



GSBSM-1-UL
ISBN: 978-1-56051-724-5

Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members

2018
First Edition



Documents
Document Sharing Hub



American Association of State Highway and Transportation Officials
444 North Capitol Street, NW, Suite 249
Washington, DC 20001
202-624-5800 phone/202-624-5806 fax
www.transportation.org

Cover photos: Left: Blue Water Bridge, Port Huron, MI. The M&M-designed Blue Water Bridge carries I-69/94 over the St. Clair River between Michigan and Canada. The original bridge is a continuous steel cantilever through truss with an 871-ft main span. The twin bridge is a 922-ft tied arch bridge. Courtesy of Thomas Murphy, Modjeski & Masters, Inc. **Right:** Point Marion Bridge, Point Marion, PA. Courtesy of Anthony Ream, HDR, Inc.

© 2018 by the American Association of State Highway and Transportation Officials. All rights reserved.
Duplication is a violation of applicable law.

ISBN: 978-1-56051-724-5

Pub Code: GSBSM-1



© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.



AMERICAN ASSOCIATION
OF STATE HIGHWAY AND
TRANSPORTATION OFFICIALS
AASHTO

GSBSM-1-UL
ISBN: 978-1-56051-724-5

Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members

2018
First Edition



Documents
Document Sharing Hub

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Full document details: AASHTO GSBSM-1-2018

This page intentionally left blank.

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
444 North Capitol Street, N.W. Suite 249
Washington, D.C. 20001

**EXECUTIVE COMMITTEE
2018-2019**

OFFICERS:

PRESIDENT: Carlos Braceras, Utah*

VICE PRESIDENT: Patrick McKenna, Missouri*

SECRETARY-TREASURER: Scott Bennett, Arkansas*

EXECUTIVE DIRECTOR: Bud Wright, Washington, D. C.

REGIONAL REPRESENTATIVES:

REGION I: Jennifer Cohan, Delaware
Victoria Sheehan, New Hampshire

REGION II: Russell McMurry, Georgia

REGION III: Joe McGuinness, Indiana
Mark Lowe, Iowa

REGION IV: Roger Millar, Washington
Marc Luiken, Alaska

IMMEDIATE PAST PRESIDENT: John Schroer, Tennessee

***Elected at the 2018 Annual Meeting in Atlanta, Georgia**

This page intentionally left blank.

AASHTO COMMITTEE ON BRIDGES AND STRUCTURES, 2018

GREGG FREDRICK, *Chair*

BRUCE V. JOHNSON, *Vice Chair*

JOSEPH L. HARTMANN, Federal Highway Administration, *Secretary*

PATRICIA J. BUSH, *AASHTO Liaison*

ALABAMA, Eric J. Christie, William “Tim” Colquett,
Randall B. Mullins

ALASKA, Leslie Daughtery, Elmer E. Marx, Richard
A. Pratt

ARIZONA, David B. Benton,
David L. Eberhart, Pe-Shen Yang

ARKANSAS, Charles “Rick” Ellis, Mike Hill, Joe
Santini

CALIFORNIA, Thomas A. Ostrom, Gedmund
Setberg, Dolores Valls

COLORADO, Behrooz Far, Stephen Harelson, Jessica
Martinez

CONNECTICUT, Mary E. Baker, Timothy D. Fields

DELAWARE, Jason Arndt, Jason Hastings, Craig A.
Stevens

DISTRICT OF COLUMBIA, Donald L. Cooney,
Konjit C. “Connie” Eskender, Richard Kenney

FLORIDA, Sam Fallaha, William Potter, Jeff
Pouliotte

GEORGIA, Clayton Bennett, Bill DuVall, Steve
Gaston

HAWAII, James Fu, Kevin Murata, John Williams

IDAHO, Matthew Farrar

ILLINOIS, Tim A. Armbrrecht, Carl Puzey, Jayme
Schiff

INDIANA, Andrew Fitzgerald, Jeremy Hunter, Anne
M. Rearick

IOWA, Ahmad Abu-Hawash, James S. Nelson

KANSAS, Mark E. Hoppe, John P. Jones, Curt F.
Niehaus

KENTUCKY, Bart Asher, Andy Barber, Marvin
Wolfe

LOUISIANA, Arthur D’Andrea, Paul Fossier,
Zhengzheng “Jenny” Fu

MAINE, Jeffrey S. Folsom, Wayne Frankhauser,
Michael Wight

MARYLAND, Maurice Agostino, Jesse Creel, Jeffrey
L. Robert

MASSACHUSETTS, Alexander K. Bardow, Thomas
Donald, Joseph Rigney

MICHIGAN, Matthew Chynoweth, Rebecca Curtis,
Richard E. Liptak

MINNESOTA, Arielle Ehrlich, Ed Lutgen,
Kevin Western

MISSISSIPPI, Aaron Cagle, Justin Walker, Scott
Westerfield

MISSOURI, Dennis Heckman, Greg E. Sanders,
Scott Stotlemeyer

MONTANA, Amanda Jackson, Dustin E. Rouse

NEBRASKA, Mark Ahlman, Fouad Jabber, Mark J.
Traynowicz

NEVADA, Troy Martin, Jessen Mortensen

NEW HAMPSHIRE, Robert Landry, David L. Scott

NEW JERSEY, Xiaohua “Hannah” Cheng, Eddy
Germain, Dave D. Lambert

NEW MEXICO, Kathy Crowell, Jeff C. Vigil

NEW YORK, Brenda Crudele, Ernest Holmberg,
Richard Marchione

NORTH CAROLINA, Brian Hanks,
Scott Hidden, Girchuru Muchane

NORTH DAKOTA, Jon D. Ketterling, Jason R.
Thorenson

OHIO, Alexander B.C. Dettloff,
Timothy J. Keller, Jeffrey Syar

OKLAHOMA, Steven Jacobi, Walter Peters, Tim
Tegeler

OREGON, Bruce V. Johnson, Albert Nako,
Tanarat Potisuk

PENNSYLVANIA, James M. Long, Thomas P.
Macioce, Lou Ruzzi

PUERTO RICO, (Vacant)

RHODE ISLAND, Georgette Chahine, Keith Gaulin

SOUTH CAROLINA, Terry B. Koon, Jeff Sizemore

SOUTH DAKOTA, Steve Johnson, Dave Madden,
Todd S. Thompson

TENNESSEE, John S. Hastings, Ted A. Kniazewycz

TEXAS, Bernie Carrasco, Jamie F. Farris, Gregg A.
Freeby

U.S. DOT, Joseph L. Hartmann

UTAH, Rebecca Nix, Carmen Swanwick, Cheryl
Hersh Simmons

VERMONT, Kristin M. Higgins, Jim Lacroix

VIRGINIA, Prasad L. Nallapaneni,
Kendal R. Walus, Andrew M. Zickler

WASHINGTON, Tony M. Allen,
Mark A. Gaines, Bijan Khaleghi

WEST VIRGINIA, Ahmed Mongi, William Varney

WISCONSIN, Scot Becker,
William C. Dreher

WYOMING, Jeff R. Booher, Paul G. Cortez,
Gregg C. Frederick, Michael E. Menghini

DELAWARE RIVER AND BAY AUTHORITY,
Shoukry Elnahal

MDTA, Dan Williams

**MULTNOMAH COUNTY TRANSPORTATION
DIVISION**, Jon Henrichsen

N.Y. STATE BRIDGE AUTHORITY, William
Moreau

**U.S. ARMY CORPS OF ENGINEERS—
DEPARTMENT OF THE ARMY**, Phillip W.
Sausser

U.S. COAST GUARD, Kamal Elnahal
U.S. DEPARTMENT OF AGRICULTURE—
FOREST SERVICE, John R. Kattell

TRANSPORTATION RESEARCH BOARD,
Waseem Dekelbab

FOREWORD

While there is variability in the definitions found in US specifications, fracture-critical members (FCMs) are generally considered to be steel primary members or portions thereof, subject to tension, whose failure would probably cause a portion of, or the entire bridge to collapse. The decision to define members as FCMs has been made without considering internal redundancy; in other words, the redundancy within an individual member built-up from multiple components that provides mechanical separation of elements to limit crack propagation across the entire member cross-section. Prior to the development of these Guide Specifications, no standards existed on how to establish whether adequate internal redundancy exists in a mechanically-fastened built-up steel member. Transportation Pooled Fund Project TPF-5(253) “Evaluation of Member Level Redundancy in Built-up Steel Members” was intended to address issues in performing a “credible” analysis, identifying members with adequate internal redundancy to resist complete cross-section failure should one component suddenly fail (Hebdon et al., 2015; Lloyd et al., 2018). The resistance to such a failure mode is referred to as cross-boundary fracture resistance (CBFR). Existing and newly designed members satisfying the provisions of these Guide Specifications need not be classified as FCMs, but rather may be classified as Internally Redundant Members (IRMs).

These provisions also include a methodology to establish the interval for inspections specifically intended to identify whether any of the tension components have failed. In the context of this Guide Specification, this inspection is referred to as a Special Inspection for IRMs and must be of sufficient depth to reliably detect a severed component. This is a major departure from the current calendar-based approach to setting inspection intervals, and is a first step toward the development of an integrated fracture control plan by AASHTO. It is also important to note that the primary objective of the in-service special inspections of IRMs is to detect fully severed components in a built-up member rather than very small fatigue cracks emanating from any one of thousands of fastener holes. Probability of detection (POD) studies suggest that it is unrealistic to assume that such small cracks can be identified with a high level of reliability using traditional visual inspection techniques. Considering the sheer number of possible locations where a crack can initiate in a built-up member (i.e., at each fastener), this finding is not very surprising. In contrast, the likelihood of detecting a severed component in a built-up member is much higher.

The user is cautioned and must recognize that traditional hands-on or in-depth inspections may be desirable for other reasons, such as detecting corrosion. Thus, the Owner may wish to consider such inspections at shorter intervals than indicated herein. Further, it is noted that these provisions are not intended to justify leaving a severed component of a member in service for an extended period once discovered. Such damage, once found, should be repaired as quickly as practical.

Throughout these Guide Specifications, the condition during which an individual component of the built-up member is assumed to have severed is referred to as the “faulted state”. It is in this condition (i.e., with an assumed severed component) that these Guide Specifications are intended to apply. The provisions are applicable for both evaluating existing members and for use during the design of new members. They are not applicable for ratings or other evaluations when all components are intact.

A new reliability-based load combination used to evaluate internal redundancy of built-up members was developed to achieve a target reliability index in the faulted state. This load combination is based on the research described in NCHRP Report 883 (Connor et al., 2018), which used the same procedures that were previously used to create the various load combinations specified in the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) (referred to as LRFD Design hereafter). Criteria have also been developed for the strength and service limit states to determine if the bridge demonstrates sufficient performance in the faulted state.

The commentary directs attention to other documents that provide references and suggestions for carrying out the requirements and the intent of these Guide Specifications. The commentary is not intended to provide every detail related to the development of these Guide Specifications, nor is it intended to provide a detailed summary of the studies and research data reviewed in formulating these Guide Specifications. Detailed examples showing the application of these Guide Specifications are provided in Appendix B.

This page intentionally left blank.

TABLE OF CONTENTS

SECTION 1—GENERAL	1-1
1.1—GENERAL	1-1
1.2—DEFINITIONS	1-1
1.3—NOTATION	1-2
1.4—GENERAL REQUIREMENTS	1-3
1.5—REQUIREMENTS FOR FLEXURAL MEMBERS	1-4
1.5.1—Requirements for New Design of Internally Redundant Built-up Flexural Members	1-6
1.6—REQUIREMENTS FOR AXIALLY LOADED TENSION MEMBERS	1-6
1.6.1—Requirements for New Design of Internally Redundant Built-up Axially Loaded Tension Members	1-7
1.6.2—Specific Requirements for Built-up Tension Tie Members	1-8
1.6.3—Specific Requirements for Multi-Eyebars	1-9
1.7—APPLIED LOAD IN THE FAULTED STATE	1-9
1.7.1—Flexural Members	1-9
1.7.2—Axially Loaded Tension Members	1-10
1.8—IN-SERVICE SPECIAL INSPECTIONS	1-11
SECTION 2—PERFORMANCE IN THE FAULTED STATE	2-1
2.1—CALCULATED FLEXURAL STRESS IN THE FAULTED STATE	2-1
2.1.1—General	2-1
2.1.2—Noncomposite Flexural Members	2-1
2.1.3—Composite Flexural Members	2-2
2.2—CALCULATED STRESSES IN THE FAULTED STATE DUE TO AXIAL TENSION	2-3
2.2.1—Multi-component Truss Members: Plate Failures	2-3
2.2.1.1—Multi-component Truss Members: Failed Angle Check	2-7
2.2.2—Two-component Truss Members	2-8
2.2.3—Multi-eyebars and Two-eyebars Truss Members	2-9
2.2.4—Members Acting as Tension Ties	2-9
2.3—STRENGTH DESIGN CRITERIA IN THE FAULTED STATE	2-9
2.4—DEFLECTION	2-10
2.5—FATIGUE DESIGN CRITERIA IN THE FAULTED STATE	2-10
2.5.1—Case Ia: Infinite Fatigue Life Check for Members Satisfying Case I Requirements in the Faulted State	2-13
2.5.2—Case Ib: Finite Life Fatigue Check for Members Satisfying Case I Requirements in the Faulted State	2-13

2.5.3—Finite Life Fatigue Check for Members Satisfying Case II Requirements in the Faulted State	2-14
2.6—FASTENER HOLE TYPE—EXISTING MEMBERS	2-15
SECTION 3—MAXIMUM INTERVAL FOR SPECIAL INSPECTIONS FOR IDENTIFYING INDIVIDUAL SEVERED COMPONENTS IN IRMS	3-1
3.1—GENERAL	3-1
3.2—UPDATING THE FUTURE SPECIAL INSPECTION INTERVAL	3-2
REFERENCES	4-1
Appendix A: IRM Procedures Flowchart.....	A-1
Appendix B: Examples.....	B-1

SECTION 1

GENERAL

1.1—GENERAL

These provisions shall be used for the evaluation of internal redundancy in built-up members traditionally designated as Fracture Critical Members (FCM). These provisions shall be used during the design of new members or the evaluation of existing members built-up from plates and rolled shapes that are mechanically fastened using high-strength bolts or rivets and subject to flexure or axial tension.

Members satisfying the requirements of these provisions shall be considered to possess sufficient internal redundancy to provide safety against complete brittle fracture of the entire member, in the event sudden fracture of an individual tension load-carrying component occurs. Such members shall be classified as Internally Redundant Members (IRMs). In new designs, IRMs shall be identified on the contract plans.

A member shall be considered to be in the faulted state when any single tension load-carrying component of the built-up member, e.g., a single angle, cover plate, or an individual component of an axial member, is assumed to be completely severed.

1.2—DEFINITIONS

Bearing Ratio	= The maximum allowable ratio of rivet bearing stress to plate tension stress.
Built-up Member	= A member in which the plates and angles comprising the portion of the member in tension are mechanically-fastened together with rivets or high-strength bolts.
Collapse	= A major change in the geometry of the bridge rendering it unfit for use, as defined in LRFD Design (AASHTO, 2017) and the MBE (AASHTO, 2018).
Component (of a Primary Member)	= A portion of a (primary) member with a specific design function; for example, a cover plate attached to the flange of a girder, the web plate of a girder, or an angle used to attach components together.
Cross Boundary Fracture Resistance (CBFR)	= The ability of an IRM to resist complete failure of the cross-section should one component suddenly fail.
Faulted State	= State of a member in which a single component is assumed to be severed, as opposed to the unfaulted state.
Fracture Critical Member (FCM)	= A steel primary member or portion thereof subject to tension whose failure would probably cause a portion of or the entire bridge to collapse.

C1.1

These provisions describe a procedure for evaluating the internal redundancy of tension components of built-up flexural members and tension members. Internal redundancy is also often referred to as member-level redundancy (MLR). To facilitate implementation, a flowchart describing the overall procedures is provided in Appendix A.

The term Internally Redundant Member (IRM) applies to the entire member if it satisfies the provisions specified herein; i.e., a member cannot have a component designated as a FCM, and another as an IRM.

The term “shall” denotes a requirement for compliance with these Specifications.

The term “should” indicates a strong preference for a given criterion.

The term “may” indicates a criterion that is usable, but other local and suitably documented, verified, and approved criterion may also be used in a manner consistent with the approaches described in the current Guide Specification.

Fracture Critical Member Inspection.	= A hands-on inspection of a fracture-critical member or member component that may include visual and other nondestructive evaluation as defined in 23 CFR 650.305–Definitions (FHWA, 2017).
Internal Redundancy	= Ability of a member to provide an alternate resistance mechanism after the failure of an individual component through load redistribution within the member.
Internally Redundant Member (IRM)	= A built-up steel member which meets the requirements set forth in these Guide Specifications. An IRM shall not be classified as an FCM, but shall be inspected using an appropriate technique at an interval specified in Article 3 of these Guide Specifications. IRMs shall be identified on the plans as IRMs.
Minimum or Safe Fatigue Life	= The fatigue life corresponding to a probability of failure approximately equal to 2.5%, as calculated in Article 7.2 of the MBE (AASHTO, 2018). This is also consistent with the fatigue resistance used in design, as specified in LRFD Design (AASHTO, 2017).
Primary Member	= A steel member or component that transmits gravity loads through a necessary as-designed load path. These members are therefore subjected to more stringent fabrication and testing requirements. Synonymous with the term “main member”.
Special Inspection for IRMs	= In the specific context of these Guide Specifications, an inspection of sufficient depth to reliably detect severed or fractured components and other conditions deemed important in a member identified as an IRM. To meet the objectives of the Special Inspection for IRMs, it is understood that special access equipment may be required for some bridges, while for others it may be possible to reliably detect broken components from the ground using normal visual inspection techniques.

1.3—NOTATION

A_{AFG}	= Gross area of the member in the faulted state (in. ²).
A_{AFN}	= Net area of the member in the faulted state, including the effect of fastener holes (in. ²).
A_N	= Net section area of the entire axial member in the unfaulted state (in. ²).
D	= Depth of web plate (in.).
f_{AFG}	= Factored total gross section stress in the faulted state (ksi).
f_{AFN}	= Factored total net section stress in the faulted state (ksi).
F_u	= Specified minimum tensile strength (ksi).
F_y	= Specified minimum yield strength (ksi).
l_f	= Edge distance calculated from the center of the fastener hole to the edge of the material, perpendicular to the force (in.).
M_{DC}	= Bending moment due to dead load of structural components and nonstructural attachments (kip-in.).
M_{DW}	= Bending moment due to dead load of wearing surfaces and utilities (kip-in.).
M_{LL+IM}	= Bending moment due to vehicular live load and dynamic load allowance (kip-in.).
M_u	= Total factored bending moment (kip-in.).
N_{AX}	= Number of plates contributing to the cross-section area of an axial member in the unfaulted state, as shown in Table 2.2.1-1. This includes full and partial-depth plates, but does not include angle legs.

N_{FL}	=	Number of cover plates on the tension flange in the unfaulted state not including the horizontal leg of the angles.
N_f	=	Remaining fatigue life in years.
N_u	=	Number of accumulated years in the unfaulted state.
P_{DC}	=	Axial tensile force due to dead load of structural components and nonstructural attachments (kip).
P_{DW}	=	Axial tensile force due to dead load of wearing surfaces and utilities (kip).
P_{LL+IM}	=	Axial tensile force due to vehicular live load and dynamic load allowance (kip).
P_u	=	Applied factored axial tensile force (kip).
P_{u-L}	=	Applied factored axial tensile force in a single angle of an axial member (kip).
S_{x-AFG}	=	Elastic section modulus of the gross section of the member in the faulted state, (in ³).
S_{x-AFN}	=	Elastic section modulus of the net section of the member in the faulted state, including the effect of fastener holes (in ³).
$S_{x-AFG,NC}$	=	Elastic section modulus of the non-composite gross section of the member in the faulted state (in ³).
$S_{x-AFN,NC}$	=	Elastic section modulus of the non-composite net section of the member in the faulted state, including the effect of fastener holes (in ³).
$S_{x-AFG,COMP}$	=	Elastic section modulus of the composite gross section of the member in the faulted state (in ³).
$S_{x-AFN,COMP}$	=	Elastic section modulus of the composite net section of the member in the faulted state, including the effect of fastener holes (in ³).
t_p	=	Thickness of the full-depth cover plate (in.).
t_L	=	Thickness of the angle (in.).
Y_f	=	Total finite fatigue life in years in the faulted state.
Y_u	=	Total finite fatigue life in years in the unfaulted state.
β_{AF}	=	Cover plate adjustment factor.
γ_{LL}	=	Vehicular live load factor for the Internal Redundancy load combination specified in Table 1.7.1-1.
γ_{DC}	=	Structural components and non-structural attachments dead load factor for the Internal Redundancy load combination specified in Table 1.7.1-1.
γ_{DW}	=	Wearing surface and utilities dead load factor for the Internal Redundancy load combination specified in Table 1.7.1-1.
ε_B	=	Fatigue stress range amplification factor accounting for localized bending effects in a built-up axial member in the faulted state.
ε_{VL}	=	Fatigue stress range amplification factor accounting for localized shear lag effects in a built-up axial member in the faulted state.
ϕ_u	=	Resistance factor for fracture on the net section of tension members (LRFD Design Article 6.5.4.2 (AASHTO, 2017)).
ϕ_y	=	Resistance factor for yielding on the gross section of tension members (LRFD Design Article 6.5.4.2 (AASHTO, 2017)).

1.4—GENERAL REQUIREMENTS

C1.4

For these provisions to apply, the built-up member shall satisfy the following general requirements in the as-built unfaulted state:

- Members shall be proportioned to meet all applicable strength requirements;
- Existing members to be evaluated shall possess remaining fatigue life in the unfaulted state. The

It is not permitted to utilize the procedures included in Article 7.2.7.2.3 of the MBE (AASHTO, 2018), which

provisions of Article 7.2.7.2.3 contained in the MBE shall not be applied.

- New members shall be designed for infinite life using the Fatigue I load combination in the unfaulted state;
- Tack welds commonly found on existing members shall be deemed acceptable and shall not disqualify the member from evaluation by these provisions;
- For existing bridges, members to be evaluated shall have undergone a traditional fracture-critical member inspection within the past 24 months with no cracks, significant corrosion, impact damage, etc., identified in the components subjected to tension;
- For newly designed yet to be constructed bridges, a hands-on inspection shall be performed within 24 months after the bridge is opened to traffic with no cracks, significant corrosion, impact damage, etc., identified in the components subjected to tension;
- For existing bridges, members to be evaluated shall not exhibit severe corrosion, impact damage, or other forms of damage in the regions being evaluated by these provisions;
- Members need not be evaluated for internal redundancy at mechanically fastened splices.

permit revising the calculated fatigue life when a negative fatigue serviceability index is obtained since that provision presumes field inspections are capable of reliably detecting the existing or lack thereof, of fatigue cracks. Since fatigue cracks that exist beneath the head of a fastener are non-detectable, it would be inappropriate to apply this provision of the MBE when evaluating the internal redundancy of a built-up member.

Prior to the first application of these specifications, the condition of existing bridges needs to be documented through a traditional fracture-critical member inspection to ensure there are not conditions on the bridge which would disqualify the member from being evaluated using these provisions. Examples include corrosion that is significant enough to require load posting, and cracking or unmitigated damage due to impact in primary members.

For new bridges, the intent of the hands-on inspection is to find damage that could affect the integrity of the member should a fault occur. While the inspection will need to be hands-on, it does not need to satisfy all of the requirements associated with a traditional fracture-critical inspection.

1.5—REQUIREMENTS FOR FLEXURAL MEMBERS C1.5

For these provisions to apply, built-up flexural members shall satisfy the following requirements in the as-built unfaulted state:

- At least one cover plate shall be present along the entire length of the tension flange, except within a distance of $1.5D$ from the end of the member where

In many girders, cover plates are terminated a short distance from the ends of girders at abutments, over piers in simple spans, or at pin-and-hanger details in continuous spans. However, in these regions, the girder is

D is the depth of the web plate. This region, where only flange angles are present, need not be evaluated using the provisions contained herein to establish or alter the interval for special inspections for IRMs calculated for the member in Article 3;

- The web plate(s) shall be mechanically fastened to the tension flange cover plates with angles and high-strength bolts or rivets;
- Welds shall not be used to attach the web(s) to the tension flange angles or to attach the cover plates together or to the flange angles;
- All components comprising the portion of the tension flange shall be adequately fastened together with high-strength bolts or rivets satisfying all strength, stitch, and sealing requirements in effect at the time of the design;
- Any single tension flange component, i.e., flange angle or cover plate, shall not exceed 60% of the total tension flange gross area. For this specific check, the calculation of the tension-flange gross area shall only include the area of each tension flange cover plate and the area of the horizontal legs of the tension flange angles including fastener holes as shown in the shaded portion of Figure 1.5-1. The web and vertical legs of tension-flange angles shall not be included in the tension-flange area;

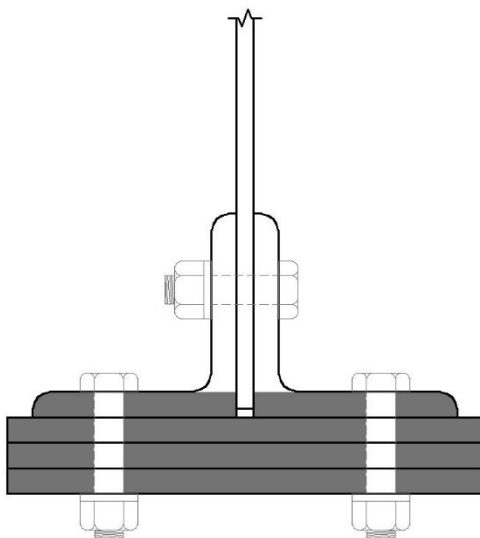


Figure 1.5-1—Total Tension-Flange Gross Area for a Built-up Flexural Member

primarily subjected to shear stress and not flexure. Further, the stress ranges in this region are very low. To prevent girders from being needlessly disqualified from these provisions, cover plates are not required within a distance of $1.5D$ from the end of a member.

For flexural members, these provisions are intended to only be applied to built-up tension flanges of primary steel I- or box-shaped members, as shown in Figure C1.5-1; e.g., girders, stringers, and floor beams. The compression flange may be welded to the web. For flexural members, fracture of the web need not be explicitly considered in the calculations because it will be less critical than the failure of a cover plate, due to the limited contribution of the web to the flexural capacity of the section.

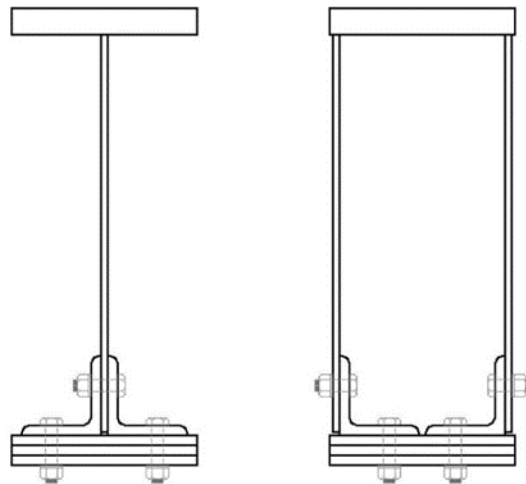


Figure C1.5-1—Typical Built-up Bottom Flange of an I-Shaped (left) and Box-Shaped (right) Flexural Member

Depending on the geometry of the cover plates and angles on flexural members, failure of an interior cover plate may result in greater stress in the remaining components. Therefore, multiple scenarios would need to be evaluated to identify the worst-case stress increase in the remaining net section. A pilot parametric study has shown that assuming failure of the outermost cover plate generally provides a reasonable estimate of the worst-case conditions for cases where the angles and cover plates are of proportions typically used in the past. In this study, various girder geometries ranging from 66- to 120-inch-deep webs, with 1 to 4 cover plates ranging from 14 to 22 in. wide and $\frac{7}{16}$ to $\frac{3}{4}$ in. thick, were examined. For these cases, it was revealed that assuming the failure of the outermost cover plate resulted in an underestimate of

1.5.1—Requirements for New Design of Internally Redundant Built-up Flexural Members

The following requirements shall apply for the new design of internally redundant built-up flexural members in the as-built unfaulted state:

- The compression flange may be welded to the web. In continuous spans, regions subjected to net stress reversal shall be built-up and mechanically fastened together;
- The area of any individual cover plate shall not exceed 1.5 times the area of any adjacent cover plate;
- Cover plates of equal thickness shall be used when possible. When unequal thickness cover plates must be used, they shall diminish in thickness toward the outside of the cross-section;
- The thickness of individual cover plates should not exceed twice the thickness of the connection angles when possible, but in no case shall the thickness of the cover plates exceed 2.25 times the connection angle thickness;
- The cover plates and web plate shall be fabricated using grades of steel satisfying the AASHTO M 270M/M 270 (ASTM A709/A709M) Charpy V-notch requirements for fracture-critical members. The flange angles need only satisfy the Charpy V-notch requirements for non-fracture-critical members.

1.6—REQUIREMENTS FOR AXIALLY LOADED TENSION MEMBERS

For these provisions to apply, built-up members subject to axial tension shall satisfy the following requirements in the as-built unfaulted state:

- For members comprised of only two components, the area of an individual component

the resulting stress increase by less than 1 percent (Hebdon et al., 2015). Therefore, it is recommended that the procedure specified herein be applied with the outermost cover plate as the assumed severed component and the cover plate adjustment factor, β_{AF} , applied.

C1.5.1

The requirements on proportions of flexural members provided herein are intended to ensure that a new girder is detailed using components that are reasonably similar in size. The proportioning limits are based on historic built-up girder proportions that have been observed in service, early AASHTO guidance for built-up flexural members used in railroad bridges, and the limitations and results of the research conducted during the development of these provisions (Hebdon et al., 2015).

During the large-scale testing performed to develop these provisions, specimens were subjected to test temperatures as low as -120°F to ensure that the fracture toughness of all components of the tension flange and web was very low. During all testing, the Charpy V-notch (CVN) impact energy of all components was limited to 10 ft-lbs or less, and was only 3 to 5 ft-lbs for the majority of the specimens. This was a conscious decision during the research to ensure that the specimens were representative of “worst-case” scenarios in terms of fracture toughness. As a result, a minimum CVN value is actually not required to utilize the provisions specified herein for evaluating older steel members regardless of the age of the steel when evaluating an existing bridge. However, for new designs, it is deemed prudent to require the satisfaction of the AASHTO M 270M/M 270 (ASTM A709/A709M) CVN impact criteria for fracture-critical members for elements in the cross-section comprised of plates. The portions of the cross-section comprised of rolled sections, such as flange angles, that satisfy the CVN requirements for fracture-critical members may not be readily available. Hence, the CVN requirements for non-fracture-critical members need only be satisfied for these sections as a matter of convenience.

C1.6

There are effectively two classes of axially loaded tension members; multi-component and two-component members. Common examples of each class of member are illustrated in Figures C1.6-1 and C1.6-2.

shall not exceed 50 percent of the gross area of the entire member. For multi-component members, any individual component shall not exceed 40 percent of the gross area of the entire member;

- For the calculation of gross and net areas, the following shall not be included:
 - Lacing;
 - Batten or stay plates;
 - Area removed by hand holes.
- All components shall be fastened together with high-strength bolts or rivets satisfying all strength, stitch and sealing requirements in effect at the time of the design;
- At gusset plates and other connections, only primary longitudinal load-carrying components need to be connected to, or through, the joint. Only the vertical legs of angles and the webs of channels need to be connected to the gusset plates, as long as that portion of the member is sufficiently developed into the joint.

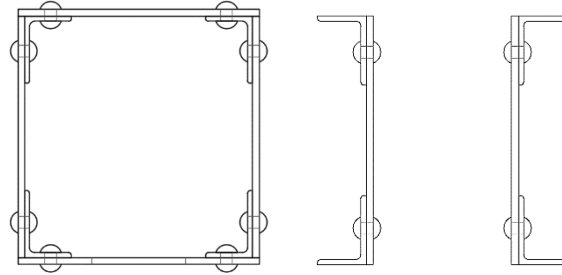


Figure C1.6-1—Typical Multi-component Axially Loaded Tension Members; Box-shaped Member (with or without hand holes) (left), and Built-up Channel Member (lacing or batten plates not shown for clarity) (right)

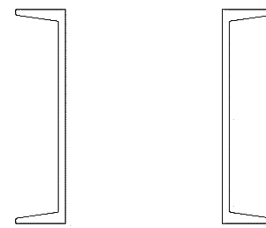


Figure C1.6-2—Typical Two-component Axially Loaded Tension Member; Rolled Channel Member (lacing or batten plates not shown for clarity)

1.6.1—Requirements for New Design of Internally Redundant Built-up Axially Loaded Tension Members

The following requirements shall apply for the new design of internally redundant built-up axially loaded tension members in the as-built unfaulted state:

- All primary plate components comprising the cross-section shall be fabricated using grades of steel satisfying the AASHTO M 270M/M 270 (ASTM A709/A709M) Charpy V-notch requirements for fracture-critical members. Other components in the cross-section, such as angles, channels, or WTs, need only satisfy the Charpy V-notch requirements for nonfracture-critical members. Batten plates, stay plates and lacing also need only satisfy the Charpy V-notch requirements for nonfracture-critical members;

C1.6.1

By definition, members satisfying these provisions are not classified as FCMs. Hence, there is no need to require the primary plate components to satisfy the fracture-critical Charpy V-notch (CVN) requirements contained in the AASHTO M 270M/M 270 (ASTM A709/A709M) specification. Nevertheless, for these components, satisfaction of the CVN requirements for fracture-critical members adds little cost. Hence, it is considered good practice to request the fracture-critical CVN requirements for plate elements when designing new built-up members. Because rolled shapes, such as angles and channels, are not typically produced to meet these more stringent requirements, the CVN requirements for nonfracture-critical members need only be satisfied for these sections as a matter of convenience.

Out-of-plane bending stresses resulting from failure of a single component are reduced when the component with the largest axial stiffness is positioned closer to the

- When lattice or batten plates are used to connect individual built-up channels comprised of a single web plate and two angles, the area of a web plate should not exceed 1.5 times the area of an individual angle;
- When multiple web plates are used, the plates shall be of equal thickness when possible. When unequal thickness plates must be used, the thickness of any individual plate should not exceed 1.5 times the thickness of any adjacent plate. The plates shall diminish in thickness toward the outside of the cross-section.

angles. Figure C1.6.1-1 illustrates the preferred plate detailing with the thinner plates positioned outward of the thicker plate or by using multiple plates of equal thickness.

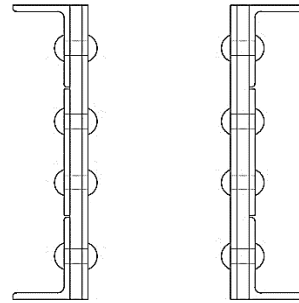


Figure C1.6.1-1—Multi-Component Built-up Axially Loaded Channel Member (lacing or batten plates not shown for clarity) with Plate Thicknesses Diminishing Toward the Outside of the Cross-Section

In the context of these provisions, an interior web plate on an axially loaded member refers to a full-depth plate having components, either plates or angles, on both sides. An exterior web plate is a full-depth plate that has at least one surface entirely exposed. Figure C1.6.1-2 illustrates two typical examples to help clarify.

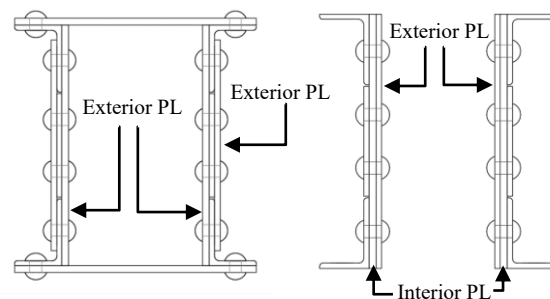


Figure C1.6.1-2—Illustrations of Typical Built-up Members. (Left) Box Member with all Exterior Plates, (Right) Channel Member (lattice removed) with Interior and Exterior Plates

1.6.2—Specific Requirements for Built-up Tension Tie Members

Any portion of a tension member that is welded together shall be considered one component of the built-up member. In such cases, component failure analysis to determine internal member redundancy shall assume that all components welded together, fail simultaneously.

C1.6.2

Some designs have been observed where small plates were welded to larger plates effectively replacing the connection angles. In such cases, the plates welded together cannot be considered independent components since the weld connecting them provides no cross-boundary fracture resistance (CBFR).

1.6.3—Specific Requirements for Multi-Eyebars Members

Members made up of eyebars in trusses shall meet the following requirements in order to be evaluated with these provisions:

- Two-eyebars diagonal or hanger members: These members shall be connected into joints in which rotation of the joint is minimal after failure of one of the two eyebars by any combination of the following:
 - Primary chord members framing into the joint or pin, if fabricated using eyebars, have a minimum of four eyebars;
 - The joint or pin to which the two-eyebars member is attached is adequately restrained from rotation by the floorbeam assembly.
- Multi-eyebars members: These members must have a minimum of four separate eyebars.

The pin shall not be considered an internally redundant member, and reliable inspections shall be programmed separately from the inspections required by these provisions.

1.7—APPLIED LOAD IN THE FAULTED STATE

1.7.1—Flexural Members

In the faulted state, the factored moment in a flexural member, M_u , for the evaluation of internal redundancy shall be calculated as follows:

$$M_u = \gamma_{DC}M_{DC} + \gamma_{DW}M_{DW} + \gamma_{LL}M_{LL+IM} \quad (1.7.1-1)$$

where:

γ_{DC} , γ_{DW} , γ_{LL} = load factors for the Internal Redundancy load combination specified in Table 1.7.1-1.

M_{DC} = bending moment due to the dead load of structural components and nonstructural attachments (kip-in.)

M_{DW} = bending moment due to the dead load of wearing surfaces and utilities (kip-in.)

M_{LL+IM} = bending moment due to the vehicular live load and dynamic load allowance (kip-in.)

C1.6.3

Connections and adjoining members play a critical role in the member-level redundancy of members comprised of eyebars. This type of member should be considered as an assembly in context with its connection to adjacent members. Of particular concern is rotational restraint of the connection when a single eyebars fails in a member comprised of only two eyebars. Significant rotation of connecting pins following failure of a single eyebars could allow the remaining eyebars, as well as other members sharing the pin, to slip off the pin. Some multi-eyebars members are known to possess as many as eight separate eyebars. In these cases, experience has shown that failure of a single eyebars does not result in catastrophic failure of the member. Another case is when a two-eyebars member, such as a tension diagonal, connects into a chord member comprised of several eyebars (e.g., 4 eyebars), a built-up chord and/or a built-up floorbeam and hanger offering high rotational stiffness. In these cases, the assembly would likely resist rotations caused by failure of one of the two eyebars. Thus, the internal redundancy of the eyebars member is highly dependent upon its boundary conditions. Note that the pin is not to be considered an internally redundant member (Connor et al., 2018). Tension chords comprised of only two eyebars are explicitly excluded from these provisions regardless of the rotational restraint at the pin.

C1.7

C1.7.1

Equation 1.7.1-1 presents the basic load combination for calculating the total factored moment in a flexural member for the evaluation of internal redundancy in the faulted state. This load combination may be directly applied to noncomposite sections. For composite sections, the effects of the locked-in dead load moment applied to the noncomposite section must be differentiated from the effects of the moments applied to the composite section. The treatment of composite and noncomposite sections in flexural members is addressed further in Article 2.1.

The load factors for the internal redundancy load combination specified in Table 1.7.1-1 represent the loads that may be expected to occur in the faulted state and that a member in the faulted state must be able to withstand prior to discovery and repair. These load factors, which correspond to the Redundancy II load combination in NCHRP Report 883 (Connor et al., 2018), were calibrated to the Strength I limit state at target reliability indices of 1.5 and 2.5 for bridges built to the *AASHTO/AWS D1.5M/D1.5 Bridge Welding Code*

Table 1.7.1-1—Load Factors for the Internal Redundancy Load Combination

Application	γ_{DC}	γ_{DW}	γ_{LL}
Bridges built to AASHTO/AWS D1.5 FCP	1.05	1.05	1.30
All other bridges	1.15	1.25	1.50

The applied live load shall be the HL-93 design vehicular live load specified in LRFD Design Article 3.6.1.2.1. The dynamic load allowance, IM, shall be taken as 15 percent, and shall not be applied to the design lane load. The design truck or design tandem shall be positioned longitudinally and transversely such that the load effects are maximized in the faulted state. When multiple lanes are loaded, the multiple presence factors specified in LRFD Design Table 3.6.1.1.2-1 shall be applied.

Additional live loads and load combinations may be considered if deemed appropriate by the Owner.

1.7.2—Axially Loaded Tension Members

In the faulted state, the factored load in an axially loaded tension member, P_u , for the evaluation of internal redundancy shall be calculated as follows:

$$P_u = \gamma_{DC}P_{DC} + \gamma_{DW}P_{DW} + \gamma_{LL}P_{LL+IM} \quad (1.7.2-1)$$

where:

γ_{DC} , γ_{DW} , γ_{LL} = load factors for the Internal Redundancy load combination specified in Table 1.7.1-1.

Fracture Control Plan (FCP) (AASHTO/AWS, 2015), and bridges that were not, respectively (Connor et al., 2018). The AASHTO AWS D1.5 FCP was first introduced into the Bridge Welding Code in 1995. However, it is recognized that forms of the AASHTO FCP were contained in AASHTO guide specifications as early as 1978. During these transition years, Owners should confirm that the members were fabricated to meet the overall objectives of the FCP; for example, it should be ensured that unauthorized weld repairs were not made on the components.

The justification for using an apparent lower reliability index for bridges meeting the modern FCP is intended to recognize that the probability of fracture of any given component is significantly less due to the improved material toughness, fabrication practices, and shop inspection required by the AASHTO/AWS FCP for non-redundant members. Further, such members would be fastened using highly fatigue resistant high-strength bolted connections, rather than rivets, and would be designed for infinite life in the unfaulted state. Hence, the decreased likelihood associated with an individual component fracture is exploited by using the lower load factors, but without compromising the actual reliability of the member.

In order to include possible dynamic amplification associated with the passage of traffic, the lower dynamic load allowance, IM, of 15 percent utilized in the fatigue load combinations in LRFD Design (AASHTO, 2017) is applied to the live load.

Owners may wish to consider other live loads to evaluate the strength of the damaged structure, such as permit loads if deemed appropriate. Because the HL-93 notional load envelopes the response produced by most vehicles, there may not be a need to evaluate the effects associated with other vehicles in the faulted state.

C1.7.2

Equation 1.7.2-1 presents the basic load combination for calculating the total factored tensile load in an axially loaded tension member for the evaluation of internal redundancy in the faulted state. Methods for calculating the redistribution of load in the faulted state for typical axially loaded tension-member cross-sections are specified in Article 2.2.

P_{DC} = axial tensile load due to the dead load of structural components and nonstructural attachments (kip)

P_{DW} = axial tensile load due to the dead load of wearing surfaces and utilities (kip)

P_{LL+IM} = axial tensile load due to the vehicular live load and dynamic load allowance (kip)

1.8—IN-SERVICE SPECIAL INSPECTIONS

The maximum interval between in-service special inspections for members classified as IRMs using these provisions shall be determined as specified in Article 3.1. These provisions shall not be taken to alter the interval for normal safety inspections.

C1.8

For built-up members that satisfy the provisions specified herein, instantaneous full-depth fracture is not a plausible failure limit state. Hence, these members need not be subjected to the same in-service fracture-critical member inspections required for other flexural members traditionally classified as fracture-critical members (FCMs) and are subsequently classified as internally redundant members (IRMs). Built-up members satisfying the proportioning requirements specified in Article 1 also possess sufficient internal redundancy to carry applied loads in the faulted state without compromising the safety of the structure. Therefore, these members should not be classified as FCMs.

As discussed in Article 3, fatigue in the faulted state must be evaluated to determine an appropriate maximum interval between the special inspections calculated in Article 3 to ensure that premature cracking of the remaining components near a localized component failure does not go undetected.

This page intentionally left blank.