent moment and maximum moment. The truck load placement to cause maximum shear is the same as that to cause maximum moment at the critical section.

Table 4. Summary of shear and moment at critical section (without distribution factors or impact factors).

Component	Maximum Shear (kips)	Concurrent Moment (kip-ft)	Maximum Moment (kip-ft)
Self-weight (DC)	37.6	119.4	119.4
Barrier (DC)	3.8	12.1	12.1
Wearing surface (DW)	5.6	17.6	17.6
Diaphragm (DC)	1.3	4.2	4.2
HL-93 Lane Load	28.5	87.3	90.0
HL-93 Truck	62.6	192.0	192.0

The ultimate shear is as follows. A live load factor of 1.75 is used, which represents the inventory level rating of the HL-93 design load.

Shear due to dead load: $V_{u(DC,DW)} = 1.25V_{DC} + 1.50V_{DW} = 1.25(37.6\text{ kips} + 3.8\text{ kips} + 1.3\text{ kips}) + 1.50(5.6\text{ kips}) = 61.8\text{ kips}$

Maximum shear due to live load: $V_{u(LL+IM)inv}$ = 1.75(0.443)(28.5 kips + 1.33(62.6 kips))
= 86.6 kips

Total shear: $V_u = V_{u(DC,DW)} + V_{u(LL+IM)inv}$ = 61.8 kips + 86.6 kips = 148.4 kips

The ultimate dead load moment is as follows.

Moment due to dead load: $M_{u(DC,DW)} = 1.25M_{DC} + 1.50M_{DW}$ = 1.25(119.4 kip-ft + 12.1 kip-ft
+ 4.2 kip-ft) + 1.50(17.6 kip-ft) = 196.2 kip-ft

The ultimate live load and total moment occurring concurrently with the maximum shear is as follows.

Moment due to live load occurring concurrently to maximum shear:
$M_{u(LL+IM)inv}$ = 1.75(0.287)(87.3 kip-ft + 1.33(192.0 kip-ft)) = 172.1 kip-ft

Total moment occurring concurrently to maximum shear:
$M_u = M_{u(DC,DW)} + M_{u(LL+IM)inv}$ = 196.2 kip-ft + 172.1 kip-ft = 368.3 kip-ft = 4,420 kip-inch

The ratio of live load moment to live load shear is as follows. The live load factor cancels out in this equation.

Live load moment to shear ratio: $\eta_{LL} = M_{u(LL+IM)inv} / V_{u(LL+IM)inv}$ = 23.8 inch

The shear reinforcement at this location is (2) legs of No. 4 bars spaced at 12 inches on center.

- Area of transverse reinforcement: A_v = (2)(0.2 inch^2) = 0.4 inch^2
- Spacing of transverse reinforcement: s = 12 inch

The maximum spacing requirement even for regions of high stress is satisfied by s = 12 inch (AASHTO LRFD BDS Article 5.7.2.6).

Max. spacing for $v_u \geq 0.125f'_c$: $s_{max} = 0.4d_v$ = 0.4(33.81 inch) = 13.5 inch ≤ 12.0 inch

The minimum transverse reinforcement associated with the 12-inch spacing is as follows (AASHTO LRFD BDS Eqn. 5.7.2.5-1).

Min. transverse reinforcement: $A_v \geq 0.0316\lambda\sqrt{(f'_c)} \times (b_v s) / f_y$

$A_v \geq 0.0316(1.0)\sqrt{(5.0\text{ ksi})} \times (10\text{ inch})(12\text{ inch}) / (60\text{ ksi}) = 0.141\text{ inch}^2$

The provided transverse reinforcement at this location (A_v = 0.4 inch^2) is greater than the minimum transverse reinforcement requirement.

Sectional Shear Resistance

The sectional shear resistance was calculated using the iterative process described in Chapter 2.

Step 1: Assume a live load and the associated V_u, M_u, and N_u. The first assumed live load should be equal to HL-93 loading or the appropriate rating truck or inventory loading. Future iterations can be equal to this load times a multiplier that would change during each iteration.

Assume $V_{u(LL)}$ = 86.6 kips for Iteration #1.

Associated V_u: $V_u = V_{u(DC,DW)} + V_{u(LL+IM)} = 148.4$ kips

Associate M_u: $M_u = M_{u(DC,DW)} + \eta_{LL} V_{u(LL+IM)} = 4{,}420$ kip-inch

Step 2: Calculate the associated net longitudinal strain in the section at the centroid of the tension reinforcement, ε_s, using AASHTO LRFD BDS Eqn. 5.7.3.4.2-4.

From the definition for $|M_u|$, $|M_u| \geq |V_u - V_p|d_v$.

Lower limit for $|M_u / d_v|$: $|M_u / d_v|$ = (4,420 kip-inch / 33.81 inch) = 131.2 kips ≥ $|V_u - V_p|$ = 148.4 kips → use $|M_u / d_v| = |V_u - V_p|$ = 148.4 kips in ε_s.

Assume that $\varepsilon_s < 0$ for this prestressed concrete section. If this is the case, then $\varepsilon_s = 0$ can be used and the iterative procedure is not required. The actual ε_s will be calculated and used since it will allow for a higher shear resistance to be calculated.

Longitudinal tensile strain in the section at centroid of tension reinforcement (if $\varepsilon_s < 0$):

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

$$\varepsilon_s = \frac{|148.4\text{ kips}| + |148.4\text{ kips}| - (4.437\text{ inch}^2)(189\text{ ksi})}{(28{,}500\text{ ksi})(4.437\text{ inch}^2) + (4{,}592\text{ ksi})(406.5\text{ inch}^2)}$$

$\varepsilon_s = -0.00027 \geq -0.0004$ (lower limit on ε_s)

$\varepsilon_s = -0.00027$

This longitudinal tensile strain is used for this iteration.

Step 3: Calculate the associated β and θ using AASHTO LRFD BDS Eqn. 5.7.3.4.2-1, Eqn. 5.7.3.4.2-2, and Eqn. 5.7.3.4.2-3. Use Eqn. 5.7.3.4.2-1 since $(f_{pc} / f'_c) = 0.236 \geq 0.02$..

Concrete shear factor (w/min. transverse reinforcement): $\beta = 4.8 / (1 + 750\varepsilon_s)$

$\beta = 4.8 / (1 + 750(-0.00027)) = 6.03$

Angle of inclination of the diagonal compressive stresses: $\theta = 29 + 3500\varepsilon_s$

$\theta = 29 + 3500(-0.00027) = 28.0°$

Step 4: Calculate the nominal shear resistance, V_n, using AASHTO LRFD BDS Eqn. 5.7.3.3-1 through Eqn. 5.7.3.3-5 with ϕ from AASHTO LRFD BDS Article 5.5.4.2. For shear and torsion in monolithic prestressed concrete sections, ϕ = 0.9.

Nominal shear resistance provided by concrete: $V_c = 0.0316\beta\lambda\sqrt{(f'_c)}\, b_v d_v$

$V_c = 0.0316(6.03)(1.0)\sqrt{(5.0\text{ ksi})} \times (10\text{ inch})(33.81\text{ inch}) = 144.0\text{ kips}$

Nominal shear resistance provided by transverse reinforcement (α = 90°):

$V_s = [(A_v f_y d_v \cot\theta) / s]\, \lambda_{duct}$

$V_s = [((0.40\text{ inch}^2)(60\text{ ksi})(33.81\text{ inch}) \cot(28.0°)) / (12\text{ inch})](1.0)$

$V_s = 126.9\text{ kips}$

Nominal shear resistance: $V_n = V_c + V_s + V_p \leq 0.25 f'_c b_v d_v + V_p$

$V_n = 144.0\text{ kips} + 126.9\text{ kips} + 0\text{ kips} = 270.9\text{ kips}$

$V_n \leq 0.25 f'_c b_v d_v + V_p = 0.25(5\text{ ksi})(10\text{ inch})(33.81\text{ inch}) + 0\text{ kips} = 422.6\text{ kips}$

$V_n = 270.9\text{ kips}$

Step 5: Check to see if $\phi V_n = V_u$. If $\phi V_n = V_u$, then proceed to Step 6. Otherwise, return to Step 1 and assume a new live load.

Factored nominal shear resistance: $\phi V_n = (0.9)(270.9\text{ kips}) = 243.9\text{ kips}$

Check for iteration: $\phi V_n = 243.9\text{ kips} \neq V_u = 148.4\text{ kips}$ → **Return to Step 1**.

The live load shear to assume in the next iteration can be based on the rating factor calculated in the previous iteration. A relaxation factor of 1.0 was used for this example, $R_f = 1.0$.

Rating factor from iteration 1: $RF_{(1)} = (\phi V_{n(1)} - V_{u(DC,DW)}) / V_{u,(LL+IM)}$

$RF_{(1)} = (243.9\text{ kips} - 61.8\text{ kips}) / 86.6\text{ kips} = 2.10$

Guess for next iteration: $V_{u,(LL+IM),(2)} = ((1 - R_f) \cdot RF_{(0)} + R_f \cdot RF_{(1)}) \cdot V_{u,(LL+IM)}$

$V_{u,(LL+IM),(2)} = [(1 - 1.0)(1.0) + (1.0)(2.10)](86.6\text{ kips}) = 182.0\text{ kips}$

A summary of five iterations for this example is shown in Table 5. The solution converged after five iterations with $R_f = 1.0$ for this example.

Table 5. Summary of iterations for calculation sectional shear resistance.

i	$RF_{(i-1)}$	$V_{u(LL+IM)}$ (kips)	V_u (kips)	ε_s	β	θ (deg)	V_c (kips)	V_s (kips)	ϕV_n (kips)	$RF_{(i)}$
1	1.00	86.6	148.4	-0.00027	6.03	28.0	144.0	126.9	243.9	2.10
2	2.10	182.0	243.9	-0.00018	5.53	28.4	132.1	125.1	231.5	1.96
3	1.96	169.7	231.5	-0.00019	5.59	28.3	133.5	125.4	233.0	1.98
4	1.98	171.2	233.0	-0.00019	5.58	28.3	133.4	125.3	232.8	1.97
5	1.97	171.0	232.8	-0.00019	5.58	28.3	133.4	125.3	232.9	1.97

Check for iteration: ϕV_n = 232.8 kips ≈ V_u = 233.0 kips → **Continue to Step 6**.

A solver (e.g., Goal Seek in Excel) can be used to change $V_{u(LL+IM)}$ until $\phi V_n = V_u$.

Step 6: $\phi V_n = V_u$ for sectional shear.

The load rating factor for the design HL-93 loading at the inventory level associated with this sectional shear resistance can be calculated as follows.

Load rating factor associated with sectional shear resistance:

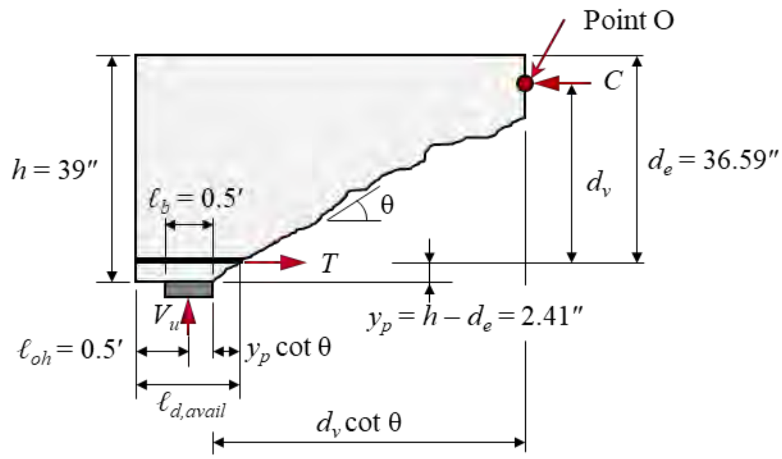
$$RF = (\phi V_n - V_{u(DC,DW)}) / V_{u(LL+IM)inv}$$

$$RF = (232.9 \text{ kips} - 61.8 \text{ kips}) / 86.6 \text{ kips} = 1.97$$

This is the load rating associated with sectional shear at the critical section (d_v from the face of the support). Continue with the longitudinal reinforcement check.

Longitudinal Reinforcement Check

The longitudinal reinforcement check will be computed where the diagonal crack extends from the inside edge of the bearing in this section, as shown in Figure 25. Per AASHTO LRFD BDS Article C5.7.3.5, the "values of V_u, V_s, V_p, and θ, calculated for the design d_v from the face of the support may be used" when completing the longitudinal reinforcement check at the inside edge of the bearing.



Source: FHWA

Figure 25. Illustration. Details related to the longitudinal reinforcement check at the inside edge of the simple end bearing for the load rating example.

Step 1: Assume a live load and the associated V_u, M_u, and N_u. The first assumed live load should be equal to HL-93 loading or the appropriate permit truck or inventory loading.

When performing this check at the inside face of the bearing, the shear and moments at the critical section should be used.

Assume $V_{u(LL+IM)}$ = 86.6 kips for Iteration #1.

Associated V_u: $V_u = V_{u(DC,DW)} + V_{u(LL+IM)} = 148.4$ kips

Associate M_u: $M_u = M_{u(DC,DW)} + \eta_{LL} V_{u(LL+IM)} = 4{,}420$ kip-inch

Step 2: Calculate net longitudinal strain, θ, and V_s.

From the definition for $|M_u|$, $|M_u| \geq |V_u - V_p| d_v$.

Lower limit for $|M_u / d_v|$: $|M_u / d_v|$ = (4,420 kip-inch / 33.81 inch) = 130.7 kips
$\geq |V_u - V_p| = 148.4$ kips → use $|M_u / d_v| = |V_u - V_p| = 148.4$ kips in ε_s.

Assume that $\varepsilon_s < 0$ for this prestressed concrete section. If this is the case, then $\varepsilon_s = 0$ can be used and the iterative procedure is not required. The actual ε_s will be calculated and used since it will allow for a higher shear resistance to be calculated.

Longitudinal tensile strain in the section at centroid of tension reinforcement (if $\varepsilon_s < 0$):

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

$$\varepsilon_s = \frac{|148.4 \text{ kips}| + |148.4 \text{ kips}| - (4.437 \text{ inch}^2)(189 \text{ ksi})}{(28{,}500 \text{ ksi})(4.437 \text{ inch}^2) + (4{,}592 \text{ ksi})(406.5 \text{ inch}^2)}$$

$\varepsilon_s = -0.00027 \geq -0.0004$ (lower limit on ε_s)

$\varepsilon_s = -0.00027$

This longitudinal tensile strain is used for this iteration.

Angle of inclination of the diagonal compressive stresses: $\theta = 29 + 3500\varepsilon_s$

$\theta = 29 + 3500(-0.00027) = 28.0°$

Nominal shear resistance provided by transverse reinforcement ($\alpha = 90°$):

$V_s = [(A_v f_y d_v \cot\theta) / s]\, \lambda_{duct}$

$V_s = [((0.40 \text{ inch}^2)(60 \text{ ksi})(33.81 \text{ inch}) \cot(28.0°)) / (12 \text{ inch})](1.0)$

$V_s = 126.9$ kips

Step 3: Perform longitudinal reinforcement check from AASHTO LRFD BDS Article 5.7.3.5.

The required transfer length and development lengths are calculated using AASHTO LRFD BDS Article 5.9.4.3.

Required transfer length: $\ell_t = 60d_b = 60(0.5 \text{ inch}) = 30$ inch

Required development length: $\ell_d = \kappa(f_{ps} - 2/3\, f_{pe})d_b = (1.6)(255.2 \text{ ksi} - 2/3(167.7 \text{ ksi}))(0.5 \text{ inch}) = 114.7$ inch

Available development length (when crack extends from inside edge of bearing):

$\ell_{d,avail} = \ell_{oh} + 0.5\ell_b + y_p \cot\theta$

$\ell_{d,avail} = 6 \text{ inch} + 0.5(6 \text{ inch}) + (2.41 \text{ inch})\cot(28.0°) = 13.5$ inch

The available development length is less than the transfer length in this case, so the stress in the strands is calculated using AASHTO LRFD BDS Eqn. 5.9.4.3.2-2.

Strand stress if $\ell_{d,avail} < \ell_t$: $f_{px} = (f_{pe} \times l_{d,avail}) / (60d_b)$

$f_{px} = (167.7 \text{ ksi})(13.5 \text{ inch}) / (60(0.5 \text{ inch})) = 75.6$ ksi

The left-hand side of AASHTO LRFD BDS Eqn. 5.7.3.5-2 is as follows.

$LHS = A_s f_y + A_{ps} f_{ps} = (0 \text{ inch}^2) + (4.437 \text{ inch}^2)(75.6 \text{ ksi}) = 335.4$ kips

The right-hand side of AASHTO LRFD BDS Eqn. 5.7.3.5-2 is as follows.

$RHS = (V_u / \phi - 0.5V_s - V_p) \cot\theta = (148.4 \text{ kips}/0.9 - 0.5(126.9 \text{ kips})) \cot(28.0°) = 190.5$ kips

Step 4: Check to see if the left-hand side of AASHTO LRFD BDS Eqn. 5.7.3.5-2 (or Eqn. 5.7.3.5-2 if not at the inside edge of the bearing) is equal to the right-hand side. If they are equal, progress to Step 5. Otherwise, return to Step 1 and assume a new live load.

The *LHS* = 335.4 kips is not equal to the *RHS* = 190.5 kips. Another iteration needs to be performed.

The load rating factor is calculated for this first iteration based on the equation included in the revised AASHTO MBE as follows.

$$RF = \frac{\left(A_{ps}f_{ps} + A_s f_y\right) - \left[\frac{|M_{DL}|}{d_v\phi_f} + \frac{0.5N_{DL}}{\phi_c} + \left(\left|\frac{V_{DL}}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta\right]}{\left(\left(\frac{|M_{LL+IM}|}{d_v\phi_f}\right) + \frac{0.5N_{LL+IM}}{\phi_c} + \left(\frac{V_{LL+IM}}{\phi_v}\right)\cot\theta\right)}$$

This equation simplifies at the inside edge of the bearing, where $M_{DL} = M_{LL+IM} = 0$ kip-inch. Some of the other components are also equal to zero since there is no axial load applied ($N_{DL} = N_{LL+IM} = 0$ kips) and no harped prestressing ($V_p = 0$ kips).

$$RF = \frac{((4.437\text{ in}^2)(75.6\text{ ksi})) - \left[\left(\left|\frac{61.8\text{ kips}}{0.9}\right| - 0.5(126.9\text{ kips})\right)\cot(28.0°)\right]}{\left(\left(\frac{86.6\text{ kips}}{0.9}\right)\cot(28.0°)\right)} = 1.80$$

The live load to use for the next iteration is calculated as follows. The relaxation factor is assumed to be equal to 1.0, $R_f = 1.0$, for this process.

$V_{u(LL+IM),2} = ((1 – 1)(1.0) + (1.0)(1.8))(86.6\text{ kips}) = 156.1\text{ kips}$

A summary of three iterations for this example is shown in Table 6. The solution converged after three iterations in this example.

Table 6. Summary of iterations for calculation shear resistance associated with longitudinal reinforcement check.

i	$RF_{(i-1)}$	$V_{u(LL+IM)}$ (kips)	V_u (kips)	ε_s	θ (deg)	V_s (kips)	RHS (kips)	f_{ps} (ksi)	LHS (kips)	$RF_{(i)}$
1	1.00	86.6	148.4	-0.00027	28.0	126.9	190.5	75.6	335.4	1.80
2	1.80	156.1	218.0	-0.00020	28.3	125.6	333.2	75.3	334.3	1.81
3	1.81	156.7	218.5	-0.00020	28.3	125.6	334.3	75.3	334.3	1.81

Check for iteration: (*LHS* = 334.3 kips) = (*RHS* = 334.3 kips) → **Continue to Step 5**.

A solver could also be used to change $V_{u(LL+IM)}$ until *LHS* = *RHS*.

Step 5: Shear resistance associated with the longitudinal reinforcement check, ϕV_n, is equal to the associated V_u when the right-hand and left-hand sides of the equation are equal.

Associated shear resistance: $\phi V_n = V_u$ = 218.5 kips (see Table 6)

The load rating associated with this longitudinal reinforcement check can be calculated as follows.

Load rating factor associated with sectional shear resistance:

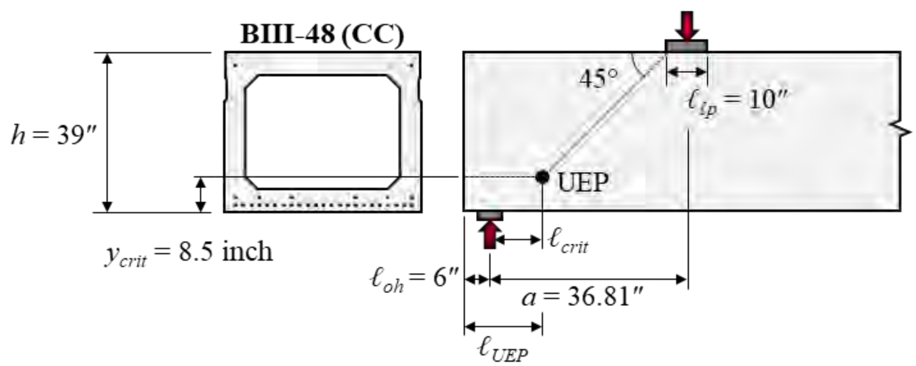
$$RF = (\phi V_n - V_{u(DC,DW)}) / V_{u(LL+IM)inv}$$

$$RF = (218.5 \text{ kips} - 61.8 \text{ kips}) / 86.6 \text{ kips} = 1.81$$

This is the load rating factor for the design HL-93 loading at the inventory level associated with the longitudinal reinforcement check when the shear crack extends from the inside face of the bearing. Continue with the horizontal shear check.

Horizontal Shear Resistance

The horizontal shear resistance will be calculated in this section of the example when the rear axle of the Design Truck is located at the critical section, i.e., where $a = x_{cr} = 0.5\ell_b + d_v$ = 36.81 inch, as shown in Figure 26. The procedure described in Chapter 2 and in Hovell et al. (2013) will be used in this example.



Source: FHWA

Figure 26. Illustration. Details related to the horizontal shear check for the load rating example.

The distance between the end of the beam and the UEP is calculated as follows.

Distance from support centerline to UEP: $\ell_{UEP} = a + \ell_{oh} - 0.5\ell_{\ell p} - h + y_{crit}$

$$\ell_{UEP} = 36.81 \text{ inch} + 6 \text{ inch} - 0.5(10 \text{ inch}) - 39 \text{ inch} + 8.5 \text{ inch} = 7.3 \text{ inch}$$

The horizontal shear resistance includes the concrete interface area and steel reinforcement crossing the interface area within the distance ℓ_{UEP} from the beam end. This distance is less than the transfer length, so only one region of interest is assumed for this calculation. It is assumed that (2) No. 4 ties with (2) legs each are provided within this ℓ_{UEP}.

Concrete area in ℓ_{UEP}: $A_{cv} = \ell_{UEP}\, b_v$ = (7.3 inch)(10 inch) = 73 inch^2

Steel area in ℓ_{UEP}: A_{vf} = (2)(2)(0.2 inch^2) = 0.8 inch^2

The total force being transferred includes the area of all the prestressing strands (including the top strands ignored for previous steps of the example).

Total force in prestressing strands at transfer: P_{PS} = (4.743 inch^2)(202.5 ksi) = 960.5 kips

The horizontal shear friction plane here is monolithically placed normal weight concrete: c = 0.40 ksi, μ = 1.4, K_1 = 0.25, and K_2 = 1.5 ksi. The beam shape factor for a box beam is k_d = 1.0. The horizontal shear resistance is calculated as follows.

Horizontal shear resistance: $V_{ni} = k_d[cA_{cv} + \mu(A_{vf}f_y - 0.04P_{PS})]$

V_{ni} = (1.0)[(0.40 ksi)(73 inch^2) + (1.0)((0.8 inch^2)(60 ksi) – 0.04(960.5 kips))] = 42.7 kips

This is checked against the upper limits for the horizontal shear resistance.

Resistance limit #1: $V_{ni} \leq K_1 f'_c A_{cv}$ = (0.25)(5.0 ksi)(73 inch^2) = 91.4 kips

Resistance limit #2: $V_{ni} \leq K_2 A_{cv}$ = (1.5 ksi)(73 inch^2) = 109.7 kips

The calculated resistance, V_{ni}, is less than the minimum of $K_1 f'_c A_{cv}$ and $K_2 A_{cv}$.

Factored horizontal shear resistance: ϕV_{ni} = (0.9)(42.7 kips) = 38.4 kips

As discussed in Chapter 2, the vertical shear demand, V_u, associated with this horizontal shear resistance can be solved for directly.

Distance from support centerline to UEP: $\ell_{crit} = \ell_{UEP} - \ell_{oh}$ = 7.3 inch – 6 inch = 1.3 inch

Vertical shear demand and horizontal shear resistance: $\phi V_n = V_u = (\phi V_{ni}\, d_e) / \ell_{crit}$

$\phi V_n = V_u$ = (38.4 kips)(36.59 inch) / (1.3 inch) = 1,072 kips

The associated load rating is calculated as follows.

Load rating: $RF = (\phi V_n - V_{u(DC,DW)}) / V_{u(LL+IM)inv}$

RF = (1,072 kips – 61.8 kips) / 86.6 kips = 11.67

Controlling Shear Failure Mechanism at Critical Section

The controlling shear mechanism is based on the minimum shear resistance and load rating of the three possible failure mechanisms. A summary of the three capacities and associated shear load rating factors is shown in Table 7.

Table 7. Summary of shear resistance at critical section for three different shear failure mechanisms.

Failure Mechanism	ϕV_n (kips)	Shear Load Rating Factor
Sectional Shear	232.8	1.97
Longitudinal Reinforcement	218.5	1.81
Horizontal Shear	1,072	11.67

The shear resistance associated with the longitudinal reinforcement check controls the resistance at the critical section.

Shear resistance at critical section: $\phi V_n = 218.5$ kips

Shear load rating factor at critical section: $LR = 1.81$

This would need to be repeated at multiple points along the length of the member.

Possible Expedients for Sectional Shear Resistance

A comparison of the load rating factors calculating using the different possible expedients for the sectional shear resistance is provided in this section.

- **Expedient #1**: Use the simplified procedure for non-prestressed sections from AASHTO LRFD BDS Article 5.7.3.4.1. This article specifies $\beta = 2.0$ and $\theta = 45°$.

  Concrete contribution: $V_c = 0.0316\beta\lambda\sqrt{(f'_c)}\, b_v d_v$
  $V_c = 0.0316(2.0)(1.0)\sqrt{(5.0 \text{ ksi})} \times (10 \text{ inch})(33.81 \text{ inch}) = 47.8$ kips

  Steel contribution: $V_s = A_v f_y d_v \cot\theta / s$
  $V_s = (0.4 \text{ inch}^2)(60 \text{ ksi})(33.81 \text{ inch})\cot(45°) / (12 \text{ inch}) = 67.6$ kips

  Factored shear resistance: $\phi V_n = \phi(V_c + V_s)$
  $\phi V_n = (0.9)(47.8 \text{ kips} + 67.6 \text{ kips}) = 103.9$ kips

  The associated load rating factor for Expedient #1 is as follows.

  Rating factor: $RF = (\phi V_n - V_{u(DC,DW)}) / V_{u(LL+IM)}$
  $RF = (103.9 \text{ kips} - 61.8 \text{ kips}) / (86.6 \text{ kips}) = 0.49$

  For this example, the resistance is not sufficient to hold the inventory load when estimated using Expedient #1.

- **Expedient #2**: Use the alternate shear design approach provided in AASHTO LRFD BDS Article 5.12.5.3.8. This is a non-iterative procedure that will generally provide conservative estimates compared to the general shear procedure using MCFT.

The precompression stress, f_{pc}, is calculated first as follows, since there is no cast-in-place composite deck.

Precompression stress for non-composite sections: $f_{pc} = (f_{pbt} - \Delta f_{pT}) / A_g$

$$f_{pc} = (202.5 \text{ ksi} - 34.8 \text{ ksi})(4.743 \text{ in}^2) / (813.0 \text{ in}^2) = 0.978 \text{ ksi}$$

This stress is used to calculate the K factor using AASHTO LRFD BDS Eqn. 5.12.5.3.8c-5 as follows.

$$K = \sqrt{1 + \frac{f_{pc}}{0.0632\lambda\sqrt{f'_c}}} = \sqrt{1 + \frac{0.978 \text{ ksi}}{0.0632(1.0)\sqrt{5.0 \text{ ksi}}}} = 2.81 \leq 2.0$$

$K = 2.0$

The concrete contribution to the shear resistance can next be calculated using AASHTO LRFD BDS Eqn. 5.12.5.3.8c-3 as follows.

Concrete contribution to shear resistance: $V_c = 0.0632K\lambda\sqrt{(f'_c)}\, b_v d$

$$V_c = 0.0632(2.0)(1.0)\sqrt{(5 \text{ ksi})}\,(10 \text{ inch})(33.81 \text{ inch}) = 95.6 \text{ kips}$$

The steel contribution to the shear strength is calculated using AASHTO LRFD BDS Eqn. 5.12.5.3.8c-4, which assumes a $\theta = 45°$, as follows.

Steel contribution to shear resistance: $V_s = (A_v f_y d) / s$

$$V_s = (0.4 \text{ inch}^2)(60 \text{ ksi})(33.81 \text{ inch}) / (12 \text{ inch}) = 67.6 \text{ kips}$$

This results in the following factored shear resistance and associated load rating factor for Expedient #2.

Factored shear resistance: $\phi V_n = \phi(V_c + V_s)$

$$\phi V_n = (0.9)(95.6 \text{ kips} + 67.6 \text{ kips}) = 146.9 \text{ kips}$$

Rating factor: $RF = (\phi V_n - V_{u(DC,DW)}) / V_{u(LL+IM)}$

$$RF = (146.9 \text{ kips} - 61.8 \text{ kips}) / (86.6 \text{ kips}) = 0.983$$

For this example, the resistance is not sufficient to hold the inventory load when estimated using Expedient #2.

- **Expedient #3**: Use AASHTO LRFD Article 5.7.3.4.2 (MCFT General Procedure) and treat the load rating problem like a design problem. If the provided (A_v / s) for member in question satisfies design requirements, then the member provides adequate strength. This expedient will show if the member can safely carry the load but does not provide the peak member shear strength, which would be used for determining the shear load rating.

This expedient is the same as Iteration #1 for the section shear resistance calculations, summarized in Table 5. The calculated resistance and demand for the HL-93 live load is as follows.

Factored shear resistance: $\phi V_n = 243.9$ kips

Demand with HL-93 loading: $V_u = 148.4$ kips

Design check: $\phi V_n = 243.9 \text{ kips} \geq V_u = 148.4$ kips

The factored shear resistance exceeds the demand, so the member will be able to safely hold the load. However, this factored shear resistance is not the actual resistance of the member. The iterations in Table 5 illustrate how the actual factored shear resistance is lower than that calculated in the first iteration. The load rating factor cannot be calculated using this expedient.

- **Expedient #4**: Use $\varepsilon_s = 0$ if $\varepsilon_s < 0$, which is true if $M_u < M_{cr}$. This expedient is included in the revised AASHTO MBE. This simplification will eliminate the iterative procedure. The load rater must make sure that $M_u < M_{cr}$ for the increased load to get $\phi V_n = V_u$.

  The cracking moment would have been calculated during the flexural analysis portion of the load rating. In this example, the cracking moment at this section was calculated as the following.

  Cracking moment for example: $M_{cr} = 29{,}753$ kip-inch

  The ultimate moment due to the HL-93 loading at the critical section (Iteration #1 in Table 5) was calculated as the following.

  Moment demand for first iteration: $M_u = 4{,}420$ kip-inch

  The demand is less than the cracking moment at this section, $M_u < M_{cr}$, so Expedient #4 may be used as shown in the following calculations.

  Longitudinal tensile strain: $\varepsilon_s = 0$

  Concrete shear factor (w/min. transverse reinforcement): $\beta = 4.8 / (1 + 750\varepsilon_s)$
  $\beta = 4.8 / (1 + 750(0)) = 4.8$

  Angle of inclination of the diagonal compressive stresses: $\theta = 29 + 3500\varepsilon_s$
  $\theta = 29 + 3500(0) = 29.0°$

  Nominal shear resistance provided by concrete: $V_c = 0.0316\beta\lambda\sqrt{(f'_c)}\, b_v d_v$
  $V_c = 0.0316(4.8)(1.0)\sqrt{(5.0 \text{ ksi})} \times (10 \text{ inch})(33.81 \text{ inch}) = 114.7$ kips

  Nominal shear resistance provided by transverse reinforcement ($\alpha = 90°$):
  $V_s = [(A_v f_y d_v \cot\theta) / s]\, \lambda_{duct}$

V_s = [((0.40 inch^2)(60 ksi)(33.81 inch) cot(29.0°)) / (12 inch)](1.0)

V_s = 122.0 kips

Factored shear resistance: $\phi V_n = \phi(V_c + V_s)$

ϕV_n = (0.9)(114.7 kips + 122.0 kips) = 213.0 kips

Rating factor: $RF = (\phi V_n - V_{u(DC,DW)}) / V_{u(LL+IM)}$

RF = (213.0 kips – 61.8 kips) / (86.6 kips) = 1.75

The moment demand associated with this shear resistance is as follows.

Associated live load shear: $V_{u(LL+IM)} = \phi V_n - V_{u(DC,DW)}$

$V_{u(LL+IM)}$ = 213.0 kips – 61.8 kips = 151.2 kips

Associated moment demand: $M_u = M_{u(DC,DW)} + \eta_{LL} V_{u(LL+IM)}$

M_u = 2,354 kip-inch + (23.8 inch)(151.2 kips) = 5,953 kip-inch

The moment associated with this shear resistance is less than the cracking moment at this section, so Expedient #4 can be applied.

A summary of the results from the four different expedients is provided in Table 8.

Table 8. Summary of expedients for shear load rating for example.

Expedient	**V_c (kips)**	**V_s (kips)**	**ϕV_n (kips)**	***RF***	**Notes**
#1: β = 2.0 and θ = 45°	47.8	67.6	103.9	0.49	
#2: BDS Article 5.12.5.3.8	95.6	67.6	146.9	0.98	
#3: MCFT as Design	144.0	126.9	243.9	--	Design is safe.
#4: Use ε_s = 0 if ε_s < 0	114.7	122.0	213.0	1.75	Only if $M_u < M_{cr}$
MCFT for Sectional Shear	133.4	125.3	232.9	1.97	

For this example, Expedient #1 and Expedient #2 resulted in load rating factors less than 1.0. Expedient #3 would show that the member can safely carry the load, but not the actual resistance or rating factor. Expedient #4 can be used at this section in this example because $M_u < M_{cr}$ and resulted in a load rating factor greater then 1.0 but about 11% less than that calculated using the actual ε_s.

SHEAR LOAD RATING ALONG LENGTH OF BEAM

The shear resistance and associated load rating should be evaluated along the length of the member. A spreadsheet was developed to perform the procedure outlined in the previous section (for the critical section) at multiple points along the length of the member. Some of the key parameters are summarized in the following tables.

- Sectional shear load rating factor along the length is summarized in Table 9.

- Load rating factor controlled by longitudinal reinforcement check along the length is summarized in Table 10.
- Horizontal shear load rating factor along the length is summarized in Table 11.
- A summary of the resistance and load rating factor for each failure mechanism along the length is provided in Table 12.

For the bridge in this example, the shear load rating at the critical section was controlled by the longitudinal reinforcement check. The load rating from the critical section to $0.3L$ was controlled by the sectional shear resistance. The load rating in the middle region of the beam was controlled by the longitudinal reinforcement check.

Table 9. Sectional shear load rating for HL-93 inventory load along length of beam.

x (ft)	*x* / *L*	$V_{u(DC,DW)}$ (kips)	$V_{u(LL+IM)}$ (kips) [a]	V_u (kips)	$M_{u(DC,DW)}$ (k-inch)	η_{LL}	M_u (k-inch)	ε_s	β	θ (deg)	V_c (kips)	V_s [b] (kips)	ϕV_n (kips)	$V_{u(LL+IM)inv}$ (kips)	*RF*
3.1 [c]	0.03	61.8	171.1	232.9	2,354	23.8	6,433	-0.00019	5.58	28.3	133.4	125.3	232.9	86.6	1.97
4.5	0.05	59.8	170.6	230.4	3,399	35.0	9,366	-0.00017	5.48	28.4	131.0	125.0	230.4	84.8	2.01
7.0	0.07	56.3	166.4	222.8	5,141	54.4	14,199	-0.00010	5.18	28.7	123.8	123.7	222.8	81.7	2.04
11.5	0.12	53.6	163.8	217.4	6,460	70.0	17,923	-0.00005	4.97	28.8	118.7	122.8	217.4	79.2	2.07
17.1	0.18	45.7	130.1	175.8	9,836	114.0	24,673	0.00053	3.44	30.9	82.1	113.2	175.8	72.3	1.80
22.8	0.24	37.8	104.9	142.7	12,675	158.1	29,266	0.00134	2.39	33.7	57.2	101.4	142.7	65.6	1.60
28.5	0.30	29.9	91.5	121.4	14,977	202.2	33,487	0.00216	1.83	36.6	43.8	91.2	121.4	59.0	1.55
34.1	0.36	22.0	84.1	106.1	16,743	246.3	37,456	0.00297	1.49	39.4	35.5	82.4	106.1	52.7	1.60
39.8	0.42	14.1	80.1	94.2	17,972	290.4	41,238	0.00376	1.26	42.2	30.0	74.7	94.2	46.4	1.73
47.5	0.50	0.0	78.1	78.1	18,831	369.3	47,665	0.00513	0.99	47.0	23.6	63.1	78.1	35.7	2.19
47.5	0.50	0.0	78.1	78.1	18,831	369.3	47,666	0.00513	0.99	47.0	23.6	63.1	78.1	35.7	2.19
47.5	0.50	0.0	78.1	78.1	18,831	369.3	47,665	0.00513	0.99	47.0	23.6	63.1	78.1	35.7	2.19
55.2	0.58	14.1	80.1	94.2	17,972	290.4	41,238	0.00376	1.26	42.2	30.0	74.7	94.2	46.4	1.73
60.9	0.64	22.0	84.1	106.1	16,743	246.3	37,456	0.00297	1.49	39.4	35.5	82.4	106.1	52.7	1.60
66.5	0.70	29.9	91.5	121.4	14,977	202.2	33,487	0.00216	1.83	36.6	43.8	91.2	121.4	59.0	1.55
72.2	0.76	37.8	104.9	142.7	12,675	158.1	29,266	0.00134	2.39	33.7	57.2	101.4	142.7	65.6	1.60
77.9	0.82	45.7	130.1	175.8	9,836	114.0	24,673	0.00053	3.44	30.9	82.1	113.2	175.8	72.3	1.80
83.6	0.88	53.6	163.8	217.4	6,460	70.0	17,923	-0.00005	4.97	28.8	118.7	122.8	217.4	79.2	2.07
88.0	0.93	56.3	166.4	222.8	5,141	54.4	14,199	-0.00010	5.18	28.7	123.8	123.7	222.8	81.7	2.04
90.5	0.95	59.8	170.6	230.4	3,399	35.0	9,366	-0.00017	5.48	28.4	131.0	125.0	230.4	84.8	2.01
91.9 [c]	0.97	61.8	171.1	232.9	2,354	23.8	6,433	-0.00019	5.58	28.3	133.4	125.3	232.9	86.6	1.97

[a] $V_{u(LL)}$ column was modified using solver until ($\phi V_n - V_u$) = 0 kips.
[b] Transverse reinforcement assumed as A_v = 0.4 inch^2 at s = 12 inch along the entire length of the beam.
[c] Critical section: d_v away from the face of the support.

Table 10. Shear load rating for HL-93 inventory load associated with longitudinal reinforcement check along length of beam.

x (ft)	x / L	$V_{u(LL+IM)}$ (kips) [a]	V_u (kips)	M_u (k-inch)	ε_s	θ (deg)	V_s [b] (kips)	*RHS* (kips)	$l_{d,avail}$ (inch)	f_{px} (ksi)	*LHS* (kips)	ϕV_n (kips)	*RF*	AASHTO LRFD BDS
3.1 [c]	0.03	156.7	218.5	6,091	-0.00020	28.3	125.6	334.4	13.5	75.4	334.4	218.5	1.81	Eqn. 5.7.3.5-2
4.5	0.05	197.4	257.2	10,304	-0.00014	28.5	124.5	716.2	28.9	161.4	716.2	257.2	2.33	Eqn. 5.7.3.5-1
7.0	0.07	199.4	255.8	15,994	-0.00006	28.8	123.0	878.0	59.3	197.9	878.0	255.8	2.44	Eqn. 5.7.3.5-1
11.5	0.12	248.2	301.8	23,828	0.00133	33.6	101.6	1,132.2	118.0	255.2	1,132.2	301.8	3.13	Eqn. 5.7.3.5-1
17.1	0.18	169.9	215.6	29,216	0.00191	35.7	94.2	1,132.2	187.9	255.2	1,132.2	215.6	2.35	Eqn. 5.7.3.5-1
22.8	0.24	123.5	161.2	32,198	0.00217	36.6	91.0	1,132.2	256.7	255.2	1,132.2	161.2	1.88	Eqn. 5.7.3.5-1
28.5	0.30	94.6	124.5	34,100	0.00233	37.1	89.2	1,132.2	325.2	255.2	1,132.2	124.5	1.60	Eqn. 5.7.3.5-1
34.1	0.36	75.9	97.9	35,430	0.00243	37.5	88.1	1,132.2	393.5	255.2	1,132.2	97.9	1.44	Eqn. 5.7.3.5-1
39.8	0.42	63.5	77.7	36,419	0.00250	37.8	87.3	1,132.2	461.8	255.2	1,132.2	77.7	1.37	Eqn. 5.7.3.5-1
47.5	0.50	51.1	51.1	37,694	0.00259	38.1	86.4	1,132.2	554.4	255.2	1,132.2	51.1	1.43	Eqn. 5.7.3.5-1
47.5	0.50	51.1	51.1	37,694	0.00259	38.1	86.4	1,132.2	554.4	255.2	1,132.2	51.1	1.43	Eqn. 5.7.3.5-1
47.5	0.50	51.1	51.1	37,694	0.00259	38.1	86.4	1,132.2	554.4	255.2	1,132.2	51.1	1.43	Eqn. 5.7.3.5-1
55.2	0.58	63.5	77.7	36,419	0.00250	37.8	87.3	1,132.2	646.6	255.2	1,132.2	77.7	1.37	Eqn. 5.7.3.5-1
60.9	0.64	75.9	97.9	35,430	0.00243	37.5	88.1	1,132.2	714.4	255.2	1,132.2	97.9	1.44	Eqn. 5.7.3.5-1
66.5	0.70	94.6	124.5	34,100	0.00233	37.1	89.2	1,132.2	782.2	255.2	1,132.2	124.5	1.60	Eqn. 5.7.3.5-1
72.2	0.76	123.5	161.2	32,198	0.00217	36.6	91.0	1,132.2	849.8	255.2	1,132.2	161.2	1.88	Eqn. 5.7.3.5-1
77.9	0.82	169.9	215.6	29,216	0.00191	35.7	94.2	1,132.2	187.9	255.2	1,132.2	215.6	2.35	Eqn. 5.7.3.5-1
83.6	0.88	248.2	301.8	23,828	0.00133	33.6	101.6	1,132.2	118.0	255.2	1,132.2	301.8	3.13	Eqn. 5.7.3.5-1
88.0	0.93	199.4	255.8	15,994	-0.00006	28.8	123.0	878.0	59.3	197.9	878.0	255.8	2.44	Eqn. 5.7.3.5-1
90.5	0.95	197.4	257.2	10,304	-0.00014	28.5	124.5	716.2	28.9	161.4	716.2	257.2	2.33	Eqn. 5.7.3.5-1
91.9 [c]	0.97	156.7	218.5	6,091	-0.00020	28.3	125.6	334.4	13.5	75.4	334.4	218.5	1.81	Eqn. 5.7.3.5-2

[a] $V_{u(LL)}$ column was modified using solver until ($\phi V_n - V_u$) = 0 kips.
[b] Transverse reinforcement assumed as A_v = 0.4 inch^2 at s = 12 inch along the entire length of the beam.
[c] Critical section: d_v away from the face of the support.

Table 11. Horizontal shear load rating for HL-93 inventory load along length of beam.

x (ft)	x / L	ℓ_{UEP} (inch)	ℓ_1 (inch)	$A_{cv,1}$ (inch^2)	$A_{vf,1}$ [a] (inch^2)	$V_{ni,1}$ [b] (kips)	ℓ_2 (inch)	$A_{cv,2}$ (inch^2)	$A_{vf,2}$ [a] (inch^2)	$V_{ni,2}$ [b] (kips)	ϕV_{ni} (kips)	ℓ_{crit} (inch)	$\phi V_n = V_u$ (kips)	*RF*
3.1 [c]	0.03	7.31	7.31	73.1	0.80	42.7	0.00	0.0	0.00	0.0	38.4	1.3	1072.1	11.66
4.5	0.05	24.50	24.5	245.0	1.60	178.6	0.00	0.0	0.00	0.0	160.8	18.5	317.9	3.04
7.0	0.07	54.50	30.0	300.0	1.60	200.6	24.50	245.0	0.80	165.2	329.2	48.5	248.4	2.35
11.5	0.12	107.90	30.0	300.0	1.60	200.6	77.90	779.0	2.40	513.2	642.4	101.9	230.7	2.24
17.1	0.18	175.94	30.0	300.0	1.60	200.6	145.94	1459.4	4.80	987.0	1068.8	169.9	230.1	2.55
22.8	0.24	243.98	30.0	300.0	1.60	200.6	213.98	2139.8	6.80	1,427.1	1465.0	238.0	225.2	2.86
28.5	0.30	312.02	30.0	300.0	1.60	200.6	282.02	2820.2	9.20	1,900.9	1891.3	306.0	226.1	3.32
34.1	0.36	380.06	30.0	300.0	1.60	200.6	350.06	3500.6	11.60	2,374.6	2317.7	374.1	226.7	3.89
39.8	0.42	448.10	30.0	300.0	1.60	200.6	418.10	4181.0	13.60	2,814.8	2713.9	442.1	224.6	4.53
47.5	0.50	540.49	30.0	300.0	1.60	200.6	510.49	5104.9	16.80	3,453.2	3288.4	534.5	225.1	6.30
47.5	0.50	540.50	30.0	300.0	1.60	200.6	510.50	5105.0	16.80	3,453.2	3288.4	534.5	225.1	6.30
47.5	0.50	540.49	30.0	300.0	1.60	200.6	510.49	5104.9	16.80	3,453.2	3288.4	534.5	225.1	6.30
55.2	0.58	448.10	30.0	300.0	1.60	200.6	418.10	4181.0	13.60	2,814.8	2713.9	442.1	224.6	4.53
60.9	0.64	380.06	30.0	300.0	1.60	200.6	350.06	3500.6	11.60	2,374.6	2317.7	374.1	226.7	3.89
66.5	0.70	312.02	30.0	300.0	1.60	200.6	282.02	2820.2	9.20	1,900.9	1891.3	306.0	226.1	3.32
72.2	0.76	243.98	30.0	300.0	1.60	200.6	213.98	2139.8	6.80	1,427.1	1465.0	238.0	225.2	2.86
77.9	0.82	175.94	30.0	300.0	1.60	200.6	145.94	1459.4	4.80	987.0	1068.8	169.9	230.1	2.55
83.6	0.88	107.90	30.0	300.0	1.60	200.6	77.90	779.0	2.40	513.2	642.4	101.9	230.7	2.24
88.0	0.93	54.50	30.0	300.0	1.60	200.6	24.50	245.0	0.80	165.2	329.2	48.5	248.4	2.35
90.5	0.95	24.50	24.5	245.0	1.60	178.6	0.00	0.0	0.00	0.0	160.8	18.5	317.9	3.04
91.9 [c]	0.97	7.31	7.31	73.1	0.80	42.7	0.00	0.0	0.00	0.0	38.4	1.3	1072.1	11.66

[a] Transverse reinforcement assumed as A_v = 0.4 inch^2 at s = 12 inch along the entire length of the beam with 0.8 inch^2 located in the end region to resist bursting and spalling stresses.
[b] V_{ni} was found to be less than the two limits ($K_1 f'_c A_{cv}$ and $K_2 A_{cv}$) for both regions of interest along the length of the beam.
[c] Critical section: d_v away from the face of the support.

Table 12. Summary of shear load rating values for HL-93 inventory load and controlling failure mechanisms along length of the beam.

x (ft)	x / L	ϕV_{n1} (kips)	RF_1	ϕV_{n2} (kips)	RF_2	ϕV_{n3} (kips)	RF_3	Controlling Failure Mechanism	ϕV_n (kips)	RF
3.1 [a]	0.03	232.9	1.97	218.5	1.81	1072.1	11.66	Longitudinal Reinforcement Check	218.5	1.81
4.5	0.05	230.4	2.01	257.2	2.33	317.9	3.04	Sectional Shear	230.4	2.01
7.0	0.07	222.8	2.04	255.8	2.44	248.4	2.35	Sectional Shear	222.8	2.04
11.5	0.12	217.4	2.07	301.8	3.13	230.7	2.24	Sectional Shear	217.4	2.07
17.1	0.18	175.8	1.80	215.6	2.35	230.1	2.55	Sectional Shear	175.8	1.80
22.8	0.24	142.7	1.60	161.2	1.88	225.2	2.86	Sectional Shear	142.7	1.60
28.5	0.30	121.4	1.55	124.5	1.60	226.1	3.32	Sectional Shear	121.4	1.55
34.1	0.36	106.1	1.60	97.9	1.44	226.7	3.89	Longitudinal Reinforcement Check [b]	97.9	1.44
39.8	0.42	94.2	1.73	77.7	1.37	224.6	4.53	Longitudinal Reinforcement Check [b]	77.7	1.37
47.5	0.50	78.1	2.19	51.1	1.43	225.1	6.30	Longitudinal Reinforcement Check [b]	51.1	1.43
47.5	0.50	78.1	2.19	51.1	1.43	225.1	6.30	Longitudinal Reinforcement Check [b]	51.1	1.43
47.5	0.50	78.1	2.19	51.1	1.43	225.1	6.30	Longitudinal Reinforcement Check [b]	51.1	1.43
55.2	0.58	94.2	1.73	77.7	1.37	224.6	4.53	Longitudinal Reinforcement Check [b]	77.7	1.37
60.9	0.64	106.1	1.60	97.9	1.44	226.7	3.89	Longitudinal Reinforcement Check [b]	97.9	1.44
66.5	0.70	121.4	1.55	124.5	1.60	226.1	3.32	Sectional Shear	121.4	1.55
72.2	0.76	142.7	1.60	161.2	1.88	225.2	2.86	Sectional Shear	142.7	1.60
77.9	0.82	175.8	1.80	215.6	2.35	230.1	2.55	Sectional Shear	175.8	1.80
83.6	0.88	217.4	2.07	301.8	3.13	230.7	2.24	Sectional Shear	217.4	2.07
88.0	0.93	222.8	2.04	255.8	2.44	248.4	2.35	Sectional Shear	222.8	2.04
90.5	0.95	230.4	2.01	257.2	2.33	317.9	3.04	Sectional Shear	230.4	2.01
91.9 [a]	0.97	232.9	1.97	218.5	1.81	1072.1	11.66	Longitudinal Reinforcement Check	218.5	1.81

[a] Critical section: d_v away from the face of the support.
[b] Flexural demand, M_u, leads to the smaller load rating for the longitudinal reinforcement check toward midspan. AASHTO LRFD BDS Article 5.7.3.5 states that "…the area of longitudinal reinforcement on the flexural tension side of the member need not exceed the area required to resist the maximum moment acting alone."

The longitudinal reinforcement check using AASHTO LRFD BDS Eqn. 5.7.3.5-1 can become more of a check for flexure as the moment demand, M_u, increases.

General check for longitudinal reinforcement:

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \phi_f} + 0.5 \frac{N_u}{\phi_c} + \left(\left| \frac{V_u}{\phi_v} - V_p \right| - 0.5 V_s \right) \cot \theta$$

The shear-related term will decrease at sections toward midspan. If the axial force and shear terms of the equation are eliminated, this equation essentially becomes a check on the flexural strength.

General check for longitudinal reinforcement (without the axial force and shear terms):

$$A_{ps} f_{ps} + A_s f_y \geq |M_u| \, / \, (d_v \phi_f)$$

Multiplying both sides by $(d_v \phi_f)$:

$$\phi_f M_n \approx (d_v \phi_f)(A_{ps} f_{ps} + A_s f_y) \geq |M_u|$$

This is reflected in Table 10 as the moment demand in the midspan region approaches the nominal flexural strength of the section, ϕM_n = 38,280 kip-inch for this section.

AASHTO LRFD BDS Article 5.7.3.5 states that "…the area of longitudinal reinforcement on the flexural tension side of the member need not exceed the area required to resist the maximum moment acting alone." This essentially means that the flexural strength checks are sufficient in the midspan region, and the longitudinal reinforcement check need not be applied.

Because of this, the minimum shear load rating factor is 1.50 at 0.3L and controlled by the sectional shear resistance. This is less than the shear load rating at the critical section (1.81), which shows the importance of checking the shear load rating along the length of the beam.

The horizontal shear resistance did not control the shear load rating at any point along the beam in this example. Garber et al. (2016) showed an example of where the horizontal shear resistance controlled the estimated shear strength and led to a horizontal shear failure in a laboratory test of a full-scale bulb-tee girder.

CHAPTER 4. SUMMARY AND CONCLUSIONS

Shear load rating using MCFT can be performed using the procedure proposed by FHWA-HIF-22-025 and adopted in revisions to the AASHTO MBE. Additional details for using this procedure for pretensioned concrete elements were provided in this report along with a load rating example. Additional details for post-tensioned applications can be found in FHWA-HIF-22-025.

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