COMPANION TO THE AISC *STEEL CONSTRUCTION MANUAL*

Volume 1: Design Examples

Version 15.1

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PREFACE

The primary objective of this Companion is to provide guidance and additional resources of the use of the 2016 AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-16) and the 15th Edition AISC *Steel Construction Manual*.

The Companion consists of design examples in Parts I, II and III. The design examples provide coverage of all applicable limit states, whether or not a particular limit state controls the design of the member or connection. In addition to the examples that demonstrate the use of the AISC *Manual* tables, design examples are provided for connection designs beyond the scope of the tables in the AISC *Manual*. These design examples are intended to demonstrate an approach to the design, and are not intended to suggest that the approach presented is the only approach. The committee responsible for the development of these design examples recognizes that designers have alternate approaches that work best for them and their projects. Design approaches that differ from those presented in these examples are considered viable as long as the AISC *Specification*, sound engineering, and project specific requirements are satisfied.

Part I of these examples is organized to correspond with the organization of the AISC *Specification*. The Chapter titles match the corresponding chapters in the AISC *Specification*.

Part II is devoted primarily to connection examples that draw on the tables from the AISC *Manual*, recommended design procedures, and the breadth of the AISC *Specification*. The chapters of Part II are labeled II-A, II-B, II-C, etc.

Part III addresses aspects of design that are linked to the performance of a building as a whole. This includes coverage of lateral stability and second-order analysis, illustrated through a four-story braced-frame and moment-frame building.

The Design Examples are arranged with LRFD and ASD designs presented side-by-side, for consistency with the AISC *Manual*. Design with ASD and LRFD are based on the same nominal strength for each element so that the only differences between the approaches are the set of load combinations from ASCE/SEI 7-16 used for design, and whether the resistance factor for LRFD or the safety factor for ASD is used.

CONVENTIONS

The following conventions are used throughout these examples:

1. The 2016 AISC *Specification for Structural Steel Buildings* is referred to as the AISC *Specification* and the 15th Edition AISC *Steel Construction Manual,* is referred to as the AISC *Manual*.

2. The 2016 ASCE *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* is referred to as ASCE/SEI 7.

3. The source of equations or tabulated values taken from the AISC *Specification* or AISC *Manual* is noted along the right-hand edge of the page.

4. When the design process differs between LRFD and ASD, the designs equations are presented side-by-side. This rarely occurs, except when the resistance factor, ϕ, and the safety factor, Ω, are applied.

5. The results of design equations are presented to three significant figures throughout these calculations.

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Part I
Examples Based on the AISC *Specification*

This part contains design examples demonstrating select provisions of the AISC *Specification for Structural Steel Buildings.*

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Chapter A
General Provisions

A1. SCOPE

These design examples are intended to illustrate the application of the 2016 AISC *Specification for Structural Steel Buildings*, ANSI/AISC 360-16 (AISC, 2016a), and the AISC *Steel Construction Manual*, 15th Edition (AISC, 2017) in low-seismic applications. For information on design applications requiring seismic detailing, see the 2016 AISC *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-16 (AISC, 2016b) and the AISC *Seismic Design Manual*, 2nd Edition (AISC, 2012).

A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

Section A2 includes a detailed list of the specifications, codes and standards referenced throughout the AISC *Specification*.

A3. MATERIAL

Section A3 includes a list of the steel materials that are approved for use with the AISC *Specification*. The complete ASTM standards for the most commonly used steel materials can be found in *Selected ASTM Standards for Structural Steel Fabrication* (ASTM, 2016).

A4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

Section A4 requires that structural design drawings and specifications meet the requirements in the AISC *Code of Standard Practice for Steel Buildings and Bridges*, ANSI/AISC 303-16 (AISC, 2016c).

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CHAPTER A REFERENCES

AISC (2012), *Seismic Design Manual*, 2nd Ed., American Institute of Steel Construction, Chicago, IL.

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AISC (2016c), *Code of Standard Practice for Steel Buildings and Bridges*, ANSI/AISC 303-16, American Institute of Steel Construction, Chicago, IL.

AISC (2017), *Steel Construction Manual*, 15th Ed., American Institute of Steel Construction, Chicago, IL.

ASTM (2016), *Selected ASTM Standards for Structural Steel Fabrication*, ASTM International, West Conshohocken, PA.

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Chapter B
Design Requirements

B1. GENERAL PROVISIONS

The AISC *Specification* requires that the design of members and connections shall be consistent with the intended behavior of the framing system and the assumptions made in the structural analysis.

B2. LOADS AND LOAD COMBINATIONS

In the absence of an applicable building code, the default load combinations to be used with the AISC *Specification* are those from *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-16 (ASCE, 2016).

B3. DESIGN BASIS

Chapter B of the AISC *Specification* and Part 2 of the AISC *Manual* describe the basis of design, for both load and resistance factor design (LRFD) and allowable strength design (ASD).

AISC *Specification* Section B3.4 describes three basic types of connections: simple connections, fully restrained (FR) moment connections, and partially restrained (PR) moment connections. Several examples of the design of each of these types of connections are given in Part II of these *Design Examples.*

Information on the application of serviceability and ponding provisions may be found in AISC *Specification* Chapter L and AISC *Specification* Appendix 2, respectively, and their associated commentaries. Design examples and other useful information on this topic are given in AISC Design Guide 3, *Serviceability Design Considerations for Steel Buildings*, Second Edition (West et al., 2003).

Information on the application of fire design provisions may be found in AISC *Specification* Appendix 4 and its associated commentary. Design examples and other useful information on this topic are presented in AISC Design Guide 19, *Fire Resistance of Structural Steel Framing* (Ruddy et al., 2003).

Corrosion protection and fastener compatibility are discussed in Part 2 of the AISC *Manual*.

B4. MEMBER PROPERTIES

AISC *Specification* Tables B4.1a and B4.1b give the complete list of limiting width-to-thickness ratios for all compression and flexural members defined by the AISC *Specification*.

Except for one section, the W-shapes presented in the compression member selection tables as column sections meet the criteria as nonslender element sections. The W-shapes with a nominal depth of 8 in. or larger presented in the flexural member selection tables as beam sections meet the criteria for compact sections, except for seven specific shapes. When noncompact or slender-element sections are tabulated in the design aids, local buckling criteria are accounted for in the tabulated design values.

The shapes listing and other member design tables in the AISC *Manual* also include footnoting to highlight sections that exceed local buckling limits in their most commonly available material grades. These footnotes include the following notations for W-shapes:

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.

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[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi.

CHAPTER B REFERENCES

ASCE (2016), *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-16, American Society of Civil Engineers, Reston, VA.

West, M.A., Fisher, J.M. and Griffis, L.G. (2003), *Serviceability Design Considerations for Steel Buildings*, Design Guide 3, 2nd Ed., AISC, Chicago, IL.

Ruddy, J.L., Marlo, J.P., Ioannides, S.A. and Alfawakhiri, F. (2003), *Fire Resistance of Structural Steel Framing*, Design Guide 19, AISC, Chicago, IL.

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Chapter C
Design for Stability

C1. GENERAL STABILITY REQUIREMENTS

The AISC *Specification* requires that the designer account for both the stability of the structural system as a whole and the stability of individual elements. Thus, the lateral analysis used to assess stability must include consideration of the combined effect of gravity and lateral loads, as well as member inelasticity, out-of-plumbness, out-of-straightness, and the resulting second-order effects, P-Δ and P-δ. The effects of "leaning columns" must also be considered, as illustrated in the examples in this chapter and in the four-story building design example in Part III of these *Design Examples*.

P-Δ and P-δ effects are illustrated in AISC *Specification* Commentary Figure C-C2.1. Methods for addressing stability, including P-Δ and P-δ effects, are provided in AISC *Specification* Section C2 and Appendix 7.

C2. CALCULATION OF REQUIRED STRENGTHS

The calculation of required strengths is illustrated in the examples in this chapter and in the four-story building design example in Part III of these *Design Examples*.

C3. CALCULATION OF AVAILABLE STRENGTHS

The calculation of available strengths is illustrated in the four-story building design example in Part III of these *Design Examples*.

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EXAMPLE C.1A DESIGN OF A MOMENT FRAME BY THE DIRECT ANALYSIS METHOD

Given:

Determine the required strengths and effective length factors for the columns in the moment frame shown in Figure C.1A-1 for the maximum gravity load combination, using LRFD and ASD. The uniform load, w_D, includes beam self-weight and an allowance for column self-weight. Use the direct analysis method. All members are ASTM A992 material.

Columns are unbraced between the footings and roof in the *x*- and *y*-axes and have pinned bases.

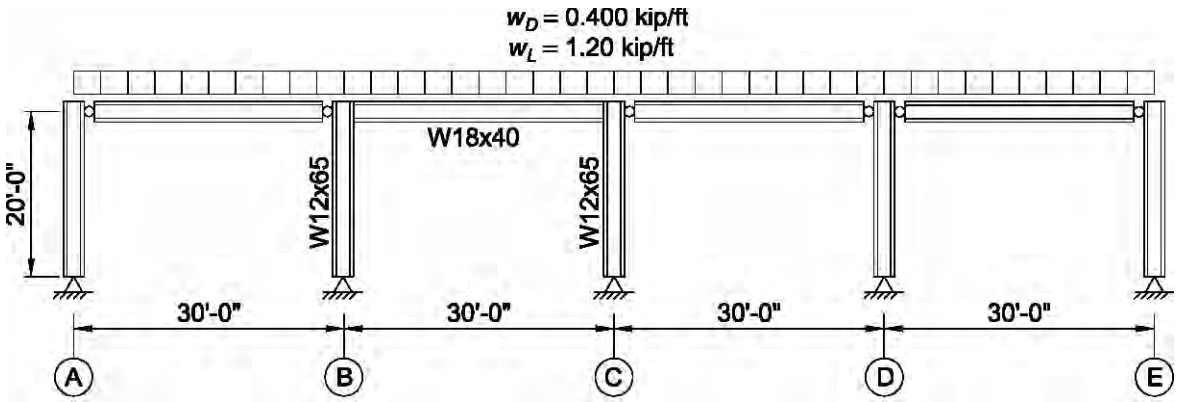


Fig. C.1A-1. Example C.1A moment frame elevation.

Solution:

From AISC *Manual* Table 1-1, the W12×65 has $A = 19.1$ in.^{2}

The beams from grid lines A to B and C to E and the columns at A, D and E are pinned at both ends and do not contribute to the lateral stability of the frame. There are no *P*-Δ effects to consider in these members and they may be designed using $L_c = L$.

The moment frame between grid lines B and C is the source of lateral stability and therefore will be evaluated using the provisions of Chapter C of the AISC *Specification*. Although the columns at grid lines A, D and E do not contribute to lateral stability, the forces required to stabilize them must be considered in the moment-frame analysis. The entire frame from grid line A to E could be modeled, but in this case the model is simplified as shown in Figure C.1A-2, in which the stability loads from the three "leaning" columns are combined into a single representative column.

From Chapter 2 of ASCE/SEI 7, the maximum gravity load combinations are:

LRFD	ASD
$w_u = 1.2D + 1.6L$ $= 1.2(0.400 \text{ kip/ft}) + 1.6(1.20 \text{ kip/ft})$ $= 2.40 \text{ kip/ft}$	$w_u = D + L$ $= 0.400 \text{ kip/ft} + 1.20 \text{ kip/ft}$ $= 1.60 \text{ kip/ft}$

Per AISC *Specification* Section C2.1(d), for LRFD, perform a second-order analysis and member strength checks using the LRFD load combinations. For ASD, perform a second-order analysis using 1.6 times the ASD load combinations and divide the analysis results by 1.6 for the ASD member strength checks.

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Frame analysis gravity loads

The uniform gravity loads to be considered in a second-order analysis on the beam from B to C are:

LRFD	ASD
$w_u' = 2.40$ kip/ft	$w_a' = 1.6(1.60 \text{ kip/ft})$ $= 2.56$ kip/ft

Concentrated gravity loads to be considered in a second-order analysis on the columns at B and C contributed by adjacent beams are:

LRFD	ASD
$P_u' = \frac{w_u' l}{2}$ $= \frac{(2.40 \text{ kip/ft})(30.0 \text{ ft})}{2}$ $= 36.0$ kips	$P_a' = \frac{w_a' l}{2}$ $= \frac{(2.56 \text{ kip/ft})(30.0 \text{ ft})}{2}$ $= 38.4$ kips

Concentrated gravity loads on the representative "leaning" column

The load in this column accounts for all gravity loading that is stabilized by the moment frame, but is not directly applied to it.

LRFD	ASD
$P_{uL}' = (60.0 \text{ ft})(2.40 \text{ kip/ft})$ $= 144$ kips	$P_{aL}' = (60.0 \text{ ft})(2.56 \text{ kip/ft})$ $= 154$ kips

Frame analysis notional loads

Per AISC *Specification* Section C2.2, frame out-of-plumbness must be accounted for either by explicit modeling of the assumed out-of-plumbness or by the application of notional loads. Use notional loads.

From AISC *Specification* Equation C2-1, the notional loads are:

LRFD	ASD
$\alpha = 1.0$ $Y_i = (120 \text{ ft})(2.40 \text{ kip/ft})$ $= 288$ kips $N_i = 0.002\alpha Y_i$ (*Spec.* Eq. C2-1) $= 0.002(1.0)(288 \text{ kips})$ $= 0.576$ kip	$\alpha = 1.6$ $Y_i = (120 \text{ ft})(1.60 \text{ kip/ft})$ $= 192$ kips $N_i = 0.002\alpha Y_i$ (*Spec.* Eq. C2-1) $= 0.002(1.6)(192 \text{ kips})$ $= 0.614$ kip

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Summary of applied frame loads

The applied loads are shown in Figure C.1A-2.

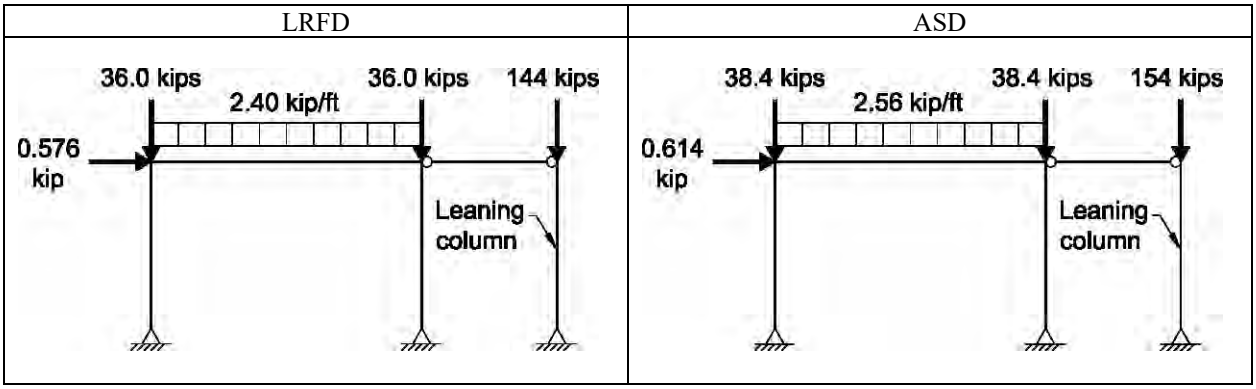


Fig. C.1A-2. Applied loads on the analysis model.

Per AISC *Specification* Section C2.3, conduct the analysis using 80% of the nominal stiffnesses to account for the effects of inelasticity. Assume, subject to verification, that $\alpha P_r / P_{ns}$ is not greater than 0.5; therefore, no additional stiffness reduction is required ($\tau_b = 1.0$).

Half of the gravity load is carried by the columns of the moment-resisting frame. Because the gravity load supported by the moment-resisting frame columns exceeds one-third of the total gravity load tributary to the frame, per AISC *Specification* Section C2.1, the effects of *P*-δ and *P*-Δ must be considered in the frame analysis. This example uses analysis software that accounts for both *P*-Δ and *P*-δ effects. (If the software used does not account for *P*-δ effects this may be accomplished by subdividing the columns between the footing and beam.)

Figures C.1A-3 and C.1A-4 show results from a first-order and a second-order analysis. (The first-order analysis is shown for reference only.) In each case, the drift is the average of drifts at grid lines B and C.

First-order results

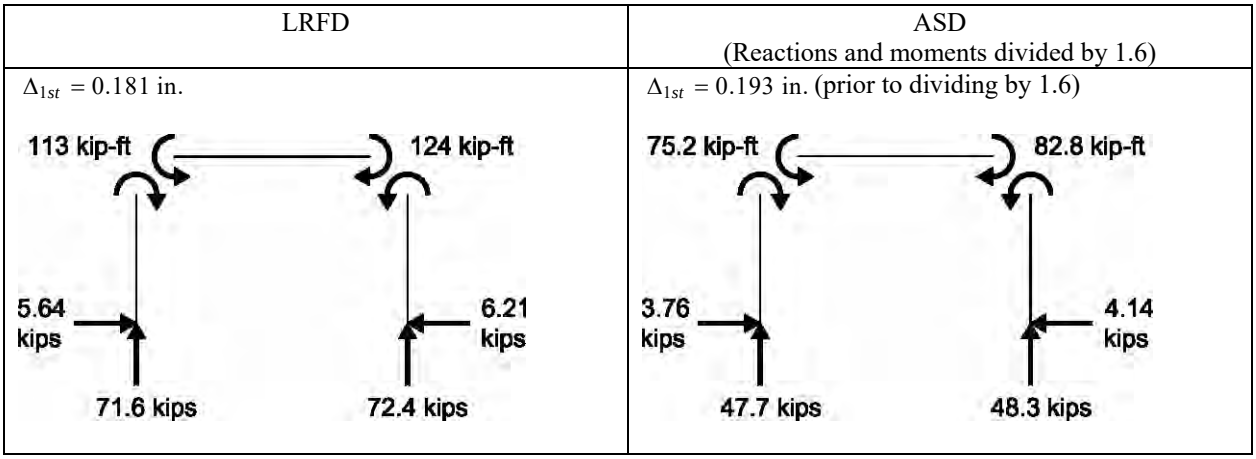


Fig. C.1A-3. Results of first-order analysis.

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Second-order results

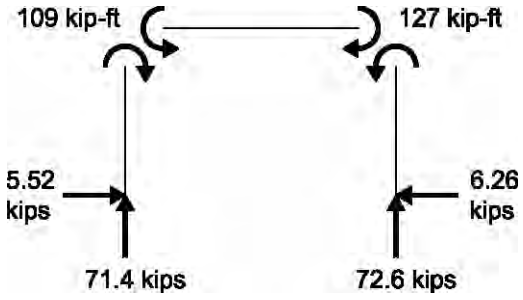
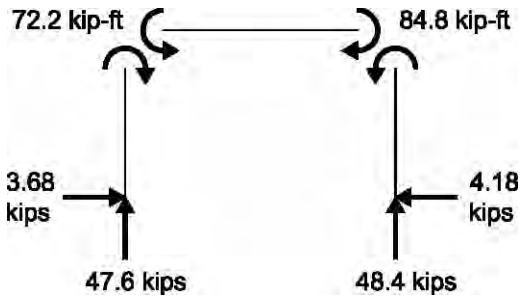
LRFD	ASD (Reactions and moments divided by 1.6)
$\Delta_{2nd} = 0.290$ in. Drift ratio: $\frac{\Delta_{2nd}}{\Delta_{1st}} = \frac{0.290 \text{ in.}}{0.181 \text{ in.}}$ $= 1.60$  	$\Delta_{2nd} = 0.321$ in. (prior to dividing by 1.6) Drift ratio: $\frac{\Delta_{2nd}}{\Delta_{1st}} = \frac{0.321 \text{ in.}}{0.193 \text{ in.}}$ $= 1.66$ 

Fig. C.1A-4. Results of second-order analysis.

Check the assumption that $\alpha P_r / P_{ns} \le 0.5$ on the column on grid line C.

Because a W12×65 column contains no elements that are slender for uniform compression,

$$P_{ns} = F_y A_g$$
$$= (50 \text{ ksi})(19.1 \text{ in.}^2)$$
$$= 955 \text{ kips}$$

LRFD	ASD
$\frac{\alpha P_r}{P_{ns}} = \frac{1.0(72.6 \text{ kips})}{955 \text{ kips}}$ $= 0.0760 \le 0.5$ **o.k.**	$\frac{\alpha P_r}{P_{ns}} = \frac{1.6(48.4 \text{ kips})}{955 \text{ kips}}$ $= 0.0811 \le 0.5$ **o.k.**

The stiffness assumption used in the analysis, $\tau_b = 1.0$, is verified.

Note that the drift ratio, 1.60 (LRFD) or 1.66 (ASD), does not exceed the recommended limit of 2.5 from AISC *Specification* Commentary Section C1.

The required axial compressive strength in the columns is 72.6 kips (LRFD) or 48.4 kips (ASD). The required bending moment diagram is linear, varying from zero at the bottom to 127 kip-ft (LRFD) or 84.8 kip-ft (ASD) at the top. These required strengths apply to both columns because the notional load must be applied in each direction.

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Although the second-order sway multiplier (drift ratio) is fairly large at 1.60 (LRFD) or 1.66 (ASD), the change in bending moment is small because the only sway moments are those produced by the small notional loads. For load combinations with significant gravity and lateral loadings, the increase in bending moments is larger.

Per AISC *Specification* Section C3, the effective length for flexural buckling of all members is taken as the unbraced length ($K = 1.0$):

$L_{cx} = 20.0$ ft
$L_{cy} = 20.0$ ft

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EXAMPLE C.1B DESIGN OF A MOMENT FRAME BY THE EFFECTIVE LENGTH METHOD

Given:

Repeat Example C.1A using the effective length method.

Determine the required strengths and effective length factors for the columns in the moment frame shown in Figure C.1B-1 for the maximum gravity load combination, using LRFD and ASD. Use the effective length method.

Columns are unbraced between the footings and roof in the *x*- and *y*-axes and have pinned bases.

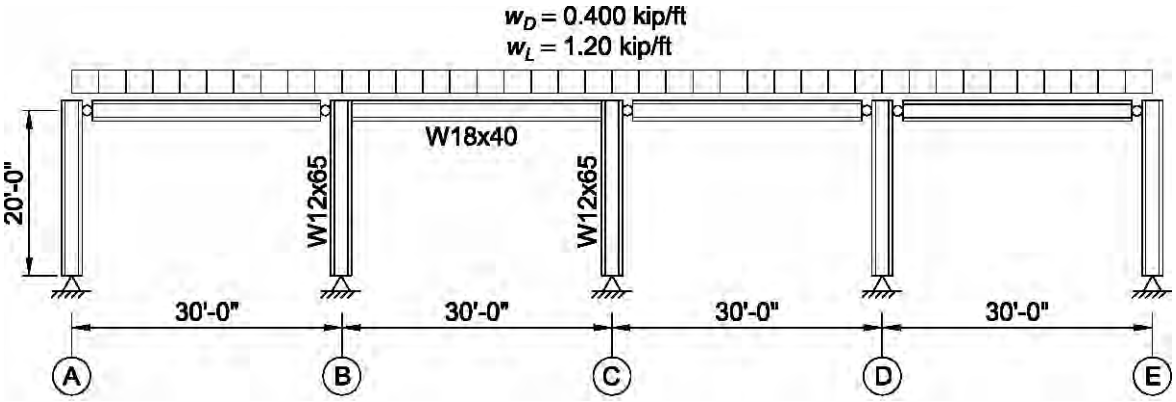


Fig. C.1B-1. Example C.1B moment frame elevation.

Solution:

From AISC *Manual* Table 1-1, the W12×65 has I_x = 533 in.^{4}

The beams from grid lines A to B and C to E and the columns at A, D and E are pinned at both ends and do not contribute to the lateral stability of the frame. There are no *P*-Δ effects to consider in these members and they may be designed using $L_c = L$.

The moment frame between grid lines B and C is the source of lateral stability and therefore will be evaluated using the provisions of Chapter C of the AISC *Specification*. Although the columns at grid lines A, D and E do not contribute to lateral stability, the forces required to stabilize them must be considered in the moment-frame analysis. The entire frame from grid line A to E could be modeled, but in this case the model is simplified as shown in Figure C.1B-2, in which the stability loads from the three "leaning" columns are combined into a single representative column.

Check the limitations for the use of the effective length method given in AISC *Specification* Appendix 7, Section 7.2.1:

(a) The structure supports gravity loads primarily through nominally vertical columns, walls or frames.

(b) The ratio of maximum second-order drift to the maximum first-order drift (both determined for LRFD load combinations or 1.6 times ASD load combinations, with stiffness not adjusted as specified in AISC *Specification* Section C2.3) in all stories will be assumed to be no greater than 1.5, subject to verification in the following.

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From Chapter 2 of ASCE/SEI 7, the maximum gravity load combinations are:

LRFD	ASD
$w_u = 1.2D + 1.6L$ $= 1.2(0.400 \text{ kip/ft}) + 1.6(1.20 \text{ kip/ft})$ $= 2.40 \text{ kip/ft}$	$w_u = D + L$ $= 0.400 \text{ kip/ft} + 1.20 \text{ kip/ft}$ $= 1.60 \text{ kip/ft}$

Per AISC *Specification* Appendix 7, Section 7.2.2, the analysis must conform to the requirements of AISC *Specification* Section C2.1, with the exception of the stiffness reduction required by the provisions of Section C2.1(a).

Per AISC *Specification* Section C2.1(d), for LRFD perform a second-order analysis and member strength checks using the LRFD load combinations. For ASD, perform a second-order analysis at 1.6 times the ASD load combinations and divide the analysis results by 1.6 for the ASD member strength checks.

Frame analysis gravity loads

The uniform gravity loads to be considered in a second-order analysis on the beam from B to C are:

LRFD	ASD
$w'_u = 2.40 \text{ kip/ft}$	$w'_a = 1.6(1.60 \text{ kip/ft})$ $= 2.56 \text{ kip/ft}$

Concentrated gravity loads to be considered in a second-order analysis on the columns at B and C contributed by adjacent beams are:

LRFD	ASD
$P'_u = \frac{w'_u l}{2}$ $= \frac{(2.40 \text{ kip/ft})(30.0 \text{ ft})}{2}$ $= 36.0 \text{ kips}$	$P'_a = \frac{w'_a l}{2}$ $= \frac{(2.56 \text{ kip/ft})(30.0 \text{ ft})}{2}$ $= 38.4 \text{ kips}$

Concentrated gravity loads on the representative "leaning" column

The load in this column accounts for all gravity loads that is stabilized by the moment frame, but not directly applied to it.

LRFD	ASD
$P'_{uL} = (60.0 \text{ ft})(2.40 \text{ kip/ft})$ $= 144 \text{ kips}$	$P'_{aL} = (60.0 \text{ ft})(2.56 \text{ kip/ft})$ $= 154 \text{ kips}$

Frame analysis notional loads

Per AISC *Specification* Appendix 7, Section 7.2.2, frame out-of-plumbness must be accounted for by the application of notional loads in accordance with AISC *Specification* Section C2.2b. Note that notional loads need to only be applied to the gravity load combinations per AISC *Specification* Section C2.2b(d) when the requirement that $\Delta_{2nd} / \Delta_{1st} \le 1.7$ (using stiffness adjusted as specified in Section C2.3) is satisfied. Per the User Note in AISC *Specification* Appendix 7, Section 7.2.2, Section C2.2b(d) will be satisfied in all cases where the effective length method is applicable, and therefore the notional load need only be applied in gravity-only load cases.
From AISC *Specification* Equation C2-1, the notional loads are:

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LRFD	ASD
$\alpha = 1.0$	$\alpha = 1.6$
$Y_i = (120\text{ ft})(2.40\text{ kip/ft})$ $= 288\text{ kips}$	$Y_i = (120\text{ ft})(1.60\text{ kip/ft})$ $= 192\text{ kips}$
$N_i = 0.002\alpha Y_i$ (*Spec.* Eq. C2-1) $= 0.002(1.0)(288\text{ kips})$ $= 0.576\text{ kip}$	$N_i = 0.002\alpha Y_i$ (*Spec.* Eq. C2-1) $= 0.002(1.6)(192\text{ kips})$ $= 0.614\text{ kip}$

Summary of applied frame loads

The applied loads are shown in Figure C.1B-2.

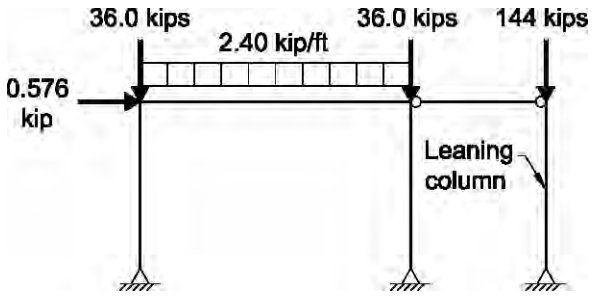
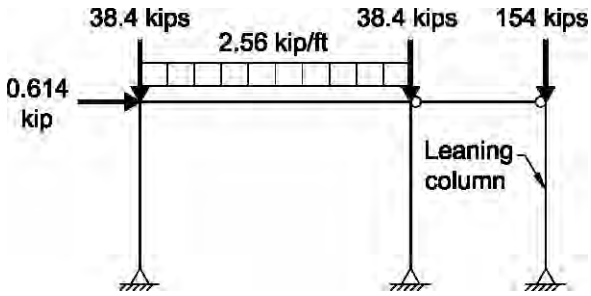
LRFD	ASD
	

Fig. C.1B-2. Applied loads on the analysis model.

Per AISC *Specification* Appendix 7, Section 7.2.2, conduct the analysis using the full nominal stiffnesses.

Half of the gravity load is carried by the columns of the moment-resisting frame. Because the gravity load supported by the moment-resisting frame columns exceeds one-third of the total gravity load tributary to the frame, per AISC *Specification* Section C2.1(b), the effects of P-δ on the response of the structure must be considered in the frame analysis. This example uses analysis software that accounts for both P-Δ and P-δ effects. When using software that does not account for P-δ effects, this could be accomplished by subdividing columns between the footing and beam.

Figures C.1B-3 and C.1B-4 show results from a first-order and second-order analysis. In each case, the drift is the average of drifts at grid lines B and C.

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First-order results

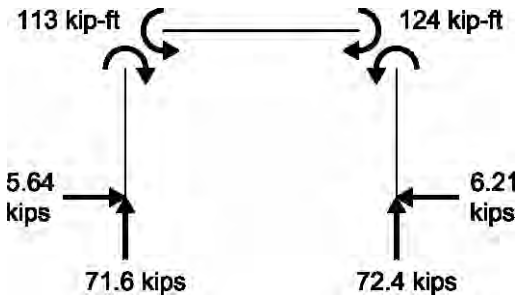
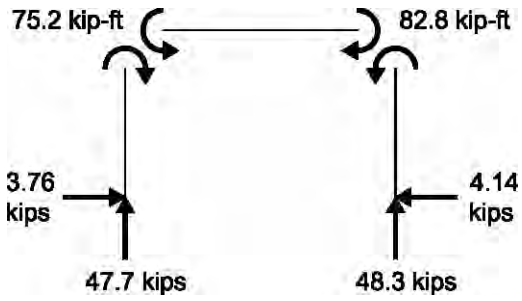
LRFD	ASD (Reactions and moments divided by 1.6)
$\Delta_{1st} = 0.145$ in.	$\Delta_{1st} = 0.155$ in. (prior to dividing by 1.6)
	

Fig. C.1B-3. Results of first-order analysis.

Second-order results

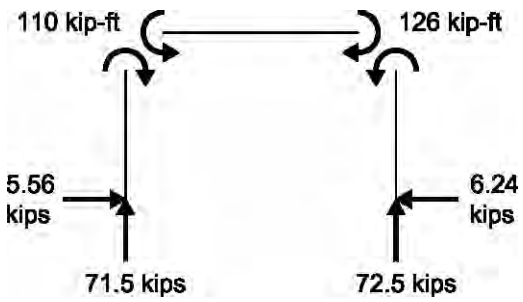
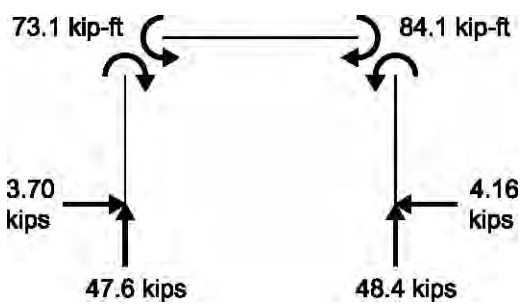
LRFD	ASD
$\Delta_{2nd} = 0.204$ in. Drift ratio: $\frac{\Delta_{2nd}}{\Delta_{1st}} = \frac{0.204 \text{ in.}}{0.145 \text{ in.}} = 1.41$	$\Delta_{2nd} = 0.223$ in. (prior to dividing by 1.6) Drift ratio: $\frac{\Delta_{2nd}}{\Delta_{1st}} = \frac{0.223 \text{ in.}}{0.155 \text{ in.}} = 1.44$
	

Fig. C-1B-4. Results of second-order analysis.

The assumption that the ratio of the maximum second-order drift to the maximum first-order drift is no greater than 1.5 is verified; therefore, the effective length method is permitted.

Although the second-order sway multiplier is fairly large at approximately 1.41 (LRFD) or 1.44 (ASD), the change in bending moment is small because the only sway moments for this load combination are those produced by the small notional loads. For load combinations with significant gravity and lateral loadings, the increase in bending moments is larger.

Calculate the in-plane effective length factor, K_x, using the "story stiffness approach" and Equation C-A-7-5 presented in AISC *Specification* Commentary Appendix 7, Section 7.2. With $K_x = K_2$:

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$$K_x = \sqrt{\frac{P_{story}}{R_M P_r}\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{HL}\right)} \geq \sqrt{\frac{\pi^2 EI}{L^2}\left(\frac{\Delta_H}{1.7 H_{col} L}\right)} \qquad \text{(Spec. Eq. C-A-7-5)}$$

Calculate the total load in all columns, P_{story}, as follows:

LRFD	ASD
$P_{story} = (2.40 \text{ kip/ft})(120 \text{ ft})$ $= 288 \text{ kips}$	$P_{story} = (1.60 \text{ kip/ft})(120 \text{ ft})$ $= 192 \text{ kips}$

Calculate the coefficient to account for the influence of P-δ on P-Δ, R_M, as follows, using AISC *Specification* Commentary Appendix 7, Equation C-A-7-6:

LRFD	ASD
$P_{mf} = 71.5 \text{ kips} + 72.5 \text{ kips}$ $= 144 \text{ kips}$	$P_{mf} = 47.6 \text{ kips} + 48.4 \text{ kips}$ $= 96.0 \text{ kips}$
$R_M = 1 - 0.15(P_{mf}/P_{story})$ (*Spec.* Eq. C-A-7-6) $= 1 - 0.15\left(\frac{144 \text{ kips}}{288 \text{ kips}}\right)$ $= 0.925$	$R_M = 1 - 0.15(P_{mf}/P_{story})$ (*Spec.* Eq. C-A-7-6) $= 1 - 0.15\left(\frac{96.0 \text{ kips}}{192 \text{ kips}}\right)$ $= 0.925$

Calculate the Euler buckling strength of one moment frame.

$$\frac{\pi^2 EI}{L^2} = \frac{\pi^2 (29{,}000 \text{ ksi})(533 \text{ in.}^4)}{[(20.0 \text{ ft})(12 \text{ in./ft})]^2}$$
$$= 2{,}650 \text{ kips}$$

From AISC *Specification* Commentary Equation C-A-7-5, for the column at line C:

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LRFD	ASD
$K_x = \sqrt{\frac{P_{story}}{R_M P_r}\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{HL}\right)}$ $\geq \sqrt{\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{1.7 H_{col} L}\right)}$ $= \sqrt{\left[\frac{288 \text{ kips}}{(0.925)(72.5 \text{ kips})}\right](2{,}650 \text{ kips}) \times \left[\frac{0.145 \text{ in.}}{(0.576 \text{ kip})(20.0 \text{ ft})(12 \text{ in./ft})}\right]}$ $\geq \sqrt{(2{,}650 \text{ kips}) \times \left[\frac{0.145 \text{ in.}}{1.7(6.21 \text{ kips})(20.0 \text{ ft})(12 \text{ in./ft})}\right]}$ $= 3.45 \geq 0.389$ Use $K_x = 3.45$	$K_x = \sqrt{\frac{1.6 P_{story}}{R_M (1.6) P_r}\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{HL}\right)}$ $\geq \sqrt{\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{1.7(1.6) H_{col} L}\right)}$ $= \sqrt{\left[\frac{1.6(192 \text{ kips})}{0.925(1.6)(48.4 \text{ kips})}\right](2{,}650 \text{ kips}) \times \left[\frac{0.155 \text{ in.}}{(0.614 \text{ kip})(20.0 \text{ ft})(12 \text{ in./ft})}\right]}$ $\geq \sqrt{(2{,}650 \text{ kips}) \times \left[\frac{0.155 \text{ in.}}{1.7(1.6)(4.14 \text{ kips})(20.0 \text{ ft})(12 \text{ in./ft})}\right]}$ $= 3.46 \geq 0.390$ Use $K_x = 3.46$

Note that the column loads are multiplied by 1.6 for ASD in Equation C-A-7-5.

With $K_x = 3.45$ and $K_y = 1.00$, the column available strengths can be verified for the given member sizes for the second-order forces (calculations not shown), using the following effective lengths:

$$L_{cx} = K_x L_x = 3.45(20.0 \text{ ft}) = 69.0 \text{ ft}$$

$$L_{cy} = K_y L_y = 1.00(20.0 \text{ ft}) = 20.0 \text{ ft}$$

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EXAMPLE C.1C DESIGN OF A MOMENT FRAME BY THE FIRST-ORDER METHOD

Given:

Repeat Example C.1A using the first-order analysis method.

Determine the required strengths and effective length factors for the columns in the moment frame shown in Figure C.1C-1 for the maximum gravity load combination, using LRFD and ASD. Use the first-order analysis method as given in AISC *Specification* Appendix 7, Section 7.3.

Columns are unbraced between the footings and roof in the *x*- and *y*-axes and have pinned bases.

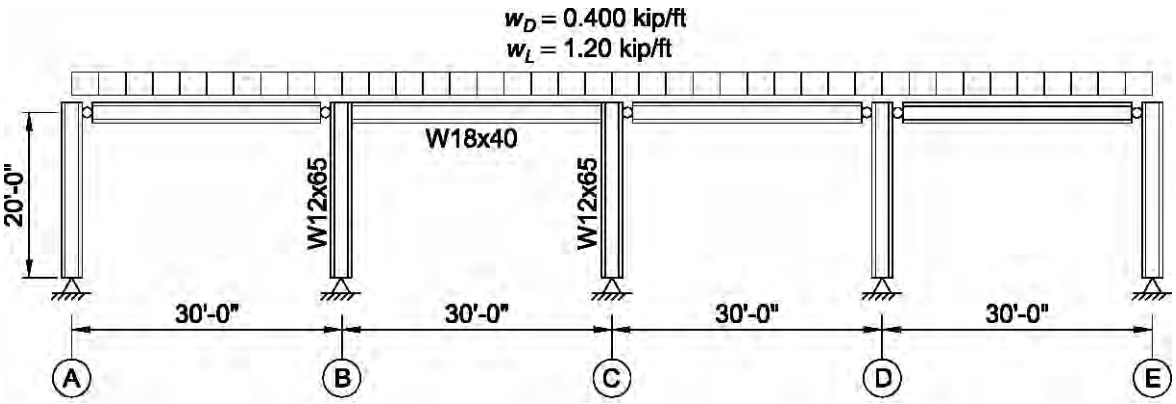


Fig. C.1C-1. Example C.1C moment frame elevation.

Solution:

From AISC *Manual* Table 1-1, the W12×65 has $A = 19.1$ in.^{2}

The beams from grid lines A to B and C to E and the columns at A, D and E are pinned at both ends and do not contribute to the lateral stability of the frame. There are no *P*-Δ effects to consider in these members and they may be designed using L_c=L.

The moment frame between grid lines B and C is the source of lateral stability and will be designed using the provisions of AISC *Specification* Appendix 7, Section 7.3. Although the columns at grid lines A, D and E do not contribute to lateral stability, the forces required to stabilize them must be considered in the moment-frame analysis. These members need not be included in the analysis model, except that the forces in the "leaning" columns must be included in the calculation of notional loads.

Check the limitations for the use of the first-order analysis method given in AISC *Specification* Appendix 7, Section 7.3.1:

(a) The structure supports gravity loads primarily through nominally vertical columns, walls or frames.
(b) The ratio of maximum second-order drift to the maximum first-order drift (both determined for LRFD load combinations or 1.6 times ASD load combinations, with stiffnesses not adjusted as specified in AISC *Specification* Section C2.3) in all stories will be assumed to be equal to or less than 1.5, subject to verification.
(c) The required axial compressive strength of all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure will be assumed to be no more than 50% of the cross-section strength, subject to verification.

Per AISC *Specification* Appendix 7, Section 7.3.2, the required strengths are determined from a first-order analysis using notional loads determined in the following, along with a B_1 multiplier to account for second-order effects, as determined from Appendix 8.

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Loads

From Chapter 2 of ASCE/SEI 7, the maximum gravity load combinations are:

LRFD	ASD
$w_u = 1.2D + 1.6L$ $= 1.2(0.400 \text{ kip/ft}) + 1.6(1.20 \text{ kip/ft})$ $= 2.40 \text{ kip/ft}$	$w_u = D + L$ $= 0.400 \text{ kip/ft} + 1.20 \text{ kip/ft}$ $= 1.60 \text{ kip/ft}$

Concentrated gravity loads to be considered on the columns at B and C contributed by adjacent beams are:

LRFD	ASD
$P_u = \frac{w_u l}{2}$ $= \frac{(2.40 \text{ kip/ft})(30.0 \text{ ft})}{2}$ $= 36.0 \text{ kips}$	$P_a = \frac{w_a l}{2}$ $= \frac{(1.60 \text{ kip/ft})(30.0 \text{ ft})}{2}$ $= 24.0 \text{ kips}$

Using AISC *Specification* Appendix 7, Section 7.3.2, frame out-of-plumbness is accounted for by the application of an additional lateral load.

From AISC *Specification* Appendix Equation A-7-2, the additional lateral load is determined as follows:

LRFD	ASD
$\alpha = 1.0$ $Y_i = (120 \text{ ft})(2.40 \text{ kip/ft})$ $= 288 \text{ kips}$ $\Delta = 0$ in. (no drift for this load combination) $L = (20.0 \text{ ft})(12 \text{ in./ft})$ $= 240 \text{ in.}$ $N_i = 2.1\alpha(\Delta/L)Y_i \geq 0.0042Y_i$ (*Spec.* Eq. A-7-2) $= 2.1(1.0)\left(\frac{0 \text{ in.}}{240 \text{ in.}}\right)(288 \text{ kips})$ $\geq 0.0042(288 \text{ kips})$ $= 0 \text{ kip} < 1.21 \text{ kips}$ Use $N_i = 1.21$ kips	$\alpha = 1.6$ $Y_i = (120 \text{ ft})(1.60 \text{ kip/ft})$ $= 192 \text{ kips}$ $\Delta = 0$ in. (no drift for this load combination) $L = (20.0 \text{ ft})(12 \text{ in./ft})$ $= 240 \text{ in.}$ $N_i = 2.1\alpha(\Delta/L)Y_i \geq 0.0042Y_i$ (*Spec.* Eq. A-7-2) $= 2.1(1.6)\left(\frac{0 \text{ in.}}{240 \text{ in.}}\right)(192 \text{ kips})$ $\geq 0.0042(192 \text{ kips})$ $= 0 \text{ kip} < 0.806 \text{ kip}$ Use $N_i = 0.806$ kip

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Summary of applied frame loads

The applied loads are shown in Figure C.1C-2.

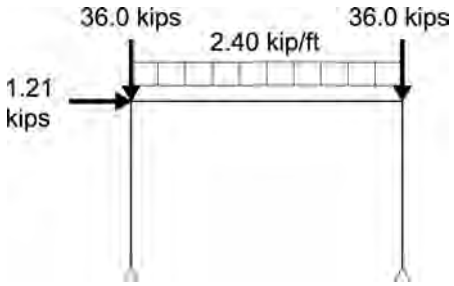
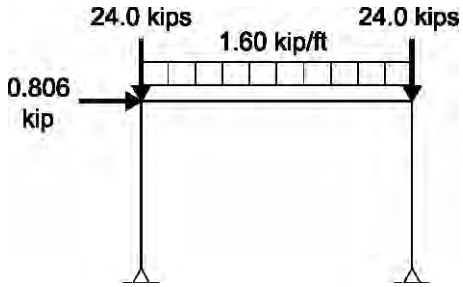
LRFD	ASD
	

Fig. C.1C-2. Applied loads on the analysis model.

Conduct the analysis using the full nominal stiffnesses, as indicated in AISC *Specification* Commentary Appendix 7, Section 7.3.

Using analysis software, the first-order results shown in Figure C.1C-3 are obtained:

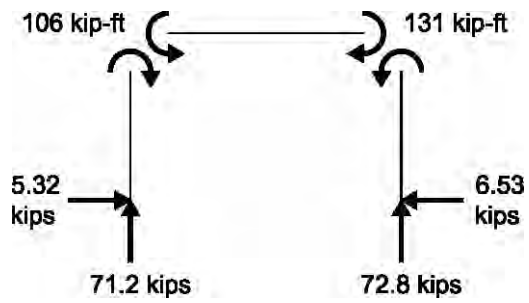
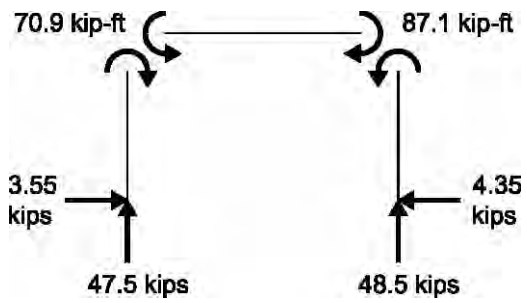
LRFD	ASD
$\Delta_{1st} = 0.304$ in. 	$\Delta_{1st} = 0.203$ in. 

Fig. C.1C-3. Results of first-order analysis.

Check the assumption that the ratio of the second-order drift to the first-order drift does not exceed 1.5. B_2 can be used to check this limit. Calculate B_2 per Appendix 8, Section 8.2.2 using the results of the first-order analysis.

LRFD	ASD
$P_{mf} = 2(36.0 \text{ kips}) + (30.0 \text{ ft})(2.40 \text{ kip/ft})$ $= 144$ kips $P_{story} = 144 \text{ kips} + 4(36.0 \text{ kips})$ $= 288$ kips	$P_{mf} = 2(24.0 \text{ kips}) + (30.0 \text{ ft})(1.60 \text{ kip/ft})$ $= 96.0$ kips $P_{story} = 96.0 \text{ kips} + 4(24.0 \text{ kips})$ $= 192$ kips

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LRFD	ASD
$R_M = 1 - 0.15\left(P_{mf}/P_{story}\right)$ (*Spec.* Eq. A-8-8) $= 1 - 0.15(144\text{ kips}/288\text{ kips})$ $= 0.925$	$R_M = 1 - 0.15\left(P_{mf}/P_{story}\right)$ (*Spec.* Eq. A-8-8) $= 1 - 0.15(96.0\text{ kips}/192\text{ kips})$ $= 0.925$
$\Delta_H = 0.304$ in.	$\Delta_H = 0.203$ in.
$H = 6.53\text{ kips} - 5.32\text{ kips}$ $= 1.21$ kips	$H = 4.35\text{ kips} - 3.55\text{ kips}$ $= 0.800$ kip
$L = (20\text{ ft})(12\text{ in./ft})$ $= 240$ in.	$L = (20\text{ ft})(12\text{ in./ft})$ $= 240$ in.
$P_{e\,story} = R_M \dfrac{HL}{\Delta_H}$ (*Spec.* Eq. A-8-7) $= 0.925\dfrac{(1.21\text{ kips})(240\text{ in.})}{0.304\text{ in.}}$ $= 884$ kips	$P_{e\,story} = R_M \dfrac{HL}{\Delta_H}$ (*Spec.* Eq. A-8-7) $= 0.925\dfrac{(0.800\text{ kip})(240\text{ in.})}{0.203\text{ in.}}$ $= 875$ kips
$\alpha = 1.0$	$\alpha = 1.6$
$B_2 = \dfrac{1}{1 - \dfrac{\alpha P_{story}}{P_{e\,story}}} \geq 1$ (*Spec.* Eq. A-8-6) $= \dfrac{1}{1 - \dfrac{1.0(288\text{ kips})}{884\text{ kips}}} \geq 1$ $= 1.48 > 1$	$B_2 = \dfrac{1}{1 - \dfrac{\alpha P_{story}}{P_{e\,story}}} \geq 1$ (*Spec.* Eq. A-8-6) $= \dfrac{1}{1 - \dfrac{1.6(192\text{ kips})}{875\text{ kips}}} \geq 1$ $= 1.54 > 1$

When a structure with a live-to-dead load ratio of 3 is analyzed by a first-order analysis the required strength for LRFD will always be 1.5 times the required strength for ASD. However, when a second-order analysis is used this ratio is not maintained. This is due to the use of the amplification factor, α, which is set equal to 1.6 for ASD, in order to capture the worst case second-order effects for any live-to-dead load ratio. Thus, in this example the limitation for applying the first-order analysis method, that the ratio of the maximum second-order drift to maximum first-order drift is not greater than 1.5, is verified for LRFD but is not verified for ASD. Therefore, for this example the first-order method is invalid for ASD and will proceed with LRFD only.

Check the assumption that $\alpha P_r \leq 0.5P_{ns}$ and, therefore, the first-order analysis method is permitted.

Because the W12×65 column does not contain elements that are slender for compression,

$$P_{ns} = F_y A_g$$

$$0.5P_{ns} = 0.5F_y A_g$$
$$= 0.5(50\text{ ksi})\left(19.1\text{ in.}^2\right)$$
$$= 478\text{ kips}$$

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$$\alpha P_r = 1.0(72.8 \text{ kips})$$
$$= 72.8 \text{ kips} < 478 \text{ kips} \quad \textbf{o.k.} \text{ (LRFD only)}$$

The assumption that the first-order analysis method can be used is verified for LRFD.

Although the second-order sway multiplier is 1.48, the change in bending moment is small because the only sway moments are those produced by the small notional loads. For load combinations with significant gravity and lateral loadings, the increase in bending moments is larger.

The column strengths can be verified after using the B_1 amplification given in Appendix 8, Section 8.2.1 to account for second-order effects (calculations not shown here). In the direction of sway, the effective length factor is taken equal to 1.00, and the column effective lengths are as follows:

$L_{cx} = 20.0$ ft
$L_{cy} = 20.0$ ft

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Chapter D
Design of Members for Tension

D1. SLENDERNESS LIMITATIONS

AISC *Specification* Section D1 does not establish a slenderness limit for tension members, but recommends limiting L/r to a maximum of 300. This is not an absolute requirement. Rods and hangers are specifically excluded from this recommendation.

D2. TENSILE STRENGTH

Both tensile yielding strength and tensile rupture strength must be considered for the design of tension members. It is not unusual for tensile rupture strength to govern the design of a tension member, particularly for small members with holes or heavier sections with multiple rows of holes.

For preliminary design, tables are provided in Part 5 of the AISC *Manual* for W-shapes, L-shapes, WT-shapes, rectangular HSS, square HSS, round HSS, Pipe, and 2L-shapes. The calculations in these tables for available tensile rupture strength assume an effective area, A_e, of $0.75A_g$. The gross area, A_g, is the total cross-sectional area of the member. If the actual effective area is greater than $0.75A_g$, the tabulated values will be conservative and calculations can be performed to obtain higher available strengths. If the actual effective area is less than $0.75A_g$, the tabulated values will be unconservative and calculations are necessary to determine the available strength.

D3. EFFECTIVE NET AREA

In computing net area, A_n, AISC *Specification* Section B4.3b requires that an extra 1/16 in. be added to the bolt hole diameter. A computation of the effective area for a chain of holes is presented in Example D.9.

Unless all elements of the cross section are connected, $A_e = A_n U$, where U is a reduction factor to account for shear lag. The appropriate values of U can be obtained from AISC *Specification* Table D3.1.

D4. BUILT-UP MEMBERS

The limitations for connections of built-up members are discussed in Section D4 of the AISC *Specification*.

D5. PIN-CONNECTED MEMBERS

An example of a pin-connected member is given in Example D.7.

D6. EYEBARS

An example of an eyebar is given in Example D.8. The strength of an eyebar meeting the dimensional requirements of AISC *Specification* Section D6 is governed by tensile yielding of the body.

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EXAMPLE D.1 W-SHAPE TENSION MEMBER

Given:

Select an ASTM A992 W-shape with 8 in. nominal depth to carry a dead load of 30 kips and a live load of 90 kips in tension. The member is 25.0 ft long. Verify the member strength by both LRFD and ASD with the bolted end connection as shown in Figure D.1-1. Verify that the member satisfies the recommended slenderness limit. Assume that connection limit states do not govern.

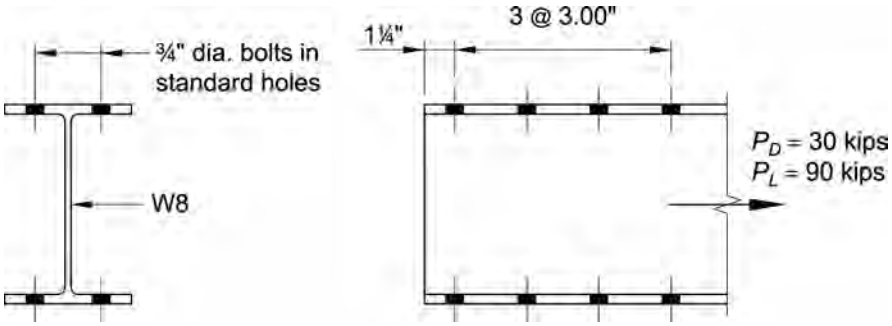


Fig D.1-1. Connection geometry for Example D.1.

Solution:

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(30 \text{ kips}) + 1.6(90 \text{ kips})$ $= 180 \text{ kips}$	$P_a = 30 \text{ kips} + 90 \text{ kips}$ $= 120 \text{ kips}$

From AISC *Manual* Table 5-1, try a W8×21.

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W8×21
A_g = 6.16 in.^{2}
b_f = 5.27 in.
t_f = 0.400 in.
d = 8.28 in.
r_y = 1.26 in.

The WT-shape corresponding to a W8×21 is a WT4×10.5. From AISC *Manual* Table 1-8, the geometric properties are as follows:

WT4×10.5
$\bar{y}$ = 0.831 in.

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Tensile Yielding

From AISC *Manual* Table 5-1, the available tensile yielding strength of a W8×21 is:

LRFD	ASD
$\phi_t P_n = 277 \text{ kips} > 180 \text{ kips}$ **o.k.**	$\frac{P_n}{\Omega_t} = 184 \text{ kips} > 120 \text{ kips}$ **o.k.**

Tensile Rupture

Verify the table assumption that $A_e / A_g \geq 0.75$ for this connection.

From the description of the element in AISC *Specification* Table D3.1, Case 7, calculate the shear lag factor, *U*, as the larger of the values from AISC *Specification* Section D3, Table D3.1 Case 2 and Case 7.

From AISC *Specification* Section D3, for open cross sections, *U* need not be less than the ratio of the gross area of the connected element(s) to the member gross area.

$$U = \frac{2 b_f t_f}{A_g}$$
$$= \frac{2(5.27 \text{ in.})(0.400 \text{ in.})}{6.16 \text{ in.}^2}$$
$$= 0.684$$

Case 2: Determine *U* based on two WT-shapes per AISC *Specification* Commentary Figure C-D3.1, with $\bar{x} = \bar{y} = 0.831$ in. and where *l* is the length of connection.

$$U = 1 - \frac{\bar{x}}{l}$$
$$= 1 - \frac{0.831 \text{ in.}}{9.00 \text{ in.}}$$
$$= 0.908$$

Case 7:

$$b_f = 5.27 \text{ in.}$$
$$\frac{2}{3} d = \frac{2}{3}(8.28 \text{ in.})$$
$$= 5.52 \text{ in.}$$

Because the flange is connected with three or more fasteners per line in the direction of loading and $b_f < \frac{2}{3} d$:

$$U = 0.85$$

Therefore, use the larger $U = 0.908$.

Calculate A_n using AISC *Specification* Section B4.3b.

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$$A_n = A_g - 4\left(d_h + \tfrac{1}{16}\text{ in.}\right)t_f$$
$$= 6.16\text{ in.}^2 - 4\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\left(0.400\text{ in.}\right)$$
$$= 4.76\text{ in.}^2$$

Calculate A_e using AISC *Specification* Section D3.

$$A_e = A_n U \qquad \text{(Spec. Eq. D3-1)}$$
$$= \left(4.76\text{ in.}^2\right)\left(0.908\right)$$
$$= 4.32\text{ in.}^2$$

$$\frac{A_e}{A_g} = \frac{4.32\text{ in.}^2}{6.16\text{ in.}^2}$$
$$= 0.701 < 0.75$$

Because $A_e/A_g < 0.75$, the tensile rupture strength from AISC *Manual* Table 5-1 is not valid. The available tensile rupture strength is determined using AISC *Specification* Section D2 as follows:

$$P_n = F_u A_e \qquad \text{(Spec. Eq. D2-2)}$$
$$= \left(65\text{ ksi}\right)\left(4.32\text{ in.}^2\right)$$
$$= 281\text{ kips}$$

From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75\left(281\text{ kips}\right)$ $= 211\text{ kips} > 180\text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $\frac{P_n}{\Omega_t} = \frac{281\text{ kips}}{2.00}$ $= 141\text{ kips} > 120\text{ kips}$ **o.k.**

Note that the W8×21 available tensile strength is governed by the tensile rupture limit state at the end connection versus the tensile yielding limit state.

See Chapter J for illustrations of connection limit state checks.

Check Recommended Slenderness Limit

$$\frac{L}{r} = \frac{\left(25.0\text{ ft}\right)\left(12\text{ in./ft}\right)}{1.26\text{ in.}}$$
$$= 238 < 300 \text{ from AISC } \textit{Specification} \text{ Section D1} \quad \textbf{o.k.}$$

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EXAMPLE D.2 SINGLE-ANGLE TENSION MEMBER

Given:

Verify the tensile strength of an ASTM A36 L4×4×½ with one line of four ¾-in.-diameter bolts in standard holes, as shown in Figure D.2-1. The member carries a dead load of 20 kips and a live load of 60 kips in tension. Additionally, calculate at what length this tension member would cease to satisfy the recommended slenderness limit. Assume that connection limit states do not govern.

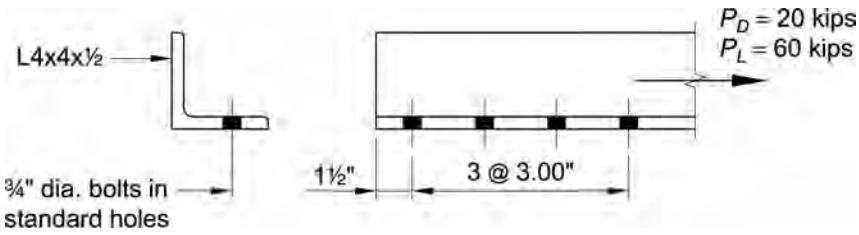


Fig. D.2-1. Connection geometry for Example D.2.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-7, the geometric properties are as follows:

L4×4×½
A_g = 3.75 in.^{2}
r_z = 0.776 in.
$\overline{x}$ = 1.18 in.

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(20 \text{ kips}) + 1.6(60 \text{ kips})$ $= 120 \text{ kips}$	$P_a = 20 \text{ kips} + 60 \text{ kips}$ $= 80.0 \text{ kips}$

Tensile Yielding

$$P_n = F_y A_g \qquad \text{(Spec. Eq. D2-1)}$$
$$= (36 \text{ ksi})(3.75 \text{ in.}^2)$$
$$= 135 \text{ kips}$$

From AISC *Specification* Section D2, the available tensile yielding strength is:

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LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(135 \text{ kips})$ $= 122 \text{ kips} > 120 \text{ kips}$ **o.k.**	$\Omega_t = 1.67$ $\frac{P_n}{\Omega_t} = \frac{135 \text{ kips}}{1.67}$ $= 80.8 \text{ kips} > 80.0 \text{ kips}$ **o.k.**

Tensile Rupture

From the description of the element in AISC *Specification* Table D3.1 Case 8, calculate the shear lag factor, *U,* as the larger of the values from AISC *Specification* Section D3, Table D3.1 Case 2 and Case 8.

From AISC *Specification* Section D3, for open cross sections, *U* need not be less than the ratio of the gross area of the connected element(s) to the member gross area. Half of the member is connected, therefore, the minimum value of *U* is:

$$U = 0.500$$

Case 2, where *l* is the length of connection and $\bar{y} = \bar{x}$:

$$\begin{aligned} U &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{1.18 \text{ in.}}{9.00 \text{ in.}} \\ &= 0.869 \end{aligned}$$

Case 8, with four or more fasteners per line in the direction of loading:

$$U = 0.80$$

Therefore, use the larger $U = 0.869$.

Calculate A_n using AISC *Specification* Section B4.3b.

$$\begin{aligned} A_n &= A_g - (d_h + 1/16 \text{ in.})t \\ &= 3.75 \text{ in.} - (13/16 \text{ in.} + 1/16 \text{ in.})(1/2 \text{ in.}) \\ &= 3.31 \text{ in.}^2 \end{aligned}$$

Calculate A_e using AISC *Specification* Section D3.

$$\begin{aligned} A_e &= A_n U \qquad \text{(Spec. Eq. D3-1)} \\ &= (3.31 \text{ in.}^2)(0.869) \\ &= 2.88 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} P_n &= F_u A_e \qquad \text{(Spec. Eq. D2-2)} \\ &= (58 \text{ ksi})(2.88 \text{ in.}^2) \\ &= 167 \text{ kips} \end{aligned}$$

From AISC *Specification* Section D2, the available tensile rupture strength is:

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LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(167 \text{ kips})$ $= 125 \text{ kips} > 120 \text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $\frac{P_n}{\Omega_t} = \frac{167 \text{ kips}}{2.00}$ $= 83.5 \text{ kips} > 80.0 \text{ kips}$ **o.k.**

The L4×4×½ available tensile strength is governed by the tensile yielding limit state.

LRFD	ASD
$\phi_t P_n = 122 \text{ kips} > 120 \text{ kips}$ **o.k.**	$\frac{P_n}{\Omega_t} = 80.8 \text{ kips} > 80.0 \text{ kips}$ **o.k.**

Recommended L_{max}

Using AISC *Specification* Section D1:

$$\begin{aligned} L_{max} &= 300 r_z \\ &= 300\left(\frac{0.776 \text{ in.}}{12 \text{ in./ft}}\right) \\ &= 19.4 \text{ ft} \end{aligned}$$

Note: The *L/r* limit is a recommendation, not a requirement.

See Chapter J for illustrations of connection limit state checks.

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EXAMPLE D.3 WT-SHAPE TENSION MEMBER

Given:

An ASTM A992 WT6×20 member has a length of 30 ft and carries a dead load of 40 kips and a live load of 120 kips in tension. As shown in Figure D3-1, the end connection is fillet welded on each side for 16 in. Verify the member tensile strength by both LRFD and ASD. Assume that the gusset plate and the weld are satisfactory.

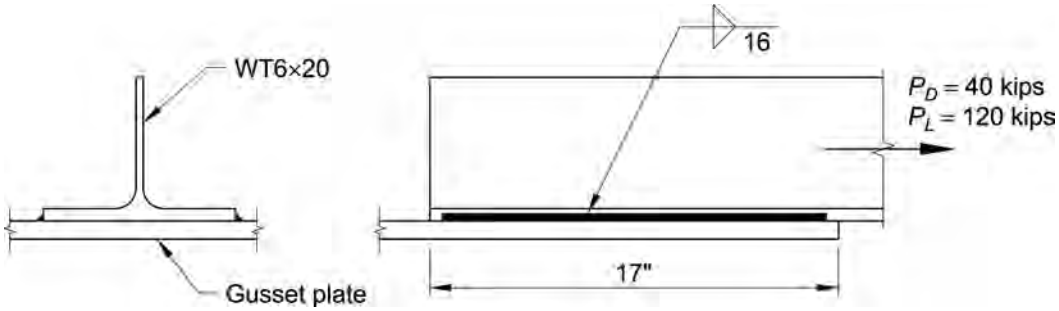


Fig. D.3-1. Connection geometry for Example D.3.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-8, the geometric properties are as follows:

WT6×20
$A_g = 5.84$ in.^{2}
$b_f = 8.01$ in.
$t_f = 0.515$ in.
$r_x = 1.57$ in.
$\bar{y} = 1.09$ in.

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(40 \text{ kips}) + 1.6(120 \text{ kips})$ $= 240$ kips	$P_a = 40 \text{ kips} + 120 \text{ kips}$ $= 160$ kips

Tensile Yielding

Check tensile yielding limit state using AISC *Manual* Table 5-3.

LRFD	ASD
$\phi_t P_n = 263 \text{ kips} > 240 \text{ kips}$ **o.k.**	$\frac{P_n}{\Omega_t} = 175 \text{ kips} > 160 \text{ kips}$ **o.k.**

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Tensile Rupture

Check tensile rupture limit state using AISC *Manual* Table 5-3.

LRFD	ASD
$\phi_t P_n = 214 \text{ kips} < 240 \text{ kips}$ **n.g.**	$\frac{P_n}{\Omega_t} = 142 \text{ kips} < 160 \text{ kips}$ **n.g.**

The tabulated available rupture strengths don't work and may be conservative for this case; therefore, calculate the exact solution.

Calculate U as the larger of the values from AISC *Specification* Section D3 and Table D3.1 Case 4.

From AISC *Specification* Section D3, for open cross sections, U need not be less than the ratio of the gross area of the connected element(s) to the member gross area.

$$U = \frac{b_f t_f}{A_g}$$
$$= \frac{(8.01 \text{ in.})(0.515 \text{ in.})}{5.84 \text{ in.}^2}$$
$$= 0.706$$

Case 4, where l is the length of the connection and $\bar{x} = \bar{y}$:

$$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right)$$
$$= \left[\frac{3(16.0 \text{ in.})^2}{3(16.0 \text{ in.})^2 + (8.01 \text{ in.})^2}\right]\left(1 - \frac{1.09 \text{ in.}}{16.0 \text{ in.}}\right)$$
$$= 0.860$$

Therefore, use $U = 0.860$.

Calculate A_n using AISC *Specification* Section B4.3. Because there are no reductions due to bolt holes or notches:

$$A_n = A_g$$
$$= 5.84 \text{ in.}^2$$

Calculate A_e using AISC *Specification* Section D3.

$$A_e = A_n U \qquad \text{(Spec. Eq. D3-1)}$$
$$= (5.84 \text{ in.}^2)(0.860)$$
$$= 5.02 \text{ in.}^2$$

Calculate P_n.

$$P_n = F_u A_e \qquad \text{(Spec. Eq. D2-2)}$$
$$= (65 \text{ ksi})(5.02 \text{ in.}^2)$$
$$= 326 \text{ kips}$$

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From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(326 \text{ kips})$ $= 245 \text{ kips} > 240 \text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $\frac{P_n}{\Omega_t} = \frac{326 \text{ kips}}{2.00}$ $= 163 \text{ kips} > 160 \text{ kips}$ **o.k.**

Alternately, the available tensile rupture strengths can be determined by modifying the tabulated values. The available tensile rupture strengths published in the tension member selection tables are based on the assumption that $A_e = 0.75A_g$. The actual available strengths can be determined by adjusting the values from AISC *Manual* Table 5-3 as follows:

LRFD	ASD
$\phi_t P_n = (214 \text{ kips})\left(\frac{A_e}{0.75A_g}\right)$ $= (214 \text{ kips})\left[\frac{5.02 \text{ in.}^2}{0.75(5.84 \text{ in.}^2)}\right]$ $= 245 \text{ kips} > 240 \text{ kips}$ **o.k.**	$\frac{P_n}{\Omega_t} = (142 \text{ kips})\left(\frac{A_e}{0.75A_g}\right)$ $= (142 \text{ kips})\left[\frac{5.02 \text{ in.}^2}{0.75(5.84 \text{ in.}^2)}\right]$ $= 163 \text{ kips} > 160 \text{ kips}$ **o.k.**

Recommended Slenderness Limit

$$\frac{L}{r_x} = \frac{(30.0 \text{ ft})(12 \text{ in./ft})}{1.57 \text{ in.}}$$
$$= 229 < 300 \text{ from AISC } \textit{Specification} \text{ Section D1} \quad \textbf{o.k.}$$

Note: The L/r_x limit is a recommendation, not a requirement.

See Chapter J for illustrations of connection limit state checks.

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EXAMPLE D.4 RECTANGULAR HSS TENSION MEMBER

Given:

Verify the tensile strength of an ASTM A500 Grade C HSS6×4×⅜ with a length of 30 ft. The member is carrying a dead load of 40 kips and a live load of 110 kips in tension. As shown in Figure D.4-1, the end connection is a fillet welded ½-in.-thick single concentric gusset plate with a weld length of 16 in. Assume that the gusset plate and weld are satisfactory.

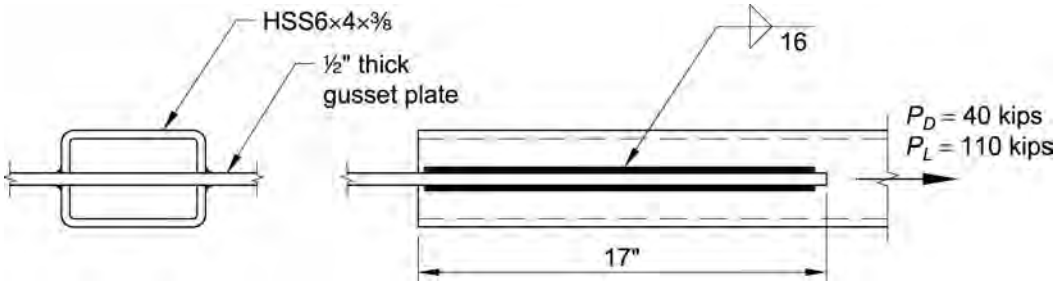


Fig. D.4-1. Connection geometry for Example D.4.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C, rectangular HSS
F_y = 50 ksi
F_u = 62 ksi

From AISC *Manual* Table 1-11, the geometric properties are as follows:

HSS6×4×⅜
A_g = 6.18 in.^{2}
r_y = 1.55 in.
t = 0.349 in.

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(40 \text{ kips}) + 1.6(110 \text{ kips})$ $= 224 \text{ kips}$	$P_a = 40 \text{ kips} + 110 \text{ kips}$ $= 150 \text{ kips}$

Tensile Yielding

Check tensile yielding limit state using AISC *Manual* Table 5-4.

LRFD	ASD
$\phi_t P_n = 278 \text{ kips} > 224 \text{ kips}$ **o.k.**	$\frac{P_n}{\Omega_t} = 185 \text{ kips} > 150 \text{ kips}$ **o.k.**

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Tensile Rupture

Check tensile rupture limit state using AISC *Manual* Table 5-4.

LRFD	ASD
$\phi_t P_n = 216 \text{ kips} < 224 \text{ kips}$ **n.g.**	$\frac{P_n}{\Omega_t} = 144 \text{ kips} < 150 \text{ kips}$ **n.g.**

The tabulated available rupture strengths may be conservative in this case; therefore, calculate the exact solution.

Calculate U from AISC *Specification* Section D3 and Table D3.1 Case 6.

$$\begin{aligned}\bar{x} &= \frac{B^2 + 2BH}{4(B+H)} \\ &= \frac{(4.00 \text{ in.})^2 + 2(4.00 \text{ in.})(6.00 \text{ in.})}{4(4.00 \text{ in.} + 6.00 \text{ in.})} \\ &= 1.60 \text{ in.}\end{aligned}$$

$$\begin{aligned}U &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{1.60 \text{ in.}}{16.0 \text{ in.}} \\ &= 0.900\end{aligned}$$

Allowing for a 1/16-in. gap in fit-up between the HSS and the gusset plate:

$$\begin{aligned}A_n &= A_g - 2(t_p + 1/16 \text{ in.})t \\ &= 6.18 \text{ in.}^2 - 2(1/2 \text{ in.} + 1/16 \text{ in.})(0.349 \text{ in.}) \\ &= 5.79 \text{ in.}^2\end{aligned}$$

Calculate A_e using AISC *Specification* Section D3.

$$\begin{aligned}A_e &= A_n U \qquad \text{(Spec. Eq. D3-1)} \\ &= (5.79 \text{ in.}^2)(0.900) \\ &= 5.21 \text{ in.}^2\end{aligned}$$

Calculate P_n.

$$\begin{aligned}P_n &= F_u A_e \qquad \text{(Spec. Eq. D2-2)} \\ &= (62 \text{ ksi})(5.21 \text{ in.}^2) \\ &= 323 \text{ kips}\end{aligned}$$

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From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(323 \text{ kips})$ $= 242 \text{ kips} > 224 \text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $\frac{P_n}{\Omega_t} = \frac{323 \text{ kips}}{2.00}$ $= 162 \text{ kips} > 150 \text{ kips}$ **o.k.**

The HSS available tensile strength is governed by the tensile rupture limit state.

Recommended Slenderness Limit

$$\frac{L}{r} = \frac{(30.0 \text{ ft})(12 \text{ in./ft})}{1.55 \text{ in.}}$$
$$= 232 < 300 \text{ from AISC } \textit{Specification} \text{ Section D1} \quad \textbf{o.k.}$$

Note: The L/r limit is a recommendation, not a requirement.

See Chapter J for illustrations of connection limit state checks.

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EXAMPLE D.5 ROUND HSS TENSION MEMBER

Given:

Verify the tensile strength of an ASTM A500 Grade C HSS6.000×0.500 with a length of 30 ft. The member carries a dead load of 40 kips and a live load of 120 kips in tension. As shown in Figure D.5-1, the end connection is a fillet welded ½-in.-thick single concentric gusset plate with a weld length of 16 in. Assume that the gusset plate and weld are satisfactory.

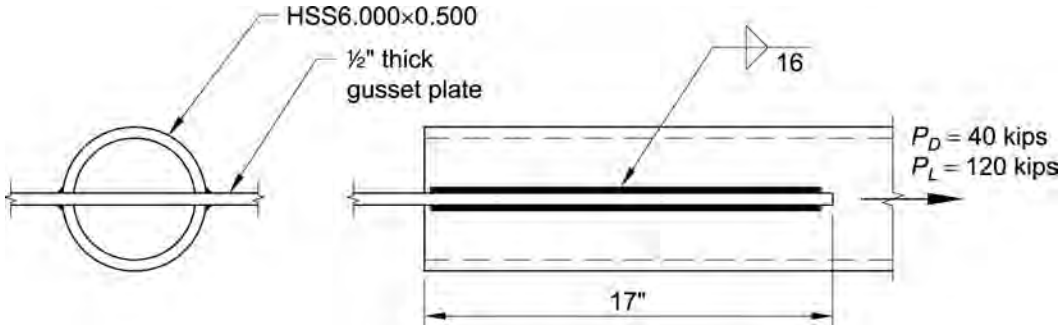


Fig. D.5-1. Connection geometry for Example D.5.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C, round HSS
F_y = 46 ksi
F_u = 62 ksi

From AISC *Manual* Table 1-13, the geometric properties are as follows:

HSS6.000×0.500
A_g = 8.09 in.^{2}
r = 1.96 in.
t = 0.465 in.

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(40 \text{ kips}) + 1.6(120 \text{ kips})$ $= 240$ kips	$P_a = 40 \text{ kips} + 120 \text{ kips}$ $= 160$ kips

Tensile Yielding

Check tensile yielding limit state using AISC *Manual* Table 5-6.

LRFD	ASD
$\phi_t P_n = 335 \text{ kips} > 240 \text{ kips}$ **o.k.**	$\frac{P_n}{\Omega_t} = 223 \text{ kips} > 160 \text{ kips}$ **o.k.**

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Tensile Rupture

Check tensile rupture limit state using AISC *Manual* Table 5-6.

LRFD	ASD
$\phi_t P_n = 282 \text{ kips} > 240 \text{ kips}$ **o.k.**	$\frac{P_n}{\Omega_t} = 188 \text{ kips} > 160 \text{ kips}$ **o.k.**

Check that $A_e/A_g \geq 0.75$ as assumed in table.

Determine U from AISC *Specification* Table D3.1 Case 5.

$$l = 16.0 \text{ in.}$$
$$D = 6.00 \text{ in.}$$
$$\frac{l}{D} = \frac{16.0 \text{ in.}}{6.00 \text{ in.}}$$
$$= 2.67 > 1.3, \text{ therefore } U = 1.0$$

Allowing for a 1/16-in. gap in fit-up between the HSS and the gusset plate,

$$A_n = A_g - 2\left(t_p + 1/16 \text{ in.}\right)t$$
$$= 8.09 \text{ in.}^2 - 2\left(1/2 \text{ in.} + 1/16 \text{ in.}\right)\left(0.465 \text{ in.}\right)$$
$$= 7.57 \text{ in.}^2$$

Calculate A_e using AISC *Specification* Section D3.

$$A_e = A_n U \qquad \text{(Spec. Eq. D3-1)}$$
$$= \left(7.57 \text{ in.}^2\right)\left(1.0\right)$$
$$= 7.57 \text{ in.}^2$$

$$\frac{A_e}{A_g} = \frac{7.57 \text{ in.}^2}{8.09 \text{ in.}^2}$$
$$= 0.936 > 0.75 \quad \textbf{o.k.}$$

Because AISC *Manual* Table 5-6 provides an overly conservative estimate of the available tensile rupture strength for this example, calculate P_n using AISC *Specification* Section D2.

$$P_n = F_u A_e \qquad \text{(Spec. Eq. D2-2)}$$
$$= \left(62 \text{ ksi}\right)\left(7.57 \text{ in.}^2\right)$$
$$= 469 \text{ kips}$$

From AISC *Specification* Section D2, the available tensile rupture strength is:

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LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(469 \text{ kips})$ $= 352 \text{ kips} > 240 \text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $\frac{P_n}{\Omega_t} = \frac{469 \text{ kips}}{2.00}$ $= 235 \text{ kips} > 160 \text{ kips}$ **o.k.**

The HSS available strength is governed by the tensile yielding limit state.

Recommended Slenderness Limit

$$\frac{L}{r} = \frac{(30.0 \text{ ft})(12 \text{ in./ft})}{1.96 \text{ in.}}$$
$$= 184 < 300 \text{ from AISC } \textit{Specification} \text{ Section D1} \quad \textbf{o.k.}$$

Note: The L/r limit is a recommendation, not a requirement.

See Chapter J for illustrations of connection limit state checks.

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EXAMPLE D.6 DOUBLE-ANGLE TENSION MEMBER

Given:

An ASTM A36 2L4×4×½ (⅜-in. separation) has one line of eight ¾-in.-diameter bolts in standard holes and is 25 ft in length as shown in Figure D.6-1. The double angle is carrying a dead load of 40 kips and a live load of 120 kips in tension. Verify the member tensile strength. Assume that the gusset plate and bolts are satisfactory.

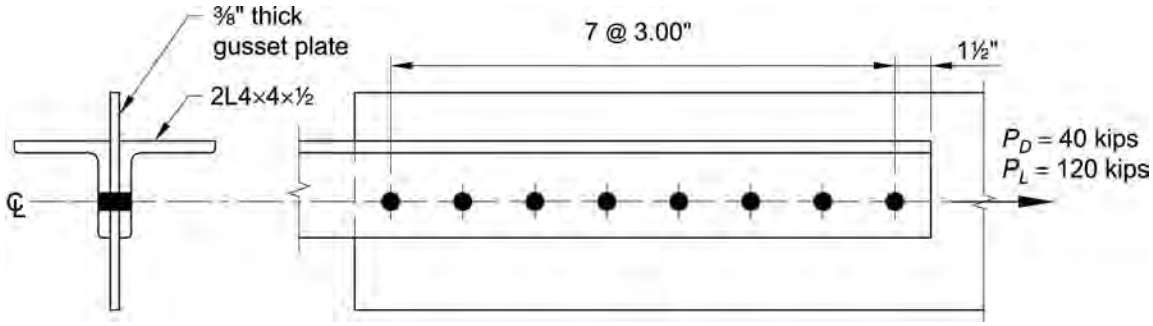


Fig. D.6-1. Connection geometry for Example D.6.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-7 and 1-15, the geometric properties are as follows:

L4×4×½
$\bar{x}$ = 1.18 in.

2L4×4×½ (s = ⅜ in.)
A_g = 7.50 in.^{2}
r_y = 1.83 in.
r_x = 1.21 in.

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(40 \text{ kips}) + 1.6(120 \text{ kips})$ $= 240$ kips	$P_a = 40 \text{ kips} + 120 \text{ kips}$ $= 160$ kips

Tensile Yielding

Check tensile yielding limit state using AISC *Manual* Table 5-8.

LRFD	ASD
$\phi_t P_n = 243 \text{ kips} > 240 \text{ kips}$ **o.k.**	$\frac{P_n}{\Omega_t} = 162 \text{ kips} > 160 \text{ kips}$ **o.k.**

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Tensile Rupture

Determine the available tensile rupture strength using AISC *Specification* Section D2. Calculate U as the larger of the values from AISC *Specification* Section D3, Table D3.1 Case 2 and Case 8.

From AISC *Specification* Section D3, for open cross sections, U need not be less than the ratio of the gross area of the connected element(s) to the member gross area. Half of the member is connected, therefore, the minimum U value is:

$$U = 0.500$$

From Case 2, where l is the length of connection:

$$\begin{aligned} U &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{1.18 \text{ in.}}{21.0 \text{ in.}} \\ &= 0.944 \end{aligned}$$

From Case 8, with four or more fasteners per line in the direction of loading:

$$U = 0.80$$

Therefore, use $U = 0.944$.

Calculate A_n using AISC *Specification* Section B4.3.

$$\begin{aligned} A_n &= A_g - 2\left(d_h + \tfrac{1}{16} \text{ in.}\right)t \\ &= 7.50 \text{ in.}^2 - 2\left(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\left(\tfrac{1}{2} \text{ in.}\right) \\ &= 6.63 \text{ in.}^2 \end{aligned}$$

Calculate A_e using AISC *Specification* Section D3.

$$\begin{aligned} A_e &= A_n U \qquad \text{(Spec. Eq. D3-1)} \\ &= \left(6.63 \text{ in.}^2\right)\left(0.944\right) \\ &= 6.26 \text{ in.}^2 \end{aligned}$$

Calculate P_n.

$$\begin{aligned} P_n &= F_u A_e \qquad \text{(Spec. Eq. D2-2)} \\ &= \left(58 \text{ ksi}\right)\left(6.26 \text{ in.}^2\right) \\ &= 363 \text{ kips} \end{aligned}$$

From AISC *Specification* Section D2, the available tensile rupture strength is:

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LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(363 \text{ kips})$ $= 272 \text{ kips}$	$\Omega_t = 2.00$ $\frac{P_n}{\Omega_t} = \frac{363 \text{ kips}}{2.00}$ $= 182 \text{ kips}$

Note that AISC *Manual* Table 5-8 could also be conservatively used since $A_e \geq 0.75A_g$.

The double-angle available tensile strength is governed by the tensile yielding limit state.

LRFD	ASD
243 kips > 240 kips **o.k.**	162 kips > 160 kips **o.k.**

Recommended Slenderness Limit

$$\frac{L}{r_x} = \frac{(25.0 \text{ ft})(12 \text{ in./ft})}{1.21 \text{ in.}}$$
$$= 248 < 300 \text{ from AISC } \textit{Specification} \text{ Section D1} \quad \textbf{o.k.}$$

Note: From AISC *Specification* Section D4, the longitudinal spacing of connectors between components of built-up members should preferably limit the slenderness ratio in any component between the connectors to a maximum of 300.

See Chapter J for illustrations of connection limit state checks.

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