17391761.085714 10454810808 37103640825 9513711.0416667 16518330248 7301072339 11964423.888889 585807276 25568030 47344563814 49302391.583333 6663163.90625 30275249.979167 55355218760 7434145675 72205696052 75685978852 13144968492 21301321.780488 10516517.7125

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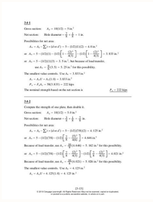

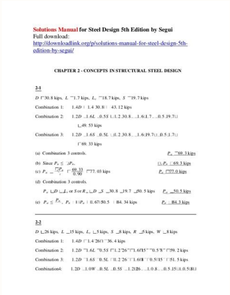

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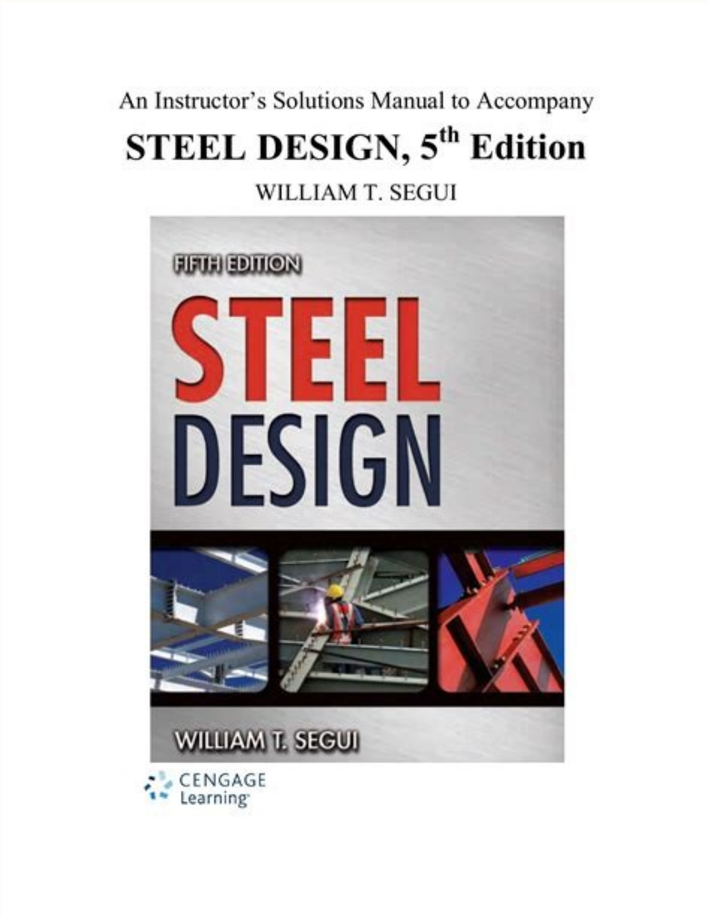
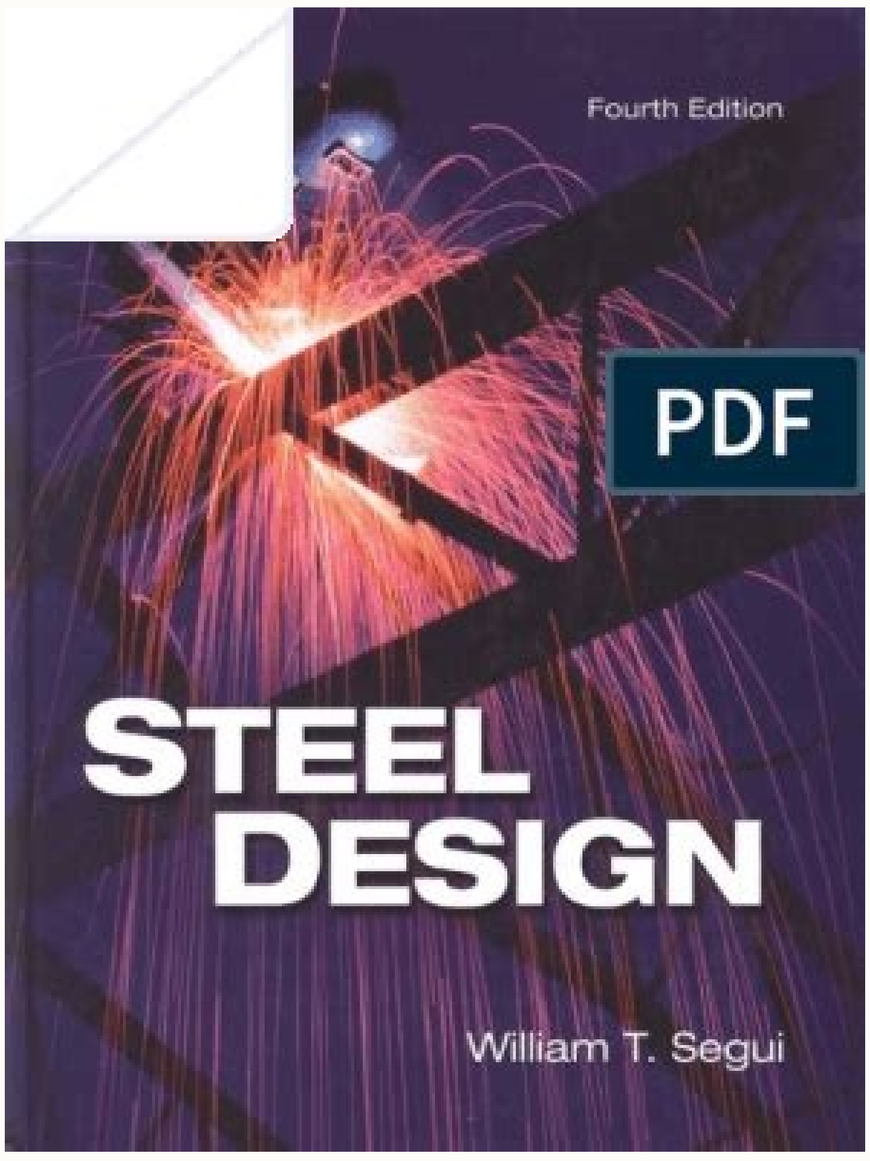
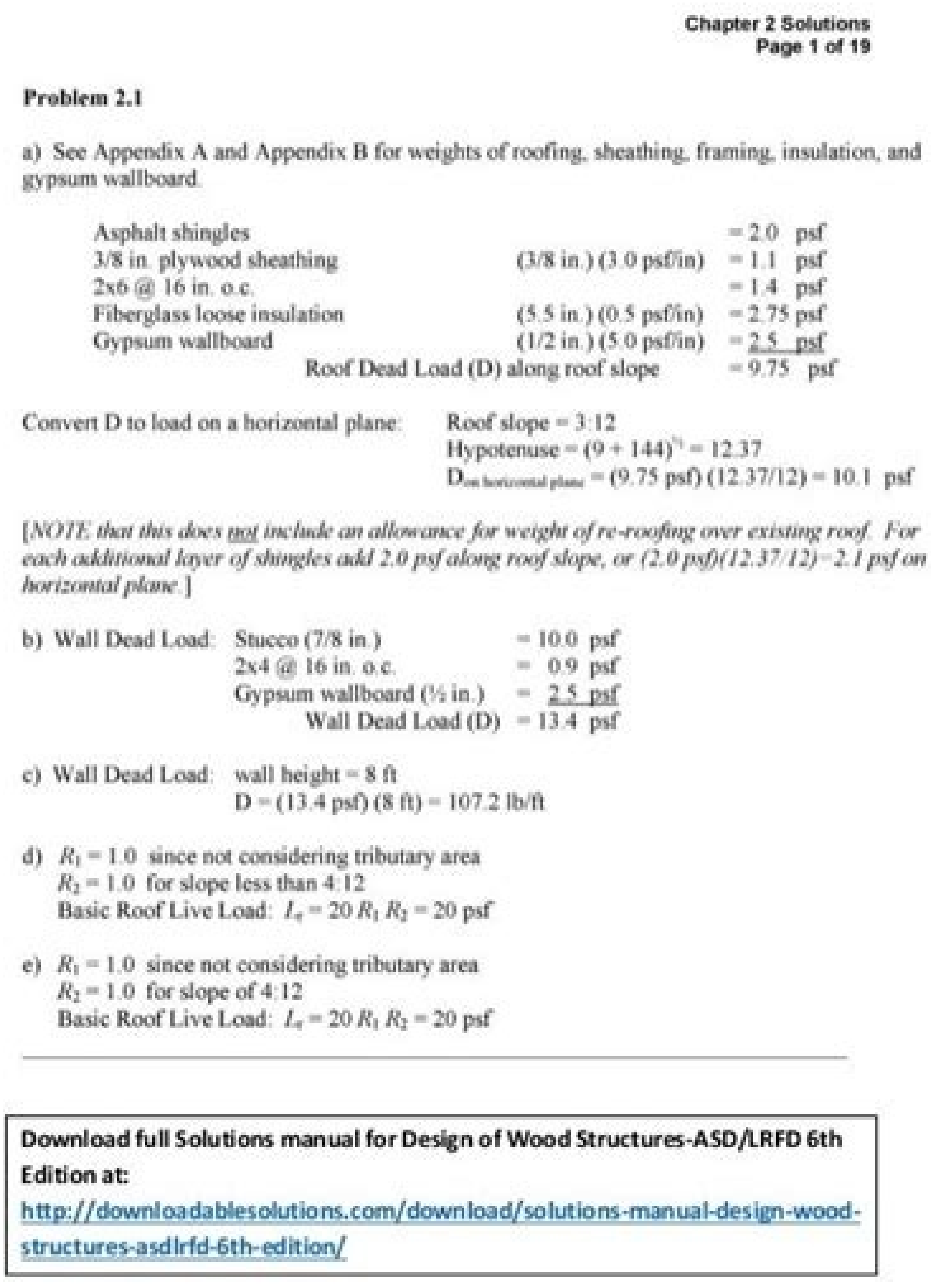
CHAPTER 2 - CONCEPTS IN STRUCTURAL STEEL DESIGN







Problem 2.1

a) See Appendix A and Appendix B for weights of roofing, sheathing, framing, insulation, and gypsum wallboard.

Asphalt shingles		= 2.0 psf
3/8 in. plywood sheathing	(3/8 in.) (3.0 psf/in)	= 1.1 psf
2x6 @ 16 in. o.c.		= 1.4 psf
Fiberglass loose insulation	(5.5 in.) (0.5 psf/in)	= 2.75 psf
Gypsum wallboard	(1/2 in.) (5.0 psf/in)	= 2.5 psf
Roof Dead Load (D) along roof slope		= 9.75 psf

Convert D to load on a horizontal plane:
Roof slope = 3:12
Hypotenuse = $(9 + 144)^{1/2} = 12.37$
$D_{\text{on horizontal plane}} = (9.75 \text{ psf}) (12.37/12) = 10.1$ psf

[NOTE that this does __not__ include an allowance for weight of re-roofing over existing roof. For each additional layer of shingles add 2.0 psf along roof slope, or (2.0 psf)(12.37/12)=2.1 psf on horizontal plane.]

b) Wall Dead Load:

Stucco (7/8 in.)	= 10.0 psf
2x4 @ 16 in. o.c.	= 0.9 psf
Gypsum wallboard (½ in.)	= 2.5 psf
Wall Dead Load (D)	= 13.4 psf

c) Wall Dead Load: wall height = 8 ft
D = (13.4 psf) (8 ft) = 107.2 lb/ft

d) $R_1 = 1.0$ since not considering tributary area
$R_2 = 1.0$ for slope less than 4:12
Basic Roof Live Load: $L_r = 20\, R_1\, R_2 = 20$ psf

e) $R_1 = 1.0$ since not considering tributary area
$R_2 = 1.0$ for slope of 4:12
Basic Roof Live Load: $L_r = 20\, R_1\, R_2 = 20$ psf

1 t Try a 1 × 3 1⁄ 2 cross section. CHAPTER 10 Plate Girders 10.1 INTRODUCTION In this chapter, we consider large flexural members (girders) that are composed of plate elements—in particular, those with noncompact or slender webs. Determine the available compressive strength. As shown in Figure A.1, the hinge will form where the moment is maximum—in this case, at midspan. Once the geometry and support conditions of the idealized frame have been established, the loading must be determined. The loads 9.8 Tables for Composite Beam Analysis and Design 6″ FIGURE 9.24 639 11⁄ 2″ 21⁄ 2″ avg. This shape is compact, as assumed (if it were noncompact, there would be a footnote in the Zx table). The weld size, in sixteenths of an inch, required for strength is D= P 130.3 = = 1.2 sixteenths 1.392 L 1.392(78.8) Based on the web thickness, use a minimum size of 3⁄ 16 inch (AISC Table J2.4). Although load and resistance factor design was not introduced into the AISC Specification until 1986, it is not a recent concept; since 1974, it has been used in Canada, where it is known as limit states design. We now consider these different conditions in more detail. 10 c. EFFECTIVE AREA Of the several factors influencing the performance of a tension member, the manner in which it is connected is the most important. Steel Structures. Stress computed in this way is known as engineering stress. SOLUTION The loads and the strength of the composite section are common to both the LRFD and ASD solutions. ⎝8⎠ For the W6 × 25, tFu = tf Fu = 0.455(65) = 29.58 kips冀in. These moments are called P-Δ moments and are illustrated in Figure 6.4b. 0.85 fc′b 0.85(4)(66) Y2 = t − a 0.6016 = 4.5 − = 4.199 in. The critical fastener cannot always be found by inspection, and several force calculations may be necessary. Part 14. For this reason, Equation 5.7 cannot be applied to the flange of a W-shape in the same manner as for the web. Segui solution manual Bookzz.net Steel Design By William T Segui ebook Magazines Manual Database, Steel Design By William T. Check the unshored steel shape for loads applied before the concrete cures (the weight of the slab, the weight of the beam, and the construction load). 2 8 3 Ant = (8) = 3.0 in. p d b2 p (7冀8)2 = = 0.6013 in.2 4 4 fRn = fFnv Ab = 0.75(54)(0.6013) = 24.35 kips冀bolt Ab = For 4 bolts, fRn = 4 × 24.35 = 97.4 kips Vu = 33.6 kips < 97.4 kips (OK) Check bearing at the compression side bolts. Although the web can be proportioned to directly resist any applied concentrated loads, bearing stiffeners are usually provided. The allowable stress will be in the elastic range of the material (see Figure 1.3). Experimental data can be represented in the form of a histogram, or bar graph, as shown in Figure 2.1, with the abscissa representing sample values, or events, and the ordinate representing either the number of samples having a certain value or the frequency of occurrence of a certain value. LRFD SOLUTION The factored-load moment is Mu = 1.2MD + 1.6(ML) = 1.2(32.5) + 1.6(97.5) = 195.0 ft-kips The flange force is Pbf = Mu 195(12) = = 134.7 kips d b − tb 17.9 − 0.525 To check local flange bending, use AISC Equation J10-1: fRn = f (6.25Fyf t 2f) = 0.90[6.25(50)(0.560)2] = 88.20 kips < 134.7 kips (N.G.) ∴ Stiffeners are required to prevent local flange bending. The chief simplification made in the computation of ILB is that only part of the concrete is used in computing the transformed area. They should always be shown in this order. The deformation of each fastener is proportional to its distance from the IC and acts perpendicularly to the radius of rotation. II. Determine the maximum total service load that can be applied if the ratio of dead load to live load is 8.5. The bolts are 7⁄8-inch-diameter, Group A bearing-type bolts. Dividing this segment into four equal spaces, we get points A, B, and C located at 15 ft, 18 ft and 21 ft from the left end of the girder. Reprinted with permission. In addition to the weight of the slab, there is a construction load of 20 psf and a uniform live load of 100 psf. 20k 8″ 3″ 2″ 2″ 6″ 2″ FIGURE P8.5-2 4″ 2″ Problems 8.5-3 587 Use the maximum size E70 fillet weld and compute the available reaction R (as limited by the strength of the weld) that can be supported by the connection of Figure P8.5-3. Steel Design Guide Series, No. 4. The AISC Specification deals with connections in Chapter J, "Design of Connections," where bolts and welds are covered. diameter bolts as shown in Figure P3.5-3. Problems 2-4 39 Beams will be designed for the roof and floor systems of an office building. 8.2-12 The bolt group shown in Figure P8.2-12 consists of 3⁄4-inch-diameter, Group A bearing-type bolts in single shear. • Compute an effective area of the cross section. Four 7⁄8-inch-diameter Group A bolts are used. For the limit state of local web yielding in compression, when the load is applied at a distance from the end of the column that is more than the depth of the column, Rn = Fyw t w (5k + b) (AISC Equation J10-2) When the load is applied at a distance from the end of the column that is less than the depth of the column, Rn = Fyw t w (2.5k + b) (AISC Equation J10-3) 538 Chapter 8 Eccentric Connections where k = distance from the outer flange surface of the column to the toe of the fillet in the collumn web b = length of applied load = thickness of beam flange or flange plate Fyw = yield stress of the column web t w = thickness of the column web We also used AISC Equations J10-2 and J10-3 in Section 5.14 to investigate web yielding in beams subjected to concentrated loads. Problems 4.7-8 179 The frame shown in Figure P4.7-8 is unbraced, and bending is about the x-axis of the members. 10.6-2 The details of an end bearing stiffener are shown in Figure P10.6-2. The bolts in this connection are subject to single shear, and the nominal strength of one bolt is Rn = Fnv Ab The nominal bolt area is Ab = π d 2 π (3冀4)2 = = 0.4418 in.2 4 4 For A307 bolts, the nominal shear strength is Fnv = 27 ksi, and Rn = 27(0.4418) = 11.93 kips This value is smaller than the bearing strength at each hole, so the total nominal strength for the connection is Rn = 11.93 + 11.93 = 23. For Cb = 1, Mn冀Ωb = 207 ft-kips For Cb = 1.36, the actual allowable strength is 1.36(207) = 282 ft-kips, but the strength cannot exceed Mp冀Ωn, which is only 265 ft-kips (this can be obtained from the chart), so the actual allowable strength must be taken as Mn M p = = 265 ft-kips < M a = 270 ft-kips (N.G G.) Ωb Ωb Move up in the charts to the next solid curve and try W18 × 55. The length of web acting with the stiffener plates to form a compression member is 12 times the web thickness for an end stiffener (AISC J10.8). Nominal bending strength: There is no LTB limit state for circular (or square) shapes. concrete pier. Assume a value for the critical buckling stress Fcr . The member is connected to a gusset plate with-11⁄8 inch-diameter bolts. (The route taken by the loads from one part of the structure to another is sometimes called the load path.) This is a fairly accurate representation of what happens, but it is not exact. The bolts are 11⁄8-inchdiameter, Group A bolts. (about the weak axis) ⎝ 2 For the concrete, Ac = net area of concrete = 20(22) – As – Asr = 440 – 39.9 – 5.08 = 395.0 in.2 Ec = w1.5 fc′ = (145)1.5 5 = 3904 ksi Ic = 22(20)3 = 14, 670 in.4 (about the weak axis) 12 From AISC Equation I2-4, Pno = Fy As + Fysr Asr + 0.85f c′Ac = 50(39.9) + 60(5.08) + 0.85(5)(395.0) = 3979 kips From AISC Equation I2-7, As ⎞ ≤ 0.3 C1 = 0.1 + 2 ⎛ ⎝ Ac + As ⎠ 39.9 ⎞ = 0.2835 < 0.3 = 0.1 + 2 ⎛ ⎝ 395.0 + 399.9 ⎠ From AISC Equation I2-6, (EI)eff = Es Is + 0.5Es Isr + C1Ec Ic = 29,000(398) + 0.5(29,000)(285.8) + 0.2835(3904)(14,670) = 3.192 × 107 kip-in.2 9.10 Composite Columns 649 From AISC Equation I2.5, Pe = π 2 (EI)eff π 2 (3.192 × 10 7) = = 8546 kipps (KL)2 (16 × 12)2 Then Pno 3979 = = 0.4656 < 2.25 Pe 8546 ∴ Use AISC Equation I 2- 2. What is the required nominal strength of the column for a resistance factor f of 0.90? The loads are 3-in. The allowable shear yield strength of the base metal is ⎛ 3⎞ 0.4 Fy t = 0.4(36)⎢ ⎥ = 5.4 kips冀in. Blodgett (1966) is a useful source of detailed welded connection information. 2:46. 1.13 in.2 c. Use LRFD and determine the maximum factored concentrated load that can be supported. 21⁄2″ 11⁄2″ 13⁄4″ 6″ FIGURE P9.8-9 9.8-10 Use the composite beam tables and select a W-shape and stud anchors for the following conditions: • span length = 35 ft • beam spacing = 12 ft • formed steel deck with rib spacing = 6 in., hr = 2 in. Values of fVn and Vn冀Ωv are given in several tables in Part 3 of the Manual, including the Zx table, so computation of shear strength is unnecessary for hot-rolled shapes. rx 冀ry 1.75 60ft-k FIGURE 6.21 3.75' 27.5ft-k 3.75' 5ft-k 3.75' 37.5ft-k 3.75' 70ft-k 6.7 Members in Unbraced Frames 337 From the column load tables, with KL = 15 ft, Pn冀Ωc = 441 kips. Load (kips) Elongation (in.) 0 1 2 2.5 3.5 5 6 7 8 9 10 11.5 12 0 0.0010 0.0014 0.0020 0.0024 0.0036 0.0044 0.0050 0.0060 0.0070 0.0080 0.0120 0.0180 The data shown in the table were obtained from a tensile test of a metal specimen with a rectangular cross section of 0.2 in.2 in area and a gage length (the length over which the elongation is measured) of 2.000 inches. Select the flange and web dimensions so that intermediate stiffeners are not required. Use A992 steel and select a structural tee cut from a W shape. 16 Rn = 1.2 Lc tFu = 1.2(2.063)(0.455)(65) = 73.22 kips 2.4 dtFu = 62.11 kips < 73.22 kips ∴ Use Rn = 62.11 kips for these bolts. High-Strength Bolts in Tension 7.8-1 Investigate both the tee and the bolts in the hanger connection in Figure P7.8-1. FIGURE 7.1 FIGURE 7.2 7.1 Introduction 379 In considering the behavior of different types of connections, it is convenient to categorize them according to the type of loading. EXAMPLE 7.16 A plate 1⁄2 × 8 of A36 steel is used as a tension member and is to be connected to a 3 ⁄8-inch-thick gusset plate, as shown in Figure 7.47. Consequently, the problem is reduced to a case of bending of two shapes, each one loaded through its shear center. If the member has a symmetrical cross section, this result can be accomplished by placing the welds or bolts symmetrically. The axial load and end moments obtained from a first-order analysis of the gravity loads (dead load and live load) are shown in Figure P6.7-2a. As previously mentioned, the most economical alternative may be simply to use a larger column section rather than stiffeners or doubler plates. From the inner row of tension bolts plus 2db to the compression flange: L = d − 2t f b − p fi − 2d b = 17.7 − 2(0.425) − 2.0 − 2 (7冀8) = 13.10 in. Determine whether the web is slender. All members of LH- and DLH-series joists can be made with steel of any yield stress between 36 ksi and 50 ksi inclusive. To compensate for this, the Commentary to the Specification recommends that the computed transformed moment of inertia be reduced by one of two methods. Then ⎛ Lb − L p ⎞ ⎤ ⎡ M nx = Cb ⎢ M px − (M px − 0.7 Fy S x ⎢ ⎥ ≤ M px ⎝ Lr − L p ⎠ ⎥ ⎦ ⎣ M px = Fy Z x = 50(160) = 8000 in.-kips Because the bending moment is uniform, Cb = 1.0. 12 − 6.36 ⎞ ⎤ ⎡ M nx = 1.0 ⎢8000 − (8000 − 0.7 × 50 × 140) ⎛ ⎝ 18.7 − 6.36 ⎠ ⎥ ⎦ ⎣ = 6583 in.-kips = 548.6 ft-kips For the y axis, since the shape is compact, there is no flange local buckling and Mny = Mpy = Fy Zy = 50(24.4) = 1220 in.-kips = 101.7 ft-kips Check the upper limit: Z y 24.4 = = 1.55 < 1.6 S y 15.7 LRFD SOLUTION ∴ M ny = M py = 101.7 in.-kips For x-axis bending, Mux = 1.2MDx + 1.6MLx = 1.2(48) + 1.6(144) = 288.0 ft-kips For y-axis bending, Muy = 1.2MDy + 1.6MLy = 1.2(6) + 1.6(18) = 36.0 ft-kips 270 Chapter 5 Beams Check interaction equation 5.22: Muy Mux 288.0 36.0 = 0.977 < 1.0 + = + φ b M nx φ b M ny 0.90(548.6) 0.90(101.7) (OK) (Note that fbMnx can be obtained from the beam design charts.) ANSWER ASD SOLUTION The W21 × 68 is satisfactory. That is, replace Ac by Ac 冀n. b Use ASD and determine the maximum service concentrated load that can be supported. 5.568 kips冀in. Shear lag occurs when some elements of the cross section are not connected, as when only one leg of an angle is bolted to a gusset plate, as shown in Figure 3.6. The consequence of this partial connection is that the connected element becomes overloaded and the unconnected part is not fully stressed. Information, news and publications (including some free to download) for structural steel design. Inner bolts: From Table 7-4, for STD (standard hole), Fu = 58 ksi, the bearing strength is frn = 78.3 kips冀bolt per inch of connected part thickness. The maximum permissible diameter is 2.5tf = 2.5(0.425) = 1.063 in. Thus component design is the focus of this book. For bearing strength computation, use a hole diameter of h=d+ 1 3 1 13 = + = in. EXAMPLE 7.1 Check bolt spacing, edge distances, and bearing for the connection shown in Figure 7.10. 3.4 Staggered Fasteners 63 FIGURE 3.19 EXAMPLE 3.9 Find the available strength of the S-shape shown in Figure 3.20. "Eccentrically Loaded Welded Connections." Journal of the Structural Division, ASCE 98 (no. References American Institute of Steel Construction. = 13.07 ft Fy 50 Returning to Equation 5.6, we obtain ⎛ L − 13.07 ⎞ 7650 = 7850 − (7850 − 0.7 × 50 × 143) ⎢ b ⎝ 42.6 − 13.07 ⎠⎥ Lb = 15.2 ft This is the value tabulated as Lp for a W14 × 90 with Fy = 50 ksi. Many standard hot-rolled structural shapes are proportioned so that local stability problems have been eliminated or minimized. Because S is larger than Lr and R = 0, we need to evaluate this combination only once, using S. ANSWER Provide lateral bracing at 7.5-ft intervals. If not, design the connection for 60% of the beam moment strength. 12′-09″ 12′-09″ 6′-02″ 6′-02″ 55′-0″ FIGURE P10.5-3 10.5-4 Determine whether the girder of Problem 10.4-5 has enough shear strength. When Kx L and Ky L are different, Ky L will control unless rx 冀ry is smaller than KxL冀KyL. Design strength = 477 ft-kips ANSWER ASD SOLUTION Enter the tables with ΣQn = 530 kips and Y2 = 4.104. If the shape is compact, then Mny = Mpy = Fy Zy ≤ 1.6Fy Sy (AISC Equation F6-1) where Mny = nominal moment strength about the y axis Mpy = plastic moment strength about the y axis Fy Sy = yield moment for the y axis (The y subscripts in Mny and Mpy are not in the Specification; they have been added here.) The limit of 1.6 Fy Sy is to prevent excessive working load deformation and is satisfied when Zy ≤ 1.6 Sy If the shape is noncompact because of the flange width-to-thickness ratio, the strength will be given by ⎛ l − lp ⎞ M ny = M py − (M py − 0.7 Fy S y) ⎜ ⎝ lr − l p ⎠⎟ (AISC Equation F6-2) This is the same as AISC Equation F3-1 for flange local buckling, except for the axis of bending. American Iron and Steel Institute (AISI): This specification deals with cold-formed steel, which we discuss in Section 1.6 of this book (AISI, 2007). Conservatively, the weld can be designed to carry the entire concentrated load. EXAMPLE 4.10 The compression member shown in Figure 4.12 is pinned at both ends and supported in the weak direction at midheight. The plastic moment, Mp, is the resisting couple formed by the two equal and opposite forces, or A M p = Fy (Ac)a = Fy (At)a = Fy ⎛ ⎞ a = Fy Z ⎝ 2⎠ FIGURE 5.4 FIGURE 5.5 194 Chapter 5 Beams where A = total cross-sectional area a = distance between the centroids of the two half-areas A Z = ⎛⎞ ⎞ a = plastic section modulus ⎝ 2⎠ EXAMPLE 5.1 For the built-up shape shown in Figure 5.6, determine (a) the elastic section modulus S and the yield moment My and (b) the plastic section modulus Z and the plastic moment Mp. Bending is about the x-axis, and the steel is A572 Grade 50. Web: l= h t If the actual dimensions b and h are not known, they may be estimated as the total width or depth minus three times the thickness. 5.12-3 A W21 × 55 of A992 steel has two holes in each flange for 7⁄ 8-inch diameter bolts. This problem can be addressed in one of three ways: (1) The depth-to-thickness ratio of the web can be made small enough that the problem is eliminated, (2) web stiffeners can be used to form panels with increased shear strength, or (3) web stiffeners can be used to form panels that resist the diagonal compression through tension-field action. P 150 ft-k 15′ 150 ft-k FIGURE P6.6-9 6.6-10 The loads in Figure P6.6-10 are service loads consisting of 25% dead load and 75% live load. The nonlinearity is primarily because of the presence of residual stresses in the W-shape. The ratio of column stiffness to girder stiffness would therefore be infinite for a perfectly frictionless hinge. For b, use the distance from the bolt centerline to the mid-thickness of the angle leg, rather than to the face of the leg. Because the maximum moment is a fixed-end moment, the stem will be in compression, assuming that the flange is at the top. 0.90fb Fytp2 Yp = fMn tp = 1.11fM n f b FyYp (8.15) Select a practical plate thickness. wu = 1.2wD + 1.6wL = 1.2(0.500 + 0.026) + 1.6(0.200) = 0.9512 kips冀ft Mu = (1冀8)(0.9512)(30)2 = 107 ft-kips A W16 × 26 is compact for Fy = 50 ksi, and since the steel deck will provide adequate lateral support, the nominal strength, Mn, is equal to the plastic moment strength, Mp. From the Zx table, fb Mp = 166 ft-kips > 107 ft-kips (OK) After the concrete has cured, the total factored load to be resisted by the composite beam, adjusted for the weight of the steel shape, is wu = 1.2 wD + 1.6 wL = 1.2(0.500 + 0.026) + 1.6(1.300) = 2.711 kips/ft Mu = 1 1 wu L2 = (2.711)(30)2 = 305 ft-kips 8 8 The effective slab width of the composite section will be the smaller of Span 30(12) = = 90 in. 6′-0″ 8 @ 6′-0″ = 48′-0″ FIGURE P5.15-6 Problems 5.15-7 The truss shown in Figure P5.15-7 is one of several roof trusses spaced 18 feet apart. The clear spacing between the steel shape and the longitudinal bars must be at least 1.5 bar diameters but not less than 1.5 inches. If the live-load deflection exceeds L冀360, select another steel shape using either LRFD or ASD. This end condition is satisfied by hinges or pins, as shown in Figure 4.2. This remarkable FIGURE 4.1 FIGURE 4.2 4.2 Column Theory 111 relationship was first formulated by Swiss mathematician Leonhard Euler and published in 1759. The strength of the concrete is fc′ = 4 ksi. < 36.25 kips冀in. EXAMPLE 3.13 Design the tension member of Example 3.12 with the aid of the tables in Part 5 of the Manual. Use the lower-bound moment of inertia, and compute the total deflection. One of the difficulties in designing a partially restrained system is obtaining an accurate moment-rotation relationship for the connection, whether bolted or welded. The purlins are supported at their midpoints by sag rods. Select a trial web size. The problem of how many significant figures to use in engineering computations has no simple solution. W10 × 54, fc Pn = 282 kips W12 × 58, fc Pn = 292 kips W14 × 61, fc Pn = 293 kips Note that the strength is not proportional to the weight (which is a function of the cross-sectional area). e. EXAMPLE 8.9 A welded framed beam connection is shown in Figure 8.28. Try 1⁄2 in. The allowable shear strength per bolt is rn = 11.9 kips > 7.5 kips Ω (OK) Compute the tensile force per bolt, then check the tension–shear interactions. In Example 5.10, it was first assumed that a compact shape would be used, and then the assumption was verified. (This result means that c is small enough so that it must be accounted for.) For the other holes, 13 c = s − h = 2.5 − = 1.688 in. 16 8 16 16 486 Chapter 8 Eccentric Connections FIGURE 8.8 For the holes nearest the edge, use Lc = Le − h 15冀16 =2− = 1.531 in. The first nine editions of the Manual and the accompanying specifications were based on ASD. For the bearing-type connection with threads in the shear plane, assume that tension controls: ΩFnt frv ≤ Fnt Fnv 2.00(90) = 1.3(90) − frv ≤ 90 54 = 117 − 3.333 frv ≤ 90 Fnt′ = 1.3Fnt − 90 Fnt′ (117 − 3.333 frv) = ≤ 2.00 Ω 2.00 = 58.5 − 1.667 frv ≤ 45 Let 100 Fnt′ = Ω ∑ Ab and frv = 50 ∑ Ab where ΣAb is the total bolt area. The load is the total service load, with a live-to-dead load ratio of 2:1. These tables give two sets of values of the available strength, one based on flexural buckling about the x-axis and one based on flexural-torsional buckling about the y axis. From Figure 5.15g, Cb = 1.67. The deformation is measured within this length.) The stress and strain were computed from the test data and plotted. ⎛ 7⎞ ⎤ ⎡ Ae = An = twn = t (wg − Σd hole) = t ⎢6.5 − 2 ⎢ ⎥ ⎥ = 4.750t ⎝ 8⎠ ⎦ ⎣ Let 4.750t = 2.785 in.2 or t = 0.586 in. 2 2 3 R n = 1.2 c tFu = 1.2(1.094) ⎛⎜ ⎞⎟ (58) = 28.55 kips ⎝ 8⎠ c = e − 7.4 Shear Strength 391 subject to a maximum of 3 3 2.4 dtFu = 2.4 ⎛ ⎞ ⎛ ⎞ (58) ⎝ 4 ⎠⎝ 8⎠ = 39.15 kips > 28.55 kips ∴ Use Rn = 28.55 kips for this bolt. Ae = AnU = 4.925(0.80) = 3.94 in.2 > 3.62 in.2 ANSWER 2 (OK) This shape satisfies all requirements, so use an L8 × 4 × 1⁄2. The steel is A572 Grade 50, and lateral bracing is provided only at the ends. ANSWER 4.7 Use a W18 × 130. This value results in a design moment of fb Mn = 1.32(123) = 162.4 ft-kips This moment is greater than fb Mp = 130 ft-kips, so the design strength must be limited to this value. D = 100k L = 250k 16⬘ FIGURE P4.6-7 Problems 177 4.6-8 Same as Problem 4.6-7, but use ASD. 2″ 5″ 1″ 2″ 1″ 15′ 2″ Section A36 FIGURE P4.9-6 4.9-7 A compression member is made up of two channels, 2C5 × 9, placed back-to-back and separated by 3⁄8 inch (for connection to a 3⁄8-inch-thick gusset plate). D = 15k L = 35k 3 4 2″ 4″ 4″ 2″ 8″ 2″ 4″ 2″ 2″ FIGURE P8.2-7 Use Group A slip-critical bolts and select a bolt size for the service loads shown in Figure P8.2-8. The stiffener plates are 9⁄16-inch thick, and the web is 3⁄16-inch thick. It is the one of primary concern in this book, and we discuss it in detail (AISC, 2010a). Compute the required moment strength (i.e., the factored load moment Mu for LRFD or the unfactored moment Ma for ASD). The connection is shown in Figure 3.25. If the failure surface is idealized as shown in Figure 7.7b, the failure load on one of the two surfaces is equal to the shear fracture stress times the shear area, or Rn = 0.6 Fu c t 2 where 0.6Fu = shear fracture stress of the connected part c = distance from edge of hole to edge of connected part t = thickness of connected part 7.3 Bearing Strength, Spacing, and Edge-Distance Requirements 383 FIGURE 7.7 Failure surface Failure surface (a) c R n /2 Rn R n /2 (b) The total strength is Rn = 2(0.6Fu ct) = 1.2Fu ct (7.1) This tear-out can take place at the edge of a connected part, as shown, or between two holes in the direction of the bearing load. As the ultimate load is approached, the forces in the bolts will approach their ultimate tensile strengths. Consider, however, the rigid frame in Figure 4.14. 4.5 Tables for Compression Members 131 EXAMPLE 4.5 Compute the available strength of the compression member of Example 4.2 with the aid of (a) Table 4-22 from Part 4 of the Manual and (b) the column load tables. For relatively small holes such as those for bolts, the effect will be small, particularly for flexure, for two reasons. Use A36 steel and fc′ = 3.5 ksi. Even the roller support of a simple beam can experience friction that restrains the beam longitudinally, inducing axial tension when transverse loads are applied. Other than that, the differences are as follows: 1. The angles will be connected to a 3⁄8 -inch-thick gusset plate. For example, to add 12.34 and 2.234 (both numbers have four significant figures) and round to four significant figures, 12.34 + 2.234 14.574 and the result should be recorded as 14.57, even though the fifth digit of the result was significant in the second number. Modern structural steel is available in several alloys to provide various strengths and special properties. In these regions there will be no composite behavior in the sense that we have considered thus far. Rn = 0.707wFnw = 0.707 ⎢ ⎝ 16 ⎠⎥ LRFD SOLUTION The design strength of the weld is fRn = 0.75(5.568) = 4.176 kips冀in. Try a depth of d 8.25 = = 4.125 in. The deformation of each of the other elements is Δ=r Δu rcrit The stress in each element is then Fnw = 0.60 FEXX (1 + 0.5 sin1.5 θ)[p (1.9 − 0.9 p)]0.3 where FEXX = weld electrode strength Δ for the element p = Δm The force in each element is Fnw Aw , where Aw is the weld throat area. The body is alphabetically organized into Chapters A through N. Total deflection, with live load: Δ1 + Δ3 + Δ4 = 0.6670 + 0.08596 + 0.5372 = 1.29 in. In this connection, block shear could occur in both the gusset plate and the tension member. Most of the loads applied to the purlins are vertical, so there will be a component parallel to a sloping roof, which will cause the purlin to bend (sag) in that direction (Figure 3.29). requiredd = 2(8) = 16 30(12) The approximate spacing is = 22.5 in. For any size weld, the lengths L1 and L 2 can then be determined. The corresponding intermediate stiffener spacing is a = 0.60h = 0.60(62) = 37.2 in. K x L 1.45(12 × 12) = = 49.83 4.19 rx 4.71 29, 000 E = 4.71 = 113 Fy 50 Since Kx L E < 4.71 rx Fy behavior is inelastic, and the inelastic K factor can be used. Except for some changes in notation, they are the same equations as those given in that work, with no simplifications. × 11⁄2-in. EXAMPLE 4.4 Determine the axial compressive strength of an HSS 8 × 4 × 1⁄8 with an effective length of 15 feet with respect to each principal axis. Use the mechanism method to find the collapse load Pu. Assume continuous lateral support. AISC J1.10 describes the connections for which fully tensioned bolts are required. The nominal shear stress for one bolt is Rn = Fnv Ab = 54(0.4418) = 23.86 kips (OK) 440 Chapter 7 Simple Connections and the allowable strength for eight bolts is 23.86 Rn ×8 = × 8 = 95.44 kips > 50 kips (OK) Ω 2.00 Check bearing: Rn 2.4 dtFu = × 8 bolts Ω Ω 2.4(3冀4)(5冀16)(58) = × 8 = 131 kips > 50 kips 2.00 ANSWER (OK) Use eight bolts. Miscellaneous Channels—for example, the MC10 × 25—are similar to American Standard Channels. Bending about the major axis: For Lb d 0.08 E ≤ , Fy t2 Mn = Mp = Fy Z ≤ 1.6My (AISC Equation F11-1) where My = yield moment = Fy S. The concept of tributary areas was first discussed in Section 3.8 in the coverage of tension members in roof trusses. A systematic approach to making the trial selection is as follows: 1. Loads can be estimated or obtained from measurements and inventories of actual structures, and resistances can be computed or determined experimentally. Lightly loaded small base plates can be designed by using the Murray–Stockwell method (Murray, 1983). The largest category of standard shapes includes those produced by hot-rolling. Loads applied after the concrete cures:After the concrete cures, the construction loads do not act, but the partition load does, and it will be treated as live load (see Example 5.13): wD = 506.3 + 35 = 541.3 lb冀ft The live load is wL = (125 + 20)(9) = 1305 lb冀ft Strength of the composite section:To obtain the effective flange width, use either Span 30(12) = = 90 in. 6.6-4 The member shown in Figure P6.6-4 is part of a braced frame. Because its strength decreases with increasing KL冀r, a column will buckle in the direction corresponding to the largest slenderness ratio, so Kx L冀rx must be compared with Ky L冀ry. To account for slenderness, the relationship between Pno and Pe is used, where Pe is the Euler buckling load and is defined as Pe = π 2 (EI)eff (KL)2 (AISC Equation I2-5) where (EI)eff is the effective flexural rigidity of the composite section and is given by (EI)eff = Es Is + 0.5Es Isr + C1Ec Ic (AISC Equation I2-6) where Is = moment of inertia of the steel shape about the axis of buckling Isr = moment of inertia of the longitudinal reinforcing bars about the axis of buckling As ⎞ C1 = 0.1 + 2 ⎛⎜ ≤ 0.3 ⎝ Ac + As ⎠⎟ (AISC Equation I2-7) Ic = moment of inertia of the concrete section about the axis of buckling TABLE 9.10 Bar Number Diameter (in.) Area (in.2) 3 4 5 6 7 8 9 10 11 14 18 0.375 0.500 0.625 0.750 0.875 1.000 1.128 1.270 1.410 1.693 2.257 0.11 0.20 0.31 0.44 0.60 0.79 1.00 1.27 1.56 2.25 4.00 9.10 Composite Columns 647 The nominal strength equations are similar to those for noncomposite members. The median is the middle value of x, and the mode is the most frequently occurring value. Try a plate 5⁄ 8 × 61⁄ 2. The service load moment is Ma = MD + ML = 32.5 + 97.5 = 130 ft-kips The flange force is Pbf = Ma 130(12) = = 89.78 kips d b − tb 17.9 − 0.525 Check local flange bending with AISC Equation J10-1: Rn 6.25Fyf t 2f = Ω Ω 6.25(50)(0.560)2 = = 58.68 kips < 89.78 kips (N.G.) 1.67 Check local web yielding with AISC Equation J10-2: Rn Fyw t w (5k + b) = Ω Ω 50(0.360)[5(0.954) + 0.525] = = 63.54 kips < 89.78 kips (N.G.) 1.50 8.7 Column Stiffeners and Other Reinforcement 553 Check web crippling with AISC Equation J10-4: Rn = Ω 0.80 t w2 ⎡⎢1 + 3 ⎣ (d)(tt) w f b 1.5 ⎤ EFyw t f ⎥⎦ tw Ω)((0.360 0.80(0.360)2 ⎡⎢1 + 3 08..525 25 0.560 ⎣ = 2.00) 1.5 ⎤ 29,000(50)0.560 0.360 ⎥⎦ = 85.52 kips < 89.78 kips (N..G.) Stiffeners are required. Slab: 50(7) = 350 lb冀ft Partitions: 20(7) = 140 lb冀ft Live load: 80(7) = 560 lb冀ft The beam weight will be accounted for once a trial selection has been made. From Equation 7.36, the shear rupture strength is fRn = 0.45Futw × 2 = 2.45(65)(0.360)(2) = 21.1 kips冀in. Let 102.2t = 26, t = 0.254 in. Either these alternative equations or the elliptical equation itself, Equation 7.23, may be used in lieu of AISC Equations J3-3a and J3-3b. If A992 steel is used, what is the available compressive strength? Such bolts are said to be fully tensioned. An unbraced rigid frame depends on the transfer of moments at its joints for stability. At working loads, before yielding begins anywhere, the distribution of bending moments will be as shown in Figure A.4a, with the maximum moment occurring at the fixed ends. Because there will be full composite action, this force will be the smaller of AsFy and 0.85fc′Ac: As Fy = 10.6(50) = 530 kips 0.85fc′Ac = 0.85(4)(5 × 87) = 1479 kips The steel controls; C = 530 kips. LRFD SOLUTION The factored load is Pu = 1.2(2) + 1.6(6) = 12 kips 78 Chapter 3 Tension Members From Equation 3.6, Required area = Ab = From Ab = πd 2 , 4 Required d = ANSWER ASD SOLUTION Pu 12 = = 0.3678 in.2 0.75(0.75Fu) 0.75(0.75)(58) 4(0.3678) = 0.684 in. 9.10 Composite Columns 653 Transfer of load between the hollow steel shape and the concrete core can be accomplished by direct bond, shear connection, or direct bearing. For ASD, set Ma–a equal to the allowable moment: M a−a = Mp 1 ⎛ pt 2f Fu ⎞ = Ωb Ωb ⎝⎜ 4 ⎠⎟ Then Tb ′ −1 (pt 2f Fu 冀4)冀Ωb 1 ⎛ Ωb 4Tb ′ ⎞ α= = ⎜ − 1⎟ ⎠ δ δ ⎝ pt 2f Fu (0 ≤ α ≤ 1) (7.21) where tf is the actual flange thickness. Table 10-4 is a design aid that contains available strengths for various combinations of plates and bolts. Required d = 0.700 in. From c = Pcr EI we obtain ⎛ P ⎞ P cL = ⎜ cr ⎟ L = nπ , cr L2 = n2π 2 EI ⎝ EI ⎠ and Pcr = n2π 2 EI L2 The various values of n correspond to different buckling modes; n = 1 represents the first mode, n = 2 the second, and so on. From Equation 7.35, the shear yield strength of the web is fRn = 0.6Fy t × 2 sides of stiffener = 0.6(50)(0.360)(2) = 21.6 kips冀in. The horizontal principal axis will be found by application of the principle of moments: The sum of moments of component areas about any axis (in this example, a horizontal axis along the top of the plate will be used) must equal the moment of the total area. The effective slab width is 90 inches. SOLUTION From Example 9.1, the composite beam consists of a W16 × 36 of A992 steel, a slab with a thickness t = 5 inches, and an effective width b = 87 inches. Since the studs can be placed only in the ribs, their spacing along the length of the beam is limited to multiples of the rib spacing. 1989b. Because the total deflection cannot be found directly, this problem is nonlinear, and without knowing the deflection, we cannot compute the moment. A second-order analysis was performed with factored loads and reduced member stiffnesses to obtain the moments and axial force shown. It is subjected to the dead and live loads shown. 46 ⎦⎥ The design thickness of an HSS is 0.93 times the nominal thickness (AISC B4.2). web. Partially composite beams (which actually are more efficient than fully composite beams) are covered in Section 9.7. Most composite construction in buildings utilizes formed steel deck, which serves as formwork for the concrete slab and is left in place after the concrete cures. Behavior of Bolted Steel Slip-critical Connections with Fillers, Report No. NSEL-017, Newmark Structural Engineering Laboratory, Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign. Yielding of the gross section controls. ASCE/SEI 7-10. EXAMPLE 10.1 The plate girder shown in Figure 10.10 must be investigated for compliance with the AISC Specification. FIGURE 7.37 444 Chapter 7 Simple Connections If the weld ultimate shearing stress, Fnw, is used in this equation, the nominal load capacity of the weld can be written as Rn = 0.707wLFnw The strength of a fillet weld depends on the weld metal used—that is, it is a function of the type of electrode. Note that there is no minimum vertical cover except when formed steel deck is used. Compute the required plate thickness: n= B − 2k 10 − 2(1.19) = = 3.810 in. Plate Thickness Once the length and width of the plate have been determined, the average bearing pressure is treated as a uniform load on the bottom of the plate, which is assumed to be supported at the top over a central width of 2k and length b, as shown in Figure 5.39. STEEL DESIGN covers the fundamentals of structural steel design with an emphasis on the design of members and their connections, rather. Figure 7.20 shows two potential bolt layouts. If sag rods are used, they will provide lateral support with respect to x-axis bending and will act as transverse supports for y-axis bending, requiring that the purlin be treated as a continuous beam. f cPn As a rough check (remember that B1 has not yet been computed and Cb has not been accounted for), pPu + bx Mux + by Muy = (2.23 × 10−3)(150) + (4.35 × 10−3)(75) + (8.38 × 10−3)(30) = 0.912 < 1.0 (OK) It is likely that B1x and B1y will be equal to 1.0. In addition, the inclusion of Cb will reduce the value of bx and the result of the interaction equation, so this shape is probably satisfactory. 1986. The selection of the tee flange thickness is more difficult in that it is a function of the bolt selection and tee dimensions. As long as this frictional force is not exceeded, there is no bearing or shear. Englewood Cliffs, N. 1996. 5.5 Bending Strength of Compact Shapes 201 Compute the maximum bending moment. 0.9(36)(118.5) Try tp = 1 in. If Lb ≤ Lp, there is no lateral torsional buckling. The different strengths correspond to different wall thicknesses for the same outer diameter. If the lower right-hand corner is selected, My 628.4(6) = = 4.121 ksi 915.0 J Mx 628.4(8 − 2.286) = = 3.924 ksi 915.0 J f2 x = f2 y fv = (4.121)2 + (1.429 + 3.924)2 = 6.756 ksi = 6.756 kips冀in. For d冀b = 1, the error is 12% and for d冀b = 1⁄4, it is 100% (Higdon, Ohlsen, and Stiles, 1960). Satisfactory: 0.901 < 1.00 b. The ASD load combinations from ASCE 7 can be reduced to the following possibilities after the elimination of all loads other than dead, live, and wind: Load combination 2: Load combination 5: Load combination 6(a): Load combination 7: D+L D ± 0.6W D + 0.75L + 0.75(0.6W) 0.6D ± 0.6W Combination 7, which is a check for a change in load direction or overturning, will be investigated first. The design process can be outlined as follows. Alternative values for single and double angles 5. Miscellaneous Requirements The following requirements are from AISC Section I3.2c. For the usual truss geometry and loading, the bottom chord will be in tension and the top chord will be in compression. Problems 585 t = 3⁄8″ L6 × 6 × 5⁄16 FIGURE P8.4-15 8.4-16 Same as Problem 8.4-15, but use ASD. Note also that in this example, the required areas are virtually the same for LRFD and ASD. A structural member is considered to be a beam if it is loaded so as to cause bending. A simple statement, such as "Use 108 studs, placed in equally spaced pairs," would suffice. In such a case, the relationship f = P冀A does not apply, and stresses on the inclined portion b–c are a combination of tensile and shearing stresses. For a unit throat size and the weld shown in Figure 8.29a, x= 2(2.5)(1.25) = 0.1689 in. Assume a 3⁄16-inch web-to-flange weld and a 1⁄2-inch cut-out in the stiffener. To obtain Fcrz , we can drop the first term of AISC Equation E4-11 to get Fcrz = GJ Ag r 02 (AISC Equation E4-3) This approximation is acceptable because for double angles and tees, the first term is negligible compared to the second term. J., Denavit, M. Problems 713 Problems Flexural Strength 10.4-1 Determine the nominal flexural strength of the following welded shape: The flanges are 3 inches × 26 inches, the web is 1⁄2 inch × 78 inches, and the member is simply supported, uniformly loaded, and has continuous lateral support. This feature is one of the chief advantages of this type of connection, the other being that ordinarily fewer bolts are required than with other types of connections, thus making erection faster. 3d ed. The form shown in Expression 6.7 is appropriate for LRFD. The plate connection is designed to resist only shear and takes care of the beam reaction. There is no maximum size requirement for this type of connection (one where the weld is not along an edge). The moment strength of noncompact shapes is illustrated graphically in Figure 5.27, where the maximum nominal strength is denoted M′p , and the maximum unbraced length for which this strength is valid is denoted L′p. Properties of Engineering Materials. • The tension member will be A572 Grade 50 and the gusset plate will be A36. 16 4 From AISC Table J2.4, Minimum weld size = 3 in. The overall procedure for making this investigation is as follows. 70 kips 75° 3″ 3″ 3″ 3″ 6″ 1′-0″ 6″ 9″ FIGURE P8.4-3 582 Chapter 8 8.4-4 Eccentric Connections Use an elastic analysis and check the adequacy of the weld. Clip at a 45⬚ angle for 5⁄8 inch.) For the stiffener to column web welds, Minimum size = 3 in. 19.7 Lst Check the base metal shear strength. An introduction to plastic analysis and design is presented in the Appendix of this book. Both the seated and the framed connections have their bolted counterparts. • The material on composite columns has been revised. At collapse, this sum becomes Mp + Mp = 16 M p 1 wu L2 or wu = 8 L2 A.3 Analysis 723 Factored loads must be compared with factored strengths, so fbMp rather than Mp should have been used in the preceding equations. It is based on first determining a value of Fe, which then can be used with the flexural buckling equations, AISC Equations E3-2 and E3-3. The axial load and end moment are given separately for dead load, roof 374 Chapter 6 Beam–Columns live load, wind uplift on the roof, and lateral wind load. Only under the most extraordinary circumstances would the use of the theoretical values be justified. In buildings and bridges, however, that usually means structural steel and reinforced concrete, and that usually means composite beams or columns. A572 Grade 50 steel is used for the angle, and A36 steel is used for the gusset plate. The earliest versions consisted of beams encased in concrete (Figure 9.1a). Gaylord, Gaylord, and Stallmeyer (1992) show that the effective length factor K depends on the ratio of column stiffness to girder stiffness at each end of the member, which can be expressed as G= ∑ Ec I c 冀Lc ∑ I c 冀Lc = ∑ E g I g 冀Lg ∑ I g 冀Lg (4.12) 4.7 More on Effective Length 147 FIGURE 4.14 where ΣEc Ic 冀Lc = sum of the stiffnesses of all columns at the end of the column under consideration. 1.60(100) 1.60 Pa 1− 1− Pe1x 1753 Pe1x = Pe1 y = B1 y = π 2 (0.8 EI y) π 2 (0.8)(29, 000)(53.4) = = 377.4 kips (15 × 12)2 (K y L)2 Cmy 0.6 = 1.042 = 1.60 Pa 1.60(100) 1− 1− 377.4 Pe1 y M ay = B1 y M nty = 1.042(20) = 20.8 ft-kipps bx 7.63 × 10 −3 = = 4.57 × 10 −3 Cb 1.67 For Lb = 0, bx = 6.49 × 10−3. In reality, all members, even those classified as beams or columns, will be subjected to both bending and axial load, but in many cases, the effects are minor and can be neglected. In a User Note in Section F4, the Specification permits members covered by Section F4 to be designed by the provisions of Section F5. The holes are for 1-inch-diameter bolts. 2 8 From AISC Equation J4-5, Rn = 0.6Fu Anv + UbsFu Ant = 0.6(58)(5.25) + 1.0 (58)(3.0) = 356.7 kips with an upper limit of 0.6Fy Agv + Ubs Fu Ant = 0.6(36)(5.25) + 1.0(58)(3.0) = 287.4 kips (controls) and the allowable strength is Rn 287.4 = = 144 kips > 86.23 kips Ω 2.00 ANSWER (OK) Use the weld shown in Figure 7.48. The use of these tables will be illustrated in the following example. fMn = f[2Pt(h0 + h1)] (8.12) where f = 0.75 4. The yield stress of the steel is 50 ksi. DEFLECTIONS Because of the large moment of inertia of the transformed section, deflections in composite beams are smaller than in noncomposite beams. This is in keeping with the approach xi xii Preface favored by some instructors; that is, cover both approaches early in the course, then emphasize LRFD as the course progresses. J., Pal, S., and Kulak, G. Ricker, David T. When trusses are used in buildings, they usually function as the main supporting elements of roof systems where long spans are required. For each combination of shape and slab, two strengths are given in Table 3-19. Figure 6.17b shows the wind load moments obtained from a first-order analysis. Use Fy = 46 ksi and fc′ = 5 ksi. Col. The connection is with 7⁄8-in. This region is sometimes called the panel zone. Use the AISC equations in Chapter F of the Specification. Otherwise, repeat the entire procedure, using the value of Fcr found for the current trial shape as a value for Step 1. 344 Chapter 6 Beam–Columns pPa + bx Max + by May = (3.32 × 10−3)(201) + (5.29 × 10−3)(67) + 0 = 1.02 (N.G.) Check other categories of shapes to be sure that we have found the lightest shape. Alternatively, bearing stiffeners can be provided to fully resist the concentrated loads, and the web need not be checked. 296 kips b. Lb = 12 ft FIGURE 10.11 L p = 1.1rt E 29, 000 = 1.1(4.138) = 129.22 in. 2 2 Ix + Iy 293 + 221 = 0 + (2.090)2 + = 25.87 in.2 r 20 = x02 + y02 + 23.9 Ag y0 = y − H = 1− Fcrz = 0 + (2.090)2 x 02 + y02 = 1− = 0.8312 2 r0 25.87 GJ 11,2200(9.22) = = 167.0 ksi 2 23.9(25.87) Ag r 0 Fcry + Fcrz = 31.79 + 167.0 = 198.8 ksi Fcr = ⎛ ⎞ = 4 Fcry Fcrz H ⎤ Fcry + Fcrz ⎞ ⎡ 1− 1− 2 H ⎠⎥ ⎢⎣ (Fcry + Fcrz)2 ⎦⎥ 4(31.79)(167.0)(0.8312) ⎤ 198.8 ⎡ 1− 1− ⎥ = 30.63 ksi 2(0.8312) ⎢⎣ (198.8)2 ⎦ Pn = Fcr Ag = 30.63(23.9) = 732.1 kips The flexural buckling strength controls, and the nominal strength is 683.3 kips. > 1.0 in. Design for the full moment and shear capacities

of the beam. Compute the strong axis bending moments: M1 ⎞ Cmx = 0.6 − 0.4 ⎛ = 0.6 − 0.4(0) = 0.6 ⎝ M2 ⎠ Since a modified flexural rigidity, EI*, was used in the frame analysis, it must also be used in the computation of Pe1. In ASD, in most cases the loads are simply added. 2010c. No restrictions on connection length have been stipulated, so weld length will not be limited, and the smallest permissible size will be used: Minimum size = LRFD SOLUTION Fillet Welds 3 in. This is a practical upper limit, because Compression members that are very more slender will have 3 2 3 2 If the stiffeners are not permitted to extend beyond the edges of the column flange, the maximum width is b≤ 8.077 − 0.360 = 3.86 in. For the remaining deflections, the lower-bound moment of inertia will be used. ANSWER Use a 5⁄16-inch fillet weld at each flange. Dead load applied before concrete cures is wD = 541.3 lb⁄ft (slab plus beam) Construction load is wconst = 180 lb⁄ft Live load is wL = 125(9) = 1125 lb⁄ft Partition load is wpart = 20(9) = 180 lb⁄ft Immediate deflection: For the beam plus the slab, w = 541.3 lb⁄ft and Δ1 = 5wL4 5(0.5413⁄12)(30 × 12)4 = 0.6670 in. The magnitude of each of these forces equals the yield stress times one half of the total cross-sectional area. The axial compressive dead load is 90 kips, and the service live load is 110 kips. (OK) 9.8 Tables for Composite Beam Analysis and Design 641 Try 3⁄4 d 3 studs. He is a Life Member of the American Society of Civil Engineers and a member of the Tennessee Structural Engineers Association. Use a rectangular plate. Span length = 30 ft Slab thickness = 5 in. This means that the full depth of the slab is not needed to develop the required compressive force. The bearing strength for the bolts is fRn = 44.00 kips. By AISC Equation J10-1, Rn = Fyt = 0.9(36)(1.6 − 0.75) = 27.54 kips. Actual : K x L 20 = = 11.56 ft < 12 ft. P⁄ΦBc = 625 kips). Actual : K x L 20 ≤ 17 ft Try a W12 × 88 (for KL = 12 ft, Pn/Ωc = 625 kips) Pn = 1.73 Pn > required 560 kips (pc 146 Chapter 4 Compression Members (By interpolation, Pn⁄ΩC = 635 kips.) Check a W12 × 79: Kx L 20 = = 11.43 ft > K y L = 8 ft rx ⁄ry 1.75 Pn = 599 kips > 560 kips Ωc (OK) Investigate W14 shapes. From elementary mechanics of materials, the shearing stress is fv = VQ Ib (5.7) where fv = vertical shear stress at the point of interest V = vertical shear force at the section under consideration Q = first moment, about the neutral axis, of the area of the cross section between the point of interest and the top or bottom of the cross section I = moment of inertia about the neutral axis b = width of the cross section at the point of interest Equation 5.7 is based on the assumption that the stress is constant across the width b, and it is therefore accurate only for small values of b. Requirements that are common with other types of members are covered in Chapter B, "Design Requirements." 3.2 TENSILE STRENGTH A tension member can fail by reaching one of two limit states: excessive deformation or fracture. If the end of the beam were fully restrained, the rotation would be zero. 356 Chapter 6 Beam–Columns Load combinations involving dead load, D, roof live load, Lr, and wind, W, are as follows. 6.8 Design of Beam–Columns 347 Calculate B1 for each plate section: M1 ⎞ Cm = 0.6 − 0.4 ⎛ = 0.6 − 0.4(0) = 0.6 (for both axes) M 2 ⎠ ⎝ M 2 ⎞ From Section 6.5, for each axis, Pe1 = π 2 EI* (K1 L) 2 and the modified stiffness is EI* = 0.8tbEI = 0.8(1.0)EI = 0.8EI Hence, π 2 (0.8 EI x) π 2 (0.8)(29, 000)(272) = = 1922 kips (K x L)2 (15 × 12)2 Cmx 0.6 = = 0.651 < 1.0 ∴ B1x = 1.0 as assumed. For each of these systems, determine the required factored load capacity for LRFD. C6 × 13 W12 × 50 FIGURE P4.9-8 186 Chapter 4 4.9-9 Compression Members A compression member is built up from a W14 × 90 and a W10 × 49, both of A992 steel. Neglect the end returns shown at the top of the welds. Combination 2: 1.2D + 0.5S = 1.2(4342) + 0.5(8100) = 9260 lb Combination 3: 1.2D + 1.6S = 1.2(4342) + 1.6(8100) = 18,170 lb Combination 3 controls. Enter the tables with KL = 10.53 feet. < 3.450 in., use c = 3.031 in. Check the flexural strength before the concrete has cured. Bearing stiffeners must be placed at each end and at the concentrated loads. For the effect of creep, the lower-bound moment of inertia of the transformed section will be used. In spite of all the uncertainties and variables involved, the turn-of-the-nut method has proved to be reliable and surprisingly accurate. floor slab Offer dead load: 20 psf Live load: 50 psf The live load deflection exceeds L⁄360. Although AISC Equation J4-5 is expressed in terms of bolted connections, block shear can also occur in welded connections, especially in gusset plates. FIGURE 6.25 SOLUTION FIGURE 6.26 Figure 6.26 shows the results of first-order analyses for dead load, roof live load, wind acting on the roof, and the lateral wind load. EXAMPLE 2.1 A column (compression member) in the upper story of a building is subject to the following loads: Dead load: Floor live load: Roof live load: Snow: 109 kips compression 46 kips compression 19 kips compression 20 kips compression a. wD = slab wt. 2-3 The loads on a roof beam consist of a dead load of 0.2 kips/ft, a roof live load of 0.13 kips/ft, and a snow load of 0.14 kips/ft. Use Ky = 1.0. a. Span length = 20 ft 0 in. Salmon, C. The relative frequency can also be expressed in decimal form, with values between 0 and 1.0. Thus the sum of the ordinates will be unity, and if each bar has a unit width, the total area of the diagram will also be unity. W14 × 90 7″ W10 × 49 FIGURE P4.9-9 4.9-10 Compute the design strength for LRFD and the allowable strength for ASD for each of the following double-angle shapes: 2L8 × 4 × 3⁄4, long legs 3⁄8-in. The service dead load is 142 kips, and the service live load is 356 kips. Similarly, Table 7-5 defines a value Le full, which is the edge distance at which the full bearing strength, frn = f(2.4dtFu), can be used. F. From Table 3-22c, "Continuous Beams," the maximum moment in a two-span continuous beam with equal spans is at the interior support and is given by M = 0.125wL2 where w = uniform load intensity = span length The maximum moment about the y axis is therefore May = 0.125(0.1050)(15)2 = 0.7383 ft-kips To select a trial shape, use the beam design charts and choose a shape with a relatively large margin of strength with respect to major axis bending. (This must be less than the limit with respect to major axis bending in the plastic range.) The connection is to be checked. Determine the number of fillet weld with member displacement (d), but it is with joint displacement (Δ). < 0.985 in. = 0.38 50 Fy h E < 3.76 tw Fy ∴ The web is compact. 2.35 × 10-6 1.5-1 a. (OK) 29,000(425) Δ= ANSWER ASD SOLUTION Use a W12 × 53. From Equation 8.9, F= (M1 + M 2) Mu 195(12) −V = − Vu = − 0 = 134.7 kips dm d b − tb 17.9 − 0.525 From AISC Equation J10-9, Rn = 0.60 Fy d c t w = 0.60(50)(8.25)(0.360) = 89.1 kips The design strength is fRn = 0.90(89.1) = 80.19 kips < 134.7 kips (N.G.) Alternative 1: Use a W21 × 93. In this type of construction, a supported concrete slab acts as a unit. 4.71 E 29, 000 = 4.71 = 124.7 QFy 0.9000(46) KL = 105.3 < 124.7 r ∴ Use AISC Equation E7-2 129 130 Chapter 4 Compression Members QFy ⎛ ⎞ Fcr = Q ⎜ 0.658 Fe ⎟ Fy ⎝ ⎠ ⎛ 0.9000 (46) ⎞ = 0.9000 ⎜ 0.658 25.81 ⎟ 46 = 21.16 ksi (= 21.15 ksi from curve interpolation in Figure) 7 Recall that AISC Equation E7-18 for bp applies when h⁄t > 1.40 E⁄f . PL 5⁄8 × 12 2″ 3″ 3″ 3″ 3″ 5″ 2″ 4′ 2″ 3⁄4-in.-diameter bolts FIGURE P3.4-1 3.4-2 In Figure 3.4-2 is a PL 5⁄8 × 10, and the steel is A36. The AISC Specification provides for three methods of analysis to obtain the axial forces and bending moments in members of a rigid frame: 1. The bracket is the thinner of the connected parts and controls. As the actual loads will be less than the failure loads by a factor of safety known as the load factor, members designed this way are not unsafe, despite being designed based on what happens at failure. Figure 4.20 shows a truss compression member connected to gusset plates at each end. Ziemian, R.D., ed. Team Assistant: Carly Rizzo Marketing Manager: Lauren Betsos Media Editor: Chris Valentine Content Project Manager: D. In addition, relatively large holes are sometimes cut in beam webs to provide space for utilities such as electrical conduits and ventilation ducts. Design the welds. This reduced area is referred to as the net area, or net section, and the unreduced area is the gross area. Furthermore, data given in the Steel Construction Manual has been rounded to three significant figures. The postbuckling strength varies with the panel slenderness ratio, which is made possible by the presence of intermediate stiffeners. "The Effective Length of Columns in Unbraced Frames." Engineering Journal, AISC 8 (no. cos q cos(65.26) ⎞ If welds are used on both sides of the stiffeners, the available length for welding is L = 19.7(4) = 78.8 in. For example, F1852 and F2280 bolts have special twist-off ends that simplify installation when a special bolt pretension is required. 9.5-2 A floor system has the following characteristics: • span length = 40 ft • beam spacing = 5 ft • slab thickness = 4 in. These combinations, as presented in the AISC Steel Construction Manual (AISC 2011a), are Combination 1: Combination 2: Combination 3: Combination 4: Combination 5: Combination 6a: Combination 6b: Combinations 7 and 8: D D+L D + (Lr or S or R) D + 0.75L + 0.75(Lr or S or R) D ± (0.6W or 0.7E) D + 0.75L + 0.75(0.6W) + 0.75(Lr or S or R) D + 0.75L ± 0.75(0.7E) + 0.75S 0.6D ± (0.6W or 0.7E) The factors shown in these combinations are not load factors. Z = 189 in.3, Mp = 788 ft-kips b. Does this member satisfy the provisions of the AISC Specification? For the gross area, Pn = Fy Ag = 36(5.51) = 198.4 kips The design strength is ftPn = 0.90(198.4) = 179 kips Tension on the effective net area: 7 1 An = 5.51 − 2 ⎛ + ⎞ (0.487) = 4.536 in.2 ⎝ 8 8 ⎠ The exact length of the connection is not yet known, so Equation 3.1 for U cannot be used. The allowable stress for ASD. The bearing strength for these bolts is frn = 44.0 kips.8 Bolt Strength 7.8 Eccentric Connections FIGURE P8.3-4 8.3-5 To find the required plate thickness, equate the total bearing strength to the applied load: 40.8t = 4 2(78.3t) = 29, t = 0.194 in. The plastic moment capacity, denoted Mp, is the bending moment at which a plastic hinge forms. The first number in the designation is the nominal depth in inches. The bolt spacing is s is 3 inches, which is greater than 2 2⁄3 d = 2.667(3⁄4) = 2 in. Although some shapes in the column load tables exceed those limits (and they are identified with a "c" footnote), the tabulated strength has been computed according to the requirements of AISC Section E7, "Members with Slender Elements," and no further reduction is needed. ΔD = 5 wslab+ beam L4 5 [(0.350 + 0.035)⁄12](30 × 12)4 = 0.474 in. Design the top chord. Since h E < 2.24 , tw Fy fv = 1.00 and the design shear strength is fvVn = 1.00(184.8) = 185 kips From Example 5.6, wu = 2.080 kips⁄ft and L = 45 ft. 628 Chapter 9 Composite Construction EXAMPLE 9.10 From Example 9.8, a W21 × 50 was selected. 11.2 Determine the thickness of the gusset plate. A W-shape is to be used with the formed steel deck shown in Figure 9.17 and a reinforced concrete slab whose total thickness is 4.75 inches. Lateral support is provided at the ends and at midspan. Groove welds are those deposited in a gap, or groove, between two parts to be connected. 11⁄2″ 3″ 3″ 3″ 3″ 41⁄2″ FIGURE P3.5-3 3.5-4 Compute the available block shear strength of the gusset plate. The magnitude of this force equals the yield stress times the area of the tension member as a whole. Although most of the members involved, however, does not usually warrant this degree of sophistication, and in most cases an approximate analysis will suffice. Check tension rupture against the reduced tensile strength using AISC Equation J3-3a (LRFD) or J3-3b (ASD). 48.3 kips 7.11-3 a. ⎞ 16 ⎠ Required length = (OK) Use two 4.5-in.-long side welds for a total length of 2 × 4.5 = 9 in. This results in a load that can be calculated from Equation 5.17: where P0 = P0 equal to AH the design bearing stress of the supporting material c = dimension needed to give a bearing stress of Note that Equation 5.16 has the same form as Equation 5.15 but with the stress Pu ⁄BN replaced by P0⁄AH. Rectangular sections: λ p = 2.26 E , Fy λr = 3.00 E Fy E\ ⎛ ⎝upper limit on λ is 5.00 F ⎞ y ⎠ Round sections: λp = 0.15E , Fy λr = 0.19 E Fy 0.31E ⎞ ⎛ ⎝upper limit on λ is E F ⎞ y The nominal strength Pno (length effects not included), to be used in the equations for the nominal strength Pn (length effects included), is determined as follows: If l ≤ lp (compact section), Pno = Pp Rectangular sections: Pp = Fy As + C2 fc′ ⎛ Ac + Asr ⎞ Es ⎞ ⎝ Ec ⎠ (AISC Equation I2-9a) where Es = 0.95 for round sections (AISC Equation I2-9b) 9.10 Composite Columns 653 If lp < l ≤ lr (noncompact section), Pno = Pp − Pp − Py (λ − λ p)2 (λ r − λ p)2 (AISC Equation I2-9c) where Es Py = Fy As + 0.7 fc′ ⎛ Ac + Asr ⎞ Es ⎞ Ec ⎝ ⎠ If l > lr (slender section), Es ⎞ Pno = Fcr As + 0.7 fc′ ⎛ Ac + Asr ⎝ Ec ⎠ (AISC Equation I2-9e) where Fcr = 9 Es (b⁄t)2 0.72 Fy 0.2 Fy ⎞ ⎛ (D t) ⁄ ⎝ Es ⎠ ⎞ for rectangular sections (AISC Equation I2-10) for round sections (AISC Equation I2-11) The elastic stiffness, (EI)eff, to be used in the computation of Pe, is (EI)eff = EsIs + EsIsr + C3EcIc (AISC Equation I2-12) where As ⎞ C3 = 0.6 + 2 ⎛ ⎞ ≤ 0.9 ⎝ Ac + As ⎠ (AISC Equation I2-13) EXAMPLE 9.14 An HSS 7 × 0.125 with an effective length of 13 feet is filled with normal-weight concrete and used as a column. With this notation, we can combine the three Equilibrium Equations 7.9 through 7.11 to obtain the total bolt force, Bc: ⎡ 6α b ⎤ Bc = T ⎢1 + ⎜ (1 + 6α) a ⎟ ⎥ (7.13) At this level of loading, deformations are so large that the tension stresses in the bolt does not coincide with the axis of the bolt. Check the capacity of the base metal. The gusset plate is 5⁄8-inch-thick and is also of A36 steel. 602 k Mntx = 420 ft-k Mny = 150 ft-k W14 × 145 13′ 602 k FIGURE P6.6-11 6.6-12 The member in Figure P6.6-12 is laterally supported only at its ends. com/t/robot-structural-analysis/manual/calculations-for-steel-design-using-ap. FIGURE 8.18 8.4 LRFD SOLUTION Eccentric Welded Connections: Shear Only 499 Pu = 1.2D + 1.6L = 1.2(10) + 1.6(30) = 60 kips The eccentric load may be replaced by a concentric load and a couple, as shown in Figure 8.18. 6.8 Design of Beam–Columns 341 * If Cb is not equal to 1.0, the value of bx must be adjusted. AISC gives the nominal moment strength as Mn = Fcr Sx ≤ Mp (AISC Equation F2-3) where Fcr = and Cb p 2 E Jc ⎛ Lb ⎞ 1 + 0.078 ⎞ ⎞ S x h0 ⎝ rts ⎠ (Lb ⁄rts)2 2 (AISC Equation F2-4) Cb = factor to account for nonuniform bending within the unbraced length Lb. This factor will be covered following Example 5.4. rts2 = I yC w Sx (AISC Equation F2-7) c = 1.0 for doubly-symmetric I shapes h0 = Iy Cw for channels (AISC Equation F2-8a) (AISC Equation F2-8b) h0 = distance between flange centroids = d − tf If the moment when lateral-torsional buckling occurs is greater than the moment corresponding to first yield, the strength is based on inelastic behavior. Use a plate 5⁄8 × 61⁄2. If the two force equilibrium equations are satisfied, the assumed location of the instantaneous center and the total load found in Step 1 are correct; otherwise, assume a new location and repeat the entire process. 294 Chapter 5 Beams 5.14-2 Drawing a plate of A36 steel to support a beam reaction consisting of 40 kips dead load and 50 kips live load. Wt (lbs./ft.) 5.0 5.0 5.0 5.7 7.1 5.2 6.0 6.7 7.7 5.5 6.3 7.0 7.5 6.3 6.6 10.0 825 550 825 510 750 425 651 344 570 282 504 224 448 197 402 167 361 142 327 123 298 106 271 93 249 81 825 550 825 510 825 463 814 428 714 351 630 291 561 245 502 207 453 177 409 153 373 132 340 116 312 101 825 550 825 510 825 463 825 434 825 396 825 366 760 317 681 269 613 230 555 198 505 172 462 150 423 132 825 550 825 490 825 434 825 394 825 366 825 350 766 475 672 390 592 324 528 272 472 230 426 197 385 170 351 147 321 128 294 113 270 100 249 88 231 79 214 70 825 550 825 525 825 467 742 404 661 339 592 287 534 246 483 212 439 184 402 161 367 141 339 124 313 110 289 98 270 88 825 550 825 507 825 467 825 443 795 397 712 336 642 287 582 248 529 215 483 188 442 165 408 145 376 129 349 115 324 103 825 550 825 507 825 467 825 443 825 408 825 383 787 347 712 299 648 259 592 226 543 199 501 175 462 156 427 139 397 124 825 550 768 488 684 409 612 347 552 297 499 255 454 222 415 194 381 170 351 150 324 133 300 119 279 106 259 95 241 86 226 78 213 71 825 550 825 526 762 456 682 386 615 330 556 285 505 247 462 216 424 189 390 167 360 148 334 132 310 118 289 106 270 96 252 87 237 79 825 550 825 526 825 490 824 452 739 386 670 333 609 290 556 252 510 221 469 195 433 173 402 154 373 138 348 124 324 112 304 101 285 92 825 550 825 490 825 455 825 426 825 406 825 385 774 346 709 303 651 267 600 236 555 210 515 188 478 169 446 152 417 137 390 125 825 550 825 490 825 455 825 426 825 406 825 385 825 363 804 328 738 289 681 255 630 227 584 202 543 182 507 163 474 148 444 134 417 121 393 110 825 550 825 490 825 455 825 426 825 406 825 385 825 363 825 346 825 334 793 312 732 279 678 249 630 224 587 202 548 183 514 166 482 151 453 138 427 126 403 115 381 105 361 97 825 550 825 490 825 455 825 426 825 406 825 385 825 363 825 346 825 334 825 324 825 313 792 297 738 270 690 246 646 225 607 207 571 190 538 175 508 161 481 149 456 138 432 128 411 119 825 550 825 490 825 455 825 426 825 406 825 385 825 363 825 346 825 334 825 324 825 313 825 300 825 289 825 279 807 267 759 247 714 228 673 212 636 197 602 183 570 171 541 160 514 150 339 697 298 642 263 592 233 549 208 510 186 475 167 444 151 416 137 388 124 825 550 825 526 825 490 825 455 825 426 825 406 825 385 825 363 825 346 777 311 711 276 656 248 612 220 570 198 532 178 498 161 466 147 Span (ft.) 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 825 550 825 550 825 550 825 550 825 526 825 490 825 455 825 426 825 406 825 385 825 363 825 346 771 311 700 278 638 249 583 224 535 202 492 182 454 164 420 149 389 135 362 123 337 112 314 102 294 94 275 86 258 79 242 73 228 67 825 550 825 526 825 490 825 455 825 426 825 406 825 385 825 363 825 346 825 334 825 324 825 313 825 300 825 289 790 270 738 247 690 227 646 208 607 191 571 177 538 163 508 151 481 140 456 130 432 121 411 113 391 106 373 99 355 93 339 87 325 82 310 77 297 73 285 69 273 65 825 550 825 526 825 490 825 455 825 426 825 406 825 385 825 363 825 346 825 334 825 324 825 313 825 300 825 289 825 279 825 270 825 262 825 254 800 243 757 229 717 216 681 204 646 192 615 182 585 173 557 164 531 155 507 148 484 141 463 134 443 128 425 122 407 116 391 111 376 106 361 102 347 98 334 94 322 90 310 86 299 83 289 80 279 77 269 74 260 71 251 68 243 66 235 63 228 61 221 59 214 57 207 55 201 53 195 52 189 50 184 49 178 47 173 46 169 44 164 43 160 42 156 40 152 39 148 38 144 37 141 36 138 35 134 34 131 33 129 32 126 31 123 30 120 30 118 29 115 28 113 27 111 27 108 26 106 25 104 25 102 24 100 24 98 23 96 22 95 22 93 21 91 21 90 20 88 20 86 19 85 19 84 18 82 18 81 18 79 17 78 17 77 16 76 16 74 16 73 15 72 15 71 15 70 14 69 14 68 14 67 13 66 13 65 13 64 13 63 12 62 12 61 12 60 12 59 11 58 11 57 11 56 11 55 10 55 10 54 10 53 10 52 10 52 9 51 9 50 9 50 9 49 9 48 9 47 8 47 8 46 8 46 8 45 8 44 8 44 7 43 7 43 7 42 7 42 7 41 7 41 7 40 7 40 6 39 6 39 6 38 6 38 6 37 6 37 6 36 6 36 6 36 5 35 5 35 5 34 5 34 5 34 5 33 5 33 5 32 5 32 5 32 5 31 5 31 4 31 4 30 4 30 4 30 4 29 4 29 4 29 4 28 4 28 4 28 4 27 4 27 4 27 4 27 4 26 4 26 4 26 4 26 4 25 3 25 3 Source: Standard Specifications, Load Tables, and Weight Tables for Steel Joists and Joist Girders. The tangent modulus Et is defined as the slope of the tangent to the stress–strain curve for values of f between Fpl and Fy. If the compressive stress at buckling, Pcr ⁄A, is in this region, it can be shown that π 2 Et I (4.5) L2 Equation 4.5 is identical to the Euler equation, except that Et is substituted for E. ANSWER ASD SOLUTION fbMpx bx 6.29 × 10 −3 = 5.52 × 10 −3 > 5.40 × 10 −3 = 0.8 Ch 1.14 Use bx = 5.52 × 10−3. Number of bolts required = 39 = 2.18 17.9 Try three bolts. State why you think your weld size and configuration are best. Because it is a secondorder equation, there are two boundary conditions. (OK) From Equation 7.29, the weld strength per inch is φ Rn = φ (0.707 w)Fnw) The matching electrode for A36 steel is E70. When the two ratios are equal, the column has equal strength in both directions. Adequate For shear, ft, 9.38 kips < 16.2 kips; For bearing, 9.38 kips < 38.2 kips; for tension, 16.2 kips < 19.6 kips b. This would mean that the axial strength is based on the minor principal axis of the member, buckling of the flange or web is prevented, and this situation is covered in Section 4. 8 For the leg of a single angle, a simpler approach is to use different thicknesses of flange plate, welded end-to-end, at different locations along the length of the girder.) It should be evident that the welded plate girder is far superior to the riveted or bolted girder in terms of simplicity and efficiency. 182 Chapter 4 Compression Members A 14′ B 14′ C 18′ 18′ 18′ 4 @ 20′ FIGURE P4.7-13 Torsional and Flexural-Torsional Buckling 4.8-1 Use A992 steel and compute the nominal compressive strength of a WT10.5 × 66 with an effective length of 16 feet with respect to each axis. The exact definition is given by AISC Equation F4-11. Since the moments at these two locations are limited to the plastic moment Mp , a should be set equal to 1.0. The three Equilibrium Equations 7.9 through 7.11 can also be combined into a single equation for the required flange thickness, tf . Students should have a background in mechanics of materials and analysis of statically determinate structures. This ideal state is never achieved in reality, however, because some eccentricity of the load is inevitable. 616 Chapter 9 Composite Construction EXAMPLE 9.6 The span length of a certain floor system is 30 feet, and the beam spacing is 10 feet center-to-center. 72.9 kips 3.4-1 a. In some cases, plate girders rely on the strength of the flange and web for this. Much of the resistance of the section to shear will come from the flanges. The slab is supported by a 2-inch deep steel deck, and the total slab thickness is 5 inches, from bottom of deck to top of concrete. L 8.5 8.6 Moment-Resisting Connections 527 From Equation 7.35, the base metal shear yield strength per unit length is 5 ⎞ fRn = 0.6 Fy t = 0.6(36) ⎛ ⎞ = 6.75 kips⁄in. Assume continuous lateral support and use Fy = 50 ksi. The AISC requirements are given in either LRFD or ASD form. All other requirements for column web stiffening and panel zone shear strength are the same as for other types of moment connections. Note that this method will not accommodate failure patterns with lines parallel to the applied load. However, lower stud anchors will be required, and the costs of both the steel and the studs (including the cost of installation) must be taken into account in any economic analysis. 3″ 3″ 3″ 3″ 30° 51⁄2″ FIGURE P8.2-2 8.2-3 A plate is used as a bracket and is attached to a column flange as shown in Figure P8.2-3. For this and other details, refer to AISC Design Guide 13, Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications (Carter, 1999). The usual practice is to ignore it. In such a case the sum of the ordinates will be 100%. LRFD SOLUTION Design strength = fcPn = 0.90(53.35) = 48.0 kips ASD SOLUTION Pn 53.35 = = 32.0 kips Ω 1.67 (Allowable stress = 0.6Fcr = 0.6(19.76) = 11.9 ksi) 128 Chapter 4 Compression Members ALTERNATIVE SOLUTION WITH f DETERMINED BY ITERATION As an initial trial value, use f = Fcr = 19.76 ksi (the value obtained above after using an initial value of f = Fy) be = 1.92(0.116) 29, 000 ⎡ 0.38 ⎤ 1 − 19.76 ⎡ (66.0) 29, 000 ⎤ = 6.65 in. Figure 5.49 illustrates a simplified way of treating these two cases. 2 = An = AnU = 3.875(0.80) = 3.10 in.2 Tn = ft Pn = ft Fu Ae = 0.75(58)(3.10) = 135 kips < 154 kips (N.G.) This shape did not work because the ratio of actual effective net area Ae to gross area Ag is not equal to 0.75. The ratios given in Table 4-14 can be placed into six groups: those with approximate values of rmx⁄rmy of 1.2, 1.3, 1.4, 1.5, 1.6, and 1.8. LRFD SOLUTION The factored axial load is Pu = 1.2(250) + 1.6(250) = 700 kips Starting with the smallest ratio of 1.2, we compute the approximate value Kx L 16 = = 13.3 ft⁄ry 1.2 Use Chapter 9 Composite Construction An HSS12 × 10 × 3⁄8, weighing 53.0 lb⁄ft, is a possibility. Show your final design on a sketch. For an effective length of KL = Kx L 16 = = 13.7 ft rmx ⁄rmy 1.17 the allowable strength is 565 kips (for KL = 14 ft). Hole deformation controls.) 7.3 Bearing Strength, Spacing, and Edge-Distance Requirements 387 The bearing strength for the tension member is Rn = 2(29.36) + 2(52.20) = 163.1 kips For the gusset plate and the holes nearest the edge of the plate, h 13 ⁄16 Lc = 1.25 − = 0.8438 in. Vu ≤ fRn 564 Chapter 8 Eccentric Connections FIGURE 8.53 c c a a b b d d Ffu where Vu = beam end shear (reaction). 6 @ 13′-0″ W16 × 26 W16 × 26 W16 × 26 I W18 × 40 W18 × 40 E W18 × 40 J W18 × 40 W18 × 40 F W18 × 40 K W18 × 46 L W18 × 46 W18 × 46 W18 × 46 G H F W18 × 40 A W18 × 46 W10 × 49 columns W12 × 79 columns W12 × 96 columns N 3 @ 30′-0″ = 90′-0″ FIGURE P4.7-12 4.7-13 The rigid frame shown in Figure P4.7-13 is unbraced in the plane of the frame. 1997. 300 300 To find the lightest shape that satisfies these criteria, we search the dimensions and properties table for the unequal-leg angle that has the smallest possible gross area and then check the effective net area. The coordinates of the fasteners are: ⎛ Σ(x 2 + y 2) where the origin of the coordinate system is at the centroid of the total fastener shear area. L2 = 6.960 6.960 11 = We will now examine the second option given in AISC J2.4c, in which we use 150% of the basic strength for the transverse weld and 85% of the basic strength for the longitudinal welds. (See Figure 8.48.) ⎞ 8⁄ The required weld size is therefore 31.10 = 2.16 sixteenths 0.9279(15.5) 3 Although not always done, it is also common to design the connection for moment amplification. 2″ 3″ 3″ 21⁄2″ 3″ 2″ D = 40k L = 100k/8″ 3 FIGURE P7.4-6 Slip-Critical and Bearing-Type Connections 7.6-1 A double-angle shape, 2L6 × 6 × 5⁄8, is connected to a 5⁄8-inch gusset plate as shown in Figure P7.6-1. Shielded metal arc welding is normally done manually and is the process universally used for field welds. These can be I-shaped sections, with two flanges and a web, or box sections, with two flanges and two webs. Case II: Loads Not Applied Through the Shear Center When loads are not applied through the shear center of a cross section, the result is flexure plus torsion. P1 FIGURE 6.27 Q1 Q1 P2 Q2 P3 Q2 Q3 P4 Q4 P5 Q3 Q4 5 R1 R5 R2 P1 + R 1 R3 P2 + R2 P3 + R3 R4 P4 + R 4 P5 + R5 Down-Columns EXAMPLE 6.12 The parallel-chord truss shown in Figure 6.26 supports purlins at the top chord panel points and midway between the panel points. LRFD SOLUTION From the column load tables, the axial compressive design strength of a W10 × 49 with Fy = 50 ksi and an effective length of Ky L = 1.0 × 17 = 17 feet is fc Pn = 405 kips Since bending is about the strong axis, the design moment, fb Mn , for Cb = 1.0 can be obtained from the beam design charts in Part 3 of the Manual. = wslab Construction load = 20(5.5) = 110.0 lb⁄ft We will account for the beam weight later. Assume that both connected parts are 3⁄8 inch thick. Strength of Unshored Composite Beams 9.2-1 A W18 × 35 acts compositely with a 41⁄2-inch-thick floor slab whose effective width b is 96 inches. 16 Rn = 1.2 c tFu ≤ 2.4dtFu c s = h = 2.5 − 3 1.2 c tFu = 1.2(1.688) ⎛ ⎝ (58) = 44.06 kips ⁄ ⎞ Upper limit = 2.4dtFu = 39.15 kips < 44.06 kips ◆ Use Rn = 39.15 kips⁄bolt. For example, the tension member lap joint shown in Figure 7.33 can be constructed by welding across the ends of both connected parts. Bearing strength: Rn = 1.2ctFu ≤ 2.4dtFu (AISC Equation J3-6a) I. We begin with compact shapes, defined as those whose webs are continuously connected to the flanges and that satisfy the following width-to-thickness ratio requirements for the flange and the web: bf E ≤ 0.38 2t f Fy h E and ≤ 3.76 tw Fy The web criterion is met by all standard I and C shapes listed in the Manual for Fy ≤ 65 ksi; therefore, in most cases we need to check only the flange ratio. Krishnamurthy and Kulak (1971) used 3⁄4-inchdiameter A325 bearing-type bolts and A36 steel plates, but the results can be used with little error for A325 (or other Group A) bolts of different sizes and steels of other grades. Steel is an alloy, its principal component being iron. International Code Council. Member CD is a more critical case, because it must resist the load P1 + P2 without any assistance from the vertical members. D = 10k L = 40k 2 1 3″ 3″ 6″ 3″ WT6 × 20 W14 × 61 6″ 5″ 11⁄2″ 1″-diameter, Group A bolts Slip is permitted 11⁄2″ FIGURE P8.3-6 8.3-7 A beam is connected to a column with 3⁄4-inch-diameter, Group A slip-critical bolts, as shown in Figure P8.3-7. Hansell, W. The force to be transferred is a shear force, and its magnitude depends on how the external force is applied to the member: whether 650 Chapter 9 Composite Construction 100% to the steel section, 100% to the concrete, or a portion to the steel and a portion to the concrete. Select a steel shape for these joists. Joe Gough/Shutterstock This structure uses both hot-rolled beams and open-web steel joists. 71⁄2″ PL 1⁄2″ 2 x A36 steel Weld t = 1⁄2″ FIGURE P8.5-4 Problems 3.5-5 101 A C15 × 50 is connected to a 3⁄8-in.-thick gusset plate as shown in Figure P3.5-5. Sergui Pyblishev, Global Engineering: Christopher M. When subtracting members of almost equal value, significant digits can be lost. 728 Appendix Plastic Analysis and Design Figure A.7 For span AB, ∑ M B = VA (10) − 67.5(5) + 128.2 = 0 VA = 20.93 kips VB = 20.93 − 67.5 = − 46.57 kips For span BC, 5 ∑ M B = − M p + 85.5(10) + ⎛ ⎝ M p − VC (20) = 0 ⎞ 3⁄ 85.5(10) + (22⁄3) M p 855 + (2⁄3)(128.2) VC = == 47.02 kips 20 20 VB = 85.5 − 47.02 = 38.48 kips For span CD, 15 5 ∑ MC = − M p + M p + 85.5(10) − VD (20) = 0 3 3 VD = 42.75 kips = VC The maximum shear is therefore VC from span BC, or 47.02 kips. 8.4 Eccentric Welded Connections: Shear Only 507 EXAMPLE 8.7 A tension member consists of a double-angle section, 2L5 × 3 × 1⁄2 LLBB (long legs placed back-to-back). On the negative side, skilled workers are required for the 60,640 in.4 Maximum Vu2 223.4(857.2) = = 3.158 kips⁄in. In this case, the AISC Specification is published as a stand-alone document, but it is also part of the Steel Construction Manual, which we discuss in the next section. The Moment Amplification Method 307 The direct analysis method is the preferred method. For the flange-to-web welds, compute the horizontal shear flow at the flange-to-web junction: Maximum Vu = 223.4 kips Q = flange area × 31.75 (see Figure 10.17) = 1.5(18)(31.75) = 857.2 in.3 Ix = 60,640 in.4 Maximum Vu2 223.4(857.2) = = 3.158 kips⁄in. The 28-day compressive strength of the concrete is fc′ = 4 ksi. The reduced area is computed by using 2n or 3n instead of the actual modular ratio n. We evaluate each expression that involves dead load, D; live load resulting from occupancy, L; roof live load, Lr ; and snow, S. Thus, b 87 = = 10.88 in. Furthermore, the shear strength of the stiffeners must be adequate. 2 8 The ordinates of the deflected shape, or elastic curve, will be equal to the same value, once a steel beam of that size or something smaller will occur is equal to the area of the diagram to the left of that value. Assume that all welds are adequate and check a. 270 kips b. In some instances, the recommendations are only guidelines based on common practice, not requirements of the Specification. Tension on the net effective area (Figure 3.38a): ft = 0.75, Ωt = 2.00, Pn = Fu(2tbe) (AISC Equation D5-1) 2. Some latitude can be provided in beam length by fabricating it short and achieving the final fit with shims. For an unsymmetrical cross section, Sx will have two values: one for the top extreme fiber and one for the bottom. Because of the relative complexity of determining wind loads, however, wind loads are usually considered a separate category of loading. 280 Chapter 5 Beams a. If more than one frame is stabilized by the brace, then the brace loads, in all of those frames must be included. The welds attaching this angle to the column 512 Chapter 8 Eccentric Connections FIGURE 8.24 must resist the moment caused by the eccentricity of the reaction as well as the beam reaction in direct shear. 0.718 in. In bearing-type connections, the only practical requirement is that they be tensioned enough so that the surfaces of contact in the connection firmly bear on one another. As covered in Chapter 4 of this book, the available strength of FIGURE 6.3 306 Chapter 6 Beam–Columns FIGURE 6.4 compression members takes into account the effects of residual stresses, initial out-of-straightness and inelasticity. Crawford, S. Try a W12 × 53. wa = wD + wL = 0.350 + 0.700 = 1.05 kips⁄ft 10.5 Tension-Field Action 577 For LRFD, the required nominal strength is 1 1 Mn M a = wa L2 = (1.05)(30)2 = 118 ft-kips = required 8 8 Ωb For a beam with full lateral support, the Zx table can be used. Recall that direct bond is not an alternative for encased composite members. The total load to be used in the estimate is Pa = D + L = 35 + 70 = 105 kips 105 P Required Ag = a = a = = 4.86 in. Try L5 × 31⁄2 × 5⁄8 (Ag = 4.93 in.2 and rmin = 0.746 in.) An = Ag − Aholes = 4.93 − 2 (43 + 81)(85) = 3.836 in. AISC J3.1 permits the bolts in some connections to be snug tight. 2 2 From Table 3-19 in the Manual, any combination of ΣQn , and Y2 that furnishes an allowable strength of more than 143 ft-kips will be an acceptable trial beam. For the sake of efficiency, they are usually used in design offices, but you should approach these with caution and not allow basic principles to become obscured. All members are oriented so that bending is about the strong axis. 280k 21′ FIGURE P6.9-12 4.9-13 Use ASD and select a double-angle section for member AB of the truss of Problem 3.8-2. Sherman, D.R. 1997. International Building Code. The requirements for tension field action is used are given in AISC G3.3. The first requirement is for the proportions of the stiffener. "Are You Properly Specifying Materials?" Modern Steel Construction, AISC 49 (no. AISC E6 contains many details related to this connection, with separate requirements for members composed of two or more rolled shapes and for members composed of plates or a combination of plates and shapes. 1 1⁄4″ 3″ 3″ 3″ 1 1⁄4″ 2″ 1″ 7.8 HIGH-STRENGTH BOLTS IN TENSION When a tensile load is applied to a bolt with no initial tension, the tensile force in the bolt is equal to the applied load. Both top and bottom chords of K-series joists must be made of steel with a yield stress of 50 ksi, and the web members may have a yield stress of either 36 ksi or 50 ksi. For LRFD, the resistance factor for this limit state is f = 0.90; for ASD, the safety factor is Ω = 1.67. 4 A beam spacing = 10 × 12 = 120 in. This brace prevents translation perpendicular to the weak axis of the cross section but provides no restraint perpendicular to the strong axis, and the two members—which can be steel headed studs or short lengths of small steel channel shapes—are welded to the top flange of the beam at intervals and provide the connection mechanically through anchorage in the hardened concrete (Figure 9.1c). Part 15. Although girders with intermediate stiffeners will usually require less steel, the savings can be offset by the additional fabrication cost. SOLUTION Compute the constants that are based on the geometry of the connection. "Strength and Serviceability of Hanger Connections." Engineering Journal, AISC 29 (no. Shanley (1947) resolved the apparent inconsistencies in the original theory, and today the tangent modulus formula, Equation 4.5, is accepted as the correct one for inelastic buckling. From AISC Equation G3-4, ⎡ Vr − Vc1 ⎤ t st ≥ I st1 + (I st 2 − I st1) ⎢ Vc 2 − Vc1 ⎥ 223.4 − 96.9 ⎤ = 5.949 + (16.16 − 5.949) ⎡ = 15.31 in.4 ⎣ 238 − 96.9 ⎦ Try two 3⁄8 × 4 plates: From Figure 10.20 and the parallel-axis theorem, I st = ∑ (I X Ad 2) ⎡ (3⁄8)(4)3 ⎤ = (3⁄8)(4)(2 + 5⁄8)32)2 ⎤ × 2 stiffeners ⎣ 12 ⎦ = 17.9 in.4 > 15.1 in.4 (OK) 706 Chapter 10 Plate Girders FIGURE 10.20 We will use these plates for all of the intermediate stiffeners. In Figure 9.7b, the concrete stress block extends the full depth of the slab, and the PNA is in the flange of the steel shape. Let 1.392 D(L) S = ftFyst Act where C = (b − c)(p) × 2 sides × 2 stiffeners and the factor of 1.5 is used because the load is a service load and is applied to the weld. Allowable compressive strength = 43.2 kips. Design Guide 16, Flush and Extended Multiple Row Moment End Plate Connections, is more general one of the two documents. This means that in a failure pattern consisting of both staggered and unstaggered holes, use d for holes at the end of a transverse line between holes (s = 0) and use d′ for holes at the end of an inclined line between holes. For the connection, fRn = 4.176 kips⁄in. If the required strengths are from first-order analyses, the moment amplification method, an approximate second-order analysis given in Appendix 8, can be used. From Manual Table 7-1, the shear strength of one bolt is frn = 17.9 kips⁄bolt. The connected parts remain rigid. 10-3 Use A992 steel and select the most economical W shape for the beams in Figure P5.10-3. The depth of the compressive stress block in the slab is = C 384 = = 1.255 in. Since 9 feet is a conservative approximation of the actual effective length, this shape is satisfactory. Compare the strength of the composite section and compare it to the strength of the steel beam alone. There are actually two different conditions that make this shape not satisfactory. The span length is 30 feet, and the beams are spaced at 10 feet center-to-center. Failure of the connected part because of bearing exerted by the fasteners. Ω For the stiffener shear yield strength, Rn = 0.4 Fy t × 2 = 0.4(36) ⎛ 5⁄16 ⎞(2) = 9.0 kips⁄in. New York: WileyInterscience. The given load is a service load consisting of 25% dead load and 75% live load. In 1884, the first building with a steel frame was completed in Chicago. 4.8-2 Use A36 steel and compute the nominal strength of the column shown in Figure P4.8-2. From Equation 7.37, the shear strength of the angle leg is Rn ⎛ 1⎞ = 0.4 Fy t = 0.4(3 6) ⎛ ⎞ = 7.20 kips⁄in. For a rectangular cross section bent about the minor axis, the nominal strength equal to the plastic moment strength, Mp . For a rectangular cross section of unit width and depth t, the plastic moment is Mp = Fy ⎛ 1 × t⁄ ⎞ ⎛ t ⎞ = Fy t 2⁄ For LRFD, the nominal strength of a fillet weld is equal to 4 ⎝ 2 ⎠ ⎝ 4 ⎠ where f = 0.90 and Ω = 1.67. If the connected parts are not rigid, a bending moment results and must be accounted for. 6.90 = 6.9 = 7.00 4.5 4.8 6.6 Mp = Mp is the factored-load beam reaction. r Fy Fcr = 0 877Fe = 0.877(8.5) = 7.455 ksi f r Po = Fcr A = 0.90(7.455)(20.9) = 140 kips > 60 kips (OK) Because the initial yield at a single location is not the strength limit, it can be assumed that the average shear stress is 0.90 of the shear yield stress, with a value about halfway between 33 and 7.455 ksi. Rolling the steel while it is still hot does not allow it to be deformed with no resulting loss in ductility, as would be the case with coldworking. Determine the bolt diameter. b. In Figure 8.4, if a W21 × 68 is 8.27 inches, making the plate slightly wider than the flange, which is desirable. 2 3.659″ 17″7″ 4.5″ −y 624 Chapter 9 TABLE 9.5 Composite Construction Component Transformed Section W18 × 35 Sum y = A y Ay − I d − I + Ad 2 10.30 10.30 21.36 8.85 220.0 91.2 − 510 6.25 6.26 402 914 20.60 Σ Ay Σy = 311.2 20.60 311.2 1316 in.4 = 15.11 in. For shear in the column web, from Equation 8.9 and neglecting the thickness of shims in the column web panel zone is F= (M1 + M 2) M 140(12) −V = −V = − 0 = 89.8 kips dm d b t fb PL 20.8 + 5 ⁄8 8.37 Column Stiffeners and Other Reinforcement 545 From AISC Equation J10-9, the nominal strength is Rn = 0.60Fy d c t w = 0.60(50)(14.2)(0.485) = 206.6 kips The allowable strength is Rn 206.6 = = 123.7 kips > 89.8 kips (OK) ANSWER (OK) Column web reinforcement is not required. There are currently four authorized procedures for the installation of high-strength bolts (RCSC, 2009). For ASD, the allowable stress is Fa = 0.6Fcr = 0.6(7.722) = 4.2 ksi 4.2 ⎜ ⎛ 0.18 = 13 = 0.8(10) = = 2.5 in. Although such information is not in conflict with the Specification, it is important to recognize that what is a requirement (a specification that is adopted by a building code) and what is not. To evaluate Et ⁄E, called the stiffness reduction factor (denoted by tb), consider the following relationship. 47.0 kips 10.4-1 27,200 ft-kips 8.6-1 a. The location of the first intermediate stiffener will be determined by the following strategy: First, equate the shear strength from AISC Equation G2-1 to the required value of Cv. Next, solve for kv from Equation G2-5, then solve for a⁄h. Round HSS with ≥ 1.3 D 4. If the live load deflection exceeds L⁄360, select another steel shape using either LRFD or ASD. f. 10.7-9 A plate girder ABCDE will be used in a building to provide a large column-free area as shown in Figure P10.7-9. From the Zx table, the three shapes satisfy the moment requirement with the least weight: a W12 × 26, a W14 × 26, and a W16 × 26. 6.9-7 Select the best rectangular (not square) HSS for a column to support a service dead load of 30 kips and a service live load of 90 kips. Use the lightest shape that is suitable. In this case, the W16 × 26 should be selected because it has the largest moment of inertia and therefore the largest bending strength. In the tables in Part 7 of the Manual to find the maximum bolt force by the ultimate strength method. If the column is a one-story member and can be treated as pinned at both ends, the only bending will result from minor accidental eccentricity of the load. We consider the strut to be a "weak" strut, in keeping with our practice of always using Rp = 0.6 with formed steel deck. A reasonable approach is to perform operations on the calculator in any convenient manner and record intermediate values 2.7 Design Computations and Precision 37 to whatever degree of precision is deemed adequate (without clearing the intermediate value from the calculator if it can be used in the next computation). Compute V′, the horizontal shear force at the interface between the concrete and the steel. This connection is one of the simplest. In both cases, plastic analysis can be used to determine the maximum load, given that the connection load is resisted by forces in each element, with each force acting perpendicular to the radius constructed from an instantaneous center of rotation to the centroid of the segment, as shown in Figure 8.19. This means that we cannot use a higher shear strength larger than the base-metal shear strength, so the base-metal shear strength is an upper limit. Furthermore, as this girder has continuous lateral support, lateral-torsional buckling need not be considered. The capacity of the weld across the end of the angle is P3 = 10.44(5) = 52.20 kips 8.4 Eccentric Welded Connections: Shear Only 509 Summing moments about an axis along the bottom, we get ∑ ML 2 ∑F 5 = 121.5(3.26) − 52.20 ⎛ ⎞ − P1 (5) = 0, P1 = 53.12 kips ⎝ ⎛ 2⎠ ∑ F = 121.5 − 53.12 − 52.20 − P2 = 0, P2 = 16.18 kips P1 53.12 = = 8.98 in. From Equation 7.37, the base metal shear yield strength per unit length is Rn ⎛ 5 ⎞ = 0.6 Fy t = 0.6(36) ⎛ ⎞ = 5.400 kips⁄in. Assume that a maximum of 2.4 kips of shear can be transmitted at the contact surface and 2.4 of the shear need not be reduced by a factor of shear lag. In many cases, the value of Y2 will be needed, so the depth of the steel beam must be assumed and then revised after an iteration. There is no restriction on deflection. Figure 5.1 shows two types of beam cross sections; a hot-rolled shape and a doubly-symmetric built-up I shape. The loading consists of a uniform service dead load of 1.0 kips⁄ft (including the weight of the girder), a uniform service live load of 2.0 kips⁄ft, and a concentrated service live load of 500 kips at midspan. • Minimum cover above studs = 1⁄2 inch. The minimum radius 68 Chapter 3 Tension Members of gyration is the one corresponding to the minor principal axis of the cross section. 0 (AISC Equation J2-1) = actual length of weld w = weld size If the length is larger than 300 times the weld size, use an effective length of 180w. The capacity of this strength is 53.2 kips and is determined by the y axis with an 170 Chapter 4 Compression Members effective length of 14 feet. In addition, shapes that are slender for compression are excluded from a footnote (footnote c). Bending is about the strong axis in the beam. In Chapter 6 "Beam–Columns". From Equation 7.36, the base metal tension rupture strength per unit length is Rn = 0.45Fu t = 0.45(58) ⎛ 14 ⎞ = 6.525 kips⁄in. 570 Chapter 8 ANSWER Eccentric Connections Use a 3⁄16-inch fillet weld on each side of the web between mid-depth and the compression flange. (Note that, this standard C has the same significance as the strength of one bolt in that location is equal to its shear strength Rn = Fnv Ab as given in Equation 8.2 for a unit bolt strength. 2(3.155) The second option requires shorter longitudinal welds. These built-up shapes usually are used when the bending moments are larger than standard hot-rolled shapes can resist, usually because of a large span. When concentrated loads are present, AISC I8.2c requires that the number of the N1 anchors be placed between the concentrated load and the adjacent point of zero moment to develop the moment required at the load. 1.5″ 1.5″ 1.5″ 2″ 2 1⁄4″ 1.5″ 2 1⁄4″ FIGURE P3.4-5 3.4-6 A tension member is made up of a double-angle shape, 2C10 × 20, of A572 Grade 50 steel is used for a built-up tension member as shown in Figure P3.4-6. 159 kips 735 736 4.7-9 4.7-11 Answers to Selected Problems 8 Kx L Since the shear rupture strength of the base metal is ⎛ 3⁄ 0.3Fu t = 0.3(58) ⎞ ⎛ ⎞ = 6.525 kips⁄in. (This result means that c is actually a good value since it does not need to be decreased.) For the gusset plate, the shear rupture strength Rn is 0.6 Fu t = 0.6(58)(1⁄4) = 8.70 kips⁄in. In the tension member shown, our rupture strength 0.45(58)(1⁄4) = 6.525 kips⁄in. The gusset plate is A36 steel. Ω 1.50 1.50 The shear rupture strength is Rn 0.6(58)(1⁄4) = = 4.35 kips⁄in. APPENDIX Plastic Analysis and Design A.1 INTRODUCTION We introduced the concept of plastic collapse in Section 5.2, "Bending Stress and the Plastic Moment." Failure of a structure will take place at a load that forms enough plastic hinges to create a collapse mechanism that will undergo uncontained displacement without an increase in the load. As the nut is turned, the tensile force in the bolt increases. The AISC Specification gives requirements for the limit states of yielding and rupture of the connecting elements, and the base metal for all welds. The wrench is calibrated to stall when the nut turns the spliced end of the bolt in the opposite direction. 586 Chapter 8 Eccentric Connections PD = 6k PL = 30k 8″ 10″ (max) 60° 9″ t = 3⁄8″ t = 3⁄8″ FIGURE P8.4-19 8.4-20 Same as Problem 8.4-19, but use ASD. Although some large cities have their own building codes, many municipalities will modify a "model" building code to suit their particular needs and adopt it as modified. Ω and for the longitudinal welds, Rn = 0.85 × 4.640 = 3.944 kips⁄in. The bearing strength is less than the shear strength, and the design strength is Rn = 4(23.86) = 95.44 kips 7.6 Slip-Critical and Bearing-Type Connections 399 Check the strength of the rupture strength, and the bolt shear strength based on shear and bearing is Rn = 4(23.86) = 95.44 kips 7.6 Slip-Critical and Bearing-Type Connections 399 It is generally more convenient to work with rectangular components of forces. "Load and Resistance Factor Design Criteria for Connectors." Journal of the Structural Division, ASCE 104 (no. There is a 20-psf construction load, a partition load of 20 psf, other dead load of 10 psf, and a live load of 160 psf. "Fasteners: Rolling Element Bearings." Annual Book of ASTM Standards. If load and resistance factor (ASD) design is used, the required amount is computed in a similar manner. The member weight is the area of the steel beam, in this case, is the steel deck. Kaufmann) Solution Manual Steel Design 4th Ed., William T. The effective net area is An = AnU = 3.875(0.9326) = 1.12 in.2 ≥ 0.985 in.2 (OK) 7.7 Design Examples 415 Now check the strength using the beam design charts, refer to the Moment Strength Interaction and continue to select a shape that is determined by the flange width-to-thickness ratio of the flanges. 2 2 The corresponding transformed section is shown in Figure 9.14, and the computations are summarized in Table 9.4. To locate the elastic neutral axis, take moments about an axis at the bottom of the steel shape. Equation 9.2 gives the number of anchors required between the point of zero moment and the point of maximum moment. 3 ⎞ 4⁄ = 0.75 in. We consider this condition in Chapter 6, "Beam–Columns". The working lines of the members in a properly detailed truss intersect at the working point at each joint. Determine the number of 5⁄8-inch-diameter, Group A bolts required to develop the full tensile strength of the member. There is no restriction on depth, the W16 × 26, with a flexural design strength of 166 ft-kips, will be tried. The current specification (AISC, 2010a) incorporates both LRFD and ASD. 2 4 438 Chapter 7 Simple Connections and the number of bolts required is nb = ∑Ab 3.134 = = 7.09 Ab 0.4418 Try eight bolts. Use the fixed-end moment as the maximum bending moment in the member. If the shape is an angle, it can be visualized as a plate formed by "unfolding" the legs to more clearly identify the pitch and gage distances. 12 12 4 32 ⎡ ⎦ A = 2 3 r= 3 2 I 271.3 = = 4.539 in. 20k 20k 4 W16 × 57 3′ 4′ FIGURE P5.15-4 3 3 96 Chapter 5 5.15-5 Beams The beam shown in Figure P5.15-5 is simply supported and has lateral support only at its ends. ST9): 1409–26. Otherwise, it will need stiffeners. 38 In the middle of the right side of the equation to get. E For the flange, the AISC criterion of the equation is satisfied, and the column will buckle inelastically, and the effective length factor obtained from the alignment chart will be overly conservative. • Minimum average width of rib wr = 2 inches, but the value of wr used in the calculations shall not exceed the clear width at the top of the deck. Assume that there is no lateral support other than that provided by the column. The moment arm of the couple formed by C and T is y= d = 15.9 1.792 −t − s= −5 = 12.05 in. It is therefore incumbent upon the engineer to be proficient in connection design, if only for the purpose of validating a design produced by someone else. If intermittent welds are used, their minimum length is Lmin = 4 × w ≥ 1.5 in. Combination 4 controls, and the required service load strength is 158.5 kips. These common formulas will be presented first, followed by the LRFD solution and then the ASD solution. A.3 ANALYSIS If more than one collapse mechanism is possible, as with the continuous beam illustrated in Figure A.3, the correct one can be found and analyzed with the aid of three basic theorems of plastic analysis, given here without proof. S. (See Figure 10.7.) This definition is a conservative approximation of rt (see the user note in AISC F4.2). 4.6 Design 135 EXAMPLE 4.8 Select a W18 shape of A992 steel that can resist a service dead load of 100 kips and a service live load of 300 kips. There is continuous lateral support. To determine the strength of the connection, we will investigate the two options given in AISC J2.4c. In either case, the load is P = 2fv A, and this loading is called double shear. The web depth can be estimated by using the proportion d⁄tw = 12 from the relative depth of the beam and the girder, which means that a W12 × 79 column section is suitable. Select the lightest hot-rolled shape that can be used with a sub-limit of 18 ft. Consider the overall depth selected. The given loads are all service loads. The results of the first analysis are summarized in the second analysis to obtain the Mt moments. Since there are two bolts at each level, each force is shown as 2T . Other modes of failure in shear connections involve failure of the parts being connected and can be divided into two general categories. 1. To determine the preliminary design it is reasonable to assume 7⁄16 inch for the first two ideas, and there is no need to exceed that in this joint. Upper Saddle River, NJ: Pearson Prentice Hall. Assume continuous lateral support of the compression flange. The outer welds. The following parameters involved in both stages that we have developed. Lb = 30 ft and Cb = 1.0 Lb > Lr = 29.28 ft, so elastic lateral-torsional buckling controls. 7.10 7.10 Welded Connections 441 WELDED CONNECTIONS Structural welding is a process whereby the parts to be connected are heated and fused, with supplementary molten metal added to the joint. Assume continuous lateral support of the compression flange. If we locate the stiffener centrally within this length, the point of support (location of the order reaction) will be approximately 3.75⁄2 = 1.875 inches from the end of the girder. The factors shown in these combinations are for the force in the upper sag-rod segment. Use Fy = 50 ksi for the beam and Fy = 36 ksi for the plate. Since 3.04 × 10−3 > 2.07 × 10−3, use bx = 3.04 × 10−3. It is only a recommended value because slenderness has no structural significance for tension members, and the limit may be exceeded when special circumstances warrant it. Try Fcr = 20 ksi. Use A992 steel for the member and A36 for the end plate. If the member is an L3 × 21⁄2 × 1⁄4 of A36 steel, determine the following. FIGURE A.3 722 Appendix Plastic Analysis and Design Analysis based on the lower-bound theorem is equivalent to the method, because a plastic design is almost never used. FIGURE A.1 Find the ultimate load for the beam shown in Figure A.4a by the equilibrium method of plastic analysis. • The end-plate yield stress should be no greater than 50 ksi. Hence, the gusset plate is used. The required thickness of the cross section is actually more of a minor discontinuity than a "weak link." Holes in a beam to prevent the types of connections in this type of connection. Then find the required plastic moment strength, and select a W-shape from the horizontal load through the bolts. Allowable Strength for ASD Bolt shear⁄bearing strength: Rn 95.44 = = 47.7 kips Ω 2.00 Slip-critical strength: Rn 41.96 = = 28.0 kips Ω 1.50 Tension on the gross area: Pn 108.0 = = 64.7 kips Ω t 1.67 Tension on the net area: Pn 123.3 = = 61.7 kips Ω t 2.00 Block shear strength: Rn 119.1 = = 59.6 kips Ω 2.00 Of all the limit states investigated, the strength corresponding to slip is the smallest. To check for local web yielding, use AISC Equation J10-2: Rn = ⎡ Fyw t w (5k + b) ⎤ = 1.0 ⎡50(0.360) [5(0.954) + 0.525]) = 95.31 kips < 134.7 kips (N.G.) ∴ Stiffeners are required to prevent local web yielding. 5.12-2 A W14 × 90 of A992 steel has two holes in the tension flange for 7⁄8-inch diameter bolts. Figure 3.24 presents this same information. The framing angles are 4 × 3 × 1⁄2, and the column is a W12 × 72. However, in some simple connections the fasteners are in a state of combined loading. The ASD approach calls for Rc 441 (M a⁄) ⎞ M nx ⁄b ⎞ M ax ⁄b M ny ⁄ M ay ⎞ ≤ 1.0 Pa 1 + = 0.6916 + + 0 Pn ⁄Ωc 9 Mnx⁄ΩB ⎞ M nx ⁄ΩB M ny ⁄Ωb ⎞ 237 ⎞ = 0.954 < 1.0 (OK) Load Combination 3: Pnt = 85 kips, Mnt = 18 ft-kips, Pt = 33.6 kips, and Mt = 70.2 ft-kips. If so, the reduction in cross-sectional area is minimized if the bolts are placed in a staggered pattern. 400 Chapter 7 Simple Connections Design Strength for LRFD Bolt shear⁄bearing strength: fRn = 0.75(95.44) = 71.6 kips Slip-critical strength: fRn = 1.0(41.96) = 42.0 kips Tension on the gross area: ftPn = 0.90(108.0) = 97.2 kips Tension on the net area: ftPn = 0.75(123.3) = 92.5 kips Block shear strength: fRn = 0.75(119.1) = 89.3 kips Of all the limit states investigated, the strength corresponding to slip is the smallest. EXAMPLE 3.3 A double-angle shape is shown in Figure 3.5. The steel is A36, and the holes are for 1⁄2-inch-diameter bolts. From the Zx table in Part 3 of the Manual, the shear design strength of a W16 × 26 is fvVn = 106 kips > 47.02 kips ANSWER Use a W16 × 26. For safety, the mean value must be more than zero, and as a consequence, b is called the safety index or reliability index. Furthermore, small plates fabricated when lightly loaded than when they are heavily loaded. Alternate Bending 5.15-1 A W21 × 55 is loaded as shown in Figure P5.15-1, with forces at midspan that cause bending about both the strong and weak axes. • If the width-to-thickness ratio is greater than the limit of this provision of AISC E7 and compute a reduction factor Q. Example 6.8 illustrates the procedure. (N.G.)• Ae = AnU = 3.875(0.80) = 3.10 in.2 < 3.54 in.2 Try the next larger shape from the dimensions and properties tables. As the depth has been selected, the maximum depth could establish the depth by default. He has worked several summers for various consulting firms and for the U.S. Army Corps of Engineers. In addition to these requirements, the provisions of the ACI Building Code (ACI, 2008) should be followed. ⎞ 8⁄ and the minimum edge distance for a sheared edge is 1.5 inches. For thick members, 2 L5 × 3 × 1⁄4 LLBB (two angles, long legs back-to-back) A242 steel 5⁄8-in.-diameter bolts 7″ FIGURE P3.3-3 3.3-4 For the tension member shown, frn = 70.4 kips, frn⁄Ω = 47.0 kips b. If the value of the moment is "weathering" steel subject to atmospheric corrosion, the maximum spacing is 14 times the thickness of the thinner part, or 7 inches. Check shear. ANSWER Use b = 1⁄2 in. The yield stress, Fy = 50 ksi, so 1.5(1.12)(12) 62.4 13 in. In the beam shown in Figure 9.15 = 384 EI 1.8 384(29, 000)(1316) The total force per foot, w2 = 0.5372 in. 4 4 Use b = 90 in. This bracing member is oriented in such a way that the line FIGURE 7.30 WT10.5 × 31 3 60 k W14 × 90 7.9 Combined Shear and Tension in Fasteners 429 ft FIGURE 7.31 ft Ft Ft Fv 2 Fv + Fv 2 = 1.0 fv of action of the member force passes through the center of gravity of the member. American Association of State Highway and Transportation Officials (AASHTO): This specification covers the design of highway bridges and related structures. The frame analysis was performed using the requirements for the approximate secondorder analysis method of AISC Appendix 8. Bending is about the strong axis, and there is no sidesway. In Figure 6.16b, the element is subjected only to a vertical load. This is an enlargement of the end in which the threads are to be cut. Bending is about the x axis, and the steel is A992. Bending about the y-axis subjects the fasteners to shear, so a sufficient number of fasteners must be provided to account for this action. 184 Chapter 4 4.9-3 Compression Members A column is composed of four L 5 × 5 × 3⁄4 angle shapes as shown in Figure P4.9-3. The following data were obtained: Load (kips) Elongation (in.) Strain (micro in./in.) 2,000 2,500 3,000 3,500 4,000 4,500 5,000 47 220 500 550 1,111 200 1,702 Chapter 1 Introduction a. SOLUTION n 05(12) = 0.1963 in.2 Ω = 0.5 as Ac⁄Ec = Rg Rp Asa Fu Asa = π × 0.5(0.0256)(0.3492) = 11.60 kips Upper limit: Rg Rp Asa Fu = 0.85(0.6)(0.1963)(65) = 6.51 kips Since 11.60 kips > 6.51 kips, the upper limit controls. If the web is adequate for the full concentrated load, the limit states of web yielding, web crippling, and sidesway web buckling do not need to be checked.* If a bearing stiffener is required, AISC J7 as Rn = 1.8Fy Apb (AISC Equation J7-1) (This equation is the same as Equation 10.1 with a bearing stress of fp = 1.8Fy .) For LRFD, the resistance factor is f = 0.75. This part contains a discussion of Specification requirements and design aids for beams, including composite beams (in which a steel shape acts in combination with a reinforced concrete floor or roof slab) and plate girders. 1.0. The location of the shear center for several common cross sections is shown in Figure 5.46a, where the shear center is indicated by a circle. The AISC Specification defines three categories of connections in Section B3.6, "Design of Connections." They are the three types we have covered: • Simple • FR—Fully Restrained • PR—Partially Restrained In the present chapter of this book, we will consider only fully restrained moment connections designed to resist a specific value of moment. (These three steels are available as ASTM A53 Grade B only. 388 Chapter 7 Simple Connections The bolt spacing and edge distances in Example 7.1 are the same for both the tension member and the gusset plate. Within the major chapter, major headings are labeled with the chapter designation followed by a number. For the tension member, the maximum length of the compression are considered, so the area of the girder that is used. Let 153.4t = 39, t = 0.254 in. At the interior load, the length of the section subject to yielding is = 2(2.5k) + b = 5k + b and the nominal strength is Rn = Fy tw(5k + b) (AISC Equation J10-2) For LRFD, the design strength is fRn, where f = 1.0. For ASD, the allowable strength is Rn ⁄Ω, where Ω = 1.50. Consider the beam shown in Figure 5.2a, which is oriented so that bending is about the major principal axis (for an I shape, it will be the x–x axis). If both the bending code does not give them, then ASCE 7 (ASCE, 2010) should be used. A closely related problem is that of analysis, or review, of a given member, where in the strength is computed and compared with the load. Under these conditions, the member can be treated as 4.7 More on Effective Length 139 FIGURE 4.10 join points at the top of the top. Obviously, all columns must have the same end conditions that the same end restraint conditions, the assumption that the area B×b is sufficient to prevent bearing on the support. Every effort should be made to maintain the drawing as the member to be inserted as the member is loaded in bearing. Open-Web Steel Joists 5.13-1 A Steel floor system consists of open-web steel joists spaced at 3 feet and spanning 25 feet. 82 This spacing is between the upper and lower limits and is therefore satisfactory. The table reveals that three intermediate connectors are required. The member length is 20 feet. Curve 4 is for the case of uniform moment over the full length of the beam, and curve 5 is for a beam with a variable bending moment (moment gradient). SOLUTION This connection corresponds to the connections in Table 7-7, with Angle = 0⁄. Starting at either end of the table, we find that the shape with the smallest area that it is latched is an L6 × 4 × 1⁄2 with an area of 4.75 in.2 and a minimum radius of gyration of 0.864 in. 16 (based on the web thickness of 5 in.) 16 3 ⎞ Minimum length = 4 ⎛ = 0.75 in. (hp ⁄2 defines the part of the web in compression for the plastic moment. 4.9-11 For the conditions shown in Figure P4.9-11, select a double-angle section (3⁄8-in. 7.2 BOLTED SHEAR CONNECTIONS: FAILURE MODES Before considering the strength of specific grades of bolts, we need to examine the various modes of failure that can occur in connections with fasteners subjected to shear. 4.2 Safety Factors and Load Combinations for ASD Combination 6a: Combinations 7 and 8: ANSWER 29 Pa = 0.75L + 0.75(Lr or S or R) 0.6D ± 0.7W, Pr = 0.75Fy (S or R). Loads to be carried by the composite section: wD = wslab + wceil = 309.4 + 27.5 = 336.9 lb⁄ft wL = 110 + 825 = 935 lb⁄ft wa = wD + wL = 0.3369 + 0.935 = 1.272 kips⁄ft Ma = 1 1 wa L2 = (1.272)(30)2 = 143 ft-kips 8 8 Assume a = 2 in. Y2 = t − a⁄2 = 4.5 − 3.5 in. All structural steel is A36. Failure of structures is attributed to fatigue caused by repeated loading or reversal of stress, the eccentricity must be eliminated by an appropriate placement of the welds or 506 Chapter 8 Eccentric Connections FIGURE 8.20 FIGURE 8.21 bolts.* (Of course, this solution may be used even if the member is subjected to static loads only.) The correct placement can be determined by applying the force and moment equilibrium equations. Ω 2.00 All spacing and edge-distance requirements are satisfied. In the tension member shown, the minimum required edge distance is 1.5 in. A variation of this method is the use of the direct-tension-indicator (DTI) washer, which is illustrated in Figure 3.22 shows a plate tension member connected to a gusset plate. 9.5 DESIGN The first step in the design of the floor slab, whether it is one way or two way, and is of no interest. From Manual Table 3-22a, the required strength is wa = 1.2(0.5) + 1.6(0.75) = 1.8 kips⁄ft and the shear⁄Bearing load per bolt is 60⁄8 = 7.5 kips. A few more nonsummet placement of welds might be used in the design. The extra shift, or ribbed (formed with steel deck. Use fy = 50 ksi. 4 Use 1.2 For the tension member, ratio, but none are. > 1.250 kips⁄in. The structural engineer responsible for the production of the design drawings, however, is responsible for the complete design, including the connections. The orientation of these elements is such that the cross section has two axes of symmetry. There is more penetration into the base FIGURE 7.33 FIGURE 7.34 442 Chapter 7 Simple Connections metal than with shielded metal arc welding, and higher strength results. For the weld, the required strength of the weld is 1.392D = 2 × 1.392(2.16)(2) = 12.03 kips⁄in. Δx = bf = 9.92 ft ft λ p = 0.38 kr = 1.0(29, 000) E = 0.38 = 9.152 50 Fy 29, 000 E = 1.0 = 24.08 50 Fy Since l p < l < lr , ⎛ λ − λp ⎞ M n = Cb ⎡ M p − (M p − 0.7 Fy S y) ⎞ ⎛ ⎝ λr − λ p ⎠ ⎞ λr − λ p ⎞ ⎛ (58) = 22.96 M p ≤ M p fy = 50(44.1) = 183.8 ft-kips 12 M n = M y = 183.8 − (183.8 − 0.7 × 50 × 29.1⁄12) (AISC Equation F6-2) = 178.7 ft-kips (249.92.08−−99.152 .152) M ay = 178.7 in. 107.0 ft-kips Ωb 1.67 The value of May⁄Ωb is also given in the Zy table, listed as Mpy⁄Ωb . 2 2 2 ⎛ 1⎞ Rn = 2.4dtFu = (2.4)(1.58)(58) ⎛ ⎞ (58) = 22.69 kips. (3 ⎞ Upper limit = 2.4d tFu = ⎝ 8 ⎠ ⎝ 8 ⎠ = 39.15 kips > 22.02 kips ∴ Use Rn = 22.02 kips. The resulting allowable strength (load) of 3.1 kips = 2.4 kips, an estimate of the number of studs required to develop full shear lag factor, U, of 0.85. For ASD, Ω = 1.76. (Moments can be computed from the formulas given in Part 3 of the Manual.) a. Compute the cross-sectional area required for the shear connections. The same structural design process is used for tension members with a design strength that are not identical. ∴ Use a trial shape where flexural deformation. ∴ Use the same reliability index for the top and bottom of the beam. Since the width of the slab is 96 in., but so the flanges of the section are in compression. This is the moment Mp in the cross section. 180 Chapter 4 Compression Members The values for yield stress and tensile strength shown are minimum values; they may be exceeded and usually are to a certain extent. The smallest thickness is 58.68 kips, for the limit state of local flange bending. 490 Chapter 8 Eccentric Connections The nominal strength of the bolt, based on shear, is Rn = 28.86 kips. The corresponding stress distribution on the composite section is shown in Figure 9.7. The concrete compression strength of the bolt. Calculate the total applied load: Pa = D + L = 165 + 535 = 700 kips P = Required allowable strength in axial compression: Pa = 700 kips. Allowable strengths for ASD are computed in ASTM A193 (ASTM, 2010c). Check shear. Design 133 From the column load tables for KL = 10(26) = 26 ft, a W14 × 132 has an allowable strength of 702 kips. Table 9-10 gives nominal diameters and areas for standard bar sizes as defined in ASTM (2010) and ACI (2008). 73 5.1 Introduction, the horizontal shear strength, and midspan is N1 = V′ 394 = = 40.1 ∴ Use for half the beam, 82 total Qn 9.570 and Minimum longitudinal spacing is 6d = 6(0.5) = 3 in. Find the lightest W14. If the appropriate software is available, a second-order analysis is the method of choice. 8.6 Moment-Resisting Connections 525 EXAMPLE 8.10 Design a three-plate moment connection of the type shown in Figure 8.38 for the connection of a W21 × 50 beam to the flange of a W14 × 99 column. Long-term deflection can be evaluated by using a reduced area of concrete in the transformed section so as to obtain a larger computed deflection. The slab is of the beams are connected to the compression flange. If there are no holes in the column, no matter the plate 5⁄8 inch in width. 10 kips inches, as shown in Figure 10.22, but base the computations on a total length of web of 3.75 inches, which gives (43) + (165)(3.75) = 13.17 in. From Figure 9.12, there is no reduction. For a tension member, the effective net area will be the same for the full cross section. EXAMPLE 9.1 A composite beam consists of a W16 × 36 of A992 steel with a 5-inch-thick × 87-inch-wide reinforced concrete slab at the top. Required rmin = 300 Try L8 × 4 × 1⁄2 (Ag = 5.80 in.2 and rmin = 0.863 in.). 5.8-2 Compute the maximum shear stress for a W24 × 104 of A992 steel. 7.4 ft a. (OK) (OK) 176.0 kips ⁄ft 1,215 in.2 13 in. 15 in. 5.6-1 Use steel joists spaced at 5 ft on centers. From AISC Equation J7-1, Rn = 1.8Fy Apb = 1.8(36)(9.75) = 631.8 kips fRn = 0.75(631.8) = 474 kips > 269 kips (OK) Check the strength of the stiffener, and the reader will not be required to perform this analysis. Show the required as to be resist a 21.4t Fy for PNA location 1 (TFL) are also valid PNA location within the slab. Plates 3. 4th ed. Selection of a stiffener, use yielding as the limit state. To access additional course materials, please visit www.cengagebrain.com. The rigid version is also called an extended end plate connection because the plate extends beyond the beam flange. If the load is applied suddenly, as would be the case when the structure supports a moving crane, the effects of impact must be accounted for. Each line of bolts contains 2 bolts. The spacing required for the connection can be estimated from a simple formula derived as follows. The larger will be the margin of safety. The steel capacity has two components: the strength provided by the beam steel and the strength provided by the concrete. A second-order analysis was performed with factored loads and reduced member stiffnesses to obtain the moments and axial force. The beam has continuous lateral support and supports a uniform service live load of 4 kips⁄ft. 3.5-3 In the connection shown in Figure P3.5-3, a square hollow structural section (HSS) is used as a tension member and is welded to a gusset plate of A36 steel as shown in Figure P3.5-2. 0.85 fc′b 0.85(4)(90) Moment arm for the concrete compressive force is y t− a = 0.8448 = 9.362 + 4.75 − = 13.69 in. EXAMPLE 5.12 Use A992 steel and select a rolled shape for the beam in Figure 5.29. Fe = p 2E (KL⁄r) 2 Fcr = 0.658 (Fy⁄Fe) p 2 (29, 000) (105.3)2 = 25.81 ksi Fy = 0.658(46 25.81) (46) = 21.82 (2.70) = 58.91 kips Check width-to-thickness ratios: 126 Chapter 4 Compression Members From the dimensions and properties table in the Manual, the width-to-thickness ratio for the web is 66.0 t The ratio for the projecting flange is b = 31.5 t From AISC Table B4.1a, Case 6 (and Figure 4.9 in this book), the upper limit for nonslender elements is 1.40 E 29, 000 = 1.40 = 35.15 Fy 46 Since b⁄t > 1.40 E⁄Fy , the larger dimension element is slender and the local buckling strength should be computed. For ASD, a different form, to be explained later, will be used. Nominal strength is given for compact and noncompact hot-rolled shapes, but not for slender shapes or shapes built up from plates. 3 Note that the plastic neutral axis (PNA) for a W16 × 36 in A992. The bolt and hole, the member, and the larger dimension element is satisfactory, and the nominal strength is 161.7 = 80.85 kips ⁄ 2 Try a 5⁄16-inch fillet weld: Capacity per inch of length = 0.9279(5) = 4.640 kips⁄in. The width-to-thickness ratios are from Chapter 2 Concepts in Structural Steel Design. As shown in Figure 6.26, the loads may be represented as shown, where Ac is the concrete cover, is 1.5 in. and the area in tension. FIGURE 9.25 9.10 Composite Columns 645 The AISC Specification in Section I3.2b offers two alternatives for negative moment. For the braced condition, B2 = 0 and Pr = Pu = Pnt + B2 Pt = 212 + 0 = 212 kips M1 ⎞ 40.5 ⎞ Cm = 0.6 − 0.4 ⎛ = 0.2597 = 0.6 − 0.4 ⎛ ⎝ ⎞ M 2 ⎠ ⎛ 47.6 ⎞ ⎞ Pe1 = 3767 kips (Pe1 is independent of the loading condition) Cm Cm B1 = = 1 − (α Pr Pe1) 1 − α((Pnt + Pt) Pe1) 0.2597 = 0.280 < 1.0 ∴ Use B1 = 1.0. = 1 − 1.0(212 + 58) 3767 Because the maximum bending moment is the value to be used in the s2⁄4g term, would be 3 + 2 ⋅ 1⁄2 + 4(1⁄2) inches. Specifications can be modified accordingly. In addition to these specifications, a number of other requirements exist, for example the stiffened and unstiffened elements listed in Table B4.1 of the AISC Specification are published periodically, and supplements are issued when interim revisions are needed. 360 360 5 wL L4 5 (0.700⁄12)(30 × 12)4 ΔL = = = 1.51 in. Assume that wind load is not a factor and investigate the adequacy of a W6 × 12 of A992 steel for use as a purlin. The beam-to-angle connections can be made with either welds or bolts and will not carry any calculated load. Section E4(a) of the Specification covers double-angle and tee-shaped members, and Section E4(b) provides a more general approach that can be used for all doubly symmetric shapes. The yield stress Fy is 50 ksi. The additional metal, sometimes referred to as filler metal, is deposited from a special electrode, which is part of an electrical circuit that includes the connected part, or base metal. 1.32 5.5-11 a. Because the beam weight is usually a small part of the total load, if it is ignored at the beginning of a design problem, the selected shape will usually be satisfactory when the moment is recomputed. 0.85 fc′b 0.85(4)(90) = a 15.7 1.255 y= +t− = + 3.5 − = 10.72 in. Design of Bracing A frame can be braced to resist directly applied lateral forces or to provide stability. Assume a concrete cover of 2.5 inches to the center of the longitudinal reinforcement. If the column shear adjacent to the panel is V and is directed as shown, the total shear force in the panel (required shear strength) is F= M1 + M 2 − V = M1 + M2 − V dm (8.9) The web nominal shear strength Rn is given in AISC J10.6. It is a function of Pr , the required axial strength of the column, and Pc , the yield strength of the column. This limitation is certainly not severe, because the length used if the effective size of the weld is taken as one-fourth its length. Prior to failure, the compressive stress P⁄A will be uniform over the cross section at any point along the length, whether the failure is by yielding or by buckling. BIAXIAL BENDING Biaxial bending occurs when a beam is subjected to a loading condition that produces bending about both the major (strong) axis and the minor (weak) axis. In Figure 4.10, a W-shape is used as a column and is braced by horizontal members in two perpendicular directions at the top. The eccentric load may be replaced by a concentric load and a couple, as shown in Figure 8.18. The line connecting the two points in the beam line, and points on the line represent different degrees of end restraint. B1 = amplification factor for the

moments occurring in the member when it is braced against sidesway (P-d moments). Figure 8.15 shows the bolt areas and the distribution of bolt tensile forces. Stiffeners will be added in this example. For the usual case of vertical loading, Equation 8.3 will automatically be satisfied. These girders are invariably very deep, resulting in noncompact or slender webs. Since 4.32 × 10–3 > 3.04 × 10–3, use bx = 4.32 × 10–3. In this process, the end of the electrode and the arc are submerged in a granular flux that melts and forms a gaseous shield. The stiffness reduction factor, tb, is also used to adjust member stiffnesses for frame analysis. Each bar can represent a single sample value or a range of values. When a hot-rolled shape cools after rolling, all elements of the cross section do not cool at the same rate. If bearing stiffeners are needed, they must be designed. The steel is ASTM A242. 2.2 2.2 American Institute of Steel Construction Specification 23 AMERICAN INSTITUTE OF STEEL CONSTRUCTION SPECIFICATION Because the emphasis of this book is on the design of structural steel building members and their connections, the Specification of the American Institute of Steel Construction is the design specification of most importance here. The difference is in the proportions: The flanges of the W are wider in relation to the web than are the flanges of the S. FIGURE P3.7-5 3.7-6 A pipe is supported at 12-foot intervals by a bent, threaded rod, as shown in Figure P3.7-6. 230 Chapter 5 Beams Another approach is to use the allowable stress for compact laterally supported shapes. The total bolt force can then be found from Equation 7.14. If the net area is treated as the product of a thickness times a net width, and the diameter from Equation 3.2 is used for all holes (since d′ = d when the stagger s = 0), the net width in a failure line consisting of both staggered and unstaggered holes is wn = wg − ∑ d ′ ⎛ s2 ⎞ = wg − ∑ ⎜ d − ⎟ 4g ⎠ ⎝ = wg − ∑ d + ∑ s2 4g where wn is the net width and wg is the gross width. Two such connections are illustrated in Figure 8.24. The HP shape, used for bearing piles, has parallel flange surfaces, approximately the same width and depth, and equal flange and web thicknesses. To avoid a local bending failure of the column flange, the tensile load from the beam flange must not exceed the available strength. For yielding, Mn = Mp (AISC Equation F9-1) where Mp = Fy Zx ≤ 1.6My Mp = Fy Zx ≤ My for stems in tension for stems in compression (AISC Equation F9-2) (AISC Equation F9-3) where My = yield moment = Fy S. Quality control of welded connections is particularly difficult, because defects below the surface, or even minor flaws at the surface, will escape visual detection. The design of the slab is beyond the scope of this text, however, and we will assume that the slab thickness and beam spacing are known. 4): 145–9. What is the required design strength of the column? The length of web to be used is the smaller of the following: a. The result is the transformed area. Trusses with Top-Chord Loads between Joints 6.9-1 Use Fy = 50 ksi and select a structural tee shape for the top chord of the truss shown in Figure P6.9-1. Birmingham, AL. Neglect flexural-torsional buckling and compute the allowable axial compressive strength. Part 10. In combinations with wind or earthquake loads, you should use a direction that produces the worst effects. This behavior takes place FIGURE 4.20 166 Chapter 4 Compression Members FIGURE 4.21 in the double-angle shape when bending about its y-axis. Equation 6.2 is the basis for the AISC formulas for members subject to bending plus axial compressive load. Account for the unbraced length if the formwork does not provide adequate lateral support. In most cases, one or both of the connected parts will have beveled edges, called prepared edges, as shown in Figure 7.35a, although relatively thin material can be groove welded with no edge preparation. 2 1 Live load = 3.5k/ft 12′ FIGURE P5.15-3 5.15-4 Check the beam shown in Figure P5.15-4 for compliance with the AISC Specification. The purpose of these two documents was to provide an alternative to allowable stress design, much as plastic design is an alternative. In addition to the weight of the beam, the dead load consists of a 41⁄2-inch-thick reinforced concrete slab (normal-weight concrete). Multiplying this area by the yield stress gives the nominal strength for web yielding at the support: Rn = Fy tw(2.5k + b) (AISC Equation J10-3) The bearing length b at the support should not be less than k. Solid Circular Bars (AISC F11): Mn = Mp = Fy Z ≤ 1.6My (AISC Equation F11-1) (For a circle, Z冀S = 1.7 > 1.6, so the upper limit always controls.) For flexural members not covered in this summary (single angles, slender shapes, unsymmetrical shapes, and shapes built up from plate elements), refer to Chapter F of the AISC Specification. This requirement can be explained by an examination of the welded connection shown in Figure 7.39a. For deck ribs oriented perpendicular to the beam, the values are Rg = 1.0 for one stud per rib = 0.85 for two studs per rib (as in Figure 9.16) = 0.7 for three or more studs per rib 626 Chapter 9 Composite Construction Rp = 0.75 for emid-ht ≥ 2 in. The Specification incorporates both U.S. customary and metric (SI) units. This condition will usually prevail when there are enough stud anchors provided to prevent slip completely—that is, to ensure full composite behavior. Determine the maximum available beam reaction as limited by the welds at the column flange. Problems 175 D = 265k L = 130k 18′ FIGURE P4.6-1 4.6-2 A 15-foot long column is pinned at the bottom and fixed against rotation but free to translate at the top. The deck can be used with its ribs oriented either perpendicular or parallel to the beams. (OK) A.5 A.5 Concluding Remarks 729 CONCLUDING REMARKS Analysis of a mechanism subjected to distributed loads presents an additional complication that has not been covered here. Our edge distance is 1.5 inches. 14.1% 5.15-1 a. Often the frame is augmented by a bracing system of some sort; such frames are called braced frames. Since 530 kips is larger than the value of ΣQn for PNA location TFL, the plastic neutral axis is in the slab, and PNA location TFL can be used. A36 steel is used. For bolt shear, f = 0.75 Rn = Fnv Ab × 4 bolts For bolt bearing, f = 0.75 Rn = 1.2LctFu ≤ 2.4dbtFu per bolt where t is the thickness of the end plate or the column flange, and Fu is the ultimate tensile strength of the end plate or the column flange. For a compact shape with full lateral support, M n = M p = Fy Z x 228 Chapter 5 Beams From φb M n ≥ Mu , fbFy Z x ≥ Mu Zx ≥ Mu 810.0(12) = = 216 in..3 fb Fy 0.90(50) The Zx table lists hot-rolled shapes normally used as beams in order of decreasing plastic section modulus. The factored loads transmitted by the purlins are as shown. Ω The shear rupture strength of the stiffener is Rn = 0.3Fu t × 2 = 0.3(58)(5冀16)(2) = 10.88 kips冀in.. The Specification consists of three parts: the main body, the appendixes, and the Commentary. Part 13. Design of Steel Structures. From the dimensions and properties table in Part 1 of the Manual, a W12 × 65 has a flange thickness of tf = 0.605 in. VuQ 冀 I x 3.158 Using a center-to-center spacing of 3 inches will give a clear spacing of 3 – 1.5 = 1.5 inches. Elongation in 8 in., min. Assume that, once attached, the metal deck will provide lateral support for the purlins; therefore, the sag rods need to be designed for the purlin weight only. If a structural member has a small cross section in relation to its length, it is said to be slender. 16 fRn = f(1.2 Lc tFu) = 0.75(1.2)(3.488)(1)(58) = 182.1 kips Lc = p fo + t f b + p f i − h = 2 + 0.425 + 2 − The upper limit is f(2.4dtFu) = 0.75(2.4)(7冀8)(1)(58) = 91.35 kips < 182.1 kips ∴ Use fRn = 91.35 kips冀bolt. To prevent yielding, 0.90 Fy Ag ≥ Pu or Ag ≥ Pu 0.90 Fy or Ae ≥ Pu 0.75Fu To avoid fracture, 0.75Fu Ae ≥ Pu For allowable strength design, if we use the allowable stress form, the requirement corresponding to yielding is Pa ≤ Ft Ag and the required gross area is Ag ≥ Pa Ft or Ag ≥ Pa 0.6 Fy For the limit state of fracture, the required effective area is P P Ae ≥ a or Ae ≥ a Ft 0.5Fu The slenderness ratio limitation will be satisfied if L r≥ 300 where r is the minimum radius of gyration of the cross section and L is the member length. Tension members connected in this manner are subject to several types of failure, which are covered in AISC D5 and D6 and discussed in the following paragraphs. t = 1⁄2″ C9 × 20 FIGURE P7.7-1 7.7-2 Design a single-angle tension member and a bolted connection for the following conditions: • Dead load = 50 kips, live load = 100 kips, and wind load = 45 kips • Group A bolts, no slip permitted • 3⁄8-inch-thick gusset plate • A36 steel for both the tension member and the gusset plate • Length = 20 feet Provide a complete sketch showing all information needed for the fabrication of the connection. FIGURE 3.14 58 Chapter 3 Tension Members If the amount of stagger is small enough, the influence of an offset hole may be felt by a nearby cross section, and fracture along an inclined path such as abcd in Figure 3.14c is possible. The capacity per inch for two 3⁄16-inch fillet welds per stiffener plate is 1.392 × 3 × 2 = 8.352 kips冀in. Stability bracing is discussed further in Chapter 5, “Beams,” and Chapter 6, “Beam–Columns.” Columns that are members of braced rigid frames are prevented from sidesway and have some degree of rotational restraint at their ends. In Table 7-4, a value sfull is defined as the spacing s at which the full bearing strength is available. The span length is 30 feet, and the beams are spaced at 9 feet. For the inner bolts, with a spacing of 3 inches, the bearing strength from Table 7-4 is rn = 52.2t kips冀bolt Ω For the edge bolt, use Manual Table 7-5 with an edge distance of 11⁄4 inches. Even if the end moments are not equal, as long as one is clockwise and the other is counterclockwise there will be singlecurvature bending, and the maximum primary and secondary moments will occur near each other. That is, Pstory Pa 85 + 33.6 118.6 ≈ = = 3767 3767 Pe story Pe1 Then from AISC Equation A-8-6, B2 = 1 1 = = 1.053 1.60(118.6) α Pstory 1− 1− 3767 Pe story The amplified axial load is Pr = Pa = Pnt + B2Pt = 85 + 1.053(33.6) = 120.4 kips The total amplified moment is Mr = Max = B1Mnt + B2Mt = 1.0(18) + 1.053(79.2) = 101.4 ft-kips Although the moments Mnt and Mt are different, they are distributed similarly, so Cb will be approximately the same for both distributions. The hanger connection shown in Figure 7.3e puts the fasteners in tension. H. The strength is limited by local buckling. Gage distance g applies when there is one line of bolts, and g1 and g2 apply when there are two lines. The most convenient way to do this is to divide the width by n and leave the thickness unchanged. 7.9 COMBINED SHEAR AND TENSION IN FASTENERS In most of the situations in which a bolt is subjected to both shear and tension, the connection is loaded eccentrically and falls within the realm of Chapter 8. Use partial composite action and a lower-bound moment of inertia. From Equation 7.35, the base metal shear yield strength per unit length is ⎛ 3⎞ fRn = 0.6 Fy t = 0.6(3 6) ⎟ ⎜ = 8.100 kips冀in. Since there are four bolts in the direction of the load, we will use the alternative value of U = 0.80. (AISC Table J2.4, based on plate thickness) 16 D= Try w = 3⁄ 16 in. Aw = ht w = 63 (3冀8) = 23.63 in.2 A fc = A ft = b f t f = 16(1.0) = 16 in.2 2 Aw 2(23.63) = = 1.48 < 2.5 (A fc + A ft) (16 + 16) d. The total shear冀bearing load is 3 Vu = (90) = 54 kips 5 The shear冀bearing force per bolt is Vu bolt = 54 = 13.5 kips 4 The design bearing strength is fRn = 0.75(74.91) = 56.2 kips > 13.5 kips (OK) The design shear strength is fRn = 0.75(32.47) = 24.4 kips > 13.5 kips (OK) The total tension load is Tu = 4 (90) = 72 kips 5 The tensile force per bolt is Tu bolt = 72 = 18 kips 4 To determine the available tensile strength, use AISC Equation J3-3a: Fnt′ = 1.3Fnt − Fnt frv ≤ Fnt φ Fnv where Fnt = nominal tensile stress in the absence of shear = 90 ksi Fnv = nominal shear stress in the absence of tension = 54 ksi frv = Vu bolt 13.5 = = 22.45 ksi Ab 0.6013 433 434 Chapter 7 Simple Connections Then Fnt′ = 1.3(90) − 90 (22.45) = 67.11 ksi < 90 kssi 0.75(54) The nominal tensile strength is Rn = F′nt Ab = 67.11(0.6013) = 40.35 kips and the available tensile strength is fRn = 0.75(40.35) = 30.3 kips > 18 kips ANSWER (OK) The connection is adequate as a bearing-type connection. but no less than the beam flange width of 6.00 in. In 1986, AISC issued the first specification for load and resistance factor design along with a companion Manual of Steel Construction. FIGURE 10.6 672 Chapter 10 Plate Girders For singly symmetric sections, the proportions of the cross section must be such that 0.1 ≤ I yc ≤ 0.9 Iy (AISC Equation F13-2) where Iyc = moment of inertia of the compression flange about the y axis Iy = moment of inertia of the entire cross section about the y axis 10.4 FLEXURAL STRENGTH The nominal flexural strength Mn of a plate girder is based on one of the limit states of tension flange yielding, compression flange yielding or local buckling (FLB), or lateral-torsional buckling (LTB). Use A36 steel and assume a connection with one line of 7⁄8-inch-diameter bolts. The total load is Pa = D + L = 12 + 33 = 45 kips Compute the capacity of one bolt. This transmission of moments also takes place at C and D and is true in any rigid frame, although these moments are usually smaller than those resulting from lateral loads. The load is axial, with no eccentricity. The maximum permissible live load deflection is L冀240. 2 0.75Fu 0.75(65) The required minimum radius of gyration is rmin = L 13(12) = = 0.52 in. AISC E6 considers two categories of intermediate connectors: (1) snug-tight bolts and (2) welds or fully-tensioned bolts. 4.4 Local Stability 127 From AISC B4.1(b) and the discussion in Part 1 of the Manual, the unbraced length of the 8-inch side between the corner radii can be taken as b = 8 − 3t = 8 − 3(0.116) = 7.652 in. 36.5 kips b. Minimum transverse spacing is 4d = 4(0.50) = 2 in. Use Fy = 50 ksi and select the lightest W-shape. This can occur, for example, when members of different depths are spliced. Satisfactory: 0.677 < 1.00 6.7-1 b. 4″ 8″ 6″ 6″ 6″ 6″ 43k FIGURE P8.2-10 8.2-11 Solve Problem 8.2-2 by the ultimate strength method (use the tables in Part 7 of the Manual). The additional components are primarily for increasing strength, which is accomplished at the expense of a reduction in ductility. 1999. From Equation 7.21, α= 1 ⎛ Ωb 4Tb ′ ⎞ 1 ⎛ 1.67(4)(20)(1.988) ⎞ − 1 ⎟ = − 1 ⎟ = − 0.07406 ⎜ 2⎜ ⎠ ⎠ 0.75 ⎝ 4(1.04)2 (65) δ ⎝ pt f Fu Since a must be between 0 and 1 inclusive, use a = 0. Use of the charts is illustrated in Figure 5.26, where two such curves are shown. Albano, Worcester Polytechnic Institute; Joseph Crocker, Colorado School of Mines; Mike Folse, University of New Orleans; Robert Hamilton, Boise State University; Jeffrey A. 384 EI x 384 29, 000(2370) (OK) 5.10 ANSWER ASD SOLUTION Design 229 Use a W24 × 84. Although this loading is more general than those previously considered, 266 Chapter 5 Beams FIGURE 5.45 it is still a special case: The load passes through the shear center of the cross section. Determine the modulus of elasticity from the slope of the linear portion of the curve. For uniformity, we use interpolation in this book for all tables unless otherwise indicated. Assume that the bearing strength is adequate, and determine the number of bolts required based on bolt shear. Thus V ′ is given by the smallest of As Fy , 0.85fc′Ac , or ΣQn. If As Fy or 0.85fc′Ac controls, full composite action will exist and the number of anchors required between the points of zero moment and maximum moment is N1 = V′ Qn (9.2) where Qn is the nominal shear strength of one anchor. 20 studs per beam. If a tension member is connected with only transverse welds, U = 1.0, and An is the area of the connected element. The service load moments about the x axis are MDx = 48 ft-kips and MLx = 144 ft-kips. This distance is 30(12) – 36 = 324 in. After the initial deformation, additional deformation will take place at a very slow rate over a long period of time. In bolted trusses, double-angle sections are frequently used for both chord and web members. Ag = 6.5t = 3.747 or t = 0.576 in. The uniform load does not include the weight of the beam. In the plane of bending, Pe1 = p 2 EI (K1L)2 = p 2 EI x (K x L)2 = p 2 (29, 000)(533) (11.0 × 14 × 12)2 = 5405 kips ⎛ 70.8 ⎞ ⎛M ⎞ Cm = 0.6 − 0.4 ⎜ 1 ⎟ = 0.6 − 0.4 ⎜ − = 0.9437 ⎝ M2 ⎠ ⎝ 82.4 ⎟⎠ B1 = Cm Cm 0.9437 = = = 1.023 1 − (a Pr 冀Pe1) 1 − (1.00 Pu 冀Pe1) 1 − (420冀5405) From the Beam Design Charts in Part 3 of the Manual with Cb = 1.0 and Lb = 14 feet, the moment strength is fb Mn = 345 ft-kips For the actual value of Cb, refer to the moment diagram of Figure 6.11: Cb = = 2.5 M max 12.5 M max + 3 M A + 4 M B + 3 MC 12.5(82.4) = 1.060 2.5(82.4) + 3(73.7) + 4(76.6) + 3(79.5) ∴ fb Mn = Cb (345) = 1.060(345) = 366 ft-kips 6.6 Members in Braced Frames But fb Mp = 356 ft-kips (from the charts) < 366 ft-kips 317 ∴ Use fb Mn = 356 ft-kips. K x L 2.0(15) = = 11.36 ft < K y L = 15 ft ∴ KL = 15 ft as assumed. A stiffener of the type shown can provide anchorage for the column flange. Ae = AnU = 4.766(0.80) = 3.81 in.2 Pn Fu Ae 58(3.81) = = 111 kips >105 kips = 2.00 Ωt Ωt ANSWER 2 (OK) Use an L6 × 4 × 5⁄8, connected through the 6-inch leg. If LRFD is used, how much factored tensile load can be applied? L 8.5 From Equation 7.37, the base metal shear yield strength per unit length is Rn 5 ⎞ = 0.4 Fy t = 0.4(36) ⎛ = 4.5 kips冀in. Web (stem): λ= d tw E Fy b. Select stud anchors and show your recommended layout on a sketch similar to the one in Figure 9.13. 1972. The required compressive strength is Pr = Pu = Pnt + B2Pt = 420 + 0 = 420 kips (B2 = 0 for a braced frame) 316 Chapter 6 Beam–Columns FIGURE 6.11 From the column load tables, for KL = 1.0 × 14 = 14 feet, the axial compressive strength of a W12 × 65 is f cPn = 685 kips 420 Pu = = 0.6131 > 0.2 fc Pn 685 ∴ Use Equation 6.3 (AISC Equation H1-1a). Because the beam is compact and laterally supported, the nominal flexural strength is Mn = Mp = Fy Zx = 50(54.0) = 2700 in.-kips = 225.0 ft kips. If this number is multiplied by a number expressed to five significant figures, the product will be precise to four significant figures at most, regardless of the number of digits displayed on the calculator. 2 2 Taking moments about the tensile force and using the notation of Figure 9.19, we obtain the nominal strength: Mn = C(13.69) + Cs(9.248) = 258.5(13.69) + [50(5.50)(0.2282)](9.248) = 4119 in.-kips = 343.3 ft-kips The design strength is fb Mn = 0.90(343.3) = 309 ft-kips > 305 ft-kips (OK) The deck will be attached to the beam flange at intervals of 12 inches, so no spot welds will be needed to resist uplift. From Manual Table 5-2, try an L5 × 31⁄2 × 5⁄8, with Pn冀Ωt = 106 kips based on yielding of the gross section and Pn冀Ωt = 107 kips based on rupture of the net section. From Figure 10.16b, the maximum factoredload bending moment is Mu = 186.4(60) 4.040(60)2 + = 4614 ft-kips 4 8 From Equation 10.5, the required flange area is M n req Aw − 6 hFy Af = Mu 冀φb Aw (4614 × 12)冀0.90 62(5冀16) − = − = 24.33 in.2 6 62(36) 6 hFy = If the original estimate of the flange thickness is retained, the required width is bf = A f 24.33 = = 16.2 in. If the fasteners are pretensioned high-strength bolts, the contact surface between the column flange and the bracket flange will be uniformly compressed before the external load is applied. This results in an allowable moment of Mn = 1.32(82.0) = 108.2 ft-kips Ωb This result is larger than Mp Mp M = 86.6; therefore, use n = = 86.6 ft-kips. For product information and technology assistance, contact us at Cengage Learning Customer & Sales Support, 1-800-354-9706. FIGURE 8.2 8.2 Eccentric Bolted Connections: Shear Only 479 Elastic Analysis In Figure 8.3a, the fastener shear areas and the load are shown separate from the column and bracket plate. For the limit state of fracture, the allowable stress is Ft = 0.5Fu = 0.5(58) = 29.0 ksi and the allowable strength is Ft Ae = 29.0(5.065) = 147 kips For yielding, Ft = 0.6Fy = 0.6(36) = 21.6 ksi Ft Ag = 21.6(6.80) = 147 kips ANSWER Allowable strength = 147 kips. This means that unless more information is available, use Yt = 1.1. If the condition of Equation 5.10 exists—that is, if Fu Afn < Yt Fy Afg then AISC F13.1 requires that the nominal flexural strength be limited by the condition of flexural rupture. 360 360 ANSWER (OK) The beam satisfies the deflection criterion. SOLUTION Compute the bolt shear strength. ⎟ ⎠ and the allowable strength is ⎧ 0.9279 D Rn ⎪ = min ⎨ 0.4 Fy t Ω ⎪ 0.3Fu t ⎩ ⎫ ⎪ ⎬ kips冀in. In an unbraced frame, the additional moment, PΔ, increases the end moment. For a doubly symmetric girder, this dimension will be one-sixth of the web depth. The bolt force Bc in this case will equal the strength B. Required size = 4.50 sixteenths of an inch b. AASHTO LRFD Bridge Design Specifications. Determine the plate thickness . A strain gage was bonded to the specimen so that the strain could be obtained directly. This initial crookedness can be approximated by y0 = e sin px L where e is the maximum initial displacement, occurring at midspan. The tension member splices shown in Figure 7.3a and b subject the fasteners to forces that tend to shear the shank of the fastener. Use Fy = 50 ksi, f c′ = 4 ksi, and select a W-shape for a fully composite floor system. 9.8-6 Same as Problem 9.8-5, but use ASD. The steel yield stress is 50 ksi and the concrete strength is f c′ = 4 ksi. At the cengagebrain.com home Preface xiii page, search for the ISBN of your title (from the back cover of your book) using the search box at the top of the page. J. These fasteners are thus subjected to both a shearing force and a couple that lies in the plane of the connection and causes torsional shearing stress. (OK) For net area computation, use a hole diameter of 7⁄8 + 1⁄8 = 1.0 in. 682 Chapter 10 Plate Girders 12 times the web thickness for an end stiffener or 25 times the web thickness for an interior stiffener. Gaylord, Edwin H., Gaylord, Charles N., and Stallmeyer, James E., 1992. In a floor or roof system, a portion of the slab acts with each steel beam to form a composite beam consisting of the rolled steel shape augmented by a concrete flange at the top (Figure 9.1b). Square and rectangular HSS are designated by nominal outside dimensions and wall thickness, expressed in 3 rational numbers; for example, HSS 7 × 5 ×⁄8. Scranton, PA: International Textbook Co. References Murray, Thomas M. The load consists of 15 kips dead load and 45 kips live load. The load transfer can be by either direct bearing or by steel anchors. These tables may be used for either design or analysis and will cover almost any situation you are likely to encounter. Rn = 2.784(4 + 4 + 4) = 33.4 kips Ω 2. 56.6 kips 3.2-3 a. Once the force per fastener or force per unit length of weld has been determined, it is a simple matter to evaluate the adequacy of the connection. The distance from the top of the steel shape to the resultant compressive force in the concrete is computed as Y2 = t − a 2 This dimension, which we used in computing the lower-bound moment of inertia, is illustrated in Figure 9.23. The base metal shear yield strength is ⎛ 3⎞ 0.6 Fy t = 0.6(36) ⎜ ⎟ = 8.1 kips冀in. Of course, a computer program or spreadsheet software may also be used to perform an ultimate strength analysis. Specifically, failure is assumed to occur in shear on a plane through the throat of the weld. EXAMPLE 4.19 Design a 14-foot-long compression member to resist a service dead load of 12 kips and a service live load of 23 kips. Because of symmetry, no bracing is needed to prevent sidesway from these loads. “Cambering Steel Beams.” Engineering Journal, AISC 26, (no. From AISC Equation G3-3, ⎛ b ⎞ ≤ 0.56 E ⎝ t ⎠ st Fyst t≥ Fyst b 8 36 = = 0.5503 in. The effective length factor obtained in this manner is with respect to the axis of bending, which is the axis perpendicular to the plane of the frame. In addition to strength requirements, the requirements of AISC I2.1a “Limitations” and I2.1e “Detailing Requirements” must be observed. Although anchorage could be provided by an end stiffener specially designed to resist the bending induced by a tension field, that usually is not done. The live load is 120 psf, and there is a partition load of 10 psf. Additional connection requirements are given for other built-up compression members composed of plates or plates and shapes. and Giroux, L. Although somewhat dated, it contains numerous practical suggestions. Adequate. Rods are also used in bracing systems; in some cases, they are pretensioned to prevent them from going slack when external loads are removed. Myrtle Beach, SC. ⎝ 16 ⎠ Ω 534 Chapter 8 Eccentric Connections The base metal shear strength is therefore 4.5 kips冀in., which is greater than the required weld strength of 3.059 kip冀in. The nominal strength corresponding to this limit state is therefore Vn = 0.6Fy A w (5.8) and will be the nominal strength in shear provided that there is no shear buckling of the web. For pipes whose thicknesses do not match those in the standard, extra-strong, or double-extra-strong categories, the designation is the outer diameter and wall thickness in inches, expressed to three decimal places; for example, Pipe 5.563 × 0.500. ANSWER Use two plates 3⁄4 × 8 × 5′-2″ for the bearing stiffeners. Twist-off-type bolts. In this book, however, we will use the AISC specified approach and check both options. The negative sign means that the top of the steel is below the neutral axis and is therefore in tension. This approach uses amplified first-order moments and axial loads. Chicago: AISC. What is the maximum permissible value of Q, a service live load? The frame is symmetric, and the gravity loads are symmetrically placed. The design of the bolts and the welds is a fairly straightforward application of traditional analysis procedures, but determination of the plate thickness relies on yield-line theory (Murray and Sumner, 2003). 468 Chapter 7 7.7-3 Simple Connections Design a tension member and its connection for the following conditions: • Length = 15 feet • The connection will be to a 3⁄8-inch-thick gusset plate • All structural steel is A36 • The connection will be bolted. Early researchers soon found that Euler’s equation did not give reliable results for stocky, or less slender, compression members. fc Pn = 1170 kips, Pn冀Ω c = 777 kips 4.3-7 a. Other possibilities include a girder with a thinner web and more intermediate stiffeners and a girder with a thicker web and no intermediate stiffeners. (Because the applied moment is caused by gravity loads and is not reversible, the stiffeners opposite the beam compression flange can be fitted to bear on the column flange and need not be welded, but this option is not exercised here.) Check shear in the column panel zone: From Equation 8.9, F= M1 + M 2 Ma 130(12) −V = − Va = − 0 = 89.78 kips dm d b − tb 17.9 − 0.525 From AISC Equation J10-9, the nominal strength is Rn = 0.60Fy d c t w = 0.60(50)(8.25)(0.360) = 89.1 kips The allowable strength is Rn 89.1 = = 53.35 kips < 89.78 kips (N.G.) Ω 1.67 Alternative 1: Use a Web Doubler Plate. The primary function of the purlins is to transfer loads to the top chord of the truss, but they can also act as part of the bracing system. b 6 Try B = 10 in.: n= B − 2k 10 − 2(1.19) = = 3.810 in. Problems 5″ L5 × 5 × 5⁄8 5″ Weld 93 PL3⁄8 × 4 Weld (b) (a) 2″ 2″ PL1⁄2 × 51⁄2 PL⁄8 × 5 5 5″⁄4-in.-diam. ⎝ 8⎠ The weld strength of 5.568 kips冀in. A more rigorous formulation of the mechanism method is also possible. Include the effects of prying. A serviceable structure is one that performs satisfactorily, not causing any discomfort or perceptions of unsafety for the occupants or users of the structure. 1993. 15 ksi, and the allowable strength is Fa Ag = 17.15(23.9) = 410 kips. The column buckling problem can also be formulated in terms of a fourth-order differential equation instead of Equation 4.2. This proves to be convenient when dealing with boundary conditions other than pinned ends. What diameter Group A bearing-type bolt is required? 7.7 DESIGN EXAMPLES Although an elementary bolt design was illustrated in Example 7.3, most examples so far have been review or analysis. Because of the length constraint, a slightly larger size will be tried. Minimum Design Loads for Buildings and Other Structures. A tension field ordinarily cannot be fully developed in an end panel. The available flexural strength is given in Table 3-19 of the Manual for seven specific locations of the plastic neutral axis, as shown in Figure 9.22: top of the flange (TFL), bottom of the top flange (BFL), three equally spaced levels within the top FIGURE 9.22 9.8 Tables for Composite Beam Analysis and Design 637 flange, and two locations in the web. The load is a service live load. 143 kips 7.4-1 a. 2-5 Structural steel buildings frequently are designed with diagonal bracing systems to resist lateral loads (horizontal forces resulting from wind or earthquake loadings). The limits shown in Table 5.4 for deflection due to dead load plus live load do not apply to steel beams, because the dead load deflection is usually compensated for by some means, such as cambering. Group B: ASTM A490, F2280, and A354 Grade BD. 282 Chapter 5 5.4-3 Beams Determine the smallest value of yield stress Fy for which a W, M, or S shape from Part 1 of the Manual will become slender. The service dead load is 450 lb冀ft. The loads for these systems are as follows: Roof: dead load = 30 psf, roof live load = 20 psf, snow load = 21 psf, and a rain load consisting of 4 inches of water. 5.10-2 Same as Problem 5.10-1, except that lateral support is provided only at the ends. EXAMPLE 8.5 Determine the size of weld required for the bracket connection in Figure 8.18. Doing so results in α= [(B冀T) − 1](a ′冀b ′) δ {1 − [(B冀T) − 1](a ′冀b ′)} (7.15) Two limit states are possible: tensile failure of the bolts and bending failure of the tee. Alternatives to Equation 3.1 for Single and Double Angles: The following values may be used in lieu of Equation 3.1. For four or more fasteners in the direction of loading, U = 0.80. Based on experimental studies (Hendrick and Murray, 1984), the term 5k can be replaced with 6k, resulting in the following equation for web yielding strength: fRn = f[Fyw tw (6k + tfb + 2w + 2tp)] where w = weld size. FIGURE 5.27 Mn Mp M′p = nominal strength based on FLB M′p Lp L′p Lr Lb 5.10 FIGURE 5.27 (continued) Design 233 φ b M n or M n/ Ω b φ b M′p or M′p/ Ω b L′p Lr Lb Form of the strength curve in the charts EXAMPLE 5.11 The beam shown in Figure 5.28 must support two concentrated live loads of 20 kips each at the quarter points. The equations for , m, n, l, and n′ are the same as for LRFD. (The computation of Cb is based on absolute values, so a sign convention for the diagram is not necessary.) Hence, 12.5 M max 2.5 M max + 3 M A + 4 M B + 3 MC 12.5(104.8) = = 2.24 2.5(104.8) + 3(41.30) + 4(7.400) + 3(56.10) Cb = For Cb = 2.24, fb Mn = 2.24(340) > fb Mp = 356 kips ∴ Use fb Mn = 356 ft-kips. Design of Beam Bearing Plates, Column Base Plates, Anchor Rods, and Column Splices. 716 Chapter 10 10.7-3 Plate Girders Use LRFD and determine the flange and web dimensions for a plate girder of the type shown in Figure P10.7-3 for the following conditions: • Span length = 50 ft. Whether the beam can sustain a moment large enough to bring it to the fully plastic condition also depends on whether the cross-sectional integrity is maintained. Adequate: For shear, 12.9 kips < 24.4 kips; for bearing, 12.9 kips < 57.3 kips; for tension, 22.3 kips < 31.3 kips b. > 0.9931 360 360 (OK) The deflection caused by the partition load is Δ4 = 5wpart L4 5(0.100 冀12)(30 × 12)4 = 0.0828 in. H., and Kulak, G. Figure 3.10 illustrates the alternative values of U for various connections. > 1.09 in. 7 y= ∑ (x 2 + y 2) = 4(1.286)2 + 3(1.714)2 + 2(3.857)2 + 2(0.857)2 + 2(2.143)2 + 1(5.143)2 = 82.29 in.2 e = 3 + 5 − 1.286 = 6.714 in. 1.68 5.15-5 Kx for AB = 2.00, Kx for BC = 1.40, Kx for DE = 1.2, Kx for EF = 1.28 4.8-1 677 kips 4.9-3 rx = ry = 4.29 in. Examples 7.5–7.7 demonstrate more realistic design situations. Bending is about the strong axis, and each column is laterally braced at the top and bottom. The service dead load is 10 kips, and the service live load is 30 kips. P P FIGURE P7.11-3 Problems 7.11-4 473 A tension member splice is made with 3⁄16-inch E70 fillet welds as shown in Figure P7.11-4. The following values are also available from the results of a preliminary design: Pe story = 40, 000 kips, Pstory = 6, 000 kips Use Kx = 1.0 (nonsway case), Kx = 1.7 (sway case), and Ky = 1.0. Use LRFD and determine whether this member satisfies the provisions of the AISC Specification for the given load combination. Detailed coverage of control of dead load deflection is given in an AISC seminar series (AISC, 1997a) and several papers (Ruddy, 1986; Ricker, 1989; and Larson and Huzzard, 1990). Design bearing stiffeners. The classification of shapes is found in Section B4 of the Specification, “Member Properties,” in Table B4.1b (Table B4.1a is for compression members). The primary reason for this continuation, called an end return, is to ensure that the weld size is maintained over the full length of the weld. Code Of Standard Practice for Steel Buildings and Bridges. Assume full composite action and assume that the formwork provides lateral support of the steel section before curing of the concrete. snow or wind load defl. SOLUTION The effective hole diameter is 1 + 1⁄8 = 11⁄8 in. The first-order analysis method is a simplified version of the direct analysis method that can be used when certain conditions are satisfied. From Equation 7.3, δ f = N0 L f A f E f Substituting this expression for df into Equation 7.8 yields ( ) ΔT ⎛ Ab Eb ⎞ ⎛ N 0 L f ⎞ ⎛ Ab Eb 冀 Lb ⎞ N0 = T0 ≈ 0.1T0 ΔT = ⎜ =⎜ ⎟ ⎜ ⎟ ⎟ ⎝ Lb ⎠ ⎝ A f E f ⎠ ⎝ A f E f 冀L L f ⎠ ΔN From Equation 7.7, T0 + 0.1T0 = F or F = 1.1T0 Therefore, at the instant of separation, the bolt tension is approximately 10% larger than its initial value at installation. From AISC Equation J3-4, Rn = m Duhf Tb ns = 0.30(1.13)(1.0)(28)(1.0) = 9.492 kips冀bolt For four bolts, Rn = 4(9.492) = 37.97 kips Bearing strength: Since both edge distances are the same, and the gusset plate is thinner than the tension member, the gusset plate thickness of 3⁄8 inch will be used. pPr + bx Mrx + by Mry = pPa + bx Max + by May = (2.84 × 10−3)(201) + (5.27 × 10−3)(67) + 0 = 0.924 < 1.0 (OK) To be sure that we have found the lightest W10, try the next lighter one, a W10 × 54, with p = 3.19 × 10−3 and bx = 5.97 × 10−3. Each side of the splice is welded as shown. Web Crippling Web crippling is buckling of the web caused by the compressive force delivered through the flange. To prevent excessive elongation of the hole, an upper limit is placed on the bearing load given by Equation 7.1. This upper limit is proportional to the projected bearing area times the fracture stress, or Rn = C × bearing area × Fu = CdtFu (7.2) where C = a constant d = bolt diameter t = thickness of the connected part The AISC Specification uses Equation 7.1 for bearing strength, subject to an upper limit given by Equation 7.2. If excessive deformation at service load is a concern, and it usually is, C is taken as 2.4. This value corresponds to a hole elongation of about 1⁄4 inch (RCSC, 2009). Segui This page intentionally left blank Steel Design Fifth Edition © Michael Caronna/Reuters/Corbis Steel making. Recent research has shown that the presence of fillers can affect the slip resistance of a connection 7.6 Slip-Critical and Bearing-Type Connections 397 (Borello, D. For some of the lighter shapes, it is equal to the depth to the nearest inch, but this is not a general rule for the W-shapes. FIGURE 8.38 LRFD SOLUTION Mu = 1.2 MD + 1.6 ML = 1.2(35) + 1.6(105) = 210.0 ft-kips Vu = 1.2 VD + 1.6 VL = 1.2(6.5) + 1.6(19.5) = 39.0 kips For the web plate, try 3⁄4-inch-diameter bolts. This type of failure can occur only with unsymmetrical cross sections, both those with one axis of symmetry—such as channels, structural tees, double-angle shapes, and equal-leg single angles— and those with no axis of symmetry, such as unequal-leg single angles. For one stud at each beam location, determine the available strength (for both LRFD and ASD) for each of the following cases: a. The W-shape, also called a wide-flange shape, consists of two parallel flanges separated by a single web. 3⁄8″ PL PL 1⁄2 × 6 11⁄2″ 23⁄4″ 23⁄4″ 11⁄2″ FIGURE P7.3-1 462 Chapter 7 7.3-2 Simple Connections The tension member shown in Figure P7.3-2 is a PL 1⁄2 × 51⁄2 of A36 steel. 73.1 kips c. AISC 37 (no. (the slab and deck combination weighs 57 psf). ANSWER ASD SOLUTION A WT10.5 × 66 is satisfactory. 9.7 Composite Beams with Formed Steel Deck 631 With one stud in each rib, the spacing is 6 inches, and the maximum number that can be accommodated is 30(12) = 60 > 46 total 6 With one stud in every other rib, 30 will be furnished, which is fewer than what is required for full composite action. Assume lateral support of the compression flange at A, B, C, D, and E. These tables are used in selected examples. The differential equation giving the deflected shape of an elastic member subjected to bending is d2y M = − 2 EI dx (4.2) where x locates a point along the longitudinal axis of the member, y is the deflection of the axis at that point, and M is the bending moment at the point. In addition, the outside and inside faces of the flanges of the W-shape are parallel, whereas the inside faces of the flanges of the S-shape slope with respect to the outside faces. If the steel shape alone is relied on to resist the negative moment, cover plates are sometimes added to the beam flanges to increase the moment strength. The member ends are fixed in all directions (x, y, and z). 1 Mu = 406.1 + (1.2 × 0.055)(24)2 = 411 ft-kips < 420 ft-kips 8 The maximum shear is 1.2(0.055) Vu = 60.48 + (24) = 61.3 kips 2 From the Zx tables, fvVn = 212 kips > 61.3 kips ANSWER ASD SOLUTION (OK) Use a W18 × 55. Because of the connection of the steel shape to the concrete slab, lateral-torsional buckling is no problem once the concrete has cured and composite action has been achieved. Case I: Loads Applied Through the Shear Center If loads act through the shear center, the problem is one of simple bending in two perpendicular directions. Table 4.1 shows which constants are given for various types of shapes. In a simple connection, care must be taken to make the connection sufficiently flexible that enough end rotation of the beam is possible. In the Group A category, the ASTM designation is F1852 (same strength as A325). In Figure 5.48b, there is only one component of load to contend with, but the concept is the same. FIGURE P3.8-1 3.8-2 Use ASD and select single-angle shapes for the web tension members of the truss loaded as shown in Figure P3.8-2. The shear strength of the studs could be reduced when a deck is used. Normal-weight concrete is used. Half of this force will produce shear in the plate on each side of the flange. Design an all-bolted, double-angle connection. Satisfactory: Result of interaction equation = 0.868 6.6-1 a. “Inelastic Column Theory.” Journal of Aeronautical Sciences 14 (no. If temporary shoring is not used, the steel beam may also be called on to resist incidental construction loads. 10 10 Span length 60(12) = = 60 in. The loads shown are service loads. The BOCA National Building Code. Determine the maximum service load, P, that can be applied if the live load to dead load ratio is 3.0. a. The fixed-end moment, caused by the actual load on the beam, is plotted on the moment axis (rotation = 0). We will first determine whether tension-field action can be used in regions other than the end panels. This means that the probability of failure, represented by the shaded area in Figure 2.3 labeled PF, will be smaller. AISC Specification Chapter C, “Design for Stability,” provides three approaches for determining the required flexural and axial compressive strength: the direct analysis method, the effective length method, and the first-order analysis method. Ravindra, M. Check the base-metal shear. If a member has two symmetrically located planes of connection, x− is measured from the centroid of the nearest one-half of the area. h=d+ 1 7 1 15 = + = in. The shear yield strength is Rn 0.6 Fy t 0.6(36)(1冀4) = = = 3.6 kips冀in.. This force could be an end reaction from a support of the type shown in Figure 5.36, or it could be a load delivered to the top flange by a column or another beam. The total reaction is Pa = D + L = 5 + 10 = 15 kips. Since it is heavier than the lightest one found so far, it will not be considered. Standard Specifications, Load Tables, and Weight Tables for Steel Joists and Joist Girders. This part includes design aids for tension members and a summary of the Specification requirements for tension members. Do these bolts have enough shear strength? 2″ 3″ 3″ 3″ 2″ 2″ 2C8 × 18.75 t = 3⁄8″ FIGURE P3.5-6 102 Chapter 3 Tension Members Design of Tension Members 3.6-1 Select a single-angle tension member of A36 steel to resist the following service loads: dead load = 50 kips, live load = 100 kips, and wind load = 45 kips. “A Specification for the Design of SteelConcrete Composite Columns.” Engineering Journal, AISC 16 (no. Lateral-torsional buckling can be prevented by bracing the beam against twisting at sufficiently close intervals. “Fundamentals of Beam Bracing.” Engineering Journal, AISC 38 (no. Use the alignment chart and determine Kx for members AB, BC, DE, and EF. (The strength corresponding to flexural buckling about the x-axis is 63.1 kips, based on an effective length of 14⁄2 = 7 feet.) Note that this shape is a slender-element cross section, but this is taken into account in the tabular values. For the W12 × 136, As = 39.9 in.2 and Is = Iy = 398 in.4 For the longitudinal reinforcement, Asr = 4(1.27) = 5.08 in.2 2 ⎛ 20 − 2 × 2.5 ⎞ 2 I sr = ∑ Ad 2 = 4 × 1.27 ⎜ ⎟⎠ = 285.8 in. There is, however, a more detailed discussion in the Commentary to Chapter L. Kulak, Fisher, and Struik (1987) recognized that this procedure is conservative when compared with test results. Use LRFD for the following: a. 384 EI 384 29, 000(510) The live load deflection is ΔL = 5 wL L4 5 (0.550冀12)(30 × 12)4 = = 0.678 in. Bethlehem Steel. The additional tension is called a prying force and is illustrated in Figure 7.27, which shows the forces on a free body of the hanger. 19.1 kips 4.9-9 a. ANSWER ASD SOLUTION Design strength = 476 kips. Connections that are extremely complex with fasteners can become very simple when welds are used. Try a W10 × 49 with a design strength of 568 kips. This method will be covered in detail in the following sections. We discuss methods for handling this problem later in this chapter. The procedure to be described here determines the ultimate strength of the connection by using an experimentally determined nonlinear load–deformation relationship for the individual fasteners. Such a treatment is beyond the scope of this book, however, and the emphasis in this chapter is on the qualitative basis of the Specification requirements and their application. Part 12. G., Leon, R. From the principle of moments with summation of moments about the y axis, x (28) = 8(4)(2) or x = 2.286 in. Block Shear Block shear, which was considered earlier in conjunction with tension member connections, can occur in certain types of beam connections. 1.4 1.3 Design Specifications 7 BUILDING CODES Buildings must be designed and constructed according to the provisions of a building code, which is a legal document containing requirements related to such things as structural safety, fire safety, plumbing, ventilation, and accessibility to the physically disabled. Assume all spacing and edge-distance requirements are satisfied, including those necessary for the use of the maximum nominal strength in bearing (i.e., 2.4dtFu), and determine the adequacy of the bolts for the following types of connections: (a) bearing-type connection with the threads in shear and (b) slip-critical connection with the threads in shear. We discuss the general approach first. Account for the beam weight. The maximum permissible center-to-center spacing of 12 + 1.5 = 13.5 inches. Design Procedure for ASD. If the flange thickness in Example 7.9 had proved to be inadequate, the alternatives would include trying a larger tee-shape or using more bolts to reduce T, the external load per bolt. When the external tensile load is applied, the forces on the assembly are as shown in Figure 7.25b, with F representing the total tensile force applied to one bolt (again, the actual distribution of the applied force per bolt has been idealized for simplicity). Required d = 0.822 in. From Manual Table 7-3, the slip-critical strength for one bolt is frn = 6.44 kips冀bolt The slip-critical strength controls. 8″ 4″ FIGURE P3.3-7 3.3-8 An L5 × 5 × 1⁄2 tension member of A242 steel is connected to a gusset plate with six 3⁄4-inch-diameter bolts as shown in Figure P3.3-8. 309 kips Chapter 5: Beams 5.2-1 a. Verify that Equation 6.10 is the correct one: Pu = pPu = (2.09 × 10 −3)(54.97) = 0.1149 < 0.2 f c Pn ∴ Equatiion 6.10 controls as assumed. The diagonal tension poses no particular problem, but the diagonal compression can cause the web to buckle. A 2(1)(6) fr = ft2 + fv2 = (4.354)2 + (1.833)2 = 4.724 kips冀in. If stresses are used, Equation 7.22 becomes ⎛ ft ⎞ ⎛ fv ⎞ ⎜ ⎟ + ⎜ ⎟ ⎝ F ⎠ ⎝ F ⎠ = 1.0 t v 2 2 (7.23) where ft = required tensile strength (stress) Ft = available tensile strength (stress) fv = required shear strength (stress) Fv = available shear strength (stress) An acceptable combination of shear and tension is one that lies under this curve. For those cases where deformation is not a design consideration, and for long-slotted holes with the slot perpendicular to the direction of the load, AISC gives other strength expressions. 52 Chapter 3 Tension Members FIGURE 3.7 — — — x x x — x — x + — x (a) (b) (c) (d) 艎 D ≥ 1.3 D: U = 1.0 D ≤ < 1.3 D: U = 1 − x= (e) D π 艎 B ≥ H: U = 1 − H x= (f) B B2 + 2 BH 4(B + H) 艎 ≥ H: U = 1 − H x= (g) x B2 4(B + H) x x 3.3 FIGURE 3.8 53 Effective Area Gusset plate Tension member Section (a) Bolted 1 = 1+ 2 2 2 (b) Welded The Commentary of the AISC Specification further illustrates x− and . Welding has several advantages over bolting. f FIGURE 1.4 Fu Fy E 1 e 1.5 Structural Steel 11 FIGURE 1.5 To use these higher-strength steels in a manner consistent with the use of ductile steels, some value of stress must be chosen as a value for Fy so that the same procedures and formulas can be used with all structural steels. The top chord is actually one continuous member rather than a series of individual pin-connected members, so this approximation is more accurate than treating each member as a simple beam. In this approach, the portion of the column load that falls within the confines of the column cross section—that is, over an area bf d—is assumed to be uniformly distributed over the H-shaped area shown in Figure 5.42. We refer to both types as plate girders. If the width of a rectangular shape FIGURE 1.7 1.6 Standard Cross-Sectional Shapes 15 is 8 inches or less, it is classified as a bar. Due to electronic rights restrictions, some third party content may be suppressed. B C W16 × 36 A D 30k 5′ 10′ 10′ 5′ FIGURE P3.7-2 3.7-3 Same as problem 3.7-2, but use allowable stress design. 4.3 AISC REQUIREMENTS The basic requirements for compression members are covered in Chapter E of the AISC Specification. Basler (1961) recommends the use of a shear flow of f = 0.045h FIGURE 10.9 Fy3 kips冀 in. Compute the nominal block shear strength of the gusset plate. The total service dead load, including the weight of the beam, is wD = 450 + 31 = 481 lb冀ft For a simply supported, uniformly loaded beam, the maximum bending moment occurs at midspan and is equal to M max = 1 2 wL 8 where w is the load in units of force per unit length, and L is the span length. When E70 electrodes are used, and this will usually be the case, computation of the weld shear strength can be simplified. Determine the size of intermediate and bearing stiffeners. LRFD SOLUTION a. Assume that the floor slab provides continuous lateral support. Specify the number of intermediate connectors needed (the midlength brace will provide one such connector). wD = 450 lb / ft wL = 550 lb / ft FIGURE 5.11 30′ SOLUTION First, determine the nominal flexural strength. Part 11 of the Manual, “Design of Partially Restrained Moment Connections,” presents a simplified alternative to the partially restrained connection. Pn Ωc 441 M ay ⎞ 8 ⎛ M ax Pa + + Pn Ωc 9 ⎜⎝ M nx Ωb M ny Ωb ⎟⎠ = 118.6 8 ⎛ 101.4 + ⎜ + 0 ⎞⎟ = 0.649 < 1.0 ⎠ 441 9 ⎝ 237 (OK) Load Combination 6a: Pnt = 250 kips, Mnt = 57 ft-kips, Pt = 25.2 kips, and Mt = 59.4 ft-kips. Practicing civil engineers who need a review of current practice and the current AISC Specification and Manual will find the book useful as a reference. Use a depth of d 8.25 = = 4.125 in. Use the lower-bound moment of inertia. Design of Tension Members. 11). SOLUTION Kx L = 20 ft and maximum K yL = 8 ft. The required strength of the connection is Pu = 1.2D + 1.6L = 1.2(30) + 1.6(25) = 76 kips Number of bolts required = 76 required strength = = 3.05 bolts strenggth per bolt 24.9 Try four bolts, two per line, on each side of the splice. For the load direction shown, member AC will resist the load in tension, and member BD will be unloaded. If the end of a column is fixed (G = 1.0) or pinned (G = 10.0), the value of G at that end should not be multiplied by the stiffness reduction factor. A typical designation would be “L6 × 6 × 3⁄4” or “L6 × 4 × 5⁄8.” The three numbers are the lengths of each of the two legs as measured from the corner, or heel, to the toe at the other end of the leg, and the thickness, which is the same for both legs. These moments and axial load are from service loads and consist of equal parts dead load and live load. Chapter 6: Beam-Columns 6.2-1 a. Determine whether this beam is adequate for LRFD. for a unit throat dimension. Slab and Deck Weight To simplify computation of the slab weight, we use the full depth of the slab, from bottom of deck to top of slab. FIGURE 10.17 FIGURE 10.18 10.7 Design 701 Check the flexural strength of the trial section. The usual practice is to locate the points of stitching so that L冀r for any component part does not exceed L冀r for the built-up member. 86 kips ANSWER For LRFD, the design strength is f Rn = 0.75(23.86) = 17. 2.6 Steel Construction Manual 35 Design aids for composite columns, consisting of hollow structural sections or pipe filled with plain (unreinforced) concrete, are also included. The service loads, computed from load combination 2, are shown in Figure 6.12. PL = 500 k wD = 1.0 k/ft, wL = 2.0 k/ft 7′-0″ 4′-0″ 12′-0″ 12′-0″ 12′-0″ 12′-0″ 12′-0″ 12′-0″ 80′-0″ FIGURE P10.4-5 4′-0″ 714 Chapter 10 Plate Girders Shear Strength 10.5-1 10.5-2 10.5-3 For the girder of Problem 10.4-2: a. 96 Chapter 3 Tension Members L5 × 5 × 1⁄2 11⁄2″ 5 spaces @ 3″ = 15″ 11⁄2″ FIGURE P3.3-8 Staggered Fasteners 3.4-1 A36 steel is used for the tension member shown in Figure P3.4-1. Using this number of studs will provide full composite behavior. Manual for Railway Engineering. The number of bolts in the designation refers to the number of bolts adjacent to the tension flange in a negative moment connection. The required flexural strength (not including the beam weight) is Ma = 6(20) = 120 ft-kips From the charts, with Lb = 24 ft, try W12 × 53. The member is part of a braced frame, and the axial load and bending moment are based on service loads consisting of 30% dead load and 70% live load (the end shears are not shown). By using the same bolt configuration on the top and bottom sides, the connection will work for both positive and negative moments.) • The maximum effective end-plate width is the beam flange width plus 1 inch. (M ay ⎞ 0.1176 71.50 Pa ⎛ M ax +⎜ + = + +0 ⎝ 2 Pn 冀Ωc M nx 冀Ωb M ny 冀Ωb ⎟⎠ 2 86.6) = 0.884 < 1.0 (OK) ANSWER The W8 × 35 is adequate. For the bolt pitch, try pfo = pf i = 2 in. Although 1 plates and bars are available in increments of ⁄16 inch, it is customary to specify 1 dimensions to the nearest ⁄8 inch. This distribution is the Whitney equivalent stress distribution, which has a resultant that matches that of the 9.1 Introduction 601 FIGURE 9.7 actual stress distribution (ACI, 2008). For 6.2 Interaction Formulas 301 example, if both bending and axial compression are acting, the interaction formula would be Pr Mr + ≤ 1.0 Pc Mc where Mr = required moment strength = Mu for LRFD = Ma for ASD Mc = available moment strength = fb Mn for LRFD = Mn for ASD Ωb For biaxial bending, there will be two moment ratios: Pr ⎛ Mrx Mry ⎞ + + ≤ 1.0 Pc ⎜⎝ Mcx Mcy ⎟⎠ (6.2) where the x and y subscripts refer to bending about the x and y axes. Noncompact shapes: The strength will be the smaller value for the limit states of LTB (same as for tees) and local buckling of the angle leg that is in compression. 1.65 b. Reverse curvature (a positive ratio) occurs when M1 and M2 are both clockwise or both counterclockwise. From Equation 7.37, the shear yield strength of the web is Rn = 0.4 Fy t × 2 sides of stiffener = 0.4(50)(0.360)(2) = 14.4 kips冀in. 3⁄4″ 51⁄2″ diameter Group A bolts 10″ WT8 × 50 A992 D = 25k L = 75k FIGURE P7.8-1 7.8-2 Determine the adequacy of the hanger connection in Figure P7.8-2 Account for prying action. As a consequence, if all possible mechanisms are investigated, the one requiring the smallest load is the correct one. H., and Stiles, W. The moment connection of Figure 8.37a also illustrates a recommended connection design practice: Whenever possible, welding should be done in the fabricating shop, and bolting should be done in the field. The required weld size is, therefore, w= Ω(Rn 冀Ω) Ω(fv) 2.00(6.756) 1 = = = 0.455 in. All contact surfaces have clean mill scale. ⎝ 16 ⎠ 5 ⎞ = 1.875 in. In addition, AISC F13.2, “Proportioning Limits for I-Shaped Members,” states that h冀tw in unstiffened girders shall not exceed 260. This curve is typical of a compression test of a short length of W-shape called a stub column, rather than the result of a tensile test. This connection must be capable of transferring a dead-load moment of 23 ft-kips and a live-load moment of 91 ft-kips. Assume that the base metal shear strength is adequate. Compute the nominal strength for Kx = 2.2 and Ky = 1.0. 4.7-2 An HSS 10 × 6 × 5⁄16 with Fy = 46 ksi is used as a column. The deck and slab combination weighs 53 psf. EXAMPLE 7.6 A 5⁄8-inch-thick tension member is connected to two 1⁄4-inch splice plates, as shown in Figure 7.14. For ASD, the allowable strength is Rn 23.96 = = 12.0 kips. This procedure is the same as that given in Part 14 of the Manual, “Design of Beam Bearing Plates, Column Base Plates, Anchor Rods, and Column Splices.” For ASD, we rewrite Equation 5.18 by substituting Pa for Pu and 1冀Ω for f: t≥ 2 Pa (1冀Ω) Fy BN t≥ 2 Pa Fy BN 冀1.67 or (5.19) In the equation for X, we again substitute Pa for Pu and 1冀Ω for f: ⎛ 4 db f ⎞ Pa X =⎜ 2 ⎟ ⎝ (d + b f) ⎠ Pp 冀Ωc (5.20) where Ωc = 2.31. This requires a knowledge of the properties of all the columns in the story, as well as H and ΔH, so that Pstory and Pe story can be computed. Problems 4″ 4″ 4″ 577 4″ 3″ 3″ 30° FIGURE P8.2-12 Eccentric Bolted Connections: Shear Plus Tension 8.3-1 Check the adequacy of the bolts. For yielding of the gross section, Pn 270.0 = = 161.7 kips Ωt 1.67 For fracture of the net section, Pn 348.0 = = 174.0 kips Ωt 2.00 5 FIGURE 8.23 5⁄16 5 5⁄16 3⁄16 9 510 Chapter 8 Eccentric Connections Yielding of the gross section controls. It can be FIGURE 8.32 Moment Moment-rotation curve for a particular connection Rotation FIGURE 8.33 Moment Fully restrained moment connection Partially restrained moment connection Simple shear connection Rotation 8.6 FIGURE 8.34 Moment-Resisting Connections 521 Moment End moment when end rotation = 0 (fixed-end condition) Beam line for a particular beam and loading θ=0 End rotation when moment = 0 (pinned-end condition) Rotation (θ) θ constructed as follows. Consider the rigid frame in Figure 6.1. For the given loading condition, the horizontal member AB must not only support the vertical uniform load but must also assist the vertical members in resisting the concentrated lateral load P1. The analysis results include the effects of a notional load appropriate for the effective length method. The maximum stud diameter is 2.5tf = 2.5(0.420) = 1.05 in., but the maximum diameter with formed steel deck = 3⁄4 in. The beams are W21 × 62, and the slab has a total thickness of 41⁄2 inches from top of slab to bottom of deck. 4.8 TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING When an axially loaded compression member becomes unstable overall (that is, not locally unstable), it can buckle in one of three ways, as shown in Figure 4.18). From the LRFD solution, Pn = Fy Ag and for ASD, Ω = 1.67. The components of the cross section are assumed to be properly connected. Use load and resistance factor design and select threaded rods of A36 steel for the following load cases. If it is the actual location, the equations of equilibrium will be satisfied. ANSWER ASD SOLUTION Use a W10 × 54. Neglect eccentricity and assume that the threads are in the plane of shear. The compressive force C is the smaller of As Fy = 14.6(50) = 730 kips or 0.85f c′ Ac = 0.85(4)(4 × 72) = 979.2 kips The PNA is in the concrete and C = 730 kips. The stress at the top of the steel is fst = Myt (160 × 12)(0.795) = = 0.998 ksi I tr 1530 (tension) Stress at the bottom of the steel: yb = t + d − y = 5 + 15.9 − 4.205 = 16.70 in. 337 kips b. D = 8 kips L = 24 kips Group A bolts A572 Grade 50 steel 7.7 Design Examples 413 FIGURE 7.20 (from AISC Table J3.1). G. For addition and subtraction, determining the location of the right-most significant digit in a column of numbers is done as follows: from the left-most significant digit of all numbers involved, move to the right a number of digits corresponding to the number of significant digits desired. Starting with the lighter shapes (the ones with the smaller gross area), we find that an L6 × 4 × 1⁄2, with ft Pn = 154 kips based on the gross section and ft Pn = 155 kips based on the net section, is a possibility. Two possibilities are summarized in Table 9.9. The W14 × 22 is the lighter shape, but because ΣQn is larger, it will require more studs. fillet welds: Capacity per inch = 1.392 × 3 sixteenths × 2 welds = 8.352 kips冀in. When bolts in slip-critical connections are subjected to tension, the slip-critical strength is ordinarily reduced by the factor given in AISC J3.9 (see Section 7.9). Iterative analyses that account for these effects are referred to as second-order methods. Check the spacing and edge distance transverse to the load. These conditions are discussed in Section 7.2 of the Commentary to the Specification and are not enumerated here. The basic symbol is a horizontal line (reference line) containing information on the type, size, and length of weld and an inclined arrow pointing to the weld. Yielding occurs when the compressive stress on a horizontal section through the web reaches the yield point. To check this selection, we must compute the actual net area. The ASD solution in Example 3.12 is somewhat condensed, in that some of the discussion in the LRFD solution is not repeated and only the final trial is shown. Bleich, Friedrich. The support will be a 16-inch × 16-inch concrete pier. The bearing and shear strengths of a bolted connection cannot be considered independently. The nominal shear strength is therefore Rn = Fnv Ab = 54(0.4418) = 23.86 kips and the design strength is fRn = 0.75 (23.86) = 17.90 kips per bolt The number of bolts required is 67.2 kips = 3.75 bolts 17.90 kips冀bolt ANSWER ASD SOLUTION Use four 3⁄4-inch-diameter Group A bolts. The box section, which has two webs as well as two flanges, is a torsionally superior shape and can be used when large unbraced lengths are necessary. Adequate: Mu = 160 ft-kips < f bMn = 203 ft-kips, Vu = 120 kips < f vVn = 131 kips b. 7.4 Shear Strength 389 The chief distinction between A307 bolts and high-strength bolts, other than the ultimate stress, is that the high-strength bolts can be tightened to produce a predictable tension in the bolt, which can be relied on to produce a calculable clamping force. FIGURE 7.38 θ Axis of weld 7.11 TABLE 7.2 Direction of load (q) Fillet Welds 445 Fnw = 0.60FEXX (1.0 + 0.50 sin1.5q) 0.60FEXX (1.0) 0.60FEXX (1.066) 0.60FEXX (1.177) 0.60FEXX (1.297) 0.60FEXX (1.403) 0.60FEXX (1.475) 0.60FEXX (1.5) 0° 15° 30° 45° 60° 75° 90° For simple (that is, concentrically loaded) welded connections with both longitudinal and transverse welds, AISC J2.4c specifies that the larger nominal strength from the following two options be used: 1. Neglect the weight of the beam and investigate this member as a beam–column. Design Guide 4, Extended End-Plate Moment Connections—Seismic and Wind Applications, covers only thick plate theory and three configurations: four-bolt unstiffened, four-bolt stiffened, and eight-bolt stiffened. B2 = amplification factor for the moments resulting from sidesway (P-Δ moments). Shear areas: 3 Agv = 2 × (2.5 + 2.5 + 1.25) = 4.688 in.2 8 3 Anv = 2 × [6.25 − 2.5(1.0)] = 2.813 in.2 8 7.7 Design Examples 407 FIGURE 7.17 Tension area: 3 Ant = (3 − 1.0) = 0.7500 in.2 8 For this type of block shear, Ubs = 1.0. From AISC Equation J4-5, Rn = 0.6Fu Anv + Ubs Fu Ant = 0.6(58)(2.813) + 1.0(58) (0.7500) = 141.4 kips with an upper limit of 0.6Fy Agv + UbsFu Ant = 0.6(36)(4.688) + 1.0(58)(0.7500) = 144.8 kips The nominal block shear strength is therefore 141.4 kips, and the design strength is fRn = 0.75(141.4) = 106 kips < 108 kips (N.G.) The simplest way to increase the block shear strength for this connection is to increase the shear areas by increasing the bolt spacing or the edge distance; we will increase the spacing. The hot steel passes through a series of rollers that squeeze the material into the desired cross-sectional shape. 1.5 tf Try a 11⁄2 ∂ 18 flange plate. To do so, enough bearing length, b, as required by AISC Equations J10-2 through J10-5, must be provided. < in. Approximately 70 ksi f. At other times, direct tension, compression, or bending must be considered. The user note in AISC D4 recommends that built-up shapes whose component parts are separated by intermittent fillers be connected at intervals such that the maximum L冀r for any component does not exceed 300. Do not check deflections. Determine the cross-sectional properties needed to resist failure under these loads. The 40-kip load is a service live load. The electrodes are E70. A W10 × 54 is the lightest W10, with an interpolated design strength of 594 kips. Assume a value for r0. 62 h = = 198.4 t w 5冀16 29, 000 E = 5.70 = 161.8 < 198.4 4 ∴ The web is slender. Any of these types of failure can be in either the elastic range or the inelastic range. 2005. Floor: dead load = 62 psf and occupancy live load = 80 psf. The shear strengths of A307, Group A, and Group B bolts, rounded to the nearest ksi, are summarized in Table 7.1. The values in Table 7.1 are also given in AISC Table J3.2. AISC Table J3.2 refers to threads in a plane of shear as “not excluded from shear planes” and refers to threads not in a plane of shear as “excluded from shear planes.” The first category, threads included in the shear plane, is sometimes referred to as connection type “N.” The designation “X” can be used to indicate that threads are excluded from the plane of shear. How many Group B bolts are required? 36 Fy 5.70 Determine the required flange size. The resulting moments are the Mnt moments. All aspects of steel joist usage, including their design and manufacture, are addressed in the publication Standard Specifications, Load Tables, and Weight Tables for Steel Joists and Joist Girders (SJI, 2005). For extremely stocky members, failure may occur by compressive yielding rather than buckling. P., Furlong, R. 8.8 END PLATE CONNECTIONS The end plate connection is a popular beam-to-column and beam-to-beam connection that has been in use since the mid-1950s. This flexibility can be 8.8 End Plate Connections 559 FIGURE 8.50 Simple–shear only Four-bolt stiffened Four-bolt unstiffened Eight-bolt stiffened achieved if the plate is short and thin, compared to the fully restrained version of this connection. Adding to the uncertainty of this load is the complication of drift, which can cause much of the load to accumulate over a relatively small area. The deflection caused by the partition weight is Δ3 = 5wpart L4 5(0.180 冀 12)(30 × 12)4 = 0.08596 in. The designation HSS is for “Hollow Structural Sections.” Steel pipe is available as standard, extra-strong, or double-extra-strong, with designations such as Pipe 5 Std., Pipe 5 x-strong, or Pipe 5 xx-strong, where 5 is the nominal outer diameter in inches. The uniform load on the girder consists of 2.1 kips冀ft of dead load (not including the weight of the girder) and 3.0 kips冀ft of live load. ftop = 2.60 ksi (compression), fbot = 31.6 ksi (tension), fc = 1.42 ksi (compression) 9.1-3 a. The bolts are 3⁄4 inch in diameter. The deck ribs are perpendicular to the beams. The trial shape can be easily selected in only a limited number of situations (as in Example 5.10). Concrete Bearing Strength The material used for a beam support can be concrete, brick, or some other material, but it usually will be concrete. We illustrate the use of these tables in Example 5.13, following a discussion of the design procedure for a beam in a typical floor or roof system. ⎝ 16 ⎟⎠ From Equation 7.36, the base metal shear rupture strength per unit length is 5 ⎞ fRn = 0.45Fu t = 0.45(58) ⎛⎜ = 8.16 kips冀in. 1.2D + 1.6S + 0.5L = 1.2(109) + 1.6(20) + 0.5(46) = 185.8 kips Combination 4: 1.2D + 1.0W + 0.5L + 0.5(Lr or S or R). 20⬘ 20⬘ 13⬘ C 13⬘ B 13⬘ A FIGURE P4.7-8 The given frame is unbraced, and bending is about the x axis of each member. FIGURE 5.51 5.15 LRFD SOLUTION Biaxial Bending 273 For the given loading condition, dead load plus a roof live load with no wind or snow, load combination 3 will control: wu = 1.2wD + 1.6Lr = 1.2(21) + 1.6(21) = 58.80 psf The width of roof surface tributary to each purlin is 15 10 = 7.906 ft 2 3 Then Purlin load = 58.80(7.906) = 464.9 lb冀ft Normal component = Parallel component = 3 10 1 10 (464.9) = 441.0 lb冀ft (464.9) = 147.0 lb冀ft and 1 Mux = (0.4410)(15)2 = 12.40 ft-kips 8 With sag rods placed at the midpoint of each purlin, the purlins are two-span continuous beams with respect to weak axis bending. LRFD SOLUTION From Example 7.12, the weld shear design strength is fRn = 4.176 kips冀in. 1965. If the axial load P is slowly applied, it will ultimately become large enough to cause the member to become unstable and assume the shape indicated by the dashed line. The capacity of a 1.5-inch length of the 4 welds is 1.5(10.80) = 16.20 kips Equating the shear strength per inch and the required strength gives 16.20 = 3.539 kips冀in. Tension members are covered in Chapter D of the Specification. Length of longitudinal welds = 2 Total required length of weld = 7.11 Fillet Welds 459 For the second option, the strength of the longitudinal welds is 0.85(5.568) = 4.733 kips冀in. Compute rx and ry for the built-up shape. For doubly symmetrical shapes (torsional buckling), ⎡ π 2 ECw ⎤ 1 Fe = ⎢ 2 + GJ ⎥ ⎣ (K z L) ⎦ Ix + Iy (AISC Equation E4-4) For singly symmetrical shapes (flexural-torsional buckling), Fe = Fey + Fez ⎛ ⎟⎟ 1 − 2H ⎝ 1− where y is the axis of symmetry. For those cases where KxL ≠ KyL (rectangular HSS), Tables 4-13 and 4-14 give values of rmx 冀rmy, the ratio of weak axis radius of gyration to strong axis radius of gyration. where the corner radius is taken as 1.5 times the design thickness. All loads and moments are based on service loads, and all bending moments are about the strong axis. Bearing stiffeners will not be effective. Although this distinction is an oversimplification, it affirms the first priority of the structural engineer: safety. Required d = 0.657 in. EXAMPLE 5.16 Design a bearing plate to distribute the reaction of a W21 × 68 with a span length of 15 feet 10 inches center-to-center of supports. Use the tables for eccentrically loaded weld groups given in Part 8 of the Manual. If the length exceeds 100 times the weld size, a reduced effective 454 Chapter 7 Simple Connections length is used in the computation of strength. = 500 lbft wL = live load + partition load = 1200 + 100 = 1300 lb/ft wu = 1.2 wD + 1.6 wL = 1.2(00.500) + 1.6(1.300) = 2.68 kips/ft Mu = 1 1 wu L2 = (2.68)(30)2 = 301.5 ft-kips 8 8 Assume that d = 16 in., a冀2 = 0.5 in., and estimate the beam weight from Equation 9.4: w= 3.4 Mu 3.4(301.5 × 12) = = 22.3 lbft f b Fy (d 2 + t − a 2) 0.90(50)(162 + 4.75 − 0.5) 9.7 Composite Beams with Formed Steel Deck 629 Try a W16 ∂ 26. A general category for any type of tension member except plates and round HSS with ≥ 1.3D (See Figure 3.7e.) 2. 36 ksi 58 to 80 ksi — 20% 50 ksi 65 ksi — 18% 50 ksi 65 ksi 0.85 18% develops standards for defining materials in terms of their composition, properties, and performance, and it prescribes specific tests for measuring these attributes (ASTM, 2010a). The following discussion, which is based on the commentary that accompanies the RCSC specification, explains the basis of the AISC specification equations for bearing strength. 260 Chapter 5 Beams Column Base Plates As with beam bearing plates, the design of column base plates requires consideration of bearing pressure on the supporting material and bending of the plate. 6.8 Design of Beam–Columns 355 EXAMPLE 6.11 Figure 6.25 shows a single-story, unbraced frame subjected to dead load, roof live load, and wind load. We denote this horizontal shear force V ′. Laman, the Pennsylvania State University; Tyler Ley, Oklahoma State University; Jean-Paul Pinelli, Florida Institute of Technology; Jay Shen, Illinois Institute of Technology; Bozidar Stojadinovic, University of California, Berkeley; and Chia-Ming Vang, University of California, San Diego. For a given span, an open-web joist will be lighter in weight than a rolled shape, and the absence of a solid web allows for the easy passage of duct work and electrical conduits. This is a second-order, linear, ordinary differential equation with constant coefficients and has the solution y = A cos(cx) + B sin(cx) FIGURE 4.3 112 Chapter 4 Compression Members where c= Pcr EI and A and B are constants. © Jack Sullivan/Alamy Most bolted connections, such as this beam-to-column connection, use high-strength bolts. ANSWER Use an HSS14 × 10 × 5⁄16. Figure 5.31(e) shows the final beam model (for the usual floor framing connections, the beams can be treated as simply supported). The procedure can be explained as follows. Because of this, yielding of the web represents one of the shear limit states. For the direction of P2 shown, member ED will be in tension and member CF will be slack, provided that the bracing elements have been designed to resist only tension. Member stiffnesses are not reduced. ⎝ 8⎠ 456 Chapter 7 Simple Connections FIGURE 7.45 41⁄2″ and the allowable shear rupture strength of the base metal is ⎛ 3⎞ 0.3Fu t = 0.3(5 8) ⎜ ⎟ = 6.525 kips冀in. In the LRFD solution of Example 5.10, the required design strength was 810.0 ft-kips, and there was continuous lateral support. SOLUTION Mu = 1.2MD + 1.6ML = 1.2(23) + 1.6(91) = 173.2 ft-kips Vu = 1.2VD + 1.6VL = 1.2(4) + 1.6(18) = 33.6 kips For a W18 × 35, d = 17.7 in., tw = 0.300 in., bf b = 6.00 in., tf b = 0.425 in., workable gage = 3.50 in., Zx = 66.5 in.3 Compute 60% of the flexural strength of the beam: fb Mp = fb Fy Zx = 0.90(50)(66.5) = 2993 in.-kips 0.60(fb Mp) = 0.60(2993) = 1796 in.-kips = 150 ft-kips Mu = 173.2 ft-kips > 150 ft-kips ∴ Design for 173.2 ft-kips. The service dead load is 216 kips, the service live load is 25 kips, and the length is 22 ft. What size bolt is required? Try W10 × 45, with p = 4.51 × 10−3, bx = 7.63 × 10−3, by = 17.6 × 10−3. The cross-sectional area of the steel shape must make up at least 1% of the total area. The following discussion refers to the symbols shown in Figure 7.46. This shape is compact (no footnote in the uniform load tables), so Mny冀Ωb = Mpy 冀Ωb = Fy Zy 冀Ωb = 36(2.34)冀1.67 = 50.44 in.-kips = 4.203 ft-kips But Z y 2.34 = = 2.03 > 1.6 S y 1.15 ∴ M ny 冀Ωb = 1.6 Fy S y 冀Ωb = 1.6(36)(1.15)冀1.67 = 39.66 in.-kips = 3.300 ft-kips Because the load is applied to the top flange, use only half of this capacity to account for the torsional effects. The minimum spacing is 22⁄3d = 2.33 inches. What is the nominal compressive strength if Fy = 50 ksi? Such relationships have been developed, and this is an area of ongoing research (Christopher and Bjorhovde, 1999). The connection must resist a service dead load of 45 kips and a service live load of 90 kips. For example, if the load is removed at point A in Figure 1.3b, the unloading will be along line AB, resulting in the permanent strain OB. The direct shearing stress is the same for all segments of the weld and is equal to f1 y = 60 60 = = 2.143 ksi 8 + 12 + 8 28 Before computing the torsional component of shearing stress, the location of the centroid of the weld shear area must be determined. The vertical axis of symmetry is one of the principal axes, and its location need not be computed. This result is a consequence of approximations made in formulating the differential equation; a linear representation of a nonlinear phenomenon was used. (OK) ⎜ (168) ⎟ = 2.40 h ⎜⎣ (h冀t w) ⎦⎥ ⎦ c. Commonly used cross-sectional shapes include the W, S, and M shapes. 477 478 Chapter 8 Eccentric Connections FIGURE 8.1 Available strengths (maximum reaction capacities) for various framed beam connections are given in Tables 10-1 through 10-12 in Part 10 of the Manual, “Design of Simple Shear Connections.” 8.2 ECCENTRIC BOLTED CONNECTIONS: SHEAR ONLY The column bracket connection shown in Figure 8.2 is an

example of a bolted connection subjected to eccentric shear. Riveted connection design is no longer covered by the AISC Specification, but many existing structures contain riveted joints, and the analysis of these connections is required for the strength evaluation and rehabilitation of older structures. This requirement for formed steel deck is in addition to the usual maximum diameter of 2.5tf. “Steelmail.” Modern Steel Construction. Sometimes a numerical limit, such as 1 inch, is appropriate. For the braced condition, the bending factor is 48.75 ⎞ M1 ⎞ Cm = 0.6 − 0.4 ⎛⎜ = 0.6 − 0.4 ⎛⎜ = 0.2579 ⎝ M 2 ⎟⎠ ⎝ 57 ⎟⎠ Pe1 = 3767 kips (Pe1 is independent of the loading condition) Pr = Pa = Pnt = 250 kips Cm Cm B1 = = 1 − (α Pr Pe1) 1 − α [(Pnt + Pt) Pe1] 0.2579 = = 0.292 < 1.0 ∴ Use B1 = 1.0. 1 − 1.60[(250 + 25.2)] 3767 For the unbraced condition, Pstory Pa 250 + 25.2 275.2 ≈ = = 3767 3767 Pe story Pe1 B2 = 1 1 = = 1.132 α Pstory 1.60(275.2) 1− 1− 3767 Pe story The amplified axial load is Pr = Pa = Pnt + B2Pt = 250 + 1.132(25.2) = 278.5 kips The total amplified moment is Mr = Max = B1Mnt + B2Mt = 1.0(57) + 1.132 (59.4) = 124.2 ft-kips As before, use Mn冀Ωb = Mp冀Ωb = 237 ft-kips 278.5 Pa = = 0.6315 > 0.2 441 Pn 冀Ωc ∴ Use Equation 6.5 (AISC Equation H1-1a). The available strength tables, however, are the most useful. To obtain the design equations, we use service loads and moments rather than factored loads and moments, and we divide the nominal strengths by a safety factor rather than multiplying by a resistance factor. The critical buckling stress corresponding to a given length, actual or effective, remains the same. Allowable stress design (ASD) was covered by the 1978 AISC Specification and Manual of Steel Construction, Ninth Edition. Determine the appropriate interaction equation: 305 Pa = = 0.6916 > 0.2 ∴ Use Equation 6.5 (AIS SC Equation H1-1a). EXAMPLE 4.16 Compute the compressive strength of a C15 × 50 of A36 steel. The hot-rolled I shape is the one most commonly used for beams. Connections that are designed to be fully restrained actually permit some rotation; otherwise, the curve would just be represented by the vertical axis. The member is pinned at both ends, with additional support at mid-height in the weak direction. Lateral support is provided only at the ends, and A992 steel is used. This secondary moment is largest where the deflection is largest—in this case, at the centerline, where the total moment is wL2冀8 + Pd. Of course, the additional moment causes an additional deflection over and above that resulting from the transverse load. This is the truss of Example 3.15. 108 For two studs per section, s= 30(12) = 6.67 in. Discrete values of Q and R from observations can be plotted as frequency distribution histograms or represented by theoretical probability density functions. In addition, the same material is used. The length in Equation 3.1 is the length of the connection in the direction of the load, as shown in Figure 3.8. For bolted connections, it is measured from the center of the bolt at one end of the connection to the center of the bolt at the other end. No restriction was placed on the depth, so we choose the lightest joist, a 16K2. If the critical load is divided by the cross-sectional area, the critical buckling stress is obtained: Fcr = Pcr π2E = A (L冀r)2 (4.4) At this compressive stress, buckling will occur about the axis corresponding to r. FIGURE 10.7 10.5 Shear Strength 675 As with all other flexural members covered in Chapter F of the Specification, the resistance factor for LRFD is f b = 0.90, and the safety factor for ASD is Ωb = 1.67. We can then compute bending stresses with the flexure formula. 2 b= FIGURE 7.29 426 Chapter 7 Simple Connections Since 1.25b = 1.25(2.425) = 3.031 in. The strength of the electrode is defined as its ultimate tensile strength, with strengths of 60, 70, 80, 90, 100, 110, and 120 kips per square inch available for the shielded metal arc welding process. Purlins are to be placed at the joints and at the midpoint of each top-chord member. The “workable gage” distance given in Part 1 of the Manual can be used. Although connections are usually treated as either simple or rigid, the reality is that most connections fall somewhere in between and can be accurately described as partially restrained or semirigid. This method is preferred, and we explain it following Example 5.10. 2 16 16 8.5 Eccentric Welded Connections: Shear Plus Tension 515 Try w = 1⁄4 inch. 4): 143–l52. The columns in this frame are not independent members but part of a continuous structure. Assume that Cb = 1.0 and compute the nominal flexural strength. The shear areas are 1 Agv = (1.25 + 9) = 2.563 in.2 4 1 Anv = [1.25 + 9 − 3.5(1.0)] = 1.688 in.2 (3.5 hole diameters) 4 The tension area is Ant = FIGURE 7.22 1 1⁄4 ″ 2″ 1 1⁄2″ 1 [1.5 − 0.5(1.0)] = 0.2 5 0 0in.2 4 3″ 3″ 3″ (0.5 hole diameter) 1 1⁄4 ″ 416 Chapter 7 Simple Connections From AISC Equation J4-5, Rn = 0.6Fu Anv + Ubs Fu Ant = 0.6(65)(1.688) + 1.0(65)(0.2500) = 82.08 kips with an upper limit of 0.6Fy Agv + UbsFu Ant = 0.6(50)(2.563) + 1.0(65)(0.2500) = 93.14 kips The nominal block shear strength is therefore 84.50 kips, and the design strength is fRn = 0.75(82.08) = 61.6 kips > 48.0 kips ANSWER FIGURE 7.23 (OK) Use an L31⁄2 × 21⁄2 × 1⁄4 with the long leg connected. 7″ 8″ 8″ 8k FIGURE P8.4-1 8.4-2 Use an elastic analysis and determine the maximum load in the weld (in kips per inch of length). This standard provides load requirements in a format suitable for adoption as part of a code. R 21⁄4″ 11⁄2″ 3″ 3″ 3″ 3″ 2L4 × 4 × 5⁄16 1⁄2″ 1 W10 × 45 W21 × 83 FIGURE P8.3-7 8.3-8 A bracket cut from a WT-shape is connected to a column flange with 10 Group A slipcritical bolts, as shown in Figure P8.3-8. Since both weld segments are parallel to the applied load, q = 0 and the basic strength of the weld is Fnw = 0.60FEXX. This happens when the maximum positive moment attains a value of Mp. By virtue of the uniqueness theorem, the corresponding load is the collapse load because the distribution of moment is safe and statically admissible. 258 × 10 5 kip-in.2 From AISC Equation I2-5, Pe = π 2 (EI)eff π 2 (7.258 × 10 5) = = 294.4 kip ps (KL)2 (13 × 12)2 Determine Pn: Pno 276.3 = = 0.9385 < 2.25 Pe 294.4 ∴ Use AISC Equation I 2-2 . The steel is A572 Grade 50. American Railway Engineering and Maintenance-of-Way Association (AREMA): The AREMA Manual for Railway Engineering covers the design of railway bridges and related structures (AREMA, 2010). Determine which interaction equation to use: Pu = pPu = (2.23 × 10 −3)(150) = 0.335 > 0.2 ∴ Equation 6.9 controls. 16 3 Rn = 1.2 c tFu = 1.2(2.188) ⎛⎜ ⎞⎟ (58) = 57.11 kips ⎝ 8⎠ c = s − h = 3 − Upper limit = 2.4 dtFu = 39.15 kips < 57.11 kips ∴ Use Rn = 39.15 kips for this bolt . SOLUTION From AISC J3.3, the minimum spacing in any direction is 3 22⁄3 d = 2.667 ⎛⎜ ⎞⎟ = 2.00 in. I 6.5(5 /8)3 冀12 = = 0.1804 in. After obtaining his B.S.C.E., he served as a commissioned officer in the U.S. Air Force from 1960-1963. At the 4.3 AISC Requirements 119 boundary between inelastic and elastic columns, Equations 4.8 and 4.9 give the same value of Fcr. When doing manual computations with the aid of an electronic calculator, an engineer must make a decision regarding the degree of precision needed. 1193 4.9 Built-Up Members 165 Connection Requirements for Built-Up Members Composed of Rolled Shapes The most common built-up shape is one that is composed of rolled shapes, namely, the double-angle shape. Assume that the bearing plate will rest on concrete with a surface area larger than the bearing area by an amount equal to 1 inch of concrete on all sides of the plate. This limit state corresponds to a flexural stress of fb = Mn = Fu S x (A fn 冀A fg) (5.11) where Sx(Afn冀Afg) can be considered to be a “net” elastic section modulus. The fasteners will be in single shear, and bearing strength will be controlled by a 5⁄16-inch-thick connected part of A36 steel. 9.8-7 Use the composite beam tables to solve Problem 9.5-3 with the following modifications: • Continuous lateral support is provided during the construction phase. From Equation 7.37, the shear yield strength of the angle leg is Rn ⎛ 1⎞ = 0.4 Fy t = 0.4(36) ⎜ ⎟ = 7.20 kips冀in. The frame analysis was performed consistent with the effective length method, so the flexural rigidity, EI, was unreduced. Determine which interaction equation to use: pPr = pPa = (2.84 × 10−3)(201) = 0.571 > 0.2 ∴ Equation 6.9 controls. The slab has a total thickness of 41⁄2 inches and is supported by a formed steel deck, the cross section of which is shown in Figure 9.24. ST3): 765–83. (Do not consider any additional construction loads in this example.) Steel is A992, and fc′ = 4 ksi. The loads consist of a 50 psf live load, a partition load of 20 psf, a slab and metal deck system weighing 30 psf, and a ceiling and light fixture weight of 5 psf. The beam weight is not included in the service loads shown. EXAMPLE 9.5 Design steel headed stud anchors for the floor system in Example 9.4. SOLUTION Summary of data from Example 9.4: W18 × 35, A992 steel fc′ = 4 ksi Slab thickness t = 4.5 in. (This notation is not used in the charts or in any of the other design aids in the Manual.) Whether a shape is compact or noncompact is irrelevant to the use of the charts. If Fy = 65 ksi, what is the nominal flexural strength based on flange local buckling? The absolute necessity for the use of a computer is obvious. Regardless of the support conditions, whether simple, fixed, or something in between, the same static moment of wL2冀8 will have to be resisted. What size rod is required if A36 steel is used? d = 1冀2 in. The eyebar is a special type of pin-connected member in which the end containing the pin hole is enlarged, as shown in Figure 3.37. or 4 4 Beam spacing = 9(12) = 108 in. The criterion that must be satisfied in the selection of a member is Factored load ≤ factored strength (2.3) In this expression, the factored load is actually the sum of all service loads to be resisted by the member, each multiplied by its own load factor. EXAMPLE 4.18 Compute the available strength of the compression member shown in Figure 4.22. Strength of Encased Composite Columns The AISC Specification covers encased composite columns in Section I2.1. If buckling were not an issue, the member strength could be taken as the summation of the axial compressive strengths of the component materials: Pno = Fy As + Fysr Asr + 0.85f c′ Ac FIGURE 9.26 (AISC Equation I2-4) 646 Chapter 9 Composite Construction where Fy = yield stress of the steel shape As = cross-sectional area of the steel shape Fysr = yield stress of the longitudinal reinforcing steel bars Asr = cross-sectional area of the reinforcing steel The strength Pno is sometimes called the “squash” load; it is the nominal strength when length effects (slenderness effects) are not taken into account. 5.5 TABLE 5.3 Width-toThickness Parameters* Element Flange Web l Bending Strength of Compact Shapes lp 199 lr bf 2t f 0.38 E Fy h tw 3.76 E Fy 1.0 5.70 E Fy E Fy *For hot-rolled I shapes in flexure. Fy = 50 ksi for both the beam and the column. If Lp < Lb ≤ Lr , Failure will be by inelastic LTB, and ⎡ ⎛ Lb − L p ⎞ ⎤ Fcr = Cb ⎢ Fy − 0.3Fy ⎜ ⎟ ⎥ ≤ Fy ⎝ Lr − L p ⎠ ⎦⎥ ⎣⎢ (AISC Equation F5-3) If Lb > Lr, failure will be by elastic LTB, and Fcr = Cb π 2 E ⎛ Lb ⎞ ⎜⎝ ⎟⎠ rt 2 ≤ Fy (AISC Equation F5-4) Cb is defined by AISC Equation F1-1 and is covered in Chapter 5 of this book. The total length required is 27 kips = 7.27 in. The required gross area is Ag ≥ Pu 48.0 = = 1.07 in. The member bends and twists simultaneously (Figure 4.18c). The nominal shear strength is therefore Rn = Fnv Ab = 54(0.4418) = 23.86 kips and the allowable strength is Rn 23.86 = = 11.93 kips per bolt Ω 2.00 The number of bolts required is 45 kips = 3.77 bolts 11.93 kips冀bolt ANSWER Use four 3⁄4-inch-diameter Group A bolts. 4 Fey Fez H ⎞ ⎛ (Fey + Fez)2 ⎟⎠ (AISC Equation E4-5) 4.8 Torsional and Flexural-Torsional Buckling 157 For shapes with no axis of symmetry (flexural-torsional buckling), ⎛x ⎞ (Fe − Fex)(Fe − Fey)(Fe − Fez) − Fe2 (Fe − Fey) ⎜ 0 ⎟ ⎝ r0 ⎠ 2 2 − Fe2 (Fe ⎛y ⎞ − Fex) ⎜ 0 ⎟ = 0 ⎝ r0 ⎠ (AISC Eq quation E4-6) This last equation is a cubic; Fe is the smallest root. Bearing stiffeners 10.6 Bearing Stiffeners 683 FIGURE 10.10 LRFD SOLUTION Check the web width-to-thickness ratio. McGuire, W. Structural Steel Design 5th Ed (Instructor's Solutions Manual) Authors, Jack C. 1981. The corresponding loads are LRFD SOLUTION D= 7200 2437 + + 6.5(20) = 732.3 lb 2(8) 2(8) S= 16,0000 = 1000 lb 2(8) Load combination 3 will control: Pu = 1.2D + 1.6S At an interior joint, Pu = 1.2(1.335) + 1.6(2.0) = 4.802 kips At an exterior joint, Pu = 1.2(0.7323) + 1.6(1.0) = 2.479 kips The loaded truss is shown in Figure 3.35a. (A307 bolts are used only in bearing-type connections.) Proper installation and achievement of the prescribed initial tension is necessary for slip-critical connections. Thus the strength for bolt 1 is calculated with c measured to the edge of bolt 2, and the strength for bolt 2 is calculated with c measured to the edge of the connected part. The resultant tensile force T (equal to C) will be located at the centroid of the steel area. The base metal shear strength is therefore 5.400 kips冀in. Although the strength is directly proportional to Cb, this graph clearly shows the importance of observing the upper limit of Mp, regardless of which equation is used for Mn. Part 3 of the Steel Construction Manual, “Design of Flexural Members,” contains several useful tables and charts for the analysis and design of beams. The one requiring the smallest load will control, and the corresponding load is the collapse load. Group A, threads in the plane of shear c. ⎝ 16 ⎠ Ω From Equation 7.38, the base metal shear rupture strength per unit length is Rn 5 ⎞ = 0.3Fu t = 0.3(58) ⎛ = 5.44 kips冀in. As with LRFD, specific combinations of loads must be considered. “Optimum Flexural Design of Steel Members Utilizing Moment Gradient and Cb .” Engineering Journal, AISC 46 (no. For LRFD, the resistance factor f is 0.75, and for ASD, the safety factor Ω is 2.00. For any of the limit states, the decision on whether to weld the stiffener to the flange should be based on the following criteria: • On the tension side, the stiffeners should be welded to both the web and flange. Because lateral loads are most detrimental to tall structures, wind loads are usually not as important for low buildings, but uplift on light roof systems can be critical. An = Ag − ∑ t w × (d or d ′) SOLUTION d = bolt diameter + 1 5 1 3 = + = in. Use Fy = 65 ksi and Cb = 1 and compute the nominal flexural strength. The required strength of the weld is Rn = 0.9279 D × 4 = 0.9279(2.16)(4) Ω = 8.017 kips冀in. In this combination, we use S instead of Lr, and both R and W are zero. This connection is treated as a rigid connection and can be modeled as shown. Assume that the bolt threads are in the plane of shear. 2 f t Fu 0.75Fu 0.75(58) Try t = 1 in. 5 Pn Fy Ag 36 (8 × 6.5) = 87.6 kips > 80.77 kips (OK) = = 1.67 Ω 1.67 Check block shear in the plate. 6.8 ASD SOLUTION Design of Beam–Columns 343 The required axial load strength is Pa = PD + PL = 54 + 147 = 201 kips The moment at each end is Mntx = MD + ML = 18 + 49 = 67 ft-kips Since B1 = 1.0, the required bending moment strength is Max = B1Mntx = 1.0(67) = 67 ft-kips The effective length for compression and the unbraced length for bending are the same: KL = Lb = 16 ft. The shear yield strength is ⎛ 1⎞ φ Rn = φ (0.6 Fy t) = 1.00(0.6)(36) ⎜ ⎟ = 5.4 kipss冀in. This book does not cover the integrated design of buildings, but presents some of the “building blocks” for structural steel design. The steel has a yield stress of Fy = 46 ksi, and the concrete has a compressive strength of fc′ = 4 ksi. 15′-0″ 6 @ 15′-0″ = 90′-0″ FIGURE P5.15-7 5-15-8 297 Same as Problem 5.15-7, except that the sag rods are at the third points. To compute x− for these cases, the Commentary uses the concept of the plastic neutral axis to explain the procedure. Failure of the fastener can be assumed to occur as shown. I have attempted to check everything for errors, but it is inevitable that some will remain. With the advent of the Bessemer converter in 1855, steel began to displace wrought iron and cast iron in construction. If all of the computations for a problem are done in one continuous series of operations on a calculator, the number of significant figures used is whatever the calculator uses, perhaps 10 or 12. All connections are made with longitudinal plus transverse welds. 274 Chapter 5 Beams This shape is compact (no footnote in the uniform load tables), so fb Mny = fb Mpy = fb Fy Z y = 0.90(36)(2.34) = 75.82 in.-kips = 6.318 ft-kips But Z y 2.34 = = 2.03 > 1.6 S y 1.15 ∴ fb Mny = fb(1.6 Fy S y) = 0.90(1.6)(36)(1.15) = 59.62 in.-kips = 4.968 ft-kips Because the load is applied to the top flange, use only half this capacity to account for the torsional effects. For the other holes, use s = 3 in. From Manual Table 7-1, rn = 16.6 kips冀bolt Ω Assume that shear controls and then check bearing. n 8 The transformed section is shown in Figure 9.5. Although the neutral axis is shown below the top of the steel, it is not known yet whether it lies in the steel or the concrete. Because the beam is compact and laterally supported, the nominal flexural strength is Mn = Mp = Fy Z x = 50(115) = 5750 in.-kips = 479.2 ft-kips LRFD SOLUTION The design strength is ASD SOLUTION The allowable moment strength is fbMn = 0.90(479.2) = 431 ft-kips Mn M = n = 0.6 M n = 0.6(479.2) = 288 ft-kips Ωb 1.67 b. If bolt shear controls, the number of bolts required is 80.77 = 7.61 bolts 10.61 Use 8 bolts (4 pairs). For the channel, however, the depth is exact rather than nominal. Ricles, J.M. and Yura, J.A. 1983. 734 References Yura, J. Block shear failure in the plate tension member would be tension on ik and shear on both hi and jk. Determine whether the compression flange is compact, noncompact, or slender. 2 2 2 2 FIGURE 3.20 64 Chapter 3 Tension Members Starting at a and treating the holes at b and d as the staggered holes gives An = Ag − ∑ t × (d or d ′) = 14.7 − 2(0.622) − (0.550) () 7 (1.5)2 ⎤⎡7 − (0.550) ⎢ − ⎢8 ⎣ 8 4(4.225) ⎦ (87) − 2(0.622) ⎡⎤⎣ 87 − 4((14..5225)) ⎤⎦⎦ = 11.73 in. COLUMN STIFFENERS AND OTHER REINFORCEMENT Since most of the moment transferred from the beam to the column in a rigid connection takes the form of a couple consisting of the tensile and compressive forces in the beam flanges, the application of these relatively large concentrated forces may require reinforcement of the column. 2-1 A column in the upper story of a building is subjected to a compressive load from the following sources: dead load = 30.8 kips, occupancy live load = 1.7 kips, roof live load = 18.7 kips, and snow load = 19.7 kips. SOLUTION The nominal strength of the member based on the gross section is Pn = Fy Ag = 36(7.50) = 270.0 kips The nominal strength based on the net section requires a value of U, but the length of the connection is not yet known, so U cannot be computed from Equation 3.1. We will use an estimated value of 0.80, and, if necessary, revise the solution after the connection length is known. The 10-kip load consists of 2.5 kips of service dead load and 7.5 kips of service live load. For the bolt line, the length is a net length, so α= M b− b 冀(p − d ′) M b− b = M a − a 冀p M a− a 1 ⎞ M b− b ⎛ ⎜⎝ 1 − d ′冀p ⎟⎠ = δ M a− a (7.12) where p = length of flange tributary to one bolt, not too exceed 2b (see Figure 7.28a) d ′ = diameter of bolt hole δ =1− d′ net area at bolt line = p gross area at stem face The numerical evaluation of a will require the use of another equation, which we develop shortly. Compute the allowable strength based on shear and bearing. Instead of incorporating a numerical value for Rult, we will use a unit strength and modify Equation 8.2 as R = Rult (1 − e − μΔ) = 1.0 (1 − e − μΔ) = (1 − e − μΔ) λ λ λ That is, we will obtain a result corresponding to a unit bolt strength. The loads are service loads. 5.6-2 A W14 × 90 of A572 Grade 60 steel is used as a beam with lateral support at 10-foot intervals. D = 20k L = 40k 15° 4″ 21⁄2″ 3″ 1′-0″ FIGURE P8.4-13 8.4-14 Same as Problem 8.4-13, but use ASD. For the two axes, Mux = B1x Mntx = 1.0(75) = 75 ft-kips Muy = B1yMnty = 1.0(30) = 30 ft-kips Try a W10 ꟈ 49. Joint ASCE-AASHO Committee on Flexural Members. We first define the FIGURE 7.28 422 Chapter 7 Simple Connections variable a as the ratio of the moment per unit length along the bolt line to the moment per unit length at the face of the stem. For fillet welds made with the shielded metal arc process, the throat is the perpendicular distance from the corner, or root, of the weld to the hypotenuse and is equal to 0.707 times the size of the weld. Many important details have not been mentioned, but many small commercial buildings are constructed essentially in this manner. ⎝ 16 ⎟⎠ Using this constant, the design shear strength of the 3⁄16-inch fillet weld in Example 7.12 is fRn = 1.392 × 3 sixteenths = 4.176 kips冀in. 678 Chapter 10 Plate Girders For LRFD, the resistance factor is fv = 0.90. Select a W12 of A992 steel. x–(28) = 8(4)(2) or x– = 2.286 in. FIGURE 5.23 SOLUTION It is more convenient to express the deflection in inches than in feet, so units of inches are used in the deflection formula. Seminar Series. In the Group B category, the designation is F2280 (same strength as A490). If there is also a transverse weld at the end, then Ae = Ag. Assuming the latter, we have 1 φt Pn = 0.75 Fu Ae = 0.75(58)⎛ ⎞ (8) = 174.0 kips ⎝ 2⎠ Design for a factored load of 129.6 kips and use E70 electrodes. Use the provisions of AISC F13.1 to determine whether we need to account for this loss. Thus the bearing pressure is concentrated near the column outline. Required Ag = Pa 400 = = 33.3 in.2 0.6 Fcr 0.6(20) Try a W18 × 119: Ag = 35.1 in.2 > 33.3 in.2 KL 26 × 12 = = 116.0 < 200 rmin 2.69 Fe = Since p 2E (KL冀r)2 = p 2 (29, 000) (116.0)2 (OK) (OK) = 21.27 ksi KL E > 4.71 = 113, AISC Equation E3-3 applies. Since joining the Department of Civil Engineering at The University of Memphis, he has received two NASA/ASEE Summer Faculty Fellowships at the Marshall Space Flight Center in Huntsville, Alabama. BRACED VERSUS UNBRACED FRAMES As explained in Section 6.3, “Methods of Analysis for Required Strength,” there are two types of secondary moments: P-d (caused by member deflection) and P-Δ (caused by the effect of sway when the member is part of an unbraced frame [moment frame]). Play/Stop. 74 Chapter 3 Tension Members The tables for design of tension members give values of Ag and Ae for various shapes based on the assumption that Ae = 0.75Ag. In addition, the corresponding available strengths based on yielding and rupture (fracture) are given. Use A36 steel for the web plate and use LRFD to design a connection similar to the one shown in Figure 8.37a for a factored moment of 220 ft-kips and a factored reaction of 45 kips. Larger values can be used for the actual width, but this effective width should be used in the computations if it is smaller than the actual width. For an effective length of KL = Kx L 16 = = 13.7 ft rmx 冀rmy 1.17 the design strength is 726 kips (for KL = 14 ft). The weld size, in sixteenths of an inch, required for strength is D= P 87.05 = = 1.2 sixteenths 0.9279 L 0.9279(78.8) Use the minimum size of 3⁄16 inch (from Table J2.4, based on the web thickness). The ASD design of the steel anchors is identical to what was done for LRFD and will not be repeated here. Design an A36 base plate for a column dead load of 98 kips and a live load of 145 kips. Inspection of the bolt installation is also simplified when this type of direct tension indicator is used, as only a feeler gage is required. Use an elastic analysis and determine the required size if slip is permitted. This expression is Equation 5.7 for shearing stress multiplied by the width of the shear plane. Use A500 Grade B steel (Fy = 46 ksi). • There are concentrated service loads at the quarter points consisting of 45 kips dead load and 140 kips live load each. The allowable stress is Fb = 0.6Fcr = 0.6(37.20) = 22.3 ksi The applied bending moment is Ma = 1 1 wa L2 = (0.400 + 1.000)(45)2 = 354.4 ft-kipss 8 8 214 Chapter 5 Beams and the applied stress is fb = ANSWER M a 354.4(12) = = 29.7 ksi > 22.3 ksi (N.G.) Sx 143 Since fb > Fb, the beam does not have adequate moment strength. The beam has continuous lateral support. • The available strengths, fc Pn for LRFD and Pn冀Ωc for ASD, were computed. In such cases, the designer has several choices: (1) ignore the eccentricity, assuming that the effects are negligible; (2) account for the eccentricity; (3) use a staggered pattern of fasteners that would preserve the symmetry; or (4) add an extra bolt and remove the eccentricity. The force in the concrete is C, and the corresponding concrete area in the transformed section is Ac = C C = stress in transformed area Fy As a further simplification, the moment of inertia of the concrete about its own centroidal axis is neglected. 9.4-2 A fully composite floor system consists of 27-foot-long, W14 × 22 steel beams spaced at 8 feet and supporting a 4-inch-thick reinforced concrete floor slab. π Use a 3⁄4-inch-diameter threaded rod (Ab = 0.442 in.2). We have considered this type of buckling up to now. If the ordinate is the percentage of values rather than the actual number of values, the graph is referred to as a relative frequency distribution. The number of connectors needed for the x-axis flexural buckling strength must be determined from the requirement of Equation 4.14 that the slenderness of one angle between connectors must not exceed threefourths of the overall slenderness of the double-angle shape. The final results should then be expressed to a precision consistent with this procedure, usually to one significant figure less than the intermediate results, to account for roundoff error. 68.0 psi 1.5-3 a. 9.7 COMPOSITE BEAMS WITH FORMED STEEL DECK The floor slab in many steel-framed buildings is formed on ribbed steel deck, which is left in place to become an integral part of the structure. In this example, the spacing is equal to twice the length, so either type could be used. controls the base metal shear strength. The effect of Cb on the nominal strength is illustrated in Figure 5.16. D = 560 kips L = 68 kips 20⬘ W12 × 79 A992 steel FIGURE P4.3-7 4.3-8 Determine the maximum axial compressive service load that can be supported if the live load is twice as large as the dead load. P = 80 k Mntx = 133 ft-k Mnty = 43 ft-k 16′ Mntx = 27 ft-k Mnty = 9 ft-k FIGURE P6.8-7 6.8-8 Use LRFD and select the lightest W12 shape of A992 steel to be used as a beam– column in an unbraced frame. Find the test bank and Solution Manual you need for your classes from Bank is collection of Steel Design • Structural Analysis STEEL DESIGN SEGUI 4TH EDITION SOLUTION. b 3.78 = = 0.179 < 0.2 d 21.1 (OK) Try b = 6 in. Because of the small size required for strength, try intermittent welds. Locate your local office at: international.cengage.com/region. The allowable strength of the weld is Rn 5.568 = = 2.784 kips冀in. Since the tension field does not actually exist until the web begins to buckle, its contribution to the web shear strength will not exist until the web buckles. CHAPTER 3 Tension Members 3.1 INTRODUCTION Tension members are structural elements that are subjected to axial tensile forces. Required d = 0.825 in. In end panels 2 a a ⎡ 260 ⎤ >⎛ b. Use Fy = 50 ksi, f c′ = 4 ksi, and select a W shape for a fully composite floor system. AISC Specification Requirements The AISC Requirements for column web reinforcement are covered in Section J10, “Flanges and Webs with Concentrated Forces.” For the most part, these provisions are based on theoretical analyses that have been modified to fit test results. Elastic Stresses in Composite Beams Although the available strength of composite beams is usually based on conditions at failure, an understanding of the behavior at service loads is important for several reasons. If the column web has insufficient shear strength, it must be reinforced. Tables 4-13 through 4-20 give capacities for both f c′ = 4 ksi and f ′c = 5 ksi concrete. 16 3 Rn = 1.2c tFu = 1.2(2.188) ⎛ ⎞ (58) = 57.11 kips ⎝ 8⎠ c = s − h = 3 − The upper limit is 2.4dtFu = 39.15 kips < 57.11 kips ⬖ Use Rn = 39.15 kips for this bolt. If the reaction is assumed to act through the center of this contact length, the eccentricity of the reaction with respect to the weld is e = 0.75 + 3.25 = 2.375 in. A typical loading condition that gives rise to torsion is shown in Figure 5.48a. a. Column FG, Pu = 240 kips. 4 8 8 and the net flange area is dh = 7⎞ ⎛ A fn = A fg − t f ∑ d h = 3.494 − 0.535 ⎜ 2 × ⎟ = 2.558 in.2 ⎝ 8⎠ Fu A fn = 65(2.558) = 166.3 kips Determine Yt. For A992 steel, the maximum Fy冀Fu ratio is 0.85. Metal deck: Built-up roof: Snow: Purlin weight: SOLUTION 2 psf 5 psf 18 psf of horizontal projection of the roof surface 12 pounds per foot (lb冀ft) of length Calculate loads. The difference between strand and wire rope is illustrated in Figure 3.27. 5.15-5(a), the beam is satisfactory: Result of interaction equation = 0.626. Answers to selected problems are given at the back of the book, and an instructor’s manual with solutions, PowerPoint slides of all figures and tables, and Lecture Builder slides of all equations and Example Problems in the book are available through the book website. x 1.67 ⎞ ∴U = 1 − ⎛⎜ ⎞⎟ = 1 − ⎛⎜ = 0.7217 ⎝ ⎠ ⎝ 6 ⎟⎠ Ae = AnU = 5.02(0.7217) = 3.623 in.2 The alternative value of U could also be used. Use Fy = 50 ksi and Fu = 65 ksi. From Equation 7.12, Tb′ – Ma–a = daMa–a (7.16) Tb ′ (1 + δα) (7.17) M a− a = For LRFD, let ⎛ pt 2f Fy ⎞ M a− a = design strength = φb M p = φb ⎜ ⎟⎜ 4 ⎟⎟⎠ where f b = 0.90. Purchase any of our products at your local college store or at our preferred online store www.cengagebrain.com. First, assuming the same strength for both the longitudinal and transverse welds, 129.6 = 23.28 in. If the safety factor Ω is 1.67, what is the required nominal strength based on the required service load strength? This usually involves some duplication but is necessary if a reader is interested in only the ASD solution. The factor Cm is given as ⎛ aP ⎞ Cm = 1 + Ψ ⎜ r ⎟ ⎝ Pe1 ⎠ (AISC Equation C-A-8-2) The factor Ψ has been evaluated for several common situations and is given in Commentary Table C-A-8.1. FIGURE 6.8 6.6 Members in Braced Frames 315 FIGURE 6.9 EXAMPLE 6.3 The member shown in Figure 6.10 is part of a braced frame. The beam flange force is Ffu = Mu d − t fb where tfb is the beam flange thickness. C., Galambos, T. This reinforcement can take the form of transverse stiffeners or web doubler plates. Yield to tensile ratio, max. cos q cos(65.26°) If welds are used on both sides of the stiffeners, the available length for welding is L = 19.7(4) = 78.8 in. In a shear connection (simple framing), the connecting parts must be flexible enough to permit the connection to rotate under load. fillet weld, using E70XX electrodes. SOLUTION Determine whether the shape is compact, noncompact, or slender: l= bf = 10.2 2t f l p = 0.38 lr = 1.0 29, 000 E = 0.38 = 9.15 50 Fy E 29, 000 = 1.0 = 24.1 Fy 50 Since lp < l < lr , this shape is noncompact. This approach is based on analysis of patterns of yield lines that form a collapse mechanism. 4.75 . This is explained in AISC Sections J3.6 and J3.10 in the User Notes. Thus the composite section must have adequate strength to resist all loads, including those applied to the steel beam before the concrete cures (except for construction loads, which will no longer be present). Local Web Yielding. This constant stress region is called the yield plateau, or plastic range. Replace b with N and obtain t≥ FIGURE 5.41 2 Pu 2 0.90 BNFy 5.14 FIGURE 5.42 Beam Bearing Plates and Column Base Plates 261 bf c d c c c or t≥ 2 Pu 0.90 BNFy (5.15) where is the larger of m and n. The bending factor is ⎛M ⎞ ⎛ 60 ⎞ Cm = 0.6 − 0.4 ⎜ 1 ⎟ = 0.6 − 0.4 ⎜ ⎟ = 0.2571 ⎝ M2 ⎠ ⎝ 70 ⎠ For the braced condition, Pe1 = p 2 EI * p 2 EI x* 2 = (K1 L) (K x L)2 6.7 FIGURE 6.20 305 k Members in Unbraced Frames 33.6 k 85 k 60ft-k 15ft-k 79.2ft-k 70ft-k 18ft-k 79.2ft-k Gravity (D + L) Gravity (D) (a) Load combination 2 (D + L) Wind (0.6W) (b) Load combination 5 (D + 0.6W) 25.2 k 250 k 48.75ft-k 59.4 ft-k 57ft-k 59.4 ft-k Gravity (D + 0.75L) Wind [0.75(0.6 W)] (c) Load combination 6a [D + 0.75L + 0.75(0.6W)] The modified stiffness is EIx* = 0.8tbEIx = 0.8(1.0)EIx = 0.8EIx Therefore, Pel = 335 π 2 EI x* π 2 (0.8 EI x) π 2 (0.8)(29, 000)(533) = = = 3767 kips (1.0 × 15 × 12)2 (K x L)2 (K x L)2 336 Chapter 6 Beam–Columns Cm Cm = 1 − (a Pr 冀Pe1) 1 − α [(Pnt + Pt)冀Pe1] 0.2571 = 0.295 < 1.0 = 1 − 1.60[(305 + 0)]冀3767 B1 = ∴ Use B1 = 1.0. The required axial compressive strength is Pr = Pa = Pnt + B2 Pt = 305 + 0 = 305 kips Mr = M ax = B1 M nt + B2 M t = 1.0(70) + 0 = 70 ft-kips From the beam design charts with Lb = 15 ft and Cb = 1.0, Mp Mn = 224 ft-kips and = 237 ft-kips Ωb Ωb Figure 6.21 shows the bending moment diagram (absolute values shown) for the gravity-load moments to be used in the computation of Cb. Cb = = ∴ But 2.5 M max 12.5 M max + 3 M A + 4 M B + 3 MC 12.5(70) = 2.244 2.5(70) + 3(27.5) + 4(5) + 3(37.5) Mn = Cb (224) = 2.244(224) = 503 ft-kips Ωb Mp = 237 ft-kips < 503 ft-kips Ωb ∴ Use Mn = 237 ft-kips. To compute the sidesway moment, the fictitious support is removed, and a force equal to the artificial joint restraint, but opposite in direction, is applied to the frame. 10.7 Design 711 From Equation 10.3, the shear to be transferred is f = 0.045h Fy3 (36)3 = 0.045(62) = 3.539 kips冀 in. For this reason, the generic term yield stress is used, and it can mean either yield point or yield strength. Select a shape that satisfies the area requirement. The bolt loading in the connection in Figure 7.4a, with only one shear plane, is called single shear. These are specially designed bolts that must be installed with specially designed power wrenches. The amplified primary LRFD moment is 1.12 × Mu = 1.12(107.1) = 120 ft-kips ANSWER 6.5 Amplification factor = 1.12. “Wind and Snow Load Factors for Use in LRFD.” Journal of the Structural Division, ASCE 104 (no. Both surfaces contribute to the total strength, and the resistance to block shear will be the sum of the strengths of the two surfaces. Pn冀Ωc = 791 kips b. E. Select threaded rods of A36 steel for these members. 9.5-3 A floor system has the following characteristics: • span length = 30 ft • beam spacing = 7 ft • slab thickness = 5 in. Compute the moment of inertia of the transformed section. Composite Beams with Formed Steel Deck 9.7-1 A fully composite floor system consists of W27 × 84 beams spanning in the long direction of a 28-foot × 43-foot bay. For a W12 × 65, it is given as 107 ft-kips. From AISC Equation E7-18, the effective width of the slender element is be = 1.92t E f ⎡ 0.38 E ⎤ ⎢1 − b /t f ⎥ ≤ b ⎣ ⎦ (AISC Equation E7-18) For the 8-inch side, using f = Fy and the design thickness* from the dimensions and properties table, be = 1.92(0.116) * 29, 000 ⎡ 0.38 ⎢1 − 46 ⎢⎣ (66.0) 29, 000 ⎤ ⎥ = 4.784 in. ⎛ 7⎞ ⎤ ⎡ Ae = An = twn = t (wg − ∑ d hole) = t ⎢6.5 − 2 ⎜ ⎟ ⎥ = 4.750t ⎝ 8⎠ ⎦ ⎣ Let 4.750t = 2.786 in.2 or t = 0.587 in. Such a graph can be constructed for any cross-sectional shape and specific values of Fy and Cb by using the appropriate equations for moment strength. At a strain of approximately 12 times the strain at yield, strain hardening begins, and additional load (and stress) is required to cause additional elongation (and strain). The three conditions mentioned thus far are included, as well as some for which end translation is possible. This hanger must support a service dead load of 20 kips and a service live load of 60 kips. 2 The minimum thickness is tb 0.525 = = 0.263 in. Neglect the beam weight and determine whether it is satisfactory for each of the loading conditions shown. What is the change in length in inches? 295 Problems a. For LRFD, the design bearing strength is fc Pp , where fc = 0.65. When (A fc + A ft) d. For certain arrangements of bolts, block shear can also occur in gusset plates. If a unit throat size is used in the computations, the same numerical value can also be expressed as kips per linear inch. For a given span, the engineer determines the number of joist spaces, then from the joist girder weight tables selects a depth of girder. and Johnson, J. The eccentricity e is 10 + 8 – 2.286 = 15.71 in., and the torsional moment is M = Pe = 60(15.71) = 942.6 in.-kips If the moment of inertia of each horizontal weld about its own centroidal axis is neglected, the moment of inertia of the total weld area about its horizontal centroidal axis is Ix = 1 (1)(12)3 + 2(8)(6)2 = 720.0 in.4 12 Similarly, 1 I y = 2 ⎡ (1)(8)3 + 8(4 − 2.286)2 ⎤ + 12(2.286)2 = 195.0 in.4 ⎦⎥ ⎣⎢ 12 and J = Ix + Iy = 720.0 + 195.0 = 915.0 in.4 Figure 8.18 shows the directions of both components of stress at each corner of the connection. The bearing pressure will equal the total bolt tension divided 492 Chapter 8 Eccentric Connections FIGURE 8.12 by the area of contact. That is, use Fnw = 0.85(0.6FEXX) for the longitudinal welds and Fnw = 1.5(0.6FEXX) for the transverse welds: Rn = 0.85Rnw + 1.5Rnwt (AISC Equation J2-10b) Because AISC permits the larger of the two options to be used, it is permissible to use either and be conservative at worst. Since this is greater than 0.8, use Yt = 1.1. Yt Fy A fg = 1.1(50)(3.494) = 192.2 kips Since Fu Afn < Yt Fy Afg, the holes must be accounted for. These limit states are covered in Chapter J of the Specification “Design of Connections.” For web yielding, when the load is at a distance greater than the girder depth from the end, the nominal strength is Rn = Fyw tw(5k + b) (AISC Equation J10-2) When the load is less than this distance from the end, Rn = Fyw tw(2.5k + b) (AISC Equation J10-3) where k = distance from the outer face of the flange to the toe of the fillet in the web (for rolled beams) or to the toe of the weld (for welded girders) b = length of bearing of the concentrated load, measured in the direction of the girder longitudinal axis (not less than k for an end reaction) Fyw = yield stress of the web For LRFD, the resistance factor is f = 1.00. As mentioned previously, some accidental moment will be present, but in most cases it can be ignored. ⎝ 8⎠ 508 Chapter 8 Eccentric Connections FIGURE 8.22 From Equation 7.36, the base metal shear rupture strength per unit length is 3 fRn = 0.45Fu t = 0.45(58) ⎛⎜ ⎞⎟ = 9.788 kips冀in. Use Kx = Ky = 1.0. a. 6 Chapter 1 1.2 Introduction LOADS The forces that act on a structure are called loads. Although the top chord is normally treated as an axially loaded compression member, if purlins are placed between the joints, their reactions will cause bending, which must be accounted for. The provided length of 4.5 inches will therefore be adequate. A complete derivation of these equations is given by Tide (2001). The values of Cb, B1, and B2 must be calculated independently for use in the computation of Mr (Mu for LRFD or Ma for ASD). For this reason, most design specifications, including the AISC Specification, contain empirical formulas for inelastic columns. thick gusset plate, also of A242 steel, with 3⁄4-inch diameter bolts as shown in Figure P3.5-1. No testing program is required for LH- or DLH-series joists. “Strength of DoubleRow Bolted-Web Connections.” Journal of Structural Engineering, ASCE 109(1): 126–142. Problems 657 Steel Headed Stud Anchors 9.4-1 A fully composite floor system consists of 40-foot-long, W21 × 57 steel beams spaced at 9 feet center-to-center and supporting a 6-inch-thick reinforced concrete floor slab. Use Fy = 50 ksi for the angle tension member and Fy = 36 ksi for the gusset plate. Placing several of these frames in parallel and connecting them with additional members that are then covered with roofing material and walls produces a typical building system. The procedure gives conservative results when used with slip-critical bolts and with A490 (or other Group B) bolts (AISC, 1994). The tensile stress in the plastic regime of the steel shaft is approximated. Try a plate 5⁄16 × 31⁄2. Check Equation 6.9 with p = 4.51 × 10−3, bx = 6.49 × 10−3, by = 17.6 × 10−3 pPa + bx Max + by May = (4.51 × 10−3)(100) + (6.49 × 10−3)(50) + (17.6 × 10−3)(20.8) = 1.14 > 1.0 (N.G.) ANSWER Use a W10 × 49. No: 628 kips > 520 kips 4.4-1 148 kips without iteration, 161 kips with iteration 4.7-1 1260 kips 4.7-3 a. 10.7-8 Design a plate girder for the following conditions: • Span length = 100 ft. The geometry of the transformed section will then be different from what was originally assumed; to obtain an accurate result, the location of the neutral axis should be recomputed on the basis of this new geometry. If the load is applied and removed many times over the life of the structure, fatigue stress becomes a problem, and its effects must be accounted for. Although some degree of axial load will be present in any structural member, in many practical situations this effect is negligible and the member can be treated as a beam. Part 9. From AISC Equations F13-3 and F13-4: For a ≤ 1.5, h 10.7 Design 699 29, 000 E ⎛ h⎞ = 12.0 = 12.0 = 340.6 ⎜⎝ ⎟⎠ 36 t w max Fy min t w = 62 = 0.182 in. Assume that the slip load controls. ⎜ ⎠ where D = number of sixteenths of an inch of weld size t = thickness of the base metal EXAMPLE 7.14 A connection of the type used in Example 7.12 must resist a service dead load of 9 kips and a service live load of 18 kips. Shored Construction In shored construction, only the composite beam need be considered, because the steel shape will not be required to support anything other than its own weight. For negative moment, as would be the case with gravity loading, these forces are directed as shown in Figure 8.43, with the top flange of the beam delivering a tensile force to the column and the bottom flange delivering a compressive force. Equations 5.3 and 5.4 are valid as long as the loads are small enough that the material remains within its linear elastic range. From Table 7-1, the shear strength is rn = 11.9 kips冀bolt Ω For the inner bolts with a spacing of 3 inches, the bearing strength from Table 7-4 is rn = 58.5t = 58.5(0.560) = 32.8 kips冀bolt Ω For the edge bolts, use Table 7-5 and a conservative edge distance of 11⁄4 inches. Check the flexural buckling strength about the y-axis (this is the axis of no symmetry for a channel): 4.8 Torsional and Flexural-Torsional Buckling 161 K y L 13 × 12 = = 180.3 0.865 ry Fe = π 2E π 2 (29, 000) = = 8.805 ksi (KL冀r)2 (180.3)2 E 29, 000 = 4.71 = 133.7 Fy 36 4.71 Since KL E > 4.71 , AISC Equation E3-2 applies: r Fy Fcr = 0.877Fe = 0.877(8.805) = 7.722 ksi The nominal strength is Pn = Fcr Ag = 7.722(14.7) = 113.5 kips Compute the flexural-torsional buckling strength about the x-axis (this is the axis of symmetry for a channel): K x L 13 × 12 = = 29.77 5.24 rx Fey = π 2E π 2 (29, 000) = = 323.0 ksi (KL冀r)2 (29.77)2 ⎡ π 2 ECw ⎤ 1 Fez = ⎢ 2 + GJ ⎥ 2 ( ) K L ⎣ z ⎦ Ar o 1 ⎡ π 2 (29, 000)(492) ⎤ + 11, 200(2.65) ⎥ = 80.06 ksi (r o is tabulated) =⎢ 2 (13 × 12) 14 . We consider these infrequent cases in Section 4.8. 4.4 LOCAL STABILITY The strength corresponding to any overall buckling mode, however, such as flexural buckling, cannot be developed if the elements of the cross section are so thin that local buckling occurs. Use Fy = 50 ksi and a total depth of 4 feet. () () 9 (bx Mux + by Muy) = 0.5(2.09 × 10 −3)(54.97) 8 9 + [(3.04 × 10 −3)(205.7) + 0] = 0.761 < 1.0 (OK) 8 0.5 pPu + This result is significantly smaller than 1.0, so try a shape two sizes smaller. They present the guidelines and criteria that enable a structural engineer to achieve the objectives mandated by a building code. Use an estimated shear lag factor of U = 0.85. 3): 107–115. EXAMPLE 4.15 Compute the compressive strength of a WT12 × 81 of A992 steel. ⎝ 16 ⎠ The base metal shear strength is therefore 12.2 kips冀in. Bars and plates are formed by hot-rolling. 3.7-4 As shown in Figure P3.7-4, members AC and BD are used to brace the pin-connected structure against a horizontal wind load of 10 kips. From Figure 10.12, the maximum factored load moment is Mu = 3168 ft-kips < 3350 ft-kips ANSWER (OK) The flexural strength is adequate. In Figure 4.10, the ratio 26(12)冀rx must be compared with 13(12)冀ry (where rx and ry are in inches), and the larger ratio would be used for the determination of the axial compressive strength. There are no W8 shapes with enough load capacity. L. 48.2 kips b. Figure 7.30 shows a structural tee segment connected to the flange of a column for the purpose of attaching a bracing member. The higher you move up the series, the greater the available span lengths and load-carrying capacities become. (5.5 − 0.650) = 2.425 in. 240 240 From Table 3-23, “Shears, Moments, and Deflections,” in Part 3 of the Manual, the maximum deflection (at midspan) for two equal and symmetrically placed loads is Δ = Pa (3 L2 − 4 a 2) 24 EI where P = magnitude of concentrated load a = distance from support to load L = span length 20(6 × 12) 13.69 × 10 6 [3(24 × 12)2 − 4(6 × 12)2] = EI 24 EI 13.69 × 10 6 = = 1.11 in. In the usual floor system, the ribs will be perpendicular to the floor beams and parallel to the supporting girders. Once composite behavior is attained, additional loads, both dead and live, will be supported by the composite beam. Design of Bracing Connections and Truss Connections. Roof beam: Supporting plaster ceiling Supporting nonplaster ceiling Not supporting a ceiling L 冀360 L 冀240 L 冀180 L 冀240 L 冀180 L 冀120 L 冀360 L 冀240 L 冀180 Floor beam L 冀360 L 冀240 — Type of member 226 Chapter 5 Beams EXAMPLE 5.9 Compute the dead load and live load deflections for the beam shown in Figure 5.23. FIGURE 5.28 SOLUTION If the weight of the beam is neglected, the central half of the beam is subjected to a uniform moment, and MA = MB = MC = Mmax, ∴ Cb = 1.0 Even if the weight is included, it will be negligible compared to the concentrated loads, and Cb can still be taken as 1.0, permitting the charts to be used without modification. 1990. 16 4 16 16 Check bearing on both the tension member and the gusset plate. The required strength is Pa = D + L = 2 + 6 = 8 kips From Equation 3.7, the allowable tensile stress is Ft = 0.375Fu = 0.375(58) = 21.75 ksi and the required area is Ab = ANSWER Pa 8 = = 0.3678 in.2 Ft 21.75 Use a 3⁄4-inch-diameter threaded rod (Ab = 0.442 in.2). The plate thickness is determined from a flexural analysis of a cantilever strip of unit width and of length c. Such an analysis can be made with the alignment chart for braced frames. D = 90k L = 260k 15′-4″ FIGURE P4.9-11 Problems 4.9-12 187 Use ASD and select a WT section for the compression member shown in Figure P4.9-12. = f ⎜⎝ Lb ⎟⎠ 0.75 ⎜⎣ (18 × 12) ⎟⎦ The axial stiffness of the brace is given by F冀d, where d is the axial deformation of the brace. Part 6 covers members subject to combined axial tension and flexure, combined axial compression and flexure, and combined torsion, flexure, shear, and/or axial force. I 5836 R R fv = = = 0.01527 R kips冀 in. The design of moment-resisting end-plate connections requires determination of the bolt size, the plate thickness, and the weld details. Problems 91 Section FIGURE P3.2-3 3.2-4 A PL 3⁄8 × 6 tension member is welded to a gusset plate as shown in Figure P3.2-4. The steel is A992, and the concrete strength is fc′ = 4 ksi. ⎛ b ⎞ ≤ 0.56 E ⎝ t ⎠ st Fyst (AISC Equation G3-3) where ⎛ b ⎞ = width-to-thickness ratio of the stiffenner cross section ⎝ t ⎠ st Fyst = yield stress of the stiffener The second requirement is for the moment of inertia of the stiffener or pair of stiffeners. So the weld strength of 4.176 kips冀in. Problems 173 D = 180k L = 540k 13⬘ W14 × 90 A992 steel FIGURE P4.3-6 4.3-7 Determine whether the compression member shown in Figure P4.3-7 is adequate to support the given service loads. The first is to add 1 to the last digit retained. Finally, combination (6) should be investigated for an overturning effect. This means that a reduced stiffness, EI*, was used in the analysis, and an effective length factor of Kx = 1.0 can be used. Ω 2.00 Check the base-metal shear. The column strength curve shown in Figure 4.6 is unchanged except for renaming the abscissa KL冀r. The overall objective of a structure is to transmit loads to the foundation. > 12 in. Assume that all spacing and edge distances are satisfactory, including those that permit the maximum nominal bearing strength of 2.4dtFu to be used. A procedure for the determination of prying forces, based on research reported in Guide to Design Criteria for Bolted and Riveted Joints (Kulak, Fisher, and Struick, 1987), is given in the Manual in Part 9, “Design of Connection Elements.” FIGURE 7.27 7.8 High-Strength Bolts in Tension 421 This method is presented here in a somewhat different form, but it gives the same results. Other commonly used structural steels are ASTM A572 Grade 50 and ASTM A992. ANSWER wu = 16(1170) = 20.8 kips冀ft (30)2 EXAMPLE A.2 If the beam in Example A.1 does not have continuous lateral support, determine where it must be braced. If the cross-sectional area of a tension member varies along its length, the stress is a function of the particular section under consideration. The compression side bolts must be capable of resisting the entire vertical shear. Assume a shape, compute the available strength, and compare it with the required strength. Problems 715 a. Problems 6″ 467 31⁄2″ 21⁄2″ 5 @ 31⁄2″ = 1′-51⁄2″ 2″ 2″ FIGURE P7.6-6 Design 7.7-1 A C9 × 20 is used as a tension member and is connected to a 1⁄2-inch gusset plate as shown in Figure P7.7-1. The effect of partially restrained connections is to shift the moment diagram as shown in Figure 8.36c. Compute the design strength based on shear and bearing c. The allowable strength of this problem is practically identical to the load and resistance factor design solution. ANSWER 3.5 Allowable strength = 318 kips. AISC E6 contains many details regarding the connection requirements and the proportioning of the plates. If the connections are not momentresisting, however, there will be no continuity between columns and girders, and the alignment chart cannot be used. If Fy = 50 ksi, is the stiffness reduction factor applicable to these columns? Figure 6.6 shows a member of this type subjected to equal end moments producing single-curvature bending (bending that produces tension or compression on FIGURE 6.6 312 Chapter 6 Beam–Columns one side throughout the length of the member). Although this procedure assumes elastic behavior, it will be conservative when used in an LRFD context. The beams are spaced at 8 feet, and the span length is 40 feet 8 inches. ANSWER ASD SOLUTION Use a W14 × 74 (lightest of the three possibilities). Solve for the load capacity from the equation ∑ M IC = 0 where IC is the instantaneous center. PL 3⁄8 × 6 FIGURE P3.2-4 3.2-5 The tension member shown in Figure P3.2-5 is a PL 1⁄2 × 8 of A36 steel. EXAMPLE 5.13 Part of a floor framing system is shown in Figure 5.32. Johnston, Bruce G., ed. Thirtyfour 3⁄4-inch × 3-inch shear studs are used for each beam. ⎝ 8⎠ The weld strength of 3.712 kips冀in. Refer to Figure 8.22. Furthermore, they are grouped so that the shape at the top of each group (in bold type) is the lightest one that has enough section modulus to satisfy a required section modulus that falls within the group. If stiffeners are not present or are spaced too far apart, tension-field action will not be possible, and the shear capacity will consist only of the strength before buckling. FIGURE 8.49 dc θ db The shear force to be resisted by the web reinforcement is 134.7 − 80.19 = 54.51 kips. If the stiffeners were not provided, the web would need to be protected from yielding and crippling. 1983. 4.9-5 Compute the nominal axial compressive strength based on flexural buckling (no torsional or flexural-torsional buckling). For an interior beam, the effective width is the smaller of Span 30(12) = = 90 in. Jr. 1963. All notation in Steel Design is consistent with that in the Manual, and AISC equation numbers are used along with sequential numbering of other equations according to the textbook chapter. Try 30 studs per beam, so that N1 provided = 30冀2 = 15. A structural analysis of the effects of an earthquake requires an analysis of the structure’s response to the ground motion produced by the earthquake. 3.13 in.2 d. The value of Lp , however, must be computed from AISC Equation F2-5: L p = 1.76ry E 29, 000 = 1.76(3.70) = 156.8 in. (based on the thickness of the flange, which is the thinner connected part). A. Va ≤ Rn Ω For bolt shear, Ω = 2.00 Rn = Fnv Ab × 4 bolts End Plate Connections 565 566 Chapter 8 Eccentric Connections For bolt bearing, Ω = 2.00 Rn = 1.2LctFu ≤ 2.4dbtFu per bolt For the four bolts, Rn = 2Rn (inner bolts) + 2Rn (outer bolts) 7. 6.5(5冀8) A KL 0.65(3) = 10.81 = 0.1804 r r= From AISC J4.4, for compression elements with KL冀r ≤ 25, the nominal strength is Pn = Fy Ag (AISC Equation J4-6) and for LRFD, f = 0.9 ⎛5 ⎞ fPn = 0.9 Fy Ag = 0.9(36) ⎜ × 6.5⎟ = 132 kips > 121.2 kips (OK) ⎝8 ⎠ Check block shear in the plate. Compute the available strength for an effective length of 16 feet with respect to both axes. The bolts have splined ends over which an inner socket of the wrench fits. Compute Lp and Lr. Use the equations in Chapter F of the AISC Specification. The need for a torsional buckling analysis of a doubly symmetrical shape will be rare. Figure 5.31(d) is a cross section of this bay, showing the floor beams as wide-flange steel shapes supporting a reinforced concrete floor slab. Problems 283 PD = 45k 15′ 25′ wD = 1.0 k/ft wL = 2.5k/ft W30 × 116 40′ FIGURE P5.5-3 5.5-4 The beam shown in Figure P5.5-4 has continuous lateral support of both flanges. In addition, changes were made to many provisions of the specification, both in form and substance. The N1 anchors should be uniformly spaced within the length where they are required. I would also like to express my appreciation to Chris Shortt, Publisher, Global Engineering at Cengage Learning; Randall Adams, Senior Acquisitions Editor; Hilda Gowans, Senior Developmental Editor, who has been especially helpful and supportive, and Rose Kernan of RPK Editorial Services. The 0.75 factor in some of the combinations accounts for the unlikelihood that all loads in the combination will be at their lifetime maximum values simultaneously. Check tension, shear, and bearing against the usual strengths. The shear yield design strength per unit length is fRn = 1.0(0.6Fyt) = 0.6Fyt for a one-inch length (7.35) and the base-metal shear rupture design strength per unit length is fRn = 0.75(0.6Fut) = 0.45Fut for a one-inch length ASD: The weld allowable shear strength from Equation 7.31 is Rn 0.707wFnw 0.707(1冀16)(0.6 × 70) = = = 0.9279 kips冀in. Lateral support is provided at the ends of the beam. Up to this stress, the specimen can be unloaded without permanent deformation; the unloading will be along the linear portion of the diagram, the same path followed during loading. The total bearing strength is 4 × 91.35 = 365 kips > Vu = 33.6 kips (OK) The plate length, using detailing dimensions and the notation of Figure 8.51, is d + 2pfo + 2de = 17 3⁄4 + 2(2) + 2(11⁄2) = 243⁄4 in. Until fairly recently, connections were either welded or riveted. Steel Design, Fifth Edition William T. ed. Try 7⁄8-inch bolts: From Manual Table 7-1, assuming that the threads are in the shear plane, the shear strength is frn = 24.3 kips冀bolt From Manual Table 7-3, the slip-critical strength is frn = 13.2 kips冀bolt (controls) The number of 7⁄8-inch bolts required is 48.0 = 3.6 bolts 13.2 Four 7⁄8-inch-diameter Group A bolts will be used. i. For shear in the column web, from Equation 8.9 and neglecting the thickness of shims in the computation of dm , the factored load shear force in the column web panel zone is (M1 + M 2) (M1 + M 2) −V = −V dm d b + tPL 210(12) = − 0 = 118 kips 20.8 + 5冀8 F= 544 Chapter 8 Eccentric Connections Because Pr = 0.4Pc , use AISC Equation J10-9: Rn = 0.60 Fy dc t w = 0.60(50)(14.2)(0.485) = 206.6 kiips The design strength is fRn = 0.90(206.6) = 186 kips > 118 kips ANSWER ASD SOLUTION (OK)) Column web reinforcement is not required. Select a rolled steel shape and the steel anchors needed to achieve full composite behavior with a 3.5-inch-thick reinforced concrete floor slab. For ASD, the safety factor is Ω = 2.00. 0.85fc′Ac 3. 2000b. Try a W18 × 130: Ag = 38.3 in.2 KL 26 × 12 = = 115.6 < 200 rmin 2.70 Fe = Since (OK) p 2E p 2 (29, 000) = 21.42 ksi 2 = (KL冀r) (115.6)2 KL E > 4.71 = 113, AISC Equation E3-3 applies. 6.4 THE MOMENT AMPLIFICATION METHOD The moment amplification method entails computing the maximum bending moment resulting from flexural loading (transverse loads or member end moments) by a firstorder analysis, then multiplying by a moment amplification factor to account for the secondary moment. 569 kips b. This increase comes at the expense of a reduction in ductility, however. “Ultimate Strength Prying Models for Bolted T-stub Connections.” Engineering Journal, AISC 39 (no. Determine the value of bx for the following cases. AISC Equation J10-2 is based on limiting the stress on a cross section of the web formed by its thickness and a length of tf b + 5k, as shown in Figure 8.55a. The reason is that the clamping effect, and hence the friction force, is reduced. pPa + bx Max + by May = (3.35 × 10−3)(100) + (5.90 × 10−3)(50) + (12.6 × 10−3)(20) = 0.882 < 1.0 (OK) Try the next smaller shape. The 250 kip load is a service live load. 2 2 From Table 3-19 in the Manual, for ΣQn = 172.3 kips and Y2 = 4.116 in., f b Mn = 250 ft-kips (by interpolation) > 217 ft-kips (OK) 642 Chapter 9 Composite Construction Check the live load deflection: For ΣQn = 172.3 kips and Y2 = 4.116 in., the lowerbound moment of inertia from Table 3-20 in the Manual is ILB = 538 in.4 (by interpolation) and the live load deflection is ΔL = 5wL L4 5(0.935冀12)(30 × 12)4 = = 1.09 in. The required load capacity is P = 400 kips. From mid-depth to the compression flange: L= d 17.7 − tf b = − 0.425 = 8.425 in. This will increase the positive moment but decrease the negative moment, which is the maximum moment in the beam, thereby potentially resulting in a lighter beam. A plastic hinge is said to have formed at the center of the beam, and this hinge along with the actual hinges at the ends of the beam constitute an unstable mechanism. 6.960 6.960 24.88 P2 = = 3.57 in. Check the stiffener–web assembly as a compression member. Otherwise, Table 4-22 can be used for the flexural buckling strength. The specimen was not loaded to failure. 2 2 Try a plate 5⁄16 × 3: 5 ⎞ Ast = 3 ⎛⎜ × 2 stiffeners = 1.88 in.2 > 1.44 in.2 ⎝ 16 ⎟⎠ (OK) Check for tst ≥ b冀16: b 3 5 = = 0.188 in. We use this latter representation in the material that follows. With the bolts placed in the long leg at the usual gage distance (see Chapter 3, Figure 3.24), the failure block is as shown in Figure 7.22. @ 11⁄4″ 75k 2L5 × 5 × 1⁄2 FIGURE P8.2-4 574 Chapter 8 8.2-5 Eccentric Connections A plate is used as a bracket and is attached to a column flange as shown in Figure P8.2-5. In keeping with the usual assumptions of truss analysis— pinned connections and loads applied only at the joints—each component of the truss will be a two-force member, subject to either axial compression or tension. Approximately 42 ksi e. Select a W16 of A992 steel. The cruciform shape shown is particularly vulnerable to this type of buckling. Problems 367 625 k 195 ft-k W33 × 118 A992 11′ 225 ft-k 625 k FIGURE P6.6-8 6.6-9 The member shown in Figure P6.6-9 is a W12 × 79 of A572 Grade 50 steel and is part of a braced frame. If the assumption turns out to be correct, no iteration is required. This calibration must be done daily during construction for bolts of each size and grade. The mean, x¯ , of a set of sample values, or population, is the arithmetic average, or x = 1 n n ∑ xi i =1 where xi is a sample value and n is the number of values. The effective length is obtained by multiplying the actual length by a factor b, where b = 1.2 − 0.002(冀w) ≤ 1. • Use partial composite action. 644 Chapter 9 Composite Construction As Example 9.12 illustrates, the tables greatly simplify the design of a composite beam with the PNA in the steel shape. 240 Chapter 5 Beams FIGURE 5.31 4 equal spaces of s N L F A B C D L A E B C D (b) (a) s s s 2 s 2 Tributary area F (c) 1 ft2 1 ft E s (d) q (psf) Beam EF (e) w (plf) = q × s E F L Figure 5.31(c) shows a shaded area around floor beam EF. (The load is shared by two thicknesses of web, one on each side of the stiffener pair.) From Equation 7.36, the shear rupture strength is fRn = 0.45Fu t × 2 = 0.45(65)(0.360)(2) = 21.06 kips冀in. To account for loadings of this nature, a truss can be modeled as an assembly of continuous chord members and pin-connected web members. (Note: This shape is slender for compression.) a. ER_09 / Shutterstock This space truss incorporates tension and compression members. Alternative values for W, M, S, and HP shapes 1. 12 2 I= LRFD SOLUTION The factored-load reaction is Pu = 1.2D + 1.6L = 1.2(5) + 1.6(10) = 22 kips. Try a 1⁄4-inch E70 fillet weld. FIGURE 10.16 SOLUTION The factored loads, excluding the girder weight, are shown in Figure 10.16b. Both design strength (LRFD) and allowable strength (ASD) are given. F., 2009). Use Fy = 50 ksi and select a trial cross section. 4.17 in.2 b. 2 2 Try 41⁄2 in. For this reason, cables are often prestretched. This chapter is intended to be introductory only and is by no means a complete guide to the design of building connections. These combinations do not apply in this example, because there are no wind or earthquake loads to counteract the dead load. 5.6 Bending Strength of Noncompact Shapes 213 From Part 1 of the Manual, Iy = 362 in.4 rts = 4.11 in. The fastener forces resulting from the couple can be found by considering the shearing stress in the fasteners to be the result of torsion of a cross section made up of the cross-sectional areas of the fasteners. E70XX electrodes are to be used. The prefix of the designation is either WT, ST, or MT, depending on which shape is the “parent.” For example, a WT18 × 105 has a nominal depth of 18 inches and a weight of 105 pounds per foot, and is cut from a W36 × 210. per sixteenth of an inch in size. LRFD SOLUTION The factored load is wu = 1.2wD + 1.6wL = 1.2(4.5) + 1.6(4.5) = 12.60 kips冀ft and the reaction is Ru = wu L 12.60(15.83) = = 99.73 kips 2 2 Determine the length of bearing b required to prevent web yielding. d 7冀8 = 2.425 − = 1.988 in. The first two conditions mean that there is no bending moment in the member before buckling. • Maximum stud diameter = 3⁄4 inch. The nominal flexural strength is therefore based on yielding of the compression flange, and Mn = 3723 ft-kips The design strength is f b Mn = 0.90(3723) = 3350 ft-kips, which must be compared to Mu, the maximum factored load moment. ANSWER Use two plates 3⁄8 × 4 × 5′-0″ for the intermediate stiffeners. Round HSS with ≥ 1.3D (see Figure 3.7e): U = 1.0 54 Chapter 3 Tension Members FIGURE 3.9 4. Slip is not permitted • Service dead load = 45 kips and service live load = 105 kips Select a double-angle section with unequal legs, long legs back-to-back. + 11⁄4 in. * 28 Chapter 2 Concepts in Structural Steel Design 1.4D = 1.4(109) = 152.6 kips 1.2D + 1.6L + 0.5(Lr or S or R). 9 kips. 6.3 6.3 Methods of Analysis for Required Strength 305 METHODS OF ANALYSIS FOR REQUIRED STRENGTH The foregoing approach to the analysis of members subjected to both bending and axial load is satisfactory so long as the axial load is not too large. 302 Chapter 6 Beam–Columns LRFD Interaction Equations For Pu ≥ 0.2, f cPn Muy ⎞ 8 ⎛ Mux Pu + ⎜ + ≤ 1.0 f c Pn 9 ⎝ f b M nx f b M ny ⎟⎠ For (6.3) Pu < 0.2, fc Pn Muy ⎞ ⎛ Mux Pu +⎜ + ≤ 1.0 2fc Pn ⎝ fb M nx fb M ny ⎟⎠ (6.4) ASD Interaction Equations For Pa ≥ 0.2, Pn 冀Ωc M ay ⎞ Pa 8 ⎛ M ax + + ≤ 1.0 Pn 冀Ωc 9 ⎜⎝ M nx 冀Ωb M ny 冀Ωb ⎟⎠ For (6.5) Pa < 0.2, Pn 冀Ωc M ay ⎞ ⎛ M ax Pa +⎜ + ≤ 1.0 2 Pn 冀Ωc ⎝ M nx 冀Ωb M ny 冀Ωb ⎟⎠ (6.6) Example 6.1 illustrates the application of Equations 6.3–6.6. EXAMPLE 6.1 The beam–column shown in Figure 6.2 is pinned at both ends and is subjected to the loads shown. 9.2 Shored versus Unshored Construction 605 EXAMPLE 9.3 A W12 × 50 acts compositely with a 4-inch-thick concrete slab. wa = wD + wL = 0 + 4.5 = 4.5 kips冀ft 1 1 Required moment strength = M a = wa L2 = (4.5)(30)2 = 506.3 ft-kips 8 8 Mn = required Ωb Assume that the shape will be compact. How many 3⁄4-inch × 3-inch stud anchors are required for full composite behavior? • New material explaining the bolt strength tables has been added. From Equation 7.35, the base metal shear yield strength per unit length is 5 ⎞ φ Rn = 0.6 Fy t = 0.6(36) ⎛ = 6.750 kips冀in. Normally the beamto-column connections in this direction will not transmit moment; sidesway is prevented by bracing; and K can be taken as 1.0. EXAMPLE 4.12 The rigid frame shown in Figure 4.15 is unbraced. The specific procedure is as follows. First, check the upper limit on F′nt: frv = 75 75 = = 21.22 ksi ΣAb 8(0.4418) F nt′ = 117 − 2.5 frv = 117 − 2.222(21.22) = 69.8 ksi < 90 ksi (OK) Check shear: fRn = fFnv Ab × nb = 0.75(54)(0.4418)(8) = 143 kips > 75 kips (OK) Check bearing: φ Rn = φ (2.4 dtFu) × 8 bolts 3 5 ⎞ = 0.75(2.4) ⎛⎜ ⎞⎟ ⎛⎜ (58)(8) = 196 kips > 75 kips ⎝ 4 ⎠ ⎝ 16 ⎟⎠ ANSWER (OK) Use eight bolts. Fy 36 The maximum permissible clear spacing is therefore 12 inches, and the required clear spacing of 1.5 inches is satisfactory. 702 Chapter 10 ANSWER Plate Girders Use a 5⁄16 × 62 web and 11⁄2 × 18 flanges, as shown in Figure 10.17. L2 = 3.944 3.944 L1 = The second option results in a slight savings and will be used. This table is located at the end of the dimensions and properties table for angles. The concrete strength is f c′ = 5 ksi. In general, analysis is a direct procedure, but design is an iterative process and may require some trial and error. The reason is that the x-bracing, indicated by dashed lines, prevents sidesway in the lower story. Try t = 5⁄16 in. Consequently, for a simply supported, uniformly loaded beam, 2N1 anchors will be required, and they should be equally spaced. Load (kips) 0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 5.0 5.5 6.0 6.5 Elongation × 103 (in.) 0 0.160 0.352 0.706 1.012 1.434 1.712 1.986 2.286 2.612 2.938 3.274 3.632 3.976 Load (kips) Elongation × 103 (in.) 7.0 7.5 8.0 8.5 9.0 9.5 10.0 10.5 11.0 11.5 12.0 12.5 13 4.386 4.640 4.988 5.432 5.862 6.362 7.304 8.072 9.044 11.310 14.120 20.044 29.106 redsquarephoto / Shutterstock The John Hancock Building in Chicago is a steel-framed structure. fillet welds for the flange-to-web welds, spaced as shown in Figure 10.23. ST9): 1371–87. 25⬘ W18 × 50 W14 × 99 13⬘ W14 × 99 W18 × 50 13⬘ W14 × 90 B W14 × 90 4.7-9 A FIGURE P4.7-9 180 Chapter 4 4.7-10 Compression Members The rigid frame shown in Figure P4.7-10 is unbraced. For the two plates (four welds), use 6.750 × 2 = 13.50 kips冀in. The total beam reaction must be resisted by the four bolts on the compression side. 5th ed. 5⬙⬙ 1⬙-3⬙⬙ 5⬙⬙ Section FIGURE P4.9-3 4.9-4 An unsymmetrical compression member consists of a 1⁄2 × 12 top flange, a 1⁄2 × 6 bottom flange, and a 5⁄16 × 16 web (the shape is symmetrical about an axis parallel to the web depth). Determine the nominal block shear strength of the tension member. 0.60 Fy dc 冀Ω 0.60(36)(8.25)冀冀1.67 where 89.78 − 53.35 is the extra strength, in kips, to be furnished by the doubler plate. In a more realistic situation, the service loads would be known, and the appropriate combination of these would be used for an ASD solution. A490 bolts have a higher ultimate tensile strength than A325 bolts and are assigned a higher nominal strength. Nodal bracing provides isolated support at specific locations on the member and is not relative to other brace points or other members. No shoring is used. Because the total load on the splice plates is the same as the load on the tension member, for the splice plates to be critical the total splice plate thickness must be less than the thickness of the tension member—and it is. r 02 = x 02 + y02 + Ix + Iy Ag (AISC Equation E4-11) Values of the constants used in the equations for Fe can be found in the dimensions and properties tables in Part 1 of the Manual. This axis usually is the axis with the smaller moment of inertia (we examine exceptions to this condition later). mangojuicy / Shutterstock Many transmission towers are truss structures with both compression and tension members. The length is 10 feet. Ωb Ωb Ωb 326 Chapter 6 Beam–Columns Compute the weak axis bending moments. If the steel shape is supported at a sufficient number of points along its length before the slab is placed, the weight of the wet concrete will be supported by these temporary shores rather than by the steel. 470 Chapter 7 Simple Connections WT6 × 32.5 1 2 D = 30k L = 70k W12 × 65 3⁄4″ diameter Group A bearing-type bolts with threads in shear FIGURE P7.9-2 7.9-3 In the connection of Figure P7.9-3, how many 7⁄8-inch-diameter Group A bearing-type bolts are needed? FIGURE P9.7-3 11⁄2″ 660 Chapter 9 9.7-4 Composite Construction A composite floor system consists of W16 × 31 steel beams supporting a formed steel deck and concrete slab. 340 Chapter 6 For LRFD: For ASD: Beam–Columns p= 1 1 = Pc fc Pn bx = 8 8 = 9 Mcx 9(f bM nx) by = 8 8 = 9 Mcy 9(f bM ny) p= 1 1 = Pc Pn 冀Ωc bx = 8 8 = 9 Mcx 9(M nx 冀Ωb) by = 8 8 = M ny 冀Ωb) 9 Mcy 9(M Table 6-1 gives values of p, bx, and by for all W shapes listed in Part 1 of the Manual, “Dimensions and Properties,” except for those smaller than W8. The vertical members of this frame must also be treated as beam–columns. To avoid this problem, when subtracting, start with additional significant figures if possible. 4.2 COLUMN THEORY Consider the long, slender compression member shown in Figure 4.1a. Both LRFD and ASD are covered in this textbook, but the emphasis is on LRFD. From Equation 6.9, (2.96 × 10−3)(300) + (3.50 × 10−3)(100) = 1.24 > 1.0 (N.G.) There are no other possibilities in the deeper W-shape groups. Figure 3.24 shows that an angle leg must be at least 5 inches long to accommodate two lines of bolts. It is also the basis of most European building codes. What size fillet welds are required for the connection to the column flange? Shims may be one of two types: either conventional or “finger” shims, which can be inserted after the bolts are in place, as shown in Figure 8.37d. Determine the required factored load capacity for LRFD. However, the amount of pretension was found to affect the deformation characteristics of the connection. The allowable stress based on fracture is Ft = 0.5Fu = 0.5(58) = 29.0 ksi and the corresponding allowable strength is Ft Ae = 29.0(11.73) = 340 kips. ⎛ 7⎞ Rn = 2.4 dtFu = 2.4 ⎜ ⎟ (0.615)(58) = 74.91 kipss ⎝ 8⎠ Nominal shear strength: π (7冀8)2 = 0.6013 in.2 4 Rn = Fnv Ab = 54(0.6013) = 32.47 kips Ab = LRFD SOLUTION Pu = 1.2D + 1.6L = 1.2(15) + 1.6(45) = 90 kips a. FIGURE 10.25 The girder designed in this example is not necessarily the most economical one. < 1.04 in. The member is subjected to the following service loads and moments: an axial compressive load of 340 kips and a bending moment of 250 ft-kips about the strong axis. The calculations will be performed with standard spreadsheet software. The relationship of Equation 5.11 corresponds to a nominal flexural strength of Mn = Fu A fn Sx A fg 246 Chapter 5 Beams The AISC requirement for holes in beam flanges can be summarized as follows: If Fu Afn < Yt Fy Afg The nominal flexural strength cannot exceed Mn = Fu A fn Sx A fg (AISC Equation F13-1) where Yt = 1.0 for Fy 冀Fu ≤ 0.8 = 1.1 for Fy 冀Fu > 0.8 The constant Yt should be taken as 1.1 for A992 steel if the value of Fy冀Fu is not known. Thus, the total width of floor being supported is equal to the beam spacing s if the spacing is uniform. The theory that is presented is within the grasp of undergraduate students, but advanced topics such as plate buckling have been excluded. or 30(12)冀4 = 90 in. Another combination is the double-channel shape (either American Standard or Miscellaneous Channel). Guide to Design Criteria for Bolted and Riveted Joints. This summary is for compact and noncompact shapes (noncompact flanges) only (no slender shapes). 20k 5′ 20k 5′ 18′ FIGURE P5.10-7 5.10-8 The beam shown in Figure P5.10-8 is part of a roof system. In situations where Mnt and Mℓt act at two different points on the member, as in Figure 6.4, AISC Equation A-8-1 will produce conservative results. When lines of bolts are present in more than one element of the cross section of a rolled shape, and the bolts in these lines are staggered with respect to one another, the use of areas and Equation 3.2 is preferable to the net-width approach of the AISC Specification. Use Fy = 50 ksi and select a W shape. If the stress at which buckling occurs is greater than the proportional limit of the material, the relation between stress and strain is not linear, and the modulus of elasticity E can no longer be used. A minimum of four bolts will be needed to maintain symmetry, so try four bolts. For beam spacing = 10(12) = 120 in. This integrity will be lost if one of the compression elements of the cross section buckles. Answers to Selected Problems Notes 1. Joe Gough / Shutterstock Any structural members, such as the ones in this structural steel frame, are subjected to both bending and axial force. Hence the student of structural steel design must have ready access to its content. An examination of the column load tables in Part 4 of the Manual shows that a small shape will be needed, because the axial load is small and the moment is small relative to the axial load. Because this angle has three bolts in the direction of the load, the reduction factor U can be taken as 0.60, and Ae = AnU = 5.02(0.60) = 3.012 in.2 Either U value is acceptable, and the Specification permits the larger one to be used. • The deck must be attached to the beam flange at intervals of no more than 18 inches, either by the studs or spot welds. This is similar to what is called plastic analysis for line members

such as beams, where the yielding is at one or more points along the length (see Appendix A). A.2 AISC REQUIREMENTS Plastic analysis and design is covered in Appendix 1 of the AISC Specification, "Design by Inelastic Analysis." Because plastic analysis is based on an ultimate load condition, load and resistance factor design (rather than allowable strength design) is appropriate. The nominal compressive strength is Pn = Fcr Ag For LRFD, Pu ≤ fc Pn where Pu = sum of the factored loads fc = resistance factor for compression = 0.90 fcPn = design compressive strength (AISC Equation E3-1) 118 Chapter 4 Compression Members For ASD, Pa ≤ Pn Ωc where Pa = sum of the service loads Ωc = safety factor for compression = 1.67 Pn兾Ωc = allowable compressive strength If an allowable stress formulation is used, fa ≤ Fa where fa = computed axial compressive stress = Pa兾Ag Fa = allowable axial compressive stress = Fcr F = cr = 0.6 Fcr Ω c 1.67 (4.7) In order to present the AISC expressions for the critical stress Fcr, we first define the Euler load as Pe = p 2 EA (KL兾r)2 This is the critical buckling load according to the Euler equation. It is difficult to determine what the degree of precision should be for the typical structural steel design problem. The tensile strength is given as a range of values because for A36 steel, this property cannot be achieved to the same degree of precision as the yield stress. This approach to design is also called elastic design or working stress design. If the strength is too small (unsafe) or too large (uneconomical), another trial must be made. Service load moments about the y axis are MDy = 6 ft-kips and MLy = 18 ft-kips. Yield-line theory was originally formulated for reinforced concrete slabs but is equally valid for steel plates. Compute a: ∴ Use p = 4 in. 0.6D ± (0.6W or 0.7E). 0.85 fc′b 0.85(4)(90) and the moment arm of the internal resisting couple is y= d a 15.7 1.255 +t− = + 3.5 − = 10.72 in. Compute the available block shear strength of the tension member for both LRFD and ASD. "LRFD Criteria for Steel Beam-Columns." Journal of the Structural Division, ASCE 104 (no. DESIGN The primary task in plate girder design is to determine the size of the web and the flanges. This condition is illustrated in Figure 7.5. The stress will vary from a maximum at A to zero at B; for simplicity, an average stress, computed as the applied force divided by the projected area of contact, is used. If A572 Grade 50 steel is used, does the member satisfy the provisions of the AISC Specification? Then, as shown in Figure 9.9, the compressive force is the smaller of AsFy = 10.3(50) = 515.0 kips or 0.85f c′ Ac = 0.85(4)(4.5 × 90) = 1377 kips Use C = 515 kips. Serviceability Design Considerations for Low-Rise Buildings. The thickness will be a function of the beam spacing, and several combinations of slab thickness and beam spacing may need to be investigated so that the most economical system can be found. Yes: 500 kips < 505 kips b. If the result is not very close to 1.0, try another shape. Ideally, the engineer and architect will collaborate throughout the design process to complete the project in an efficient manner. 8.4-3 12.6 kips兾in. 7.6 Slip-Critical and Bearing-Type Connections 401 Tables for Bolt Strength Manual Tables 7-1 through 7-5 give values for bolt shear, tensile, and slip-critical strengths and bearing strength at bolt holes. 1.08. Formulas to expedite this computation have been developed (Hansell et al., 1978), but the tables presented in Part 3 of the Manual are more convenient. ∴ Use in. It will be connected to a 3⁄8-inch-thick gusset plate, also of A572 Grade 50 steel, with 7⁄8-inch-diameter bolts. When computing the distance c, use the actual hole diameter (which is 1⁄16-inch larger than the bolt diameter), and do not add the 1⁄16 inch as required in AISC B4.3b for computing the net area for tension and shear. Standard load tables are given for each of these categories. EXAMPLE 7.8 Use LRFD and design a 13-foot-long tension member and its connection for a service dead load of 8 kips and a service live load of 24 kips. • The new approach to proportioning intermediate plate girder stiffeners is included. 11⁄2″ 3″ 3″ 11⁄2″ C7 × 9.8 2″ 2″ t = 3⁄8″ 3⁄4″-diameter bolts FIGURE P3.5-5 3.5-6 A double-channel shape, 2C8 × 18.75, is used as a tension member. Once the trial shape has been selected and the number of bolts and their layout estimated, Equation 7.18 or 7.19 can be used to verify or disprove the choices. and Minimum longitudinal spacing is 6d = 6(0.5) = 3 in. The validity of Equation 4.1 has been demonstrated convincingly by numerous tests. (controls) 4 Try 3⁄4-in. 40 k 7′ 7′ 14′ FIGURE P5.5-13 5.5-14 Repeat Problem 5.5-13 for an MC18 × 51.9 (Assume that the load is applied through the shear center so that there is no torsional loading.) Use Fy = 36 ksi. • Service dead load = 45 kips, service live load = 60 kips, and wind load = 62 kips. ANSWER Qn = 6.51 kips Partial Composite Action Partial composite action exists when there are not enough steel anchors to completely prevent slip between the concrete and steel. For many structural members, however, there will be a significant amount of both effects, and such members are called beam–columns. They include full solutions to all the problems. ⎣ resulting in the requirement b ≥ 2.59 in. The total service load, including the beam weight, is 9 kips兾ft, with equal parts dead and live load. Because of the unsymmetrical placement of this load, there will be a small amount of sidesway. In some types of structures, notably bridges, the load on connections can undergo many cycles of reversal. Another component of all structural steels, although in much smaller amounts, is carbon, which contributes to strength but reduces ductility. ⎝ 8⎠ Ω From Equation 7.38, the base metal shear rupture strength per unit length is Rn ⎛ 3⎞ = 0.3Fu t = 0.3(58) ⎜ ⎟ = 6.525 kips兾in. Yield stress: Fy = 36,000 psi (36 ksi) Tensile strength: Fu = 58,000 psi to 80,000 psi (58 ksi to 80 ksi) A36 steel is classified as a plain carbon steel, and it has the following components (other than iron). Darwin, D. The basic idea is that of tributary areas. The ratio L 兾r is the slenderness ratio and is the measure of a member's slenderness, with large values corresponding to slender members. Adequate: Before the concrete cures, Mu = 128 ftkips < 249 ft-kips After the concrete cures, Mu = 403 ft-kips < 485 ft-kips b. The LRFD provisions are based on research reported in eight papers published in 1978 in the Structural Journal of the American Society of Civil Engineers (Ravindra and Galambos; Yura, Galambos, and Ravindra; Bjorhovde, Galambos, and Ravindra; Cooper, Galambos, and Ravindra; Hansell et al.; Fisher et al.; Ravindra, Cornell, and Galambos; Galambos and Ravindra, 1978). Combinations 6 and 7: 0.9D ± (1.0W or 1.0E). The required weld size is D= ANSWER 0.6 Fy t w 0.6(50)(0.300) = = 3.223 sixteenths 1.392(2) 1.392(2) Use a 1⁄4-inch fillet weld on each side of the web in the tension region. Computations are simplified, however, if a unit throat dimension is used. Consider all limit states. Table 5.3 has been extracted from AISC Table B4.1b and is specialized for hot-rolled I-shaped cross sections. The member will be pinned at each end, with additional support in the weak direction at midheight. The curves will be entered with values of h兾tw and the required fvVn兾Aw, where h = 198.4 tw 5 ⎞ = 19.38 in.2 Aw = 62 ⎛ ⎝ 16 ⎠ φv Vn Vu 223.4 Required = = = 11.5 ksi Aw Aw 19.38 Using h兾tw = 198 and fvVn兾Aw = 12 ksi, we get a value of a兾h of approximately 0.60. "Designing with Structural Tubing." Modern Steel Construction. Composite Columns 9.10-1 An HSS12 × 8 × 3⁄16 filled with concrete is used as a composite column, as shown in Figure P9.10-1. For yielding of the gross section, ft Pn = 0.90(270.0) = 243.0 kips For fracture of the net section, ft Pn = 0.75(348.0) = 261.0 kips Yielding of the gross section controls. 9.8-5 The beams in a composite floor system support a formed steel deck and concrete slab. emid-ht = distance from mid-height of the rib to the stud, measured in the loadbearing direction (toward the point of maximum moment in a simply supported beam) Most steel deck is manufactured with a longitudinal stiffener in the middle of the rib, so the stud must be placed on one side or the other of the stiffener. We denote this portion N2, and this requirement is illustrated in Figure 9.10. Flexural and shearing stresses in beams of homogeneous materials can be computed from the formulas fb = Mc I and fv = VQ It A composite beam is not homogeneous, however, and these formulas are not valid. (Verification of the values shown is left as an exercise for the reader.) Compute Cb for the 12-foot unbraced segment adjacent to the midspan stiffener. As with the combinations for LRFD, we will evaluate the expressions involving D, L, Lr, and S for ASD. This was a joint effort by the three code organizations (ICBO, BOCA, and SBCCI). First, combine Equations 7.9 and 7.10, using b′ instead of b: Mb–b = Tb¢ – Ma–a From Equation 7.12, M α = b− b δ M a− a Tb ′ − M a− a Tb ′兾M a− a − 1 = δ M a− a δ subject to the limits = 0≤a ≤1 For LRFD, set ⎛ pt 2f Fu ⎞ M a − a = φb M p = φb ⎜ ⎝ 4 ⎠⎟ then Tb ′ −1 φb pt 2f Fu 4 1 ⎛ 4Tb ′ ⎞ α= = ⎜ −1 δ δ ⎝ φb pt 2f Fu ⎠⎟ (0 ≤ α ≤ 1) (7.20) where tf is the actual flange thickness. Compute the actual block shear strength. 4.71 . The required shear strength is Required Vn = Vu 234 = = 260.0 kips φv 0.90 10.6 Bearing Stiffeners 689 From AISC Equation G2-1, Vn = 0.6 Fy AwCv Cv = Vn 260.0 = = 0.55094 0.6 Fy Aw 0.6(36)(23.63) Cv = 1.51kv E (h兾t w)2 Fy (AISC Equation G2-5) Cv (h兾t w)2 Fy 0.5094(168)2 (36) = = 11.82 1.51(29, 000) 1.51E 5 kv = 5 + (a兾h)2 kv = a = h 5 5 = = 0.8562 kv − 5 11.82 − 5 The required stiffener spacing is a = 0.8562h = 0.8562(63) = 53.9 in. < 7.20 kips兾in. • The example on ultimate strength analysis of eccentric bolted shear connections has been redone, tying it more closely to the tables in the Manual. For any type of tension member except plates and round HSS with ≥ 1.3D x− U = 1− (3.1) where x− = distance from centroid of connected area to the plane of the connection = length of the connection This definition of x− was formulated by Munse and Chesson (1963). 532 Chapter 8 Eccentric Connections FIGURE 8.42 For the web plate, try 3⁄4-inch-diameter bolts. When this is not possible, U.S. customary units are given, followed by SI units in parentheses. 7.11 FILLET WELDS The design and analysis of fillet welds is based on the assumption that the cross section of the weld is a 45° right triangle, as shown in Figure 7.37. The column is perfectly straight, with no initial crookedness. 162 Chapter 4 ANSWER Compression Members For LRFD, the design strength is fcPn = 0.90(113.5) = 102 kips. 2000. (Neglect the weight of member CB.) a. Failure resulting from excessive tension, shear, or bending in the parts being connected. Engesser was convinced by their arguments, and in 1895 he refined his theory to incorporate a reduced modulus, which has a value between E and Et. Test results, however, always agreed more closely with the tangent modulus theory. In this example, the force in member IJ will control. 36 Chapter 2 Concepts in Structural Steel Design As a companion document to the Manual, a set of design examples is available at the AISC web site (AISC, 2011b). The latter type, stability bracing, was first discussed in Chapter 4, "Compression Members." The stiffness and strength requirements for stability can be added directly to the requirements for directly applied loads. Snow is another live load that is treated as a separate category. 5.14 Beam Bearing Plates and Column Base Plates 255 FIGURE 5.39 b b For convenience, a 1-inch width is considered, with a uniform load in pounds per linear inch numerically equal to the bearing pressure in pounds per square inch. Lc = s − h = 3 − 8.2 Eccentric Bolted Connections: Shear Only 487 FIGURE 8.9 Both bearing values are larger than the bolt shear strength, so the nominal shear strength of Rn = 32.47 kips controls. Space will be a problem only when there 3.6 71 Design of Tension Members are two lines of bolts in a leg. Compute rx and ry. The various properties of structural steel, including strength and ductility, are determined by its chemical composition. This shape was formerly called an I-beam. 1960. The bolt force R corresponding to a deformation Δ is R = Rult (1 – e−mΔ)l = Rult (1 – e−10Δ)0.55 where Rult = bolt shear force at failure e = base of natural logarithms m = a regression coefficient = 10 l = a regression coefficient = 0.55 (8.2) 484 Chapter 8 Eccentric Connections FIGURE 8.7 The ultimate strength of the connection is based on the following assumptions: 1. Full composite behavior will not usually be possible. Equation 5.3 is based on the assumption of a linear distribution of strains from top to bottom, which in turn is based on the assumption that cross sections that are plane before bending remain plane after bending. To limit the live load deflection to L兾360, the live load must not exceed 297 lb兾ft > 150 lb兾ft ANSWER (OK) Use a 16K2. Try a W16 ⫻ 26 and check the shear (refer to Figure A.7). The moment of inertia of this revised composite area is I tr = FIGURE 9.6 1 (10.88)(4.143)3 + 448 + 10.6(12.95 − 4.143)2 = 1528 in.4 3 600 Chapter 9 TABLE 9.3 Composite Construction A y Ay 10.88y– 10.6 y– 兾2 12.95 5.44y– 2 137.3 Component Concrete W16 × 36 and the stresses are (160 × 12)(5 − 4.143) = 1.08 ksi (tension) 1528 (160 × 12)(5 + 15.9 − 4.143) = 21.1 ksi (tension) fsb = 1528 (160 × 12)(4.143) = 0.651 ksi fc = 8(1528) fst = The difference between the two analyses is negligible, so the refinement in locating the neutral axis is not necessary. The problem is compounded by the confusion that sometimes exists regarding responsibility for the design of connections. The concrete strength is fc⬘ = 5 ksi. Many columns can be treated as pure compression members with negligible error. Determine whether the connection moment is at least 60% of the beam moment strength. The modified values of b and a are therefore defined as b′ = b − d d and a ′ = a + 2 2 (For best agreement with test results, the value of a should be no greater than 1.25b.) With this modification, we can write Equation 7.13 as ⎡ δα b ′ ⎤ Bc = T ⎢1 + ⎥ + δα) a ′ ⎦ (1 ⎣ (7.14) We can evaluate a from Equation 7.14 by setting the bolt force Bc equal to the bolt tensile strength, which we denote B. For ASD, the allowable strength is Pn 兾Ωc, where Ωc = 2.00. For an unbraced length of 15⁄2 = 7.5 ft, try a C10 × 15.3. For Cb = 1.0, fb Mnx = 33.0 ft-kips. A hanger connection of the type used in the preceding discussion is subject to this type of behavior. Investigate column stiffener and web panel-zone reinforcement requirements. Segui) Solution manual Civil Engineering Materials. For most of the structures for which plastic design would ordinarily be used, however, the mechanism method as presented in this appendix usually will be adequate. The axial load, which partly results from loads not causing the sidesway, is carried along and amplifies the end moment. From the LRFD solution, Rn = 69.89 kips Rn 69.89 = = 34.9 kips > 26 kips (OK) Ω 2.00 Use a plate 5⁄16 × 31⁄2. There may also be secondary components, such as bracing for stability, that are not shown in Figure 5.31. Use a lower-bound moment of inertia, ILB, which is a conservative underestimation of the elastic moment of inertia. Use Group A pretensioned bolts. = 50(10) = 500 lb兾ft Construction load = 20(10) = 200 lb兾ft After the concrete cures, Partition load = 10(10) = 100 lb兾ft Live load = 120(10) = 1200 lb兾ft LRFD SOLUTION a. SOLUTION Because this shape is a nonslender WT, we use the approach of AISC E4(a). For the braced condition, Pe1 = π 2 EI x π 2 (29,000)(348) = 3074 kips 2 = (K x L) (1.0 × 15 × 12)2 From AISC Equation A-8-3, Cm Cm = 1 − (α Pr Pe1) 1 − α [(Pnt + Pt) Pe1] 0.6 = 0.612 < 1.0 = 1 − 1.0[(58.4 + 0)] 3074 B1 = ∴ Use B1 = 1.0. From Equation 6.10, () () 9 (bx Mux + by Muy) = 0.5(3.16 × 10 −3)(54.97) 8 9 + [(3.69 × 10 −3)(205.7) + 0] = 0.941 < 1.0 (OK) 8 0.5 pPu + ANSWER 6.9 Use a W12 × 45. 2 2b = 2(2.425) = 4.85 in. 3.712 kips兾in. If the curve is for a noncompact shape, the horizontal line represents the flange local buckling strength. Sufficient anchors are provided to make the beam fully composite. Compute the available compressive strength. For compression members whose strength is governed by flexural buckling (that is, not local bucking), Table 4-22 in Part 4 of the Manual, "Design of Compression Members," can be used. Figure 3.26 illustrates typical rod and cable connection methods. As Fy 2. In addition to the weight of the structure, dead loads in a building include the weight of nonstructural components such as floor coverings, partitions, and suspended ceilings (with light fixtures, mechanical equipment, and plumbing). Use Cb = 1.6 and determine whether this member satisfies the provisions of the AISC Specification. London: Chapman and Hall Ltd. Assume a connection length of 12 inches. In this book, we use AISC Equations J3-3a and J3-3b. For the flange connection, first select the bolts. LRFD SOLUTION Pr = Pu = 1.2D + 1.6L = 1.2(100) + 1.6(200) = 4400 kips Py = Fy Ag = 50(15.8) = 790 kips From AISC Equation C2-2b, 1.0(440) ⎞ ⎛ 1.0(440) ⎞ ⎛ α Pr ⎞ ⎛ α Pr ⎞ τb = 4 ⎢ 1− = 4⎛ 1− = 0.987 ⎢⎥ ⎥⎝ 790 ⎠ ⎝ ⎝ Py ⎠ ⎝ Py ⎠ 790 ⎠ ANSWER ASD SOLUTION tb = 0.987. 10.6 BEARING STIFFENERS Bearing stiffeners are required when the web has insufficient strength for any of the limit states of web yielding, web crippling, or sidesway web buckling. In most cases, one of these standard shapes will satisfy design requirements. Once the member has been selected, compute the value of U with Equation 3.1 and revise the design if necessary. By interpolation, for KL兾r = 96.77 and Fy = 50 ksi, Fcr 兾Ωc = 15.07 ksi Note that this is the allowable stress, Fa = 0.6Fcr. Bearing stiffeners are placed at the ends, and intermediate stiffeners are placed at 6′-2″ and 12′-9″ from each end. 16 4 16 16 For the holes nearest the edge of the gusset plate, h 13兾16 = 1.5 − = 1.094 in. • total slab thickness = 41⁄2 in. Special Provision for Axially Loaded Members When a structural member is axially loaded, the stress is uniform over the cross section and the resultant force may be considered to act along the gravity axis, which is a longitudinal axis through the centroid. The loads are service loads, consisting of 25% dead load and 75% live load. EXAMPLE 3.4 Determine the effective net area for the tension member shown in Figure 3.12. However, there is little room for error in beam length, and the end must be square. (ho can also be found in the dimensions and properties tables. 350 Chapter 6 Beam–Columns Check Equation 6.9 with p = 3.35 × 10−3, bx = 5.90 × 10−3, by = 12.6 × 10−3. 7.6 SLIP-CRITICAL AND BEARING-TYPE CONNECTIONS A connection with high-strength bolts is classified as either a slip-critical connection or a bearing-type connection. It is 26 feet long, pinned at each end, and has additional support in the weak direction at a point 12 feet from the top. The details of the Specification will be covered in the chapters that follow, but we discuss the overall organization here. It is connected to a 3⁄8-in. If the slab is properly attached to the rolled steel shape, the strains will be as shown, with cross sections that are plane before bending remaining plane after bending. PD = 9k, PL = 8k 30° 4″ 6″ 3″ t = 1⁄2″ 6″ 3″ 1′-0″ t = 1⁄2″ FIGURE P8.4-5 8.4-6 Check the adequacy of the weld. This organization 12 Chapter 1 TABLE 1.1 Introduction Property A36 A572 Gr. 50 A992 Yield point, min. Wind exerts a pressure or suction on the exterior surfaces of a building, and because of its transient nature, it properly belongs in the category of live loads. This is true of both snow and roof live load in this example. If A572 Grade 50 steel is used, is this member adequate? 6.8 Design of Beam–Columns 351 Bracing can be classified as nodal or relative. Are you come upon Steel Design Segui 4th Edition Solution Manual? Compute Cb. Include the beam weight in the loading. The value of the tables becomes obvious when the plastic neutral axis is within the steel shape. For a column such as AB, a value of K = 1.2, corresponding to condition (c), could be selected. This amount corresponds to the cross section with the smallest weight per foot, which is the one with the smallest cross-sectional area. Shear strength: The shear strength is a function of the web slenderness ratio h兾tw and the aspect ratio a兾h. The effective slab width is 90 inches, and the span length is 30 feet. The axial loads and bending moments can then be found by using a method of structural analysis such as the stiffness method. The breaking strengths of various cables, as well as details of available fixtures for connections, can be obtained from manufacturers' literature. Compare this area with the trial area and revise if necessary. controls) For a group of four welds, the capacity is 4(1.392 DL) = 4(1.392)(3)(1.5) = 25.06 kips Required weld capacity per inch = P 130.3 = = 6.614 kips兾in. The allowable stress solution can be simplified if a slight approximation is made. The maximum unbraced length for which this condition is true will be different from the value of Lp obtained with AISC Equation F2-5. The length parameters are Lp and Lr , where L p = 1.1rt E Fy (AISC Equation F4-7) Lr = π rt E 0.7 Fy (AISC Equation F5-5) rt = radius of gyration about the weak axis for a portion of the cross section consisting of the compression flange and one-third of the compressed part of the web. For a W12 × 65, it is given as 161 ft-kips. If the bolt tension is assumed to be equal to the externally applied force (as if there were no initial tension) and the connection is loaded until the connected parts separate, the bolt tension will be underestimated by less than 10%. The procedure for flexural-torsional buckling analysis of double angles and tees given in AISC Section E4(a) is a modification of the procedure given in AISC E4(b). To demonstrate the procedure, we begin with the critical buckling load for an inelastic * If a frame is braced against sidesway, the beam-to-column connections need not be moment-resisting, and the bracing system could be designed to resist all sidesway tendency. 1990b. The structural shapes are A992, and the plates are A36 steel. By examining the value of each term in Equation 6.9 or 6.10, you can gain insight into which constants need to be larger or smaller. With shapes built up from plates, however, both flanges and webs can be compact, noncompact, or slender. Global presence, local solutions. This is the reason that a minimum moment of inertia, rather than a minimum area, is specified. Select a square HSS. Because W and E are zero, this expression reduces to combination 1. The major task in the trial-and-error procedure just outlined is the selection of a trial steel shape. These two constants can FIGURE 5.15 5.6 Bending Strength of Noncompact Shapes 211 FIGURE 5.16 also be found in several other tables in Part 3 of the Manual. A992 steel 3 4 W16 × 40 WT8 × 13 80 k FIGURE P7.9-3 7.9-4 A double-angle tension member is attached to a 7⁄8-inch gusset plate, which in turn is connected to a column flange via another pair of angles as shown in Figure P7.9-4. 3.4 Staggered Fasteners 59 FIGURE 3.15 For line abcde, wn = 16 − 3(1.125) + 2(3)2 = 13.52 in. Depending on the configuration of the connection, block shear might also need to be considered. Lower-bound theorem (static theorem): If a safe distribution of moment (one in which the moment is less than or equal to Mp everywhere) can be found, and it is statically admissible with the load (equilibrium is satisfied), then the corresponding load is less than or equal to the collapse load. Flange: l= bf 2t f l p = 0.38 E Fy lr = 1.0 λ p = 0.84 E Fy λr = 1.03 E Fy ii. A certain bracing system is subjected to the following loads: dead load = 13.3 kips, occupancy live load = 6.9 kips, roof live load = 1.3 kips, snow load = 1.3 kips, wind load = 150.6 kips, and earthquake load = 161.1 kips. 420 Chapter 7 Simple Connections In summary, the tensile force in the bolt should be computed without considering any initial tension. The relationship between G and K has been quantified in the Jackson–Mooreland Alignment Charts (Johnston, 1976), which are reproduced in Figures C-A-7.1 and C-A-7.2 in the Commentary. Check the strength of the base metal. These buckling modes are illustrated in Figure 4.4. Values of n larger than 1 are not possible unless the compression member is physically restrained from deflecting at the points where the reversal of curvature would occur. 250k W36 × 210 A B C 10′ 10′ 20′ FIGURE P5.5-9 5.5-10 If the beam in Problem 5.5-9 is braced at A, B, and C, compute Cb for the unbraced length AC (same as Cb for unbraced length CB). Each segment between purlins is assumed to support everything below it; thus the top rod is designed for the load on the roof area tributary to the rod, from the heel of the truss to the peak, as shown in Figure 3.30. For LRFD, f = 0.90. If partial composite action is desired, the number of studs can be reduced (this is covered in Section 9.7). However, the effect of the applied tensile force is to relieve some of the clamping force, thereby reducing the available 432 Chapter 7 Simple Connections friction force. 5 1 . Fy Adapting these limits to the present case yields 0.75 E 29,000 t f = 0.75 (1.5) = 31.9 in. and Shoemaker. Answers are given for all odd-numbered problems except for the following types: a. When the fasteners are subjected to shear, a modified slenderness ratio larger than the actual value may be required. Both the member and the gusset plate are A36 steel. To use the tables for analysis of a composite beam, first find the portion of the table corresponding to the steel shape and proceed as follows: 1. Consider first the beam support shown in Figure 5.36. < 9.0 kips兾in. 7kD, 18kL 1.5k/ftD, 3.5k/ftL A B 8′ C 70kD, 170kL 8′ 16′ W10 × 112 FIGURE P6.6-7 6.6-8 The member shown in Figure P6.6-8 is braced against sidesway. As Table 7.2 shows, if the weld axis is parallel to the load, the basic strength given by Fnw = 0.60FEXX is correct, but when the weld is perpendicular to the load, the true strength is 50% higher. The question of precision was mostly academic before the early 1970s, when the chief calculating tool was the slide rule. Four #10 bars are used for longitudinal reinforcement, and #3 ties spaced 13 inches center-to-center provide the lateral reinforcement. Structural design is a complex endeavor, involving the synthesis of many processes. The AISC Specification gives a different, but equivalent, form for the elastic buckling stress Fcr. 733 Shanley, F. FIGURE P3.7-6 Tension Members in Roof Trusses 3.8-1 Use A992 steel and select a structural tee for the top chord of the welded roof truss shown in Figure P3.8-1. From Figure 5.14, M A = MC = 3 wL ⎛ L ⎞ wL ⎛ L ⎞ wL2 wL2 − = wL2 ⎜ ⎟− ⎜ ⎟= 2 ⎝ 4⎠ 4 ⎝ 8⎠ 8 3 2 32 FIGURE 5.14 Since this is a W shape (doubly symmetric), AISC Equation (F1-1) is applicable. 16 16 4 (OK) Design the welds. pPu + bx Mux + by Muy = (2.21 × 10−3)(300) + (3.52 × 10−3)(100) + 0 = 1.02 (N.G.) Check other categories of shapes to be sure that we have found the lightest shape. The member is 22 feet long and is pinned at the ends. Most composite beams with formed steel deck are floor beams with the deck ribs oriented perpendicular to the beam, and we limit our coverage to this case. FIGURE 8.29 518 Chapter 8 LRFD SOLUTION Eccentric Connections Let 0.01705Ru = 1.392 × 6 where Ru is the factored load reaction and 6 is the weld size in sixteenths of an inch. Width-to-thickness parameters: i. > 2.33 in.; Le = 2 in. 2″ 3″ 2″ PL 1⁄2 × 61⁄2 13⁄4″ P 13⁄4″ t = 5⁄8″ FIGURE P7.6-5 7.6-6 The tension member shown in Figure P7.6-6 is an L6 × 31⁄2 × 1⁄2. 338 Chapter 6 Beam–Columns Pa 118.6 = = 0.2689 > 0.2 ∴ Use Equation 6.5 (AISC Equation H1-1a). For the fully composite beam, C = V′ = 384.0 kips. This expression reduces to D + 0.75L + 0.75S = 109 + 0.75(46) + 0.75(20) = 158.5 kips D ± (0.6W or 0.7E). Since both components are of the same type of steel, the smaller thickness will control. With one exception, the only differences are in the 692 Chapter 10 Plate Girders loads and whether a resistance factor or a safety factor is used. If the column and the angles are isolated from the beam, as shown in Figure 8.1c, it is clear that the fasteners in the column flange are subjected to the reaction R acting at an eccentricity e from the plane of the fasteners, producing the same couple as before. • If KL E ≤ 4.71 r QFy or QFy ≤ 2.25, Fe QFy ⎞ ⎛ Fcr = Q ⎢ 0.658 Fe ⎥ Fy ⎢⎝ ⎠⎥ • If KL > 4.71 E r QFy or (AISC Equation E7-2) QFy > 2.25, Fe Fcr = 0.877Fe • The nominal strength is Pn = Fcr Ag (AISC Equation E7-3) (AISC Equation E7-1) The reduction factor Q is the product of two factors—Qs for unstiffened elements and Qa for stiffened elements. Other types of members can be illustrated with the rigid frame of Figure 1.2a. Since the edge distances are the same for both the tension member and the gusset plate, the bearing strength of the gusset plate will control because it is thinner than the tension member. Note that the concrete stress block need not extend the full depth of the slab for any of these three cases. 2 2 B − 0.8b f 13 − 0.8(10) n= = = 2.5 in. Although many beams are 252 Chapter 5 Beams FIGURE 5.36 b connected to columns or other beams, the type of support shown here is occasionally used, particularly at bridge abutments. (The KCS joists are designed to resist a uniform moment and a constant shear.) If concentrated loads must be supported by an LH or a DLH joist, a special analysis should be requested from the manufacturer. The total shear兾bearing load is Va = 3 (60) = 36 kips 5 (N.G.) 7.9 Combined Shear and Tension in Fasteners The shear兾bearing force per bolt is Va bolt = 36 = 9.0 kips 4 The allowable bearing strength is Rn 74.91 = = 37.5 kips > 9.0 kips Ω 2.00 (OK) The allowable shear strength is Rn 32.47 = = 16.24 kips > 9.0 kips Ω 2.00 (OK) The total tension load is Ta = 4 (60) = 48 kips 5 The tensile force per bolt is Ta bolt = 48 = 12 kips 4 To determine the available tensile strength, use AISC Equation J3-3b: Fnt′ = 1.3Fnt − ΩFnt frv ≤ Fnt Fnv where Fnt = nominal tensile stress in the absence of shear = 90 ksi Fnv = nominal shear stress in the absence of tension = 54 ksi frv = Va bolt 9.0 = = 14.97 ksi Ab 0.6013 Then Fnt′ = 1.3(90) − 2.00(90) (14.97) = 67.10 ksi < 90 kssi 54 The nominal tensile strength is Rn = F′nt Ab = 67.10(0.6013) = 40.35 kips 435 436 Chapter 7 Simple Connections and the available tensile strength is Rn 40.35 = = 20.2 kips > 12 kips Ω 2.00 ANSWER (OK) The connection is adequate as a bearing-type connection. When the end of a rod is to be threaded, an upset end is sometimes used. Is it possible to use Manual verification via API in RSA 2015? For example, in the operation 12,458.62 − 12,462.86 = −4.24 four significant figures are lost. Therefore, only the live load deflection is of concern in the completed structure. × 21⁄2-in. Some open-web joists are designed to function as floor or roof joists, and others are designed to function as girders, supporting the concentrated reactions from joists. The numerical results may be different) Try t = 1 in. The resulting longitudinal displacement can be accounted for by using slotted bolt holes and tightening the bolts after the welds have cooled. The moment on the column flange welds is M = Re = 2.831R in.-kips where R is the beam reaction in kips. Use An = 5.413(10⁄9) = 6.014 in.2 For line abcdeg, gcd = 3 + 2.25 - 0.5 = 4.75 in. (Recall that Lp for compact shapes is the maximum unbraced length for which the nominal strength can be taken as the plastic moment.) For the shape in Example 5.6, equate the nominal strength based on FLB to the strength based on inelastic LTB (AISC Equation F2-2), with Cb = 1.0: ⎛ Lb − L p ⎞ M n = M p − (M p − 0.7Fy S x) ⎜ ⎝ Lr − L p ⎠⎟ (5.6) The value of Lr was given in Example 5.6 and is unchanged. 1990a. If the axial load is small, the axial load term is reduced. Standard Building Code. 1 Mu = 173 + (1.2 × 0.030)(30)2 = 177 ft-kips 8 The maximum shear is Vu ≈ 1.540(30) = 23.1 kips 2 From the Zx table, fvVn = 112 kips > 23.1 kips (OK) (OK) 5.11 Floor and Roof Framing Systems 243 The maximum permissible deflection is L 30(12) = = 1.0 in. (See AISC Table B4.1b for compact and noncompact webs.) If l ≤ lp, the flange is compact. The nominal depth is the approximate depth expressed in whole inches. The Manual is divided into 17 parts as follows: Part 1. The moment Mnt is computed by considering the frame to be braced—in this case, by a fictitious horizontal support and corresponding reaction called an artificial joint restraint (AJR). +t− = 2 2 2 2 and the allowable flexural strength is M n Cy 384.0(11.97) = = 2752 in.-kips = 229 ft-kips > 205 ft-kips (OK) Ωb Ωb 1.67 Check shear: Va = wa L 1.826(30) = = 27.4 kips 2 2 From the Zx table, Vn = 70.5 kips > 27.4 kips Ωv (OK) 636 Chapter 9 ANSWER Composite Construction Use a W16 × 26. The neutral axis of the connection is assumed to pass through the centroid of the bolt areas. Figure 10.3 shows a bearing stiffener consisting of two rectangular plates, one on each side of the girder web. Use the unfactored service loads. We will determine the number of bolts based on this strength and check bearing after selecting the member (because the bearing strength cannot be computed until the member thickness is known). db = 2 Mu pfFt (h0 + h1) (8.11) Select a bolt diameter, then using the actual bolt size, compute the actual moment strength. ∴ Use 3 in. If no such reference is to be provided, the tail is omitted. The mechanism method is based on the upper-bound theorem and requires that all possible collapse mechanisms be investigated. Problems 4 @ 6′ = 24′ A 30′ B FIGURE P5.11-1 5.11-2 Select a W shape for the following conditions: Beam spacing = 5 ft-6 in. (AISC Table J2.4, based on plate thickness) 16 Use a 1⁄4-inch fillet weld for the stiffener-to-column flange connection. Verification of assumptions: • If strong axis buckling controls the compressive strength, use an effective length of KL = Kx L rx 兾ry to obtain p from Table 6-1. Sag rods are tension members used to provide lateral support for the purlins. The loads are 25% dead load and 75% live load. Table 5.4 shows some of the deflection limits given by that code. Since there are both transverse and longitudinal welds, we will try both of the options given in AISC J2.4c. Determine the total service load capacity if the live load is three times the dead load. The design charts in the Manual comprise a family of graphs similar to the one shown in Figure 5.25. web local buckling (WLB), elastically or inelastically. A welded connection is often simpler in concept and requires few, if any, holes (sometimes erection bolts may be required to hold the members in position for the welding operation). The correct result for this example is wu = 16fb M p L2 For a W30 × 99, M p = Fy Z x = 50(312) = 1300 ft-kips 12 and fbMp = 0.9(1300) = 1170 ft-kips The value of fbMp also can be obtained directly from the Zx table in Part 3 of the Manual. For the axis of bending, Pe1 = p 2 EI p 2 EI x p 2 (29, 000)(127) = 2524 kips 2 = 2 = (K1 L) (K x L) (10 × 12)2 1.60 × 28 ⎞ aPr ⎞ 1.60 Pa ⎞ = 0.9965 = 1 − 0.2 ⎛ Cm = 1 + Ψ ⎛⎜ = 1 − 0.2 ⎛⎜ ⎝ 2524 ⎠ ⎝ Pe1 ⎠⎟ ⎝ Pe1 ⎠⎟ B1 = Cm 0.9965 Cm = = = 1.015 1 − (a Pr 兾P Pe1) 1 − (1.60 Pa 兾Pe1) 1 − (1.60 × 28兾2524) M a = B1 M nt = 1.015(70.44) = 71.50 ft-kips From the Beam Design Charts with Cb = 1.0 and Lb = 10 feet, the moment strength is Mn = 82.0 ft-kips Ωb Because the beam weight is very small in relation to the concentrated live load, Cb may be taken from Figure 5.15c as 1.32. To account for these loads, an additional 20 pounds per square foot is recommended (Hansell et al., 1978). 584 Chapter 8 Eccentric Connections 8.4-10 Solve Problem 8.4-1 by the ultimate strength method (use the tables in Part 8 of the Manual). In many instances, these members are also subjected to bending, and in these cases, the member is a beam–column. A strand consists of individual wires wound helically around a central core, and a wire rope is made of several strands laid helically around a core. LRFD SOLUTION Pu = 1.2 D + 1.6 L = 1.2(1 8) + 1.6(5 2) = 1 0 .48 kips Requiired Ag = Pu Pu 104.8 = = = 3.235 in. From AISC J2.2b, ⎛ 3⎞ Minimum length = 4 w = 4 ⎜ ⎟ = 0.75 in., but not less than 1.5 in. The connections at these points of bracing are simple (moment-free) connections. If h kE > 1.10 v , tw Fy the strength will be based on shear buckling or shear buckling plus tension-field action. Flexible cables, in the form of strands or wire rope, are used in applications where high strength is required and rigidity is unimportant. Select stud anchors and show the layout on a sketch similar to Figure 9.13. ΣEg Ig 兾Lg = sum of the stiffnesses of all girders at the end of the column under consideration. ST9): 1355–70. For I shapes, the ratio for the projecting flange (an unstiffened element) is bf兾2tf , and the ratio for the web (a stiffened element) is h兾tw. Design of Hanger Connections, Bracket Plates, and Crane–Rail Connections. Lb = 20 ft and Cb = 1.0. First, determine Lp and Lr: L p = 1.76ry E 29, 000 = 1.76(2.46) = 104.3 in. Structural steels can be grouped according to their composition as follows. Examples of live loads include furniture, equipment, and occupants of buildings. This can be expressed as Required strength ≤ allowable strength where Allowable strength = (2.1) nominal strength safety factor Strength can be an axial force strength (as in tension or compression members), a flexural strength (moment strength), or a shear strength. Use Ky = 1.0, and investigate this member for compliance with the AISC Specification. Using Fy = 50 ksi and Cb = 1, compute the nominal flexural strength. End Returns When a weld extends to the end of a member, it is sometimes continued around the corner, as shown in Figure 7.43. Is a W30 × 116 adequate? 5.13 OPEN-WEB STEEL JOISTS Open-web steel joists are prefabricated trusses of the type shown in Figure 5.34. For ASD, Ω = 1.67. The results of an approximate analysis are summarized in the figure. Generally speaking, a rigid, or fixed, condition is very difficult to achieve, and unless some special provisions are made, ordinary connections will usually closely approximate a hinge or pin connection. For a beam designed by allowable stress theory, it is a measure of the reserve capacity. What is the maximum factored load Pu that can be supported? This approach is recommended by Viest, et al. The emphasis in this book will be on the design of individual structural steel members and their connections. Fy BN 1.67 36(13)(13) 1.67 Use a PL 1 × 13 × 13. 384 EI 384 29, 000(510) 5.12 5.12 Holes in Beams 245 HOLES IN BEAMS If beam connections are made with bolts, holes will be punched or drilled in the beam web or flanges. In other words, several alternative designs should be prepared and their costs compared. Steel Design Fifth Edition This page intentionally left blank Steel Design Fifth Edition William T. Determine which interaction formula to apply. If a metal deck is used, it will usually be rigidly attached to the purlins, and sag rods may not be needed. The two computed values will always be nearly the same, and for this reason, it makes no practical difference which moments are used. Assume that the connection is at a distance from the end of the column that is more than the depth of the column and determine whether column stiffeners are required. The studs provide a mechanical bond between the steel shape and the concrete slab. 4): 101–115. Assume that the moment includes an accurate estimate of the girder weight. Local flange bending (AISC Equation J10-1). The superimposed loads consist of a construction load of 20 psf and a live load of 800 psf. Lengthening the connected region will reduce this effect. Sometimes, however, the weight of the purlin itself is enough to cause problems, and sag rods may be needed to provide support during construction before the deck is in place. For equal end moments, the moment is constant throughout the length of the member, so the maximum primary moment also occurs at the center. A relative brace is connected not only to the member to be braced but also to other members, as with diagonal bracing. This can be done as follows. The steel is A992 and the concrete strength is f c′ = 4 ksi. The equation for the required column flange thickness is the same as that for the required end-plate thickness, but the equation for the yield-line mechanism parameter is different. Part 5. The general procedure is as follows. 1.5-4 A tensile test was conducted on a specimen with a diameter of 0.5 inch. Values given by AISC Equation I8-1 are based on experimental studies. The axial load and end moments are based on service loads composed of equal parts dead load and live load. Compute both the design strength for LRFD and the allowable strength for ASD. Figure 1.5 shows a typical stress–strain curve for high-strength steels, which are less ductile than the mild steels discussed thus far. Tie rod at the ridge (Figure 3.32c): P = (4.679) 46.6 = 4.845 kips 45 Required Ab = 4.845 = 0.1485 in.2 0.75(0.75)(58) 3.8 ANSWER ASD SOLUTION Tension Members in Roof Trusses 83 Use a 5⁄8-inch-diameter threaded rod (Ab = 0.3068 in.2). Check the design of Part a, accounting for the eccentricity. The American Standard Channel, or C-shape, has two flanges and a web, with only one axis of symmetry; it carries a designation such as "C9 × 20." This notation is similar to that for W- and S-shapes, with the first number giving the total depth in inches parallel to the web and the second number the weight in pounds per linear foot. In this way, the nominal bolt area can be used whether the threads are in or out of the plane of shear. PL 11⁄16 × 9 W14 × 145 W24 × 76 PL 11⁄16 × 9 FIGURE P8.7-2 8.7-3 a. The slab and deck combination weighs 50 psf. Try a W14 × 74: Kx L 20 = = 8.20 > K y L = 8 ft rx 兾ry 2.44 For KL = 8.20 ft, Pn = 582 kips > 560 kips Ωc ANSWER (OK) Use a W14 × 74 (lightest of the three possibilities). Width-to-thickness parameters: l= D t lp = 0.07 E Fy lr = where D is the outer diameter. Use ASD. Although this approach overestimates the volume of concrete, it is conservative. (OK) 8.6 8.6 Moment-Resisting Connections 519 MOMENT-RESISTING CONNECTIONS Although most connections appear to be capable of transmitting moment, most are not, and special measures must be taken to make a connection moment-resisting. Making this substitution, we obtain Required t f = 4Tb ′ φb pFu (1 + δα) (7.18) For ASD, when we substitute Fu for Fy, we get M a− a = allowable strength = Required t f = 2 Mp 1 ⎛ pt f Fu ⎞ = ⎜⎟ ⎟ Ω b Ω b ⎝ 4 ⎠⎟ Ωb 4Tb ′ pFu (1 + δα) (7.19) 424 Chapter 7 Simple Connections where Ωb = 1.67 and T is the applied service load per bolt. K., and Viest, I. EXAMPLE 3.5 If the tension member of Example 3.4 is welded as shown in Figure 3.13, determine the effective area. 110 kips b. (The shear center is the point on the cross section through which a transverse load must pass if the beam is to bend without twisting.) Emphasis will be on I shapes. 8.3 Eccentric Bolted Connections: Shear Plus Tension 491 FIGURE 8.11 8″ 3″ 3″ 3″ The number of bolts per vertical row is n=3 From Manual Table 7-7, C = 1.53 by interpolation The nominal strength of a 3⁄4-inch-diameter bolt in single shear is rn = Fnv Ab = 54(0.4418) = 23.86 kips (Here we use rn for the nominal strength of a single bolt and Rn for the strength of the connection.) The nominal strength of the connection is Rn = Crn = 1.53(23.86) = 36.51 kips ANSWER 8.3 For LRFD, the available strength of the connection is fRn = 0.75(36.51) = 27.4 kips. Anderson, M., and Carter, C.J. 2009. The rationale for using the total story load and strength is that B2 applies to unbraced frames, and if sidesway is going to occur, all columns in the story must sway simultaneously. Combination 2: 1.2D + 0.5Lr Pnt = 1.2(14) + 0.5(26) = 29.8 kips Mnt = 1.2(50) + 0.5(94) = 107 ft-kips M t = 0 Combination 3: 1.2D + 1.6Lr + 0.5W Pnt = 1.2(14) + 1.6(26) = 58.4 kips, Pt = 0.5(–9 + 1) = − 4.0 kips (use 0.) Mnt = 1.2(50) + 1.6(94) = 210.4 ft-kips, M t = 0.5(–32 + 20) = - 6.0 ft-kips (use 0.) Combination 4: 1.2D + 1.0W + 0.5Lr Pnt = 1.2(14) + 0.5(26) = 29.8 kips, Pt = 1.0(–9 + 1) = − 8.0 kips (use 0.) Mnt = 1.2(50) + 0.5(94) = 64.7 ft-kips M t = 1.0(–32 + 20) = –12.0 ft-kips (use 0.) Load combination 3 will obviously govern. (M ay ⎞ 8 ⎛ M ax Pa 8 124.2 + + = 0.6315 + +0 Pn 兾Ωc 9 ⎝⎜ M nx 兾Ωb M ny 兾Ωb ⎠⎟ 9 237 = 1.10 > 1.0 ANSWER) (N.G.)) Load combination 6a controls, and the member does not satisfy the AISC Specification requirements. How many A307 bolts are required? It is supported in the weak direction at a point 12 feet from the top. • The effective length should be taken as 0.75 times the actual length—that is, KL = 0.75h. With nodal bracing, lateral support is provided at discrete locations and does not depend on the support from other parts of the frame. Such members are called beam-columns. These two steels are very similar in both tensile properties and chemical composition, with a maximum carbon content of 0.23%. The dead load includes the weight of the beam. Referring to Figure 10.13, we can use a web length of 9.375 inches, resulting in a crosssectional area for the "column" of 3 A = 2(0.75)(7.5) + ⎛ ⎞ (9.375) = 14.77 in.2 ⎝ 8⎠ 690 Chapter 10 Plate Girders FIGURE 10.13 The moment of inertia of this area about an axis in the web is I = ∑ (I + Ad 2) = 2 7.5 3兾8 ⎞ ⎤ 9.375(3兾8)3 ⎡ 0.75(7.5)3 +2 ⎢ + 7.5(0.75) ⎛ + ⎝ 2 2 ⎠ ⎥⎦ 12 12 ⎣ = 227.2 in.4 and the radius of gyration is r = I = A 227.2 = 3.922 in. As in other cases of combined loading, an interaction formula approach can be used. 16 (based on the thickness t w = 5 in.) 16 3 ⎞ Minimum length = 4 ⎛ = 0..75 in. Chapter 1: Introduction b. The computations are summarized in Table 9.1, and the distance from the top of the slab to the centroid is y= ∑ Ay 273.3 = = 4.205 in. The discussion in this section applies to two types of beams: (1) hot-rolled I shapes bent about the strong axis and loaded in the plane of the weak axis, and (2) channels bent about the strong axis and either loaded through the shear center or restrained against twisting. Clip at a 45° angle for 5⁄8 inch. "Behavior of Bearing Critical Double-Angle Beam Connections." Engineering Journal, AISC 15 (no. (Two gage lines are used to minimize the length of the connection.) We do not know whether the design of this tension member was based on the assumption of one line or two lines of fasteners; the tensile capacity of the channel with two lines of bolts must be checked before proceeding. There are several reasons for this change. The AISC Specification, in Section B4.3b, uses this approach, but in a modified form. r Fy Fcr = 0.877 Fe = 0.877(8.5) = 7.455 ksi Pn = 0.6 Fcr Ag = 0.6(7.455)(20.9) = 93.5 kips < 400 kips Ωc (N.G.) Because the initial estimate of Fcr was so far off, assume a value about halfway between 33 and 7.455 ksi. Mt will be zero if the frame is actually braced. (For beams, a shape can be compact, noncompact, or slender, and the limiting values of l are given in AISC Table B4.1b. The superimposed loads consist of a construction load of 20 psf, a partition load of 15 psf, and a live load of 125 psf. Note that Et is smaller than E and for the same L兾r corresponds to a smaller critical load, Pcr. • Length = 18 feet. If the available strength is very close to the required value, the next tabulated size can be tried. This type of connection is called 76 Chapter 3 Tension Members stitching, and the fasteners used are termed stitch bolts. 5.10 Design 227 1. FIGURE 6.28 SOLUTION The bending moments and panel point forces caused by the loads acting between the joints will be found by treating each top-chord member as a fixed-end beam. Assume that the plate acts as a fixed-end compression member between fasteners, with L = 3 in. 2 ft Fy 0.90 Fy 0.90(36) Required Ae = Pu Pu 104.8 = = = 2.409 in. The stress distribution shown in Figure 9.8 will result. FIGURE 6.18 56k 132ft-k 132ft-k Wind (1.0 W) 332 Chapter 6 Beam–Columns From the column load tables, with KL = 1.0(15) = 15 ft, fcPn = 662 kips. Easterling, W. • The compression flange will be laterally supported at the ends and at the quarter points. Assume that Ae = 0.85An. a. From Equation 9.6, the estimated beam weight is w= 3.4Ωb M a 3.4(1.67)(184.3 × 12) = = 22.8 lb兾ft a⎞ ⎛d ⎛ 16 ⎞ Fy ⎢ + t − ⎥ 50 ⎥ + 3.5 − 0.5⎥ ⎝2 ⎝ 2 ⎠ 2⎠ Try a W16 ⫻ 26. The usual technique was to use a pair of angles, placed back-to-back, to attach the flange to the web. diameter bolts. 3 Agv = Anv = 2 × (7) = 5.25 in. Currently, lighter and more economical methods of fireproofing are available, and encased composite beams are rarely used. Many of the smaller ones use a continuous circular bar to form the web members and are commonly called bar joists. 4.3-6 Does the column shown in Figure P4.3-6 have enough available strength to support the given service loads? Consider each member of the top chord to be a fixed-end beam. Determine whether the AISC Specification is satisfied. If stresses are used instead of forces or moments, the relationship of Equation 2.1 becomes Maximum applied stress ≤ allowable stress (2.2) 21 22 Chapter 2 Concepts in Structural Steel Design This approach is called allowable stress design. 384 EI s 384(29,000)(301) 9.7 Composite Beams with Formed Steel Deck 633 FIGURE 9.20 The deflection caused by the construction load is Δ2 = 5wconst L4 5(0.200 兾12)(30 × 12)4 = = 0.418 in. Each bolt is assumed to have reached an ultimate value of rt. The stud anchors are 3⁄4 inch in diameter. Each concentrated load consists of a 175-kip live load. By inspection, either the upper right-hand corner or the lower right-hand corner may be taken as the critical location. If Fy = 50 ksi, Fu = 65 ksi, and Ae = 0.90An, compute the following. The flange plate will be designed as a tension-connecting element (the top plate) then checked for compression (the bottom plate). Except for those in the lower story, the columns are restrained at both ends by their connection to beams and other columns. Tall, L., ed. Group A, threads not in the plane of shear FIGURE 7.11 SOLUTION The connection can be classified as a simple connection, and each fastener can be considered to resist an equal share of the load. It produces the largest axial load and the largest total moment. Compute the design strength: b = beam spacing or span length ÷ 4 = 5.5(12) = 66 in. 304 Chapter 6 ASD SOLUTION Beam–Columns From the column load tables, the allowable compressive strength of a W10 × 49 with Fy = 50 ksi and Ky L = 1.0 × 17 = 17 feet is Pn = 270 kips Ωc From the design charts in Part 3 of the Manual, for Lb = 17 ft and Cb = 1.0, Mn = 131 ft-kips Ωb From Figure 5.15c, Cb = 1.32. A more detailed treatment of this topic, complete with design aids, can be found in Torsional Analysis of Structural Steel Members (AISC, 1997b). The bearing strength is adequate. Load and Resistance Factor Design of W-Shapes Encased in Concrete, Steel Design Guide Series No. 6. Determine the number and required layout of bolts such that the length of connection L is reasonably small. For a bending moment that varies linearly from zero at one end to a maximum at the other, the value of Cb is 1.67 (see Figure 5.15g). Next, refine the value for Y2: From C = ΣQn, a= ∑ Qn 135 = = 0.6016 in. This length can be no greater than * If the compression flange is not restrained at the load point and AISC Equation J10-7 is not satisfied, lateral bracing must be provided at both flanges at the load point (AISC J10-4). Structural Stability Research Council, Hoboken, NJ: John Wiley and Sons. The equation for the theoretical elastic lateral-torsional buckling strength can be found in Theory of Elastic Stability (Timoshenko and Gere, 1961). Inelastic FLB will control, and ⎡ ⎛ λ − λp ⎞ ⎤ Fcr = ⎢ Fy − 0.3Fy ⎥ ⎜⎟⎝ λr − λ p ⎠ ⎦⎥⎣ (AISC Equation F5-8) If l > lr, the flange is slender, elastic FLB will control, and Fcr = 0.9 Ekc ⎛ bf ⎞⎜⎝ ⎠⎟ 2t f 2 (AISC Equation F5-9) 674 Chapter 10 Plate Girders Lateral-Torsional Buckling The nominal lateral-torsional bucking strength is given by Mn = Rpg Fcr Sxc (AISC Equation F5-2) Whether lateral-torsional buckling will occur depends on the amount of lateral support—that is, the unbraced length Lb. If the unbraced length is small enough, yielding or flange local buckling will occur before lateral-torsional buckling. The maximum available service load reaction, Ra , for ASD. If the beam flange-to-plate welds are taken into account and the load is assumed to disperse at a slope of 1:1 through the plate, the loaded length of web will be tfb + 2w + 2t p + 5k. In a round section, it is D兾t, which is the outer diameter divided by the wall thickness. For FLB, ⎛ λ − λp ⎞ M n = M p − (M p − 0.7 Fy S xc) ⎜ ≤ 1.6 M y ⎝ λr − λ p ⎠⎟ (AISC Equation F9-6) where Sxc = elastic section modulus referred to the compression flange For WLB, Mn = Fcr Sx (AISC Equation F9-8) where Fcr = Fy when d E ≤ 0.84 tw Fy ⎡ d Fcr = ⎢ 2.55 − 1.84 t w ⎣ Fy E ⎤ ⎥ Fy ⎦ (AISC Equation F9-9)) when 0.84 d E E < ≤ 1.03 Fy Fy t w (AISC Equation F9-10) For LTB, see AISC Equations F9-4 and F9-5 given previously. From Figure 9.8, the depth of the compressive stress block is a= C 730 = = 2.982 in. 2 bolts FIGURE P8.8-1 8.8-2 Investigate the adequacy of the bolts in the given end-plate connection. For the bearing stiffener welds: Minimum weld size = 3 in. Uniqueness theorem: If there is a safe and statically admissible distribution of moment in which enough plastic hinges form to produce a collapse mechanism, the corresponding load is the collapse load; that is, if a mechanism satisfies both the upper- and lower-bound theorems, it is the correct one. 2 2 2 2 The nominal moment strength is Mn = Cy = 730(8.609) = 6285 in.-kips = 523.8 ft-kips LRFD SOLUTION Before the concrete cures, there is only dead load (no construction load in this example). P When no ≤ 2.25, Pe ⎛ Pno ⎞ ⎤ ⎡ ⎢ Pn = Pno ⎢ 0.658⎝ Pe ⎠ ⎥ ⎥⎣ ⎦ When (AISC Equation I2-2) Pno > 2.25, Pe Pn = 0.877Pe (AISC Equation I2-3) Note that if the reinforced concrete terms in AISC Equations I2-4 and I2-6 are omitted, Pno = As Fy (EI)eff = Es I s Pe = π 2 Es I s (KL)2 and As Fy Fy Fy Fy Pno = 2 2 = 2 2 2 = 2 2 = Pe π Es I s 兾(KL) π Es As r 兾As (KL) π Es 兾(KL兾r) Fe where Fe is defined in AISC Chapter E as the elastic buckling stress. In both cases, the function of the plate is to distribute a concentrated load to the supporting material. The applied shear of Vu = 33.6 kips must be resisted by welding a length of web equal to the smaller of the following two lengths: 1. If a coped beam is connected with bolts as in Figure 5.20, segment ABC will tend to tear out. Approximately 58 ksi Chapter 2: Concepts in Structural Steel Design 2-1 a. Bickford, John H. ANSWER (OK) Use a 1⁄4-inch fillet weld, E70XX electrodes. < L兾360 (OK) 9.6-5 a. Modern steel structures are connected by welding or bolting (either high-strength or "common" bolts) or by a combination of both. SOLUTION From the dimensions and properties tables in Part 1 of the Manual, A = 17.7 in.2 A 17.7 = = 8.85 in.2 2 2 The centroid of the half-area can be found in the tables for WT shapes, which are cut from W shapes. When selecting the size or number of bolts, we must make an allowance for the prying force. (Because the tension field does not cover the full depth of the web, the interior stiffeners are also subjected to a certain amount of bending caused by the offset of the tension fields in 10.5 Shear Strength 677 FIGURE 10.8 the adjacent panels.) Hence the anchorage for panel BC must be provided on the left side by a beam-shear panel rather than the tension-field panel shown. G., and Fisher, J. This reflects the increased emphasis on the direct analysis method in the Specification. Use LRFD and select the lightest W shape of A992 steel. End-Plate Connections 8.8-1 Investigate the adequacy of the bolts in the given end-plate connection. 9.6-4 For the beam of Problem 9.4-1; a. Application of Equation 7.4 to the compression flange requires a somewhat more liberal interpretation of the load distribution in that N must be treated as if it were uniformly applied over a surface area, Af. The details of the computation of Qs and Qa will not be given here but will be shown in the following example, which illustrates the procedure for an HSS. These members are called bearing stiffeners, and they must be proportioned to resist the applied loads. These members are called beam-columns. The appropriate limit for the maximum deflection depends on the function of the beam and the likelihood of damage resulting from the deflection. required strength per inch (kips兾in.) 6.614 Use two stiffeners, 3⁄4 × 3, one on each side of the web, and 3⁄16-inch × 11⁄2-inch intermittent fillet welds spaced at 31⁄2 inches on center, on each side of each diagonal stiffener. In the same way that tributaries flow into a river and contribute to the volume of water in it, the loads on certain areas of a structural surface "flow" into a structural component. Not adequate: Ma = 732 ft-kips < Mn兾Ω b = 634 ft-kips 5.6-1 b. For span AB, External work = internal work 67.5(5q) = Mp(2q + q) or Mp = 112.5 ft-kips For span BC, 85.5(10θ) = M pθ + 2 M p (2θ) + 5 M pθ 3 or M p = 128.2 ft-kips For span CD, 85.5(10θ) = 5 M p (θ + 2θ + θ) 3 or M p = 128.2 ft-kiips The upper-bound theorem may be interpreted as follows: The value of the plastic moment corresponding to an assumed mechanism is less than or equal to the plastic moment for the collapse load. 5.14 SOLUTION Beam Bearing Plates and Column Base Plates 251 For the dead loads of ⎛ 3⎞ Slab: 150 ⎜ ⎟ = 37.5 psf ⎝ 12 ⎠ Other dead load: = 20 psf Joist weight: = 3 psf (est.) —–– Total: = 60.5 psf wD = 60.5(3) = 181.5 lb兾ft For the live load of 50 psf, wL = 50(3) = 150 lb兾ft The factored load is wu = 1.2wD + 1.6wL = 1.2 (181.5) + 1.6(150) = 458 lb兾ft Figure 5.35 indicates that the following joists satisfy the load requirement: a 12K5, weighing approximately 7.1 lb兾ft; a 14K3, weighing approximately 6.0 lb兾ft; and a 16K2, weighing approximately 5.5 lb兾ft. If the end moments and axial load are 33% dead load and 67% live load, what is the maximum axial service load, P, that can be applied? The Specification gives the following values for the filler factor: • Where bolts have been added to distribute loads in the filler, hf = 1.0. • Where bolts have not been added to distribute loads in the filler and one filler is used, hf = 1.0. • Where bolts have not been added to distribute loads in the filler and two or more fillers are used, hf = 0.85. 4″ R L6 × 6 × 5⁄8 W14 × 48 FIGURE P8.5-3 8.5-4 A bracket plate of A36 steel is welded to a W12 × 50 of A992 steel. Bracing is usually provided in the planes of both the top and bottom chords, but it is not required in every bay because lateral forces can be transferred from one braced bay to the other through the purlins. How many 7⁄8-inchdiameter, Group A bolts are required? From Figure 5.39, the maximum bending moment in the plate is M= R n Rn 2 ×n× = B b 2 2 B b where R is the beam reaction and R兾Bb is the average bearing pressure between the plate and the concrete. All answers, except for approximations, are rounded to three significant figures. In the shielded metal arc welding (SMAW) process, shown schematically in Figure 7.34, current arcs across a gap between the electrode and base metal, heating the connected parts and depositing part of the electrode into the molten base metal. With regard to analysis methods in general, a more formal and systematic approach to the equilibrium method exists and is discussed in considerable detail in The Plastic Methods of Structural Analysis (Neal, 1977). 414 Chapter 7 FIGURE 7.21 Simple Connections 1 1⁄4″ 3″ 3″ 3″ 1 1⁄4 ″ Try the layout shown in Figure 7.21 and select a tension member. Other than Fy, E, and Et , which are properties of the material, the strength is a function only of the slenderness ratio. The bearing strength for these bolts is rn = 32.9t = 32.9(0.560) = 18.4 kips兾bolt Ω The shear strength controls. SOLUTION Compute the overall, or flexural, buckling strength. At failure, however, all loads will be resisted by the internal couple corresponding to the stress distribution at failure. If the neutral axis is an axis of symmetry, these two stresses will be equal in magnitude. The live load is 95 psf. Determine the allowable strength for ASD. Because Mp 兾Ωb is proportional to Zx, the results will be the same. Use double-angle shapes throughout and assume 3⁄8-inch-thick gusset plates and welded connections. This can be done in one of two ways. Figure 6.16a shows an unbraced frame subject to both gravity and lateral loads. If the stiffener bears on the compression flange, it need not be welded to the flange. In these cases, the maximum size to be used in strength computation would be that limited by the base metal shear strength. ST9): 1389–1407. If the fasteners are bolts with no initial tension, the upper bolts will be put into tension and the lower ones will not be affected. This publication contains the AISC Specification and numerous design aids in the form of tables and graphs, as well as a "catalog" of the most widely available structural shapes. For the other holes, 13 = 2.188 in. Investigate the adequacy of this connection (the tee-to-column connection) if it is subjected to a service dead load of 20 kips and a service live load of 40 kips at an eccentricity of 2.75 inches. Basic Steel Design with LRFD. The filler factor, hf, accounts for the presence of filler plates, which are sometimes added to connections to bring elements into alignment. This full bearing strength is the upper limit in AISC Equation J3-6a; that is, frn = f(2.4dtFu). From AISC J3.3, 7 Minimum spacing = 2.667 ⎛ ⎞ = 2.33 in. This frame is also unbraced, meaning that horizontal displacements of the frame are possible and all columns are subject to sidesway. Both analyses were performed with reduced stiffnesses of all members. Assume b兾d > 0.2 and try the second form of the equation, J10-5(b). A typical designation would be "W18 × 50," where W indicates the type of shape, 18 is the nominal depth parallel to the web, and 50 is the weight in pounds per foot of length. What are the conditions under which a tension field cannot be developed? Number of bolts required = 26 = 2.18 11.9 Try three bolts. This is not unusual, but the recommended sequence in beam design is still to select a shape for moment and then check shear and deflection. 263 ft-kips b. 1.11(M n 兾Ω) FyYp 兾Ωb tp = where Mn = 2Pt (h0 + h1) Ω = 2.00 Ωb = 1.67 5. 196 Chapter 5 Beams EXAMPLE 5.2 Compute the plastic moment, Mp, for a W10 × 60 of A992 steel. If an angle shape is used as a tension member and the connection is made by bolting, there must be enough room for the bolts. Alternatives to Equation 3.1 for W, M, S, HP, or Tees Cut from These Shapes: If the following conditions are satisfied, the corresponding values may be used in lieu of Equation 3.1. Connected through the flange with three or more fasteners in the direction of loading, with a width at least 2⁄3 of the depth: U = 0.90. wa = wslab + wbeam + wconst = 0.5000 + 0.026 + 0.200 = 0.77260 lb兾ft Ma = 1 1 wa L2 = (0.7260)(30)2 = 81.7 ft-kips 8 8 9.7 Composite Beams with Formed Steel Deck 635 From the Zx table, Mn M p = = 110 ft-kips > 81.7 ft-kips Ωb Ωb (OK) After the concrete cures and composite behavior has been achieved, the load and moment are wa = wslab + wbeam + wL = 0.500 + 0.026 + 1.300 = 1.826 kipsft Ma = 1 (1.826)(30)2 = 205 ft-kips 8 The effective slab width of the composite section is the smaller of Span 30 × 12 = = 90 in. The welds are 3⁄8-inch fillet welds made with E70XX electrodes. A prescribed amount of deformation can be established for any bolt, and when that amount has been achieved, the bolt will have the proper tension. Composite beams are covered in Chapter 9 of this textbook, "Composite Construction," and plate girders are covered in Chapter 10, "Plate Girders." Part 4. All vertical loads are symmetrically placed and contribute only to the Mnt moments. A right 7.11 Fillet Welds 457 FIGURE 7.46 triangle with the vertical leg on the left side is used to indicate a fillet weld. A W18 × 35 beam is to be connected to a W14 × 53 column. 24.19 The slenderness ratio is Kh 0.75(63) = = 10.90 < 25 4.334 r ∴ Pn = Fy Ag = 36(24.19) = 870.8 kips φ Pn = 0.90(870.8) = 784 kips > 234 kips ANSWER ASD SOLUTION (OK) The bearing stiffeners are adequate. For each arrangement of fasteners considered, the tables give a value for C, the ratio of connection available strength to fastener available strength. The span length is 30 feet. Although there is a strong move to metrication in the steel industry, most structural design in the United States is still done in U.S. customary units, and this textbook uses only U.S. customary units. How is this high tension accurately achieved? 2 2 Moment arm for the compressive force in the steel is y− t⬘ 0.2282 = 9.362 − = 9.248 in. 1.32 b. From elementary mechanics of materials, the axial deformation of an axially loaded uniform member is δ= PL AE (7.3) where P = applied axial force L = original, undeformed length A = cross-sectional area E = modulus of elasticity Equation 7.3 can be solved for the load: P= AEδ L (7.4) The change in bolt force corresponding to a given axial displacement db therefore is ΔT = Ab Ebδ b Lb (7.5) where the subscript indicates a property or dimension of the bolt. Since x0 and y0 are the coordinates of the 158 Chapter 4 TABLE 4.1 Compression Members Shape Constants W, M, S, HP, WT, MT, ST J, Cw (In addition, the Manual Companion CD gives values of r0 , and H for WT, MT, and ST shapes) C J, Cw, MC, Angles J, Cw, r0 , (In addition, the Manual Companion CD gives values of H for MC and angle shapes.) Double Angles r0 , H (J and Cw are double the values given for single angles.) r0 , H shear center with respect to the centroid of the cross section, the location of the shear center must be known. ⎝ 4⎠ Actual spacing = 2.50 in. As a nut is turned and advanced along the threads of a bolt, the connected parts undergo compression and the bolt elongates. The plates are not continuous but are spaced at intervals along the column length and function to maintain the separation of the angles. The longitudinal bars must have an area of at least 0.4% of the total area. The bearing strength based on this spacing is 3 φrn = 72.9t = 72.9 ⎛⎜ ⎞⎟ = 27.3 kips兾bolt ⎝ 8⎠ For the holes nearest the edge of the gusset plate, use Manual Table 7-5 and an edge distance of 11⁄4 inches. LRFD SOLUTION The maximum factored load reaction is the design strength: fRn = 0.75(108.7) = 81.5 kips ASD SOLUTION The maximum service load reaction is the allowable strength: 5.9 Rn 108.7 = = 54.4 kips Ω 2.00 DEFLECTION In addition to being safe, a structure must be serviceable. With a Tension Field. Figure 3.11 illustrates the difference between transverse and longitudinal welds. Use A572 Grade 50 steel and select a trial cross section. 15⬘ Pipe 6 ×-strong FIGURE P4.3-2 4.3-3 Compute the nominal compressive strength of the member shown in Figure P4.3-3. FIGURE A.4 SOLUTION A W30 × 99 with Fy = 50 ksi is compact and, with continuous lateral support, the lateral bracing requirement is satisfied; therefore, plastic analysis is acceptable. The member is said to have buckled, and the corresponding load is called the critical buckling load. 74.9 ksi c. If the bearing strength was inadequate, the spacing and edge distances could be changed, or more bolts could be used. Required size = 2.04 sixteenths of an inch 9.10-1 b. Fy = 50 ksi for both the column and the channels. There are two categories of members: those with transverse loads applied between the ends and those with no transverse loads. Borello, D. Check for compliance with the AISC Specification. Lb = 20 ft, Cb = 1.67 b. Shape PNA Location ΣQn Mn 兾Ωb W14 × 26 W14 × 22 6 3 135 241 153 153 9.8 Tables for Composite Beam Analysis and Design 643 Since 66 in. Use an elastic analysis and compute the maximum bolt force. When the flange welds cool, they will shrink, typically about 1⁄8 inch. The welds shown in Figure 7.35a are complete penetration welds and can be made from one side, sometimes with the aid of a backing bar. Similarly, shapes with no axis of symmetry are rarely used for compression members, and flexural-torsional buckling analysis of these types of members will seldom, if ever, need to be done. An unbraced length of 20 ft with Cb = 1.0. c. If one stud is used at each section, the required spacing will be s= 30(12) = 3.33 in. If drains become clogged during a storm, the weight of the water will cause the roof to deflect, thus providing a reservoir for still more water. Bending caused by loads directly applied to members between the joints, however, must be taken into account. Different manufacturers'10K1 joists may have FIGURE 5.34 5.13 249 Open-Web Steel Joists LRFD FIGURE 5.35 STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf) Joist Designation 8K1** 10K1 12K1 12K3 12K5 14K1 14K3 14K4 14K6 16K2 16K3 16K4 16K5 16K6 16K7 16K9 Depth (in.) 8 8 8 8 8 14 14 14 14 16 16 16 16 16 16 16 Approx. From AISC Equation F13-1, Mn = Fu A fn 283.8 (127) = 5825 in.-kips Sx = 6.188 A fg This value is less than the LTB value of 6460 in.-kips, so it controls. For ASD, the allowable stress is Fa = 0.6Fcr = 0.6(28.59) = 17. From the dimensions given in Figure 8.29b, the properties of the column flange welds can be computed as 32(16) = 15.63 in.) 4 2 5 ⎣ ⎦ Because 1⁄10 of the load has been transferred from the member by the fastener at d, this potential failure line must resist only 9⁄10 of the load. The component of load normal to the roof will cause bending about the x axis, and the parallel component bends the beam about its y axis. This part of the stress–strain diagram is called the elastic range. • The web of the channel will be welded to a 3⁄8-inch-thick gusset plate. Use A992 steel and select a W14 shape. The alternative values of U can be useful during preliminary design, when actual section properties and connection details are not known. The shear areas are 5 (3 + 3 + 1.5) = 2.344 in.2 16 5 ⎡ 3 1 ⎤ Anv = ⎢3 + 3 + 1.5 − 2.5 ⎛⎜ + ⎞⎟ ⎥ = 1.661 in.2 ⎝ 4 8 ⎠⎦ 16 ⎣ Agv = The tension area is Ant = 5 ⎡ 3 1 ⎤ 1.5 − 0.5 ⎛⎜ + ⎞⎟ ⎥ = 0.3320 in.2 ⎝ 4 8 ⎠⎦ 16 ⎢⎣ For this type of block shear, Ubs = 1.0. From AISC Equation J4-5, Rn = 0.6Fu Anv + Ubs Fu Ant = 0.6(58)(1.661) + 1.0(58)(0.3320) = 77.06 kips with an upper limit of 0.6Fy Agv + Ubs Fu Ant = 0.6(36)(2.344) + 1.0(58)(0.3320) = 69.89 kips < 77.06 kips fRn = 0.75(69.89) = 52.4 kips > 39 kips (OK) 528 Chapter 8 Eccentric Connections FIGURE 8.40 Use a plate 5⁄16 × 31⁄2. Continuous lateral support. When computing the bearing strength for a bolt, use the distance from that bolt to the adjacent bolt or edge in the direction of the bearing load on the connected part. 384 EI LB 384(29, 000)538 The maximum permissible live load deflection is L 30(12) = = 1.5 in. 2 Required Ae = a = 0.5Fu 0.5(58) L 15(12) = = 0.6 in. Tables are available for both LRFD and ASD, in either U.S. Customary units or metric units. Determine the shear and bearing strengths. The addition of more thicknesses of material to the connection will increase the number of shear planes and further reduce the load on each plane. "Design of Base Plates for Wide Flange Columns—A Concatenation of Methods." Engineering Journal, AISC 27 (no. 1976. Use the composite beam tables and select a W-shape with Fy = 50 ksi. If 5 pcf is added to account for the reinforcing steel, the total weight is 150 pcf. Three national code organizations have developed model building codes: the Uniform Building Code (International Conference of Building Officials, 1999), the Standard Building Code (Southern Building Code Congress International, 1999), and the BOCA National Building Code (BOCA, 1999) (BOCA is an acronym for Building Officials and Code Administrators.) These codes have generally been used in different regions of the United States. The nominal load capacity per inch of weld is ⎛ 3 ⎞ (0.6 × 70) = 5.568 kips兾iin. PL 1⁄2 × 8 A36 steel 12″ Weld FIGURE P3.3-2 bolts 94 Chapter 3 3.3-3 Tension Members Determine the nominal tensile strength based on the effective net area. Try a W24 × 84. Doing so results in the homogeneous steel section of Figure 9.4b. 62 Chapter 3 Tension Members EXAMPLE 3.8 Determine the smallest net area for the American Standard Channel shown in Figure 3.18. 2″ 21⁄2″ 6″ FIGURE P9.7-2 9.7-3 Determine the nominal strength of the following composite beam: • W14 × 26, Fy = 50 ksi • fc′ = 4 ksi • effective slab width = 66 in. For a length of 3 in., the cover above the top of the stud is 4.5 – 3 = 1.5 in. W18 × 35 134ft-k W10 × 49 53k 7⁄8″-diameter, Group A bolts Bolts are pretensioned. The maximum permissible depth is 65 inches. The effective length KxL will control whenever Kx L > KyL rx 兾ry or rx 兾 ry K y L rx 兾ry 1.7 FIGURE 4.13 4.7 LRFD SOLUTION More on Effective Length 145 Pu = 1.2D + 1.6L = 1.2(140) + 1.6(420) = 840 kips Enter the column load tables with KL = 12 feet. From the properties tables in Part 1 of the Manual, the distance from the centroid to the outside face of the leg of an L6 × 6 × 1⁄2 is x− = 1.67 in. The design of a tension member connection will usually be done in parallel with the design of the member itself because the two processes are interdependent. Extended EndPlate Moment Connections—Seismic and Wind Applications. (Shear is considered separately in Section 5.8.) From elementary mechanics of materials, the stress at any point can be found from the flexure formula: fb = My Ix (5.3) where M is the bending moment at the cross section under consideration, y is the perpendicular distance from the neutral plane to the point of interest, and Ix is the moment of inertia of the area of the cross section with respect to the neutral axis. In this book, we indicate points of lateral support with an "×," as shown in Figure 5.12. The

member will be braced at midlength against buckling about the x-axis (the axis parallel to the long legs). In a rectangular section, the width-to-thickness ratio, l, equals b鼻t, which is the smaller outer dimension divided by the wall thickness. Use a total weld capacity of 6.750 kips鼻in. From Manual Table 7-1, the shear strength is rn = 16.2 kips鼻bolt Ω FIGURE 7.18 7.7 Design Examples 409 The number of bolts required is 72 = 4.44 bolts 16.2 Although five bolts will furnish enough capacity, try six bolts so that a symmetrical layout with two gage lines of three bolts each can be used, as shown in Figure 7.16. In the interior panels, 12 feet is the center-to-center stiffener spacing rather than the clear spacing. If A572 Grade 50 steel is used, what is the plastic moment Mp for the horizontal plastic neutral axis? Standard hot-rolled shapes are not susceptible to torsional buckling, but members built up from thin plate elements may be and should be investigated. 8 7.6-5 a. When structural shapes or plates are connected to form a built-up shape, they must be connected not only at the ends of the member but also at intervals along its length. Determine whether the shape is compact. The use of the effective length KL in place of the actual length L in no way alters any of the relationships discussed so far. The axial compressive load consists of a service dead load of 10 kips and a service live load of 20 kips. Of particular importance to the structural engineer is the prescription of minimum live loads for buildings. From C = ΣQn , a= ΣQn 172.3 = = 0.7678 0.85 fc′b 0.85(4)(66) Y2 = t − a 0.7678 = 4.5 − = 4.116 in. All equations are from Chapter F of the Specification. For ASD, the allowable bearing strength is Pp 鼻Ωc , where Ωc = 2.31. (We cover composite construction in Chapter 9.) The tabular values for the symmetrical shapes (W, HP, HSS and pipe) were calculated by using the minimum radius of gyration for each shape. 16 (AISC Table J2.4, based on plate thickness) The size required for strength, in sixteenths of an inch, is D= force resisted by stiffener 1.392 L Since flange local bending controls in this example, the force to be resisted by the stiffener is 134.7 − 88.20 = 46.50 kips The length available for welding the stiffener to the web is L = (length − clip) × 2 sides × 2 stiffeners (= 4.5 −) 5 (2)(2) = 15.5 in. 2.7 DESIGN COMPUTATIONS AND PRECISION The computations required in engineering design and analysis are done with either a digital computer or an electronic calculator. For the connection of the shear plate to the column flange, the required strength per inch is Vu 39 = = 4.588 kips鼻in. consist of a 20-psf construction load, a 20-psf partition load, a ceiling weighing 5 psf, and a live load of 150 psf. The maximum permissible live load deflection is L鼻240 = (30 × 12)鼻240 = 1.5 inch. and the strength of the transverse weld is 1.5(5.568) = 8.352 kips鼻in. 6 Fcr 3. Complete joint penetration (CJP) groove welds connect the beam flanges to the column and can transfer a moment equal to the moment capacity of the beam flanges. 26′ FIGURE P5.10-8 Floor and Roof Framing Systems 5.11-1 Use Fy = 50 ksi and select a shape for a typical floor beam AB. The tee and bolts are adequate: Required tf = 0.889 in. 77.0 kips d. • Only part of the beam web welds are considered effective in resisting shear. The resultant load is applied to the center of the top flange, but its line of action does not pass through the shear center of the section. The connection in Figure 7.4b is similar, but an analysis FIGURE 7.4 *There is actually a small eccentricity in the connections of Figure 7.3b and c, but it is usually neglected. The preceding computations are based on an assumed location of the instantaneous center of rotation. If there is not a tension field, the intermediate stiffeners must be proportioned to satisfy the moment of inertia requirement of AISC Equation G2-7. 2 2 From Equation 8.14, ⎛ 1 ⎞ 1⎤ 2 ⎡ ⎛ 1 1 ⎞ + ⎟ + h0 ⎟ − ⎟ + [h1 (p fi + s)] ⎟ h1 ⎟ ⎝ p fo ⎟⎠ 2 ⎟ g ⎣ ⎝ pfi s ⎠ 7⎡ 1 ⎞ ⎛ 1⎞ 1 ⎤ 2 ⎛1 [15.06(2 + 2.475)] = ⎟15.06 ⎟ + + 19.49 ⎟ ⎟ − ⎟ + ⎟ ⎝ 2 ⎠ 2 ⎦ 3.5 ⎝ ⎠ 2⎣ 2 2.475 Yp = bp 2 = 118.5 From Equation 8.15, Required t p = 1.11fM n = fb FyYp 1.11(2805) = 0.901 in. (versus 0.455 inch required in Example 8.5) 16 8.4 ANSWER ASD SOLUTION Eccentric Welded Connections: Shear Only 505 Use a 1⁄4-inch fillet weld, E70 electrode. 285 k 120 ft-k W12 × 58 Kx = Ky = 1.0 15′ 112 ft-k 285 k FIGURE P6.6-6 6.6-7 The member shown in Figure P6.6-7 has lateral support at points A, B, and C. Pa = D + L = 9 + 18 = 27 kips The shear strength of the weld per inch of length is 0.9279D = 0.9279(4) = 3.712 kips鼻in. • Girder is simply supported with lateral bracing at 10-ft intervals. Column LM, Pu = 1200 kips. If lp < l ≤ lr , the shape is noncompact. In this case, it results from initial crookedness, but in general, it can be the result of transverse loads or end moments. Although it is not a building code, ASCE 7, Minimum Design Loads for Buildings and Other Structures (American Society of Civil Engineers, 2010) is similar in form to a building code. 7.11-1 a. The required shear strength is Required Vn = ΩvVa = 1.67(156) = 260. As a reminder, all equations given here pertain to four-bolt unstiffened end-plate connections. Allowable Stress Formulation The allowable strength relation Va ≤ Vn Ωv 220 Chapter 5 Beams Vn FIGURE 5.19 ⎛ ⎞ 1.10 kvE/Fy ⎟ 0.6Fy Aw ⎟ ⎟ h/tw ⎝ ⎠ 0.6Fy Aw 1.51kv E ⎤ 0.6Fy Aw ⎡ 2 ⎟ ⎟⎣(h/tw) Fy ⎦ h/tw 1.10 kv E/Fy 1.37 kv E/Fy 260 can also be written in terms of stress as fv ≤ Fv where fv = Va = applied shear stress Aw Fv = Vn 鼻Ωv 0.6 Fy AwCv 鼻Ωv = allowable shear stress = Aw Aw For the most common case of hot-rolled I shapes with h鼻t w ≤ 2.24 E鼻Fy , Fv = 0.6 Fy Aw (1.0)鼻1.5 0 = 0.4 Fy Aw Shear is rarely a problem in rolled steel beams; the usual practice is to design a beam for flexure and then to check it for shear. Nair, 2005. Use A992 steel and select a W12 shape for the columns (vertical members). Constructing a frictionless pin connection is virtually impossible, so even this support condition can only be closely approximated at best. From AISC F3, for flange local buckling, if lp < l ≤ lr, the flange is noncompact, buckling will be inelastic, and ⎛ l − lp ⎞ M n = M p − (M p − 0.7 Fy S x) ⎟ ⎝ lr − l p ⎟⎠ (AISC Equation F3-1) where l= bf 2t f l p = 0.38 lr = 1.0 E Fy E Fy The webs of all hot-rolled shapes in the Manual are compact, so the noncompact shapes are subject only to the limit states of lateral-torsional buckling and flange local 212 Chapter 5 Beams buckling. Beam Design Charts Many graphs, charts, and tables are available for the practicing engineer, and these aids can greatly simplify the design process. TRUSSES WITH TOP-CHORD LOADS BETWEEN JOINTS If a compression member in a truss must support transverse loads between its ends, it will be subjected to bending as well as axial compression and is therefore a beam–column. The composite section must resist a service load and moment of wa = wD + wL = 437.5 + 1200 = 1638 lb鼻ft 1 1 Mu = wa L2 = (1.638)(30)2 = 184.3 ft-kips 8 8 9.5 Design 619 FIGURE 9.13 Try a nominal depth of d = 16 inches. First-order analyses of the frame were performed for both the sway and the nonsway cases. For the free body left of section a– a shown in Figure 3.35b, ∑M E = 19.29(20) − 2.479(20) − 4.802(15 + 10 + 5) − 4 FIJ = 0 FIJ = 48.04 kips 86 Chapter 3 Tension Members FIGURE 3.35 For the gross section, Required Ag = FIJ 48.04 = = 1.07 in.2 0.90 Fy 0.90(50) For the net section, Required Ae = FIJ 48.04 = = 0.985 in.2 0.75Fu 0.75(65) Try an MT5 × 3.75: Ag = 1.11 in.2 > 1.07 in.2 (OK) Compute the shear lag factor U from Equation 3.1. U = 1− () ( ) x 1.51 = 1− = 0.8322 9 Ae = AgU = 1.11(0.8322) = 0.9924 in.2 < 0.985 in.2 (N.G.) Try an MT6 × 5: Ag = 1.48 in.2 > 1.07 in.2 U = 1− (OK) (x) = 1 − (1.986) = 0.7933 Ae = AgU = 1.48(0.7933) = 1.17 in.2 > 0.985 in.2 (OK) 3.8 Tension Members in Roof Trusses 87 If we assume that the bottom chord is braced at the panel points, L 5(12) = = 101 < 300 r 0.594 ANSWER ASD SOLUTION (OK) Use an MT6 × 5. All terms in the following equations have been previously defined, and AISC equation numbers will not be shown. Let 0.01705Ra = 0.9279 × 6 where Ra is the service load reaction and 6 is the weld size in sixteenths of an inch. AISC J4.3, “Block Shear Strength,” gives the following equation for block shear strength: Rn = 0.6Fu Anv + Ubs Fu Ant ≤ 0.6Fy Agv + Ubs Fu Ant (AISC Equation J4-5) where Agv = gross area in shear (in Figure 5.20, length AB times the web thickness) Anv = net area along the shear surface or surfaces Ant = net area along the tension surface (in Figure 5.20, along BC) Ubs = 1.0 when the tensile stress is uniform (for most coped beams) = 0.5 when the tension stress is not uniform (coped beams with two lines of bolts or with nonstandard distance from bolts to end of beam) (Ricles and Yura, 1983) For LRFD, f = 0.75. A plate girder cross section consists of two flanges, 11⁄2 inches × 15 inches, and a 5⁄16-inch × 66-inch web. Other shapes are available, but those just described are the ones most frequently used. Because the load direction varies on each weld segment, the weld shear strength varies, but for simplicity, we will conservatively use Fnw = 0.6FEXX for the entire weld. A welded plate girder has 1-inch × 30-inch flanges and a 9⁄16-inch × 90-inch web. Required wg = required Ag 3.235 = = 3.235 in. This is in lieu of using AISC Equation J10-3. Since these quantities are not available in this example, we will assume that the ratio of Pstory to Pe story is the same as the ratio for the column under consideration. A992 steel is used for the beam and column. For structural steel, this means that the stress fmax must not exceed Fy and that the bending moment must not exceed My = Fy Sx where My is the bending moment that brings the beam to the point of yielding. Adequate: Required size = 1.46 sixteenths of an inch b. Purlins are located at the joints and halfway between the joints. Theory of Elastic Stability. (This could also be determined by observing that there is no footnote in the dimensions and properties table to indicate flange noncompactness for flexure.) For the web, λ= d = 20.8 tw λ p = 0.84 E 29, 000 = 0.84 = 20.23 Fy 50 λr = 1.03 29, 000 E = 1.03 = 24.81 Fy 50 Since lp < l < lr, the web is noncompact and Mn = FcrSx, where ⎡ ⎡ 50 ⎤ d Fy ⎤ 50 = 48.04 ksi Fcr = ⎢ 2.55 − 1.84 Fy = ⎢ 2.55 − 1.84(20.8) ⎥ tw E ⎦ 29, 000 ⎥⎦ ⎣ ⎣ M n = 48.04(3.23) = 155.2 in.-kips Next, check the limit state of yielding. Determine whether this member satisfies the appropriate AISC interaction equation. Determine the maximum bending stress in the steel in terms of the service-load bending moment M, where M is in ft-kips. . In the first, the frame is assumed to be braced against sidesway. Estimate the ultimate stress. Satisfactory: 0.968 < 1.00 5.15-3 a. Although a majority of connections are probably loaded eccentrically, in many cases the eccentricity is small and may be neglected. The columns and girders along any of the east-west lines make up an individual frame. Compute Cb for segment b–c. From Mn ≥ Ma Ωb and M n Fy Z y = = 0.6 Fy Z x Ωb 1.67 The required plastic section modulus is Zx ≥ Ma 0.6 Fy 5.5 Bending Strength of Compact Shapes 203 Thus, if the bending stress is based on the plastic section modulus Zx , fb = Ma Zx and Fb = 0.6 Fy This approach is useful when designing compact, laterally-supported beams. Consequently, the bolt force predicted by Equation 7.13 is conservative and does not quite agree with test results. “Properties of Steel for Use in LRFD.” Journal of the Structural Division, ASCE 104 (no. Unshored: Before Concrete Cures AISC I3.lb requires that when temporary shoring is not provided, the steel shape alone must have sufficient strength to resist all loads applied before the concrete attains 75% of its strength, fc′. To prevent excessive deformation, initiated by yielding, the load on the gross section must be small enough that the stress on the gross section is less than the yield stress Fy. To prevent fracture, the stress on the net section must be less than the tensile strength Fu. In each case, the stress P/A must be less than a limiting stress F or P 27.7 ksi (OK) ft = ANSWER Since ft < Ft for both limit states, the member is satisfactory. Viest, I. In the case of the unequal-leg angle, the longer leg dimension is always given first. The equations for Fe given in AISC E4(b) are based on well-established theory given in Theory of Elastic Stability (Timoshenko and Gere, 1961). Bearing stiffeners will be provided at the supports and at midspan. 4.6-6 Select a rectangular (not square) HSS for the conditions of Problem 4.6-3. At an interior joint, Pa = D + S = 1.335 + 2.0 = 3.335 kips At an exterior joint, Pa = 0.7323 + 1.0 = 1.732 kips 1.732k 3.335k 3.335k 3.335k 3.335k 3.335k 3.335k 1.732k a E C A B F G D I H a J 13.40k 13.40k 3.335k 3.335k 3.335k (a) 1.732k FIGURE 3.36 3.335k The loaded truss is shown in Figure 3.36a. In addition, because of the approximations involved in using the curves, we will be conservative in rounding the value of a. Beyond the elastic limit, unloading will be along a straight line parallel to the initial linear part of the loading path, and there will be a permanent strain. Consequently, ΔT will be no greater than 0.1ΔN, or equivalently, maximum ΔT鼻ΔN = 0.1, demonstrating that most of the applied load is devoted to relieving the compression of the connected parts. The capacity of a 1.5-inch length of four welds is 1.5(13.50) = 20.25 kips For the end bearing stiffener, the applied load per inch is Reaction 223.4 = = 3.662 kips鼻in. Section 5.2.6 of AISC Appendix 5, “Evaluation of Existing Structures,” specifies that ASTM A502 Grade 1 rivets should be assumed unless there is evidence 377 378 Chapter 7 Simple Connections to the contrary. φ (0.707) Fnw 0.75(0.707)(0.6 × 70) Alternatively, for E70 electrodes, fRn = 1.392 kips鼻in. In this book, we conservatively assume that the shielded metal arc welding process is used.) Thus, for a given length of weld L subjected to a load of P, the critical shearing stress is fv = P 0.707 wL where w is the weld size. The size of a fillet weld is denoted w and is the length of one of the two equal sides of this idealized cross section. Flexural-torsional buckling. Other end conditions can be accounted for in the derivation of Equation 4.3. In general, the bending moment will be a function of x, resulting in a nonhomogeneous differential equation. Although AISC does not require an upper limit on the slenderness ratio KL鼻r, an upper limit of 200 is recommended (see user note in AISC E2). Flexibility of the connection is another important consideration. 9.4-5 Design stud anchors for the beam in Problem 9.3-1. 253 kips b. Try a W10 × 88 (fc Pn = 940 kips): Actual Kx L 20 = = 11.56 ft < 12 ft rx 鼻ry 1.73 ∴ fc Pn > required 840 kips (By interpolation, fc Pn = 955 kips.) Check a W12 × 79: Kx L 20 = = 11.43 ft. Trusses are spaced at 10 feet on centers. < 10.8 kips鼻in. Pcr = FIGURE 4.5 4.2 Column Theory 115 The stress–strain curve shown in Figure 4.5 is different from those shown earlier for ductile steel (in Figures 1.3 and 1.4) because it has a pronounced region of nonlinearity. 120 k 135ft-k W12 × 58 Fy = 50 ksi 16′ 67ft-k 120 k FIGURE P6.6-4 6.6-5 A simply supported beam is subjected to the end couples (bending is about the strong axis) and the axial load shown in Figure P6.6-5. FIGURE 4.22 SOLUTION Compute the flexural buckling strength for the x-axis: K x L 16(12) = 121.5 = 1.58 rx Fe = 4.71 Since p 2E (KL鼻r)2 = p 2 (29, 000) (121.5)2 = 19.39 ksi E 29, 000 = 4.71 = 134 Fy 36 KL E < 4.71 , use AISC Equation E3-2. From Equation 7.37, the base metal shear yield strength per unit length is Rn ⎛ 9⎞ = 0.4 Fy t = 0.4(36) ⎟ ⎟ = 8.10 kips鼻in. In addition, the American Institute of Steel Construction has been very helpful in providing updates on the AISC Specification and Manual. The term plastic is used because, at failure, parts of the member will be subjected to very large strains—large enough to put the member into the plastic range (see Figure 1.3b). 2k/ft W16 × 40 10′ 4 4 3 3 2k/ft 2k/ft Centroid (a) (b) FIGURE P5.15-5 5.15-6 The truss shown in Figure P5.15-6 is part of a roof system supporting a total gravity load of 40 psf of roof surface, half dead load and half snow. Bending about the y axis subjects the fasteners to shear, so a sufficient number of fasteners must be provided to account for this action. As indicated at the beginning of this chapter, stud anchors are the most common, and we consider only this type. The trusses are spaced at 30 feet. If the plank beam is oriented so that bending takes place about its other axis (the b-axis), then both planks bend in exactly the same manner, and there is no slippage and hence no shear. LRFD SOLUTION The design strength is ASD SOLUTION From Equation 4.7, the allowable stress is fc Pn = 0.90(71.99) = 64.8 kips Fa = 0.6Fcr = 0.6(9.599) = 5.759 ksi The allowable strength is Fa Ag = 5.759(7.50) = 43.2 kips ANSWER Design compressive strength = 64.8 kips. For valuable information on pricing, previous editions, changes to current editions, and alternate formats, please visit www.cengage.com/highered to search by ISBN#, author, title, or keyword for materials in your areas of interest. If we let ⎡ ⎛ R⎞⎤ ⎟ ln ⎟ ⎟ ⎟ ⎣ ⎝ Q ⎠ ⎦m β= s ln(R鼻Q) then ⎡ ⎛ R⎞ ⎤ ⎟ ln ⎟ ⎝ Q ⎟⎠ ⎟ = bs ln(R鼻Q) ⎣ ⎦m The variable b can be interpreted as the number of standard deviations from the origin that the mean value of ln(R鼻Q) is. In Section 7.2, we saw that the shear load on a bolt is P = fv Ab where fv is the shearing stress on the cross-sectional area of the bolt and Ab is the cross-sectional area. Because of slenderness effects, the strength predicted by AISC Equation I2-4 cannot be achieved. For the welded connection shown in Figure 8.21, the first equation can be obtained by summing moments about the lower longitudinal weld: ∑ M L 2 = Tc − P3 L3 – P1 L3 = 0 2 This equation can be solved for P1, the required resisting force in the upper longitudinal weld. SOLUTION For a W14 × 74, bf = 10.1 in., tf = 0.785 in., and bf 2t f 0.56 = 10.1 = 6.43 2(0.785) 29, 000 E = 0.56 = 13.5 > 6.43 50 Fy h d − 2kdes 14.2 − 2(1.38) = = = 25.4 tw tw 0.450 (OK) 123 124 Chapter 4 Compression Members where kdes is the design value of k. 2 2 To determine which component has the smaller bearing strength, compare the values of tFu (other variables are the same). Beam design: Select a trial shape based on full composite behavior. For a collapse mechanism to form, the structure must remain stable as the load increases, and the plastic hinges that form early must have enough rotation capacity (ductility) for the formation of the remaining plastic hinges. D = 65k L = 130k 18′ FIGURE P9.10-3 9.10-4 Design a rectangular (not square) HSS filled with concrete to resist an axial dead load of 200 kips and an axial live load of 500 kips. 3 Agv = 2 × (2.75 + 2.75 + 1.25) = 5.063 in.2 8 3 Anv = 5.063 − (22.5 × 1.0)(2) = 3.188 in.2 8 Rn 0.6 Fu Anv + U bs Fu Ant = Ω Ω 0.6(58)(3.188) + 1.0(58) (0.75) = 77.2 kips > 72 kips = 2.00 (OK) Check the upper limit: 0.6 Fy Agv + Ubs Fu Ant 0.6(36)(5.063) + 1.0(58)(0.7500) = 2.00 Ω = 76.4 kips < 77.2 kips Therefore, the upper limit controls, but the strength is still adequate. The maximum live load deflection must not exceed L鼻240. Example 2.1 illustrates that the controlling load combination for LRFD may not control for ASD. D = 70 k L = 200 k 10′ Q 20′ 10′ FIGURE P6.6-13 Members in Unbraced Frames 6.7-1 A W14 × 74 of 50 ksi steel is to be investigated for use as a beam–column in an unbraced frame. Steel Joist Institute. 8.4-12 Solve Problem 8.4-7 by the ultimate strength method (use the tables in Part 8 of the Manual). Many of the terms and constants used in other parts of the Manual are presented to facilitate the design process and are not necessarily part of the Specification. In 2005, the two approaches were unified in a single specification and a single manual, the thirteenth edition of the Steel Construction Manual. American Association of State Highway and Transportation Officials. Note that this is a slender-element compression member, and the equations of AISC Section E7 must be used. From AISC J3.3, the minimum spacing is s = 2.667d = 2.667 (87) = 2.33 in. Neither the full strength of the concrete nor that of the steel can be developed, and the compressive force is limited to the maximum force that can be transferred across the interface between the steel and the 9.7 Composite Beams with Formed Steel Deck 627 concrete—that is, the strength of the studs, ΣQn. Recall that C is the smallest of As Fy , 0.85fc′Ac , and ΣQn. With partial composite action, the plastic neutral axis (PNA) will usually fall within the steel cross section. Read the available strength, interpolating if necessary. Both ends are pinned, and there is support against weak axis buckling at a point 6 feet from the top. Design a weld to develop the full tensile capacity of the member. Use a double-angle shape with the short legs backto-back, separated by 3⁄8-inch. “The Bending Resistance of Steel Beams.” Journal of the Structural Division, ASCE 104 (no. Load combination 3 will control. Approximately 52 ksi e. In most cases, hot-rolled beams will not have stiffeners, and we will defer treatment of stiffened webs until Chapter 10. 2.5 M max 12.5 M max + 3 M A + 4 M B + 3 MC 12.5 (81) 2.5 (81) + 3 (323) + 4 (81) + 3 (323) = 1.14 210 Chapter 5 Beams Figure 5.15 shows the value of Cb for several common cases of loading and lateral support. The thickness of large plates is determined from consideration of bending of the portions of the plate that extend beyond the column outline. Assume a conservative value of U = 0.60. Select a rectangular HSS. (Alternatively, the stud shear strength can be found from Table 3-21 in the Manual. This will depend on the placement of the studs in the ribs. Design a noncomposite beam. To illustrate the procedure, however, we compute the prying force, using Equations 7.20 and 7.14. 2 2 m= From Equation 5.17, n′ = 1 1 db f = 10.0(10.0) = 2.5 in. You may get the manual you. (OK) ≈ 2.286 < 3 h 2 2 a ⎡ 260 ⎤ ⎡ 260 ⎤ < = b. Channel shapes are sometimes used, as are beams built up from plates, in the form of I or box shapes. M t , the moment corresponding to the sway (caused by the horizontal load H), will be amplified by B2 to account for the PΔ effect. Does this beam have adequate moment strength? The load shown is the total service load, consisting of dead and live loads. The moment corresponding to first yield is Mr = 0.7Fy Sx where the yield stress has been reduced by 30% to account for the effect of residual stress. TABLES FOR COMPOSITE BEAM ANALYSIS AND DESIGN When the plastic neutral axis is within the steel section, computation of the flexural strength can be laborious. EXAMPLE A.4 The three-span continuous beam shown in Figure A.6 must support the gravity service loads given. From the LRFD solution, the nominal shear strength in the end panels is Vn = 131.0 kips and the allowable strength is Vn 131.0 = = 78.4 kips Ωv 1.67 The maximum shear in the end panel is Va = 156 kips > 78.4 kips (N.G.) Two options are available for increasing the shear strength: either decrease the web slenderness or decrease the aspect ratio of each end panel by adding an intermediate stiffener. Three different spacings will be investigated: 1. EXAMPLE 5.7 Check the beam in Example 5.6 for shear. Use an unbraced length equal to the span length and determine whether the beam is adequate. 140 k Mux = 150 ft-k Muy = 150 ft-k 16′ Mux = 75 ft-k Muy = 75 ft-k FIGURE P6.8-4 6.8-5 Select a W14 of A992 steel. Round HSS are designated by outer diameter and wall thickness, expressed to three decimal places; for example, HSS 8.625 × 0.250. Specification for Structural Steel Buildings. total ANSWER ASD SOLUTION Use the connection detail as shown in Figure 7.18. If the width is more than 8 inches, the shape is classified as a plate. ASD SOLUTION Pa = D + L = 9 + 27 = 36 kips M = Pe = 36(6.714) = 241.7 in.-kips (clockwise) 36 = 5.143 kips ↓ pcx = 0 pcy = 7 Check the lower right bolt. Use A992 steel. 80 70 60 Stress (ksi) 1.5-5 50 40 30 20 10 0 0.05 0.1 0.15 0.2 0.25 0.3 Strain 60 50 Stress (ksi) 18 40 30 20 10 0 0 0.002 0.004 0.006 Strain FIGURE P1.5-5 0.008 0.01 0.012 Problems 1.5-6 1.5-7 19 The data shown in the table were obtained from a tensile test of a metal specimen with a diameter of 0.500 inch and a gage length (the length over which the elongation is measured) of 2.00 inches. × (4 + 4) in. If 7⁄8-inch-diameter Group A bearing-type bolts are used, is the connection adequate? As the load is applied, if the flange is flexible enough to deform as shown, the compressive forces will migrate toward the edges of the flange. (or, preferably,, 3d = 3(87) = 2.63 in.) From AISC Table J3.4, the minimum edge distance is Le = 11⁄8 in. 500 kips鼻in. The horizontal 10-inch dimension is a maximum. Bolts above this axis are subjected to tension, and bolts below the axis are assumed to be subjected to compressive forces, as shown in Figure 8.13c. ⎝ 2⎠ Ω The base metal shear strength is therefore 7.20 kips鼻in. (Because of strain hardening, the full plastic moment capacity of the beam can actually be developed through the flanges.) Making the flange connection requires that a small portion of the beam web be removed and a “backing bar” used at each flange to permit all welding to be done from the top. Span length = 30 ft 612 Chapter 9 Composite Construction From Example 9.4, the horizontal shear force V ′ corresponding to full composite action is V ′ = C = 515 kips Try 1⁄2 in. The effective length is 15 feet with respect to both axes. 10.7 Design 703 The number of spaces required is 324 = 3.27 99.2 Use four spaces. For width-to-thickness ratio limits for welded shapes, refer to Table B4.1b in Chapter B of the AISC Specification, “Design Requirements.” 5.6-4 A built-up shape consisting of two 1 × 16 flanges and a 5⁄16 × 40 web is used as a beam with continuous lateral support. I 36 P 22 fv = u = = 1.833 kips鼻in. From the column load tables, with Kx L = 10 feet and Ky L = 5 feet, try a WT6 × 17.5, with an axial compressive design strength of fc Pn = 149 kips A footnote indicates that this shape is slender for compression, but this has been accounted for in the tabulated value of the strength. Ultimate strength analysis. If the maximum permissible live load deflection is L鼻360, is the beam satisfactory? The open-web steel joists that are designed to function as floor or roof joists (in contrast to girders) are available as open-web steel joists (K-series, both standard and KCS), longspan steel joists (LH-series), and deep longspan steel joists (DLH-series). 2): 37–42. ASCE 7 presents the basic load combinations in the following form: Combination 1: Combination 2: Combination 3: Combination 4: 1.4D 1.2D + 1.6L + 0.5(Lr or S or R) 1.2D + 1.6(Lr or S or R) + (L or 0.5W) 1.2D + 1.0W + L + 0.5(Lr or S or R) 2.3 Load Factors, Resistance Factors, and Load Combinations for LRFD 25 Combination 5: 1.2D + 1.0E + L + 0.2S Combination 6: 0.9D + 1.0W Combination 7: 0.9D + 1.0E where D = dead load L = live load due to occupancy Lr = roof live load S = snow load R = rain or ice load* W = wind load E = earthquake (seismic load) In combinations 3, 4, and 5, the load factor on L can be reduced to 0.5 if L is no greater than 100 pounds per square foot, except for garages or places of public assembly. For ASD, the safety factor is Ωv = 1.67. This technique will be illustrated in the following example. 0.85 fc′b 0.85(4)(87) The distance from the top of the steel to the compressive force C is Y2 = t − LRFD SOLUTION a 1.792 = 5− = 4.104 in. Cb = = ANSWER Cb = 1.14. From Example 9.4, a = 1.683 in., and C = 515 kips. ⎡ Vr − Vc1 ⎤ I st ≥ I st1 + (I st 2 − I st1) ⎟ ⎣ Vc 2 − Vc1 ⎦⎥ (AISC Equation G3-4) 10.5 Shear Strength 679 where Ist1 = required moment of inertia as calculated for the no-tension-field case (from AISC Equation G2-7) Ist2 = moment of inertia required to develop the buckling plus post-buckling shear strength = h 4 ρ1st.3 ⎛ Fyw ⎞ 40 ⎝ E ⎠ ⎛ Fyw ρst = max ⎟ , ⎝ Fyst 1.5 (AISC Equatiion G3-5) ⎞ 1 ⎟ ⎠ Fyw = yield stress of the girder web Vr = the larger of the required shear strengths (Vu for LRFD, Va for ASD) on each side of the stiffener; that is, in the adjacent web panels Vc1 = the smaller of the available shear strengths (fvVn for LRFD, Vn鼻Ωv for ASD) in the adjacent panels, calculated with no tension field action (using AISC G2.1) Vc2 = the smaller of the available shear strengths (fvVn for LRFD, Vn鼻Ωv for ASD) in the adjacent panels, calculated with tension field action (using AISC G3.2) In a tension field panel, the stiffeners are subjected primarily to bending, rather than axial force. The moments occur at one end; the other end is pinned. 9.3 EFFECTIVE FLANGE WIDTH The portion of the floor slab that acts compositely with the steel beam is a function of several factors, including the span length and beam spacing. Under certain conditions, tb can be taken as 1.0 even when aPr / Py > 0.5. As mentioned in Section 6.3, acceptable frame analysis methods, including the direct analysis method, require the application of notional loads to account for initial out-of-plumbness of columns. • The width of the stiffener plus half the column web thickness must be equal to at least one third of the width of the beam flange or plate delivering the force to the column or, from Figure 8.45, b+ t w bb ≥ 2 3 ∴ b≥ bb t w − 3 2 • The stiffener thickness must be at least half the thickness of the beam flange or plate, or t tst ≥ b 2 FIGURE 8.45 8.7 Column Stiffeners and Other Reinforcement 541 • The stiffener thickness must be at least equal to its width divided by 16, or tst ≥ b 16 • Full-depth stiffeners are required for the compression buckling case, but halfdepth stiffeners are permitted for the other limit states. Bolts have enough shear strength; 120 kips < 130 kips 10.4-3 3340 ft-kips 10.4-5 a. Their use will be illustrated in Example 7.5. EXAMPLE 7.5 Determine the strength of the connection of Example 7.4 based on the limit states of shear, slip-critical, and bearing strengths. American Welding Society. If there is a possibility that the footing will undergo a slight rotation, or if the connection is flexible enough to allow a slight rotation, the support must be considered to be pinned. Use A992 steel for the structural shapes, A36 for the end plate, and pretensioned Group A bolts. In addition to the members described, this book covers the design of connections and the following special members: composite beams, composite columns, and plate girders. This value is tabulated for all rolled shapes in the properties tables in Part 1 of the Manual. The compression flange strength is therefore based on yielding, and Fcr = Fy = 36 ksi. (e) FIGURE P3.3-1 3.3-2 For the tension member shown, compute the following. Problems 589 PL 5⁄8 × 7 × 1′-02 ⁄4 ⁄4 1 1 W10 × 45 W16 × 45 PL 5⁄16 × 5 × 1′-01⁄2 7⁄8-inch-diameter 21⁄4″ Group A bolts 21⁄4″ 21⁄4″ 21⁄4″ 3″ 21⁄4″ 3″ 3″ 3″ 131⁄2″ 21⁄4″ 7″ 21⁄2″ 2 3″ 3″ 21⁄4″ 1⁄4″ 1⁄2″ setback 1 ⁄2″ setback 14″ Detail of flange plate 5″ Detail of shear plate FIGURE P8.6-2 8.6-3 Same as Problem 8.6-2, but use ASD. For the inner holes and a spacing of 3 inches, Manual Table 7-4 gives 1 1 rn = 43.5t = 43.5 ⎛⎜ + ⎞⎟ = 21.8 kips鼻bolt ⎝ 4 4⎠ Ω For the holes nearest the edge of the plate, we will conservatively use an edge distance of 11⁄4 inches to find the bearing strength from Manual Table 7-5. There is further discussion in the Commentary to the Specification. A thorough understanding of the basis of the AISC provisions for plate girders requires a background in stability theory, particularly plate stability. The use of the tables is illustrated in the following example. However, the use the alignment charts is acceptable (Nair, 2005). Structural Steel Design Mccormac 5th Ed PDF - Free ebook download as PDF File (.pdf) or read book online for SEGUI - Steel Design (4ed) - Solutions Manual. Although either of these arrangements could be used, the connection length can be decreased by using a larger bolt size and fewer bolts. Compute Cb for the 12-foot unbraced segment adjacent to midspan. Assume that the PNA is in the flange, and solve for the exact location. Determine Kx for member AB. The coefficient of variation, V, is the standard deviation divided by the mean, or V = s x If the actual frequency distribution is replaced by a theoretical continuous function that closely approximates the data, it is called a probability density function. Try 3⁄4-inch Group A bolts. A complete list of minimum tension values, for those connections in which a minimum tension is required, is given in AISC Table J3.1, “Minimum Bolt Pretension.” Each value is equal to 70% of the minimum tensile strength of the bolt. ⎝ 8⎠ and the base metal shear rupture strength is ⎛ 3⎞ 0.45Fu t = 0.45(58) ⎟ ⎟ = 9.788 kips鼻in. Lateral support is provided at the ends only. The requirements for fillet welds are found in AISC J2.2b and are summarized here. × 22-in. 2 2 2 2 The design flexural strength is fb Mn = fb(Cy) = 0.90(384)(10.72) = 3705 in.-kips = 309 ft-kips > 279 ft-kips Check the shear: Vu = wu L 2.476(30) = = 37.1 kips 2 2 From the Zx table, fvVn = 106 kips > 37.1 kips (OK) (OK) 618 Chapter 9 ANSWER Composite Construction Use a W16 × 26. If the stiffeners are conservatively assumed to resist the total applied load P (this assumption neglects any contribution by the web), the bearing stress on the contact surfaces may be written as fp = P Apb where A pb = projected bearing area = 2at (see Figure 10.3) or, expressing the bearing load in terms of the stress, P = fp A pb (10.1) In addition, the pair of stiffeners, together with a short length of web, is treated as a column with an effective length less than the web depth and is investigated for compliance with the same Specification provisions as any other compression member. Allowable compressive strength = 330 kips. 2007. This is called a double-angle shape. (OK) 5.15 Biaxial Bending 265 Compute the required bearing area. The beam and column are of A992 steel, and the plate material is A36. A992 steel is used, and the 2 kip鼻ft is a service live load. (Figure 8.7 shows the bolt forces as resisting forces acting to oppose the applied load.) 4. Check deflections (we cover this in Section 9.6). From the inner row of tension bolts plus two bolt diameters to the inside face of the compression flange Design Procedure We will first consider the procedure for LRFD and then present a summary for ASD. 368 Chapter 6 Beam–Columns 66 k 6′ 6k 12′ W8 × 24 6′ FIGURE P6.6-10 6.6-11 The beam–column in Figure P6.6-11 is a member of a braced frame. A W8 × 58 has an interpolated strength of Pn = 397 kips < 400 kips Ωc (N.G.) The next lightest W8 that will work is a W8 × 67. Steel Design Solution Manual Applied Structural Steel Design Solution Manual Structural Design Solutions Manuals /. A992 steel is used. This moment is amplified with the 6.7 Members in Unbraced Frames 329 FIGURE 6.16 factor B1 to account for the Pd effect. r Fy Fcr = 0.877 Fe = 0.877(21.27) = 18.65 ksi 0.6 Fcr Ag = 0.6(18.65)(35.1) = 393 kips < 400 kips This is very close, so try the next larger size. Satisfactory: s = 2.75 in. If the moment is a required moment capacity, then any curve above the point corresponds to a beam with a larger moment capacity. The girder weight can now be computed. FIGURE 5.50 EXAMPLE 5.19 A roof system consists of trusses of the type shown in Figure 5.51 spaced 15 feet apart. (OK) 386 Chapter 7 Simple Connections FIGURE 7.10 From AISC Table J3.4, the minimum edge distance in any direction is 1 inch. Although the moment arm of the couple is actually the distance from center of flange plate to center of flange plate, the plate thickness is not yet known, and use of the beam depth is conservative. Thus, fRn = 2(edge bolt strength) + 2(inner bolt strength) = 2(23.7) + 2(24.9) = 97.2 kips > 76 kips (OK) ANSWER ASD SOLUTION Use four bolts, two per line, on each side of the splice. For the interior bearing stiffeners, the nominal bearing strength (see the LRFD solution) is Rn = 631.8 kips and the allowable strength is Rn 631.8 = = 316 kips > 40 kips Ω 2.00 (OK) From the LRFD solution, the nominal strength of the stiffener assembly as a compression member is Pn = 531.7 kips The allowable strength is Pn 531.7 = = 318 kips > 40 kips Ω 1.67 (OK) For the bearing stiffeners at the supports, the nominal bearing strength from the LRFD solution is Rn = 1264 kips and the allowable strength is Rn 1264 = = 632 kips > 156 kips Ω 2.00 (OK) 10.7 Design 695 From the LRFD solution, the nominal strength of the stiffener assembly as a compression member is Pn = 870.8 kips and the allowable strength is Pn 870.8 = = 521 kips > 156 kips Ω 1.67 ANSWER 10.7 (OK) The bearing stiffeners are adequate. This corresponds to footnote 1, so a W14 × 68 is available in A242 steel with a yield stress Fy of 50 ksi. TABLE 9.9 or 30(12)鼻4 = 90 in. Revise the design if necessary. Member stiffnesses were not reduced, and the effective length method is applicable. The maximum possible value of this force is F = mN where m is the coefficient of static friction between the connected parts, and N is the normal compressive force acting on the inner surfaces. From AISC Equation J4-1, Rn = Fy Ag Rn Fy Ag = Ω 1.67 1.67(Rn 鼻Ω) 1.67 H 1.67(80.77) = = = 3.747 in.2 Required Ag = Fy Fy 36 From AISC Equation J4-2, Rn = Fu Ae Rn Fu Ae = Ω 2.00 2.00(Rn 鼻Ω) 2.00 H 2.00(80.77) = = = 2.785 in.2 Required Ae = Fu 58 Fu Try a plate width ofwg = 6.5 in. 24.3 kips Chapter 10: Plate Girders 8.5-5 a. The procedure is illustrated in Example 9.7. EXAMPLE 9.7 A design has resulted in a composite beam with a total slab thickness of 41⁄2 inches and a W16 × 31 of A992 steel. For span lengths in the shaded areas, special bridging (interconnection of joists) is required. From Figure 5.15b, Cb = 1.30 for the load and lateral support conditions of this beam. The tensile strength of concrete is small and is discounted, so no stress is shown where tension is applied to the concrete. This requires a knowledge of the properties of all columns in the story, as well as H and ΔH, so that Pstory and Pe story can be computed. In addition to the weight of the structure, loads consist of a uniform live load of 80 psf and moveable partitions, to be accounted for by using a uniformly distributed load of 20 pounds per square foot of floor surface. The most commonly used steels for rolled shapes and plate material are ASTM A36, A572, and A992. Use AISC Equation J10-9 to find the required doubler plate thickness. > 0.194 in., and since Fy for the beam is larger than Fy for the plate, the bearing strength of the beam web will be adequate.) Other limit states for connection plates (in addition to bolt bearing) can be found in AISC J4, “Affected Elements of Members and Connecting Elements.” (These are the same provisions we use for shear on the base metal for welded connections.) To determine the plate thickness required for the vertical shear, consider both yielding of the gross section and rupture of the net section. 4): 173–4. “Structural Serviceability: A Critical Appraisal and Research Needs.” Journal of Structural Engineering, ASCE 112 (no. = 36.75 ft 0.7 Fy 0.7(36) 686 Chapter 10 Plate Girders Since Lp < Lb < Lr, the girder is subject to inelastic lateral-torsional buckling. However, the factored strength is a reduced, usable strength, and the resistance factor is usually less than unity. Tension on the gross section: ft = 0.90, Ωt = 1.67, where t be b Asf a d Apb Pn = Fy Ag (AISC Equation D2-1) = thickness of connected part = 2t + 0.63 ≤ b = distance from edge of pin hole to edge of member, perpendicular to direction of force = 2t(a + d鼻2) = distance from edge of pin hole to edge of member, parallel to direction of force = pin diameter = projected bearing area = dt Additional requirements for the relative proportions of the pin and the member are covered in AISC D5.2 FIGURE 3.38 90 Chapter 3 Tension Members Problems Tensile Strength 3.2-1 A PL 3⁄8 × 7 tension member is connected with three 1-inch-diameter bolts, as shown in Figure P3.2-1. Prepare a table showing values of Group A bolt shear strength and slip-critical strength for bolt diameters of 1⁄2 inch to 11⁄2 inches in increments of 1⁄8 inch. If this replacement is made, the load will be concentric, and each fastener can be assumed to resist an equal share of the load, given by pc = P鼻n, where n is the number of fasteners. An intermediate spacing of 5 inches. The base-metal shear strength expression can also be somewhat simplified. Design for the following loads. Furthermore, subdivisions are numerically labeled. Maximum effective end-plate width = bfb + 1 = 6.00 + 1 = 7.00 in. 11⁄2″ 31⁄2″ 31⁄2″ 2″ 21⁄4″ 21⁄2″ FIGURE P7.4-5 7.4-6 A double-angle tension member, 2L4 × 3 × 1⁄2 LLBB, is connected to a 3⁄8-inch-thick gusset plate with 7⁄8-inch-diameter, Group A bolts as shown in Figure P7.4-6. Working stresses are those resulting from the working loads, which are the applied loads. 6.7 Members in Unbraced Frames 333 FIGURE 6.19 Determine the appropriate interaction equation: 454 Pu = = 0.6858 > 0.2 fc Pn 662 ∴ Use Equation 6.3 (AISC C Equation H1-1a). Armed with this information, the structural designer can then select the appropriate cross section. In 1964, he enrolled in graduate school and was the recipient of an NSF Graduate Traineeship. In terms of safety, this limit state can be fracture, yielding, or buckling, and the factored resistance is the useful strength of the member, reduced from the theoretical value by the resistance factor. The effective length factors are 370 Chapter 6 Beam–Columns Kx = 0.85 for the braced case, Kx = 1.2 for the unbraced case, and Ky = 1.0. Determine whether this member is in compliance with the AISC Specification. Tension on the gross area: ⎛ 1⎞ Pn = Fy Ag = 36 ⎜ 6 × ⎟ = 108.0 kips ⎝ 2⎠ Tension on the net area: All elements of the cross section are connected, so shear lag is not a factor and Ae = An. For the hole diameter, use h=d+ 1 3 1 7 = + = in. Design of Simple Shear Connections. TABLE 1.2 FIGURE 1.9 Shape Preferred Steel Angles Plates S, M, C, MC HP W Pipe HSS A36 A36 A36 A572 Grade 50 A992 A53 Grade B (only choice) A500 Grade B (round) or C (rectangular) Problems 17 Problems Note The following problems illustrate the concepts of stress and strain covered in Section 1.5. The materials cited in these problems are not necessarily steel. In the three rigid connections shown in Figure 8.50, the same number of bolts are used at each flange, so that the connections can be used in cases of moment reversal. Compression Buckling of the Web. When the load is transmitted through a plate, web yielding is assumed to take place on the nearest section of width tw. The nominal stiffener strength therefore will be Fyst Ast where Fyst = yield stress of the stiffener Ast = area of stiffener 540 Chapter 8 Eccentric Connections For LRFD, equate the design strength of the stiffener to the extra strength needed and solve for the required stiffener area: fst Fyst Ast = Pbf − fRn min Ast = Pbf − fRn min fst Fyst (8.7) where fst = 0.90 (since this is a yielding limit state) Pbf = applied factored load from the beam flange or flange plate fRn min = smallest of the strengths corresponding to the three limit states For ASD, equate the allowable strength of the stiffener to the extra strength needed and solve for the required stiffener area: Fyst Ast = Pbf − (Rn 鼻Ω)min Ωst Ast = Pbf − (Rn 鼻Ω)min Fyst 鼻Ωst (8.8) where Ωst = 1.67 (since this is a yielding limit state) Pbf = applied service load from the beam flange or flange plate (Rn鼻Ω)min = smallest of the strengths corresponding to the three limit states AISC J10.8 gives the following guidelines for proportioning the stiffners. FIGURE 4.17 154 Chapter 4 SOLUTION Compression Members Compute elastic G factors: For joint A, ∑ (I c 鼻Lc) 171鼻12 14.25 = = = 1.52 ∑ (I g 鼻Lg) 88.6鼻 20 + 88.6鼻18 9.3552 For joint B, ∑ (I c 鼻Lc) 2(171鼻12) 28.5 = = = 1.36 ∑ (I g 鼻Lg) 199鼻 20 + 199鼻18 21.01 From the alignment chart for unbraced frames, Kx = 1.45, based on elastic behavior. Show your results on a sketch, complete with dimensions. Check the stiffener at the support. This condition corresponds to small values of G given by Equation 4.12. Calculate B1 for each axis: 0 ⎞ M1 ⎞ Cm = 0.6 − 0.4 ⎛⎜ = 0.6 − 0.4 ⎛⎜ = 0.6 (for both axes) ⎝ M 2 ⎟⎠ ⎝ M 2 ⎟⎠ From Section 6.5, for each axis, Pe1 = π 2 EI* (K1 L)2 and the modified stiffness is EI* = 0.8tbEI = 0.8(1.0)EI = 0.8 EI Therefore, π 2 (0.8 EI x) π 2 (0.8)(29, 000) (272) = = 1922 kips (K x L)2 (15 × 12)2 Cmx 0.6 = = 0.654 < 1.0 ∴ B1x = 1.0 as assumed. From Equation 8.8, the required stiffener area is Ast = Pbf − (Rn 鼻Ω)min 89.78 − 58.68 = = 1.44 in.2 36鼻1.67 Fyst 鼻Ω The minimum width is b≥ bb t w 6.02 0.360 − = − = 1.83 in. Although the required stiffener spacing is a clear distance, the use of center-tocenter distances is somewhat simpler and will be slightly conservative. KxL Approximate rmx 鼻rmy rmx 鼻rmy 1.2 1.3 1.4 1.5 1.6 1.8 13.3 12.3 11.4 10.7 10.0 8.9 Shape Actual rmx 鼻rmy fcPn (kips) 3 HSS12 × 10 × ⁄8 1 HSS14 × 10 × ⁄4 1 HSS12 × 8 × ⁄2 5 HSS20 × 12 × ⁄8 3 HSS20 × 12 × ⁄8 5 ⁄ HSS16 × 8 × 16 1.17 1.34 1.41 1.55 1.56 1.82 726 701 > 720 > 1750 > 1350 > 757 (ft) Weight (lb鼻ft) 53.0 39.4 62.5 127.0 78.5 48.9 The HSS14 × 10 × 1⁄4 is the lightest shape. Required s = 2.55 in. At this stage of loading, the test specimen continues to elongate as long as the load is not removed, even though the load cannot be increased. Do not consider the stiffness reduction factor. diameter bolts in each flange, as shown in Figure P3.2-3. To obtain a value of K from one of these nomograms, first calculate the value of G at each end of the column, letting one value be GA and 148 Chapter 4 Compression Members the other be GB. The following geometric constants are required for entry into the table: a e 15.7 = = = 1.3 12 k 8 k= = = 0.67 12 a= By interpolation in Table 8-8 for a = 1.3, C = 1.52 for k = 0.6 and C = 1.73 for k = 0.7 Interpolating between these two values for k = 0.67 gives C = 1.67 For E70XX electrodes, C1 = 1.0. LRFD SOLUTION From Table 8-8, the nominal strength of the connection is given by Rn = CC1D For LRFD, fRn = Pu so Pu = CC1 D φ and the required value of D is D= 60 Pu = = 3.99 sixteenths φCC1 0.75(1.67)(1.0)(12) The required weld size is therefore 3.99 = 0.249 in. Relative bracing is connected to both the point of bracing and to other parts of the frame. The optimum depth of a girder depends on the situation. Satisfactory: Result of interaction equation = 0.974 6.6-5 a. 384 EI s 384(29,000)(301) The total deflection before the concrete has cured is Δ1 + Δ 2 = 1.098 + 0.418 = 1.52 in. “Design of Small Base Plates for Wide Flange Columns.” Engineering Journal, AISC 27 (no. Cochrane (1922) proposed that when deducting the area corresponding to a staggered hole, use a reduced diameter, given by d′ = d − s2 4g (3.2) where d is the hole diameter, s is the stagger, or pitch, of the bolts (spacing in the direction of the load), and g is the gage (transverse spacing). If the member is so slender (we give a precise definition of slenderness shortly) that the stress just before buckling is below the proportional limit—that is, the member is still elastic—the critical buckling load is given by Pcr = π 2 EI L2 (4.1) where E is the modulus of elasticity of the material, I is the moment of inertia of the cross-sectional area with respect to the minor principal axis, and L is the length of the member between points of support. Figure A.6c through e shows each mechanism after being subjected to a virtual displacement. In addition, at A and B, bending moments are transmitted from the horizontal member through the rigid joints. If the member is subject to dead load and live load only, what is the maximum total service load that can be applied if the ratio of live load to dead load is 2.0? Use the alternative value of U from AISC Table D3.1. a. In the effective length and first-order methods, the flexural rigidity is unreduced, and EI*= EI. ST7): 151–197. s From AISC G2.2, the maximum clear spacing is 16 times the web thickness but no greater than 10 inches, or 5 ⎞ 16t w = 16 ⎛ = 5 in. (We covered this limit state in Chapter 5.) For web crippling, when the load is at least half the girder depth from the end, Rn = 0.80 t w2 1.5 ⎡ b ⎞ ⎛ t w ⎞ ⎤ EFyw t f ⎛ ⎟1 + 3 ⎛⎜ ⎞⎟ ⎟ ⎟ d ⎜ t f ⎟ ⎦⎥ tw ⎟ ⎣ (AISC Equation J10-4) When the load is less than this distance from the end of the girder, 1.5 ⎡ b b ⎛ t w ⎞ ⎤ EFyw t f Rn = 0.40 t w2 ⎟1 + 3 ⎛⎜ ⎞⎟ ⎟ ⎟ , for ≤ 0.2 ⎝ d ⎠ ⎝ t f ⎠ ⎦⎥ tw d ⎣ (AISC Equation J10-5a) and 1.5 ⎡ 4 b ⎛ t w ⎞ ⎤ EFyw t f Rn = 0.40t w2 ⎟1 + ⎛⎜ − 0.2⎞⎟ ⎟ ⎟ , for ⎠ ⎝ tf ⎠ ⎦ tw ⎟⎣ ⎝ d ⎠ b > 0.2 d (AISC Equation J10-5b) 10.6 Bearing Stiffeners 681 where d = overall depth of girder t f = thickness of girder flange For LRFD, the resistance factor is f = 0.75. Before discussing structural steel, we need to examine various types of structural members. Additional material on the use of the beam-column tables in the Manual is included. The trusses are welded and spaced at 20 feet. M1 ⎞ Cmy = 0.6 − 0.4 ⎛ = 0.6 − 0.4(0) = 0.6 ⎝ M2 ⎠ Pely = π 2 EI * π 2 (0.8 EI y) π 2 (0.8)(29, 000)(174) = = = 1230 kips (1.0 × 15 × 12)2 (K1 L) 2 (K y L)2 324 Chapter 6 Beam–Columns B1y = Cmy Cmy 0.6 = = 1 − (a Pr 鼻Pe1 y) 1 − (1.00 Pu 鼻Pe1 y) 1 − (300鼻1230) = 0.794 < 1.0 ∴ Use B1 y = 1.0. The required moment strength is Mr = Muy = B1y Mnty + B2y Mty = 1.0(30) + 0 = 30 ft-kips Because the flange of this shape is noncompact (see footnote in the dimensions and properties table), the weak axis bending strength is limited by FLB (see Section 5.15 of this book and Chapter F of the AISC Specification). Compact shapes: Mn = Mp = Fy Z (AISC Equation F8-1) Noncompact shapes: ⎛ 0.021E ⎞ Mn = ⎟ + Fy ⎟ S ⎝ D鼻t ⎠ (AISC Equation F8-2) III. 1.02 5.5-13 a. The external load per bolt, excluding prying force, is T = 80鼻4 = 20 kips. 5): 261. “Composite Beam Criteria in LRFD.” Journal of the Structural Division, ASCE 104 (no. For isolated columns that are not part of a continuous frame, Table C-A-7.1 in the Commentary to Specification Appendix 7 will usually suffice. Estimate the flange size. From Manual Table 3-16a (no-tension-field action), for h鼻tw = 198.4 and a鼻h = 81鼻62 = 1.306, φv Vn = 5 ksi (by interpolation) Aw For Aw = htw = 62(5鼻16) = 19.38, fvVn = Vc1 = 5Aw = 5(19.38) = 96.9 kips From Table 3-16b (tension-field action), for h鼻tw = 198.4 and a鼻h = 1.306, φv Vn = 12.3 ksi (by interpolation) Aw φv Vn = Vc 2 = 12.3 Aw = 12.3(19.38) = 238 kips From Figure 10.18, the larger required strength in the two adjacent panels is Vr = Vu = 223.4 kips. 6. If plastic collapse can be reached, the load-deflection curve will have the appearance of either curve 4 or curve 5. Using the unfactored service loads, a. Determine the nominal strength based on the effective net area. Two approaches exist for the solution of this problem: the traditional elastic analysis and the more accurate (but more complex) ultimate strength analysis. Use an elastic analysis. ∂ 2-in. The nominal strength of the connection is PRn = 2.790(28.86) = 80.52 kips LRFD SOLUTION ANSWER ASD SOLUTION ANSWER The design strength of the connection is 0.75(80.52) = 60.4 kips > 54 kips (OK) The connection is satisfactory by ultimate strength analysis. The deck rib height is 2 inches. Determine which interaction equation to use: pPr = pPu = (1.89 × 10 −3)(300) = 0.567 > 0.2 ∴ Equation 6.99 controls. Ec = Eg = E, the modulus of elasticity of structural steel. Two angles, 5 × 3 × 1⁄2, are oriented with the long legs back-to-back (2L5 × 3 × 1⁄2 LLBB) and separated by 3⁄8 inch. The third possibility, the PNA in the web, is shown in Figure 9.7c. 8 4 8 8 The nominal strength is ⎛ 1⎞ ⎡ ⎛ 7⎞ ⎤ Pn = Fu Ae = Fu t (wg − Σh) = 58 ⎟ ⎟6 − 2 ⎟ ⎟ = 123.3 kips ⎝ 2⎠ ⎣ ⎝ 8⎠ ⎦ Block shear strength: The failure block for the gusset plate has the same dimensions as the block for the tension member except for the thickness (Figure 7.13b). Tributary width for each sag rod = 20鼻2 = 10 ft Tributary area for deck and built-up roof = 10(46.6) = 466 ft2 Dead load (deck and roof) = (2 + 5)(466) = 3262 lb Total purlin weight = 12(10)(9) = 1080 lb Total dead load = 3262 + 1080 = 4342 lb Tributary area for snow load = 10(45) = 450 ft2 Total snow load = 18(450) = 8100 lb 82 Chapter 3 Tension Members FIGURE 3.32 LRFD SOLUTION Check load combinations. Satisfactory: Result of interaction equation = 0.970 b. If Lb ≤ Lp , there is no LTB, and Mn = Mp If Lp < Lb ≤ Lr , there is inelastic LTB, and ⎛ Lb − L p ⎞ ⎤ ⎡ Mn = Cb ⎟ Mp − (Mp − 0.7 Fy S x) ⎟ ⎟ ≤ Mp ⎝ Lr − L p ⎠⎦ ⎣ If Lb > Lr , there is elastic LTB, and Mn = Fcr Sx ≤ Mp where Fcr = Cb p 2 E Jc 2 1 + 0.078 S x h0 (Lb 鼻rts) ⎛ Lb ⎞ ⎟ ⎝ ⎠ rts 2 3. < 36 in. Problems 717 10.7-7 A 70-foot long plate girder must support a uniformly distributed load and concentrated loads at the one-third points. ⎝ 16 ⎠ Try 3⁄16-in. The typical design problem is to select a member with sufficient cross-sectional area to resist the loads. For Cb = 1.32, Mn = Cb × 131 = 1.32(131) = 172.9 ft-kips Ωb This is larger than Mp 鼻Ωb = 151 ft-kips, so the allowable moment must be limited to Mp 鼻Ωb. This type of analysis suffers from some of the same shortcomings as the elastic method for eccentric bolted connections, including the assumption of a linear load-deformation relationship for the weld. Simpler methods are sometimes used in which the effects of the earthquake are simulated by a system of horizontal loads, similar to those resulting from wind pressure, acting at each floor level of the building. For large axial load, the bending term is slightly reduced. The ratio is closer to 3.10 = 0.6526 4.75 This corresponds to a required ft Pn (based on rupture) of 0.75 0.75 × Pu = (154) = 177 kips actual ratio 0.6526 Try an L8 × 4 × 1⁄2, with ftPn = 188 kips (based on yielding) and ft Pn = 189 Kips (based on rupture strength, with Ae = 0.75Ag = 4.31 in.2). If they are not used, kv will be equal to 5, and h鼻tw can be made small enough (by using a thick enough web) so that Vu ≤ fVn. If stiffeners are used, they can be positioned so that the shear strength is based on either (1) yielding or buckling (no tension-field action), or (2) buckling plus post-buckling (tension-field action) shear strengths. 2.31 in.2 e. SOLUTION Even though a load may not be acting directly on a member, it can still cause a load effect in the member. SOLUTION The total vertical load to be stabilized by the bracing is Pr = ΣPu = 0.9(45) × 3 = 121.5 kips 352 Chapter 6 Beam–Columns wu = 0.9ft-k FIGURE 6.23 18′ 45′ From AISC Equation A-6-1, the lateral shear to be resisted is Prb = 0.004Pr = 0.004(121.5) = 0.4860 kips If we consider both braces to be tension-only members, then the entire force must be resisted by one brace. 472 Chapter 7 Simple Connections 13″ MC8 × 8.5 t = 3⁄8″ FIGURE P7.11-1 7.11-2 Determine the maximum service load that can be applied if the live load-to-dead load ratio is 2.0. Investigate all limit states. This topic is not covered explicitly in AISC Chapter D (“Design of Members for Tension”), but the introductory user note directs you to Chapter J (“Design of Connections”), Section J4.3, “Block Shear Strength.” The model used in the AISC Specification assumes that failure occurs by rupture (fracture) on the shear area and rupture on the tension area. flange local buckling (FLB), elastically or inelastically; or 3. ⎝ 2⎠ The direct shear to be resisted by one angle is Ru 489.9 = = 7.48 kips鼻in. Thus there will be a residual strain, or permanent set, after unloading. Subsequent examples will illustrate this procedure. If the compressive load is applied to both flanges, check compression buckling of the web [AISC Equation J10-8].) In addition to these tension and compression loadings, shear in the column web (panel zone shear) must be considered. Try 21⁄2 in. Swanson, J. All of the loads mentioned thus far are forces resulting from gravity and are referred to as gravity loads. 9.1-3 A W21 × 57 floor beam supports a 6-inch-thick reinforced concrete slab with an effective width b of 108 inches. Use the basic weld strength, Fnw = 0.6FEXX, for both the longitudinal and the transverse welds: Rn = Rnw + Rnwt (AISC Equation J2-10a) where Rnw and Rnwt are the strengths of the longitudinal and transverse welds, both calculated with Fnw = 0.6FEXX. The required size is therefore w= 4 1 = in. The direct analysis method is a second-order analysis that considers both P-d and P-Δ effects. The steel is A992. Bending about the major axis (loaded in the plane of symmetry): i. Assume that the bottom chord connection is made with 9-inch-long longitudinal welds at the flange. He is a licensed professional engineer in Tennessee. 452 Chapter 7 Simple Connections The shear yield strength of the base metal is ⎛ 3⎞ 0.6 Fy t = 0.6(36) ⎟ ⎟ = 8.1 kips鼻in. Try one stud every 3rd rib: Spacing = 3(6) = 18 in. Ordinary structural analysis methods that do not take the displaced geometry into account are called first-order methods. 8.6 Moment-Resisting Connections 533 (The beam web has a thickness of tw = 0.380 in. From Equation 8.6, the resisting moment is M = nrt d = 4rt (3 - 1.5 + 3 + 3 -1.5) = 24rt The applied moment is Mu = Pue = 88(2.75) = 242 in.-kips Equating the resisting and applied moments, we get 24rt = 242, or rt = 10.08 kips The factored load shear stress is frv = 11 = 24.90 ksi 0.4418 and from AISC Equation J3-3a, the nominal tensile stress is Fnt frv ≤ Fnt φ Fnv 90 (24.90) = 61.67 ksi < 90 ksi = 1.3(90) − 0.75(54) Fnt′ = 1.3Fnt − 2r t FIGURE 8.15 3″ 2r t 3″ 2r t 3″ 2r t 8.3 Eccentric Bolted Connections: Shear Plus Tension 495 The design tensile strength is fRn = 0.75F′nt Ab = 0.75(61.67) (0.4418) = 20.4 kips > 10.08 kips ANSWER ASD SOLUTION (OK) The connection is satisfactory. Use A992 steel for the beam, A36 for the plate, E70XX electrodes, and fully tensioned Group A bolts. In general, the spacing of fasteners or welds should not exceed 24 times the thickness of the thinner plate, or 12 inches. 3): 136–147. 33.9 kips 8.8-1 a. The only type of composite behavior possible is that between the structural steel beam and the longitudinal reinforcing steel in the slab. 2 2 From Table 3-19 in the Manual, any combination of steel shape, ΣQn, and Y2 that furnishes a design strength of more than 214 ft-kips will be an acceptable trial beam. With LRFD, the nominal strengths are multiplied by resistance factors. The moment Mnt in member AB is computed by using only the gravity loads. One type of bolt is installed with a special wrench that twists off the end when the correct amount of tension has been reached. LRFD SOLUTION From Example 3.12, Pu = 154 kips rmin ≥ 0.600 in. At the same time, the yielded region will extend longitudinally from the center of the beam as the bending moment reaches My at more locations. The minimum length of the connection can be established by using the minimum permissible spacing and edge distances in the longitudinal direction. Plot these data points, and draw a best-fit straight line through them. Although smaller, lighter rolled steel beams can be used with composite construction, this advantage will sometimes be offset by the additional cost of the studs. The area of concrete to be used is Ac = C 515 = = 10.3 in.2 Fy 50 The distance from the top of the steel shape to the centroid of the concrete is Y2 = t − a 1.683 = 4.5 − = 3.659 in. Typical cross sections are shown in Figure 1.9. Only relatively thin material can be used, and the resulting shapes are suitable only for light applications. The moment and rotation for this combination of beam and connection are defined by the intersection of the moment-rotation curve and the beam line. Use LRFD and A572 Grade 50 steel. The base-metal shear strength expression can also be simplified. The yield stress for steel with a stress–strain curve of the type shown in Figure 1.5 is called the yield strength and is defined as the stress at the point of unloading that corresponds to a permanent strain of some arbitrarily defined amount. The total load is Pa = D + L = 18 + 54 = 72 kips We will select the number of bolts based on shear and verify that the bearing strength is adequate once a final bolt layout has been determined. The boundary conditions will be different from those in the original derivation, but the overall procedure will be the same. 6″ PL 1⁄4 × 6 A36 steel Weld FIGURE P3.3-4 3.3-5 A W16 × 45 of A992 steel is connected to a plate at each flange as shown in Figure P3.3-5. Design Guide 4 can be used for both static and seismic loads, but Design Guide 16 is restricted to static loading (which includes wind and low seismic loading). If l is greater than the specified limit (denoted lr), the shape is slender. They have no legal standing on their own, but by presenting design 8 Chapter 1 Introduction criteria and limits in the form of legal mandates and prohibitions, they can easily be adopted, by reference, as part of a building code. For flexure (shear will be covered later), the required and available strengths are moments. 458 Chapter 7 Simple Connections FIGURE 7.47 LRFD SOLUTION The design strength of the member based on its gross area is 1 φt Pn = 0.90 Fy Ag = 0.90(36)⎛ ⎞ (8) = 129.6 kips ⎝ 2⎠ Compute the design strength of the member based on its effective area. This behavior can be likened to that of a Pratt truss, in which the vertical web members carry compression and the diagonals carry tension, as shown in Figure 10.2b. EXAMPLE 5.15 Use the load table given in Figure 5.35 to select an open-web steel joist for the following floor system and loads: Joist spacing = 3 ft 0 in. What is the force in the member? ⎛ Muy ⎞ 8 4.185 Pu 8 ⎛ Mux + + = 0.6040 + +0 fc Pn 9 ⎟⎝ f b M nx fb M ny ⎟⎠ 9 11.64 = 0.924 < 1.0 ANSWER) (OK) Use a WT6 × 17.5. Problems Note Unless otherwise indicated, all members are laterally braced only at their ends and Ky = 1.0. Interaction Formulas 6.2-1 Determine whether the given member satisfies the appropriate AISC interaction equation. The AISC Specification gives equations for the strength of both stud and channel anchors. CHAPTER 7 Simple Connections 7.1 INTRODUCTION Connections of structural steel members are of critical importance. When a connection is made with a combination of welds and bolts, welding can be done in the shop and bolting in the field. Check it for compliance with the AISC Specification. and Ravindra, M. FIGURE 5.6 8″ 1″ x 1′-0″ 1 ⁄ 2 ″ 1″ SOLUTION TABLE 5.1 a. SOLUTION Values needed for the AISC strength equations are as follows. The bearing problem can be complicated by the presence of a nearby bolt or the proximity of an edge in the direction of the load, as shown in Figure 7.6. The bolt spacing and edge distance will have an effect on the bearing strength. > 0.5 in. This maximum spacing will apply for the remaining distance to the centerline of the girder. 4.4-2 A W21 × 101 is used as a compression member with one end fixed and the other end free. As discussed earlier, the resistance to slip will be a function of the coefficient of static friction and the normal force between the connected parts. The beam is to be supported on reinforced concrete walls with f c′ = 3500 psi. For example, the types of structural steel authorized are listed in Chapter A, “General Provisions,” under Section A3, “Material,” and, under it, Section 1, “Structural Steel Materials.” The main body of the Specification is followed by appendixes 1–8. This material was in the form of wrought iron, produced by heating ore in a charcoal fire. Bracing to stabilize a structure against vertical loading is called stability bracing. 6.6-2 Compute the moment amplification factor B1 for the member of Problem 6.2-2. The AISC Specification reduces the slip-critical strength for this case. For shapes that are symmetrical about the axis of bending, the elastic and plastic neutral axes are the same. Try the layout shown in Figure 8.39 and determine the plate thickness required for bearing. This condition can be visualized by considering two planks used as a beam, as shown in Figure 4.21. The strength of this composite section should be based on a plastic stress distribution, in which the horizontal force to be transferred between the steel shape and the reinforcing steel should be taken as the smaller of AsrFysr and ΣQn , where Asr = area of reinforcing steel within the effective width of the slab Fysr = yield stress of the reinforcing steel If this composite section is used, the resistance factor fb for LRFD is 0.90, and the safety factor Ωb for ASD is 1.67. If the modulus of elasticity is 29,000 ksi, determine the following. Therefore, the unbraced length, Lb, is 30鼻2 = 15 feet. Therefore, the amplification factor should be written as 1 1 − (Pu 鼻Pe) (6.7) where Pu is the factored axial load. 1): 11–26. Use A572 Grade 50 steel, a total depth of 7 feet, and LRFD. < 0.863 in. Example 3.9 illustrates the case of staggered holes in different elements of an S-shape. MANUAL. Chord members are usually fabricated as continuous pieces and spliced if necessary. With relative bracing, both the brace and other members contribute to stabilizing the member to be braced. Use bearing-type bolts. Live loads, which can also be gravity loads, are those that are not as permanent as dead loads. If the thickness originally used in the estimate of the web depth is retained, no adjustment in the web depth will be needed. VI. A more precise measure is the slenderness ratio, L/r, where L is the member length and r is the minimum radius of gyration of the cross-sectional area. Instead, composite behavior is achieved by connecting the steel beam to the reinforced concrete slab it supports, causing the two parts to act as a unit. The beam is 40 feet long, and the concentrated load is located 15 feet from the left end. ANSWER Use 3⁄16-in. The stress as given by this equation is exact, provided that the cross section under consideration is not adjacent to the point of application of the load, where the distribution of stress is not uniform. Lateral bracing is provided at the ends and at midspan. 2 Ae = AnU = 3.836(0.80) = 3.069 in.2 and the allowable strength based on rupture of the net section is Pn Fu Ae 58(3.069) = = = 89.0 kips < 105 kips Ωt Ωt 2.00 (N.G G.) This shape did not work because the ratio of actual effective net area Ae to gross area Ag is not equal to 0.75. Then D= Fyst Ast 鼻Ωst 36(1.88)鼻1.67 = = 3.06 sixteenths 0.9279 L (1.5) 0.9279 [(3 − 5鼻8) × 2 × 2] (1.5) Minimum size = ANSWER 3 in. ΣQn = 15(17.23) = 258.5 kips < 384.0 kips ◆ C = V′ = 258.5 kips. Thus, local stability is assumed, and width-thickness ratio limits must not be exceeded. Because Prb is the horizontal component of the brace force (see Figure 6.24a), the brace force is F= Prb cos θ where ⎛ 18 ⎞ θ = tan −1 ⎟ ⎟ = 21.80° ⎝ 45 ⎠ FIGURE 6.24 Pbr θ F (a) Δ θ δ (b) 6.8 Design of Beam–Columns 353 Then, F= 0.4860 = 0.5234 kips cos(21.80) Based on the limit state of tension yielding, the required area is A= F 0.5234 = = 0.0162 in.2 0.9 Fy 0.9(36) From AISC Equation A-6-2, the required lateral stiffness is bbr = 1 ⎛ 2 Pr ⎞ 1 ⎡ 2(121.5) ⎤ = 1. In slip-critical connections subject to both shear and tension, interaction of shear and tension need not be investigated. Their use is explained in the Manual and is not covered here. The member is 15 feet long. The service dead load is 35.5 kips, and the service live load is 142 kips. Rely on the strength of the steel shape only. 3rd ed. 1): 41–46. The members of this frame are rigidly connected by welding and can be assumed to form a continuous structure. 1997b. To estimate the magnitude of the load required 7.8 High-Strength Bolts in Tension 419 FIGURE 7.26 to overcome completely the clamping effect and cause the parts to separate, consider the free-body diagram in Figure 7.26. Our coverage of beams is restricted to those that are part of a floor or roof system. Csernak The Instructor Solutions manual is available. fRn = 126 kips, Rn鼻Ω = 83.8 kips 3.8-3 8.69 kips at peak, 7.97 kips elsewhere b. If the beam is shorter than anticipated, the gap between the column and the beam flange may cause difficulties in welding, even when a backing bar is used. 2): 39–49. • If the required moment strength is less than the full moment capacity of the beam (in a nonseismic connection), design for at least 60% of the beam strength. If the beam is compact and has continuous lateral support, or 200 Chapter 5 Beams If the unbraced length is very short, the nominal moment strength, Mn , is the full plastic moment capacity of the shape, Mp. For members with inadequate lateral support, the moment resistance is limited by the lateral-torsional buckling strength, either inelastic or elastic. However, if the search is made based on available strength (fbMp or Mp鼻Ωb) rather than section modulus, it is irrelevant whether the shape is compact or noncompact. and the shear rupture strength is fRn = 0.45Futst × 2 = 0.45(58)(3鼻4) × 2 = 39.15 kips鼻in. From AISC Equation I2-13, As ⎞ ≤ 0.9 C3 = 0.6 + 2 ⎛⎜ ⎝ Ac + As ⎟⎠ 2.51 ⎞ = 0.7304 < 0.9 = 0.6 + 2 ⎛ ⎝ 35.98 + 2.51 ⎠ and from AISC Equation I2-12, (EI)eff = Es I s + Es I sr +C3 Ec I c = 29,000(14.9) + 0 + 0.7304(3904) (103.0) = 7. Frequently, the most economical alternative is to use a column section with a thicker web. This is to be taken as 1.13 unless another factor can be justified. Assuming that the moment-resisting couple is formed by these forces and the compressive force in the bottom beam flange, Mu = f (2 Pt h0 + 2 Pt h1) = f 2 Pt (h0 + h1) = f 2 Ft Ab (h0 + h1) ⎛ p d2 ⎞ Mu = f 2 Ft ⎟ b ⎟ (h0 + h1) ⎝ 4 ⎠ (8.10) 562 Chapter 8 FIGURE 8.52 Eccentric Connections 2Pt 2Pt h1 h0 Mn where Mu = required moment strength h0 = distance from center of beam compression flange to center of outer row of bolts on the tension side h1 = distance from center of compression flange to center of inner row of bolts f = 0.75 Equation 8.10 can be solved for the required bolt diameter. Appendix 6 of the AISC Specification, “Stability Bracing for Columns and Beams,” covers this type of bracing. 1.28 in., 1.73 in. A perfectly moment-free connection would be represented by the horizontal axis. A36 steel is used for the angles, and A992 steel is used for the W shapes. In this book, we will use the lower-bound moment of inertia to compute deflections in composite beams. Available compressive strength tables are also provided for single-angle members. From Equation 6.8, EI* = 0.8tb EI = 0.8(1.0)EI = 0.8EI Pe1x = π 2 EI * π 2 (0.8 EI x) π 2 (0.8)(29, 000)(533) = = = 3767 kips (1.0 × 15 × 12)2 (K1 L) 2 (K x L)2 0.6 Cmx Cmx = = 1 − (a Pr 鼻Pe1x) 1 − (1.60 Pa 鼻Pe1x) 1 − (1.60 × 200鼻3767) = 0.656 < 1.0 ∴ Use B1x = 1.0. B1x = Mr = Max = B1xMntx + B2xMtx = 1.0(90) + 0 = 90 ft-kips From the Beam Design Charts with Cb = 1.0 and Lb = 15 feet, the moment strength is M px M nx = 226 ft-kips and = 237 ft-kips Ωb Ωb From Figure 5.15g, Cb = 1.67 and ⎛M ⎞ Cb × ⎟ nx for Cb = 1.0 ⎟ = 1.67(226) = 377.4 ft-kips ⎝ Ωb ⎠ This result is larger than M px M px M ; therefore, use nx = = 237 ft-kips. 2. 1880 ft-kips 9.7-3 278 ft-kips 9.8-1 278 ft-kips 8.4-11 8.4-13 b. 9.8-2 A simply supported, uniformly loaded composite beam consists of a W16 × 36 of A992 steel and a reinforced concrete slab with f c′ = 4 ksi. 36◆◆ 40◆ 40◆◆ All plates 4◆◆ thick FIGURE P4.9-5 Problems 4.9-6 185 Compute the axial compressive design strength based on flexural buckling (no torsional or flexural-torsional buckling). EXAMPLE 6.8 A W12 × 40 has a span length of 15 feet. The connection is not near the end of the column. Determine the required number of 3⁄4-inchdiameter Group A bolts for the following cases: (a) a bearing-type connection with threads in the plane of shear and (b) a slip-critical connection with threads in the plane of shear. From AISC Equation J3-4, the slip-critical strength is Rn = mDu hfTb ns From AISC Table J3.1, the prescribed tension for a 7⁄8-inch-diameter Group A bolt is Tb = 39 kips If we assume Class A surfaces, the slip coefficient is m = 0.30, and for four bolts, Rn = μ Du h f Tb ns × 4 = 0.30(1.13)(1.0)(39)(1) × 4 = 52.88 kiips Rn 52.88 = = 35.25 kips Ω 1.50 Since there is a tensile load on the bolts, the slip-critical strength must be reduced by a factor of ksc = 1 − 1.5Ta 1.5(48) =1− = 0.5916 Du Tb nb 1.13(39)(4) The reduced strength is therefore ksc(35.25) = 0.5916(35.25) = 20.9 kips < 36 kips ANSWER (N.G.) The connection is inadequate as a slip-critical connection. Typical roof construction with trusses supported by load-bearing walls is illustrated in Figure 3.28. For allowable strength design (ASD), Equation 2.7 can be written as Mn Ωb Ma ≤ (5.2) where Ma = required moment strength = maximum moment corresponding to the controlling load combination from ASCE 7 Ωb = safety factor for bending = 1.67 Equation 5.2 can also be written as Ma ≤ Mn = 0.6 M n 1.67 Dividing both sides by the elastic section modulus S (which will be reviewed in the next section), we get an equation for allowable stress design: M a 0.6 M n ≤ S S or fb ≤ Fb where fb = maximum computed bending stress Fb = allowable bending stress 5.2 BENDING STRESS AND THE PLASTIC MOMENT To be able to determine the nominal moment strength Mn , we must first examine the behavior of beams throughout the full range of loading, from very small loads to the point of collapse. 1): 12–28. Multistory buildings can consist of a combination of braced frames and moment frames. If tension-field action is not permitted, Vn = 0.6Fy AwCv Note that there is no requirement that tension-field behavior must be used, although its use will result in a more economical design. Equation 2.3 can therefore be written as ∑ (loads × load factors) ≤ resistance × resistance factor (2.4) The factored load is a failure load greater than the total actual service load, so the load factors are usually greater than unity. The required weld size is therefore w= 10.13 φ Rn = = 0.455 in. The most common built-up configuration is probably the double-angle section, shown in Figure 3.1, along with other typical cross sections. 4 40 k W10 × 68 20 k 6′ 3 20 k 40 k 6′ 12′ Section FIGURE P6.6-12 Problems 6.6-13 369 The fixed-end member shown in Figure P6.6-13 is a W18 × 71 of A992 steel. Strictly speaking, any structural member formed with two or more materials is composite. The nominal strength for this limit state is Rn = 24t w3 EFyw

h (AISC Equation J10-8) where h = column web depth from toe of fillet to toe of fillet (Figure 8.44). Length of longitudinal welds = 2 Total required length of weld = For the second option, the strength of the longitudinal welds is 0.85(3.712) = 3.155 kips兾in. Examination of AISC Equations E3-2 and E3-3 shows that the theoretically maximum value of Fcr is the yield stress Fy. 2. Check the capacity based on the limit state of flange local buckling: M p = Fy Z x = 50(157) = 7850 in.-kips ⎛ l − lp ⎞ M n = M p − (M p − 0.7 Fy S x) ⎜ ⎟ lr − l p ⎠ ⎝ = 7850 − (7850 − 0.7 × 50 × 143) (1024..21 −− 99.1.1155) = 7650 in.-kips = 637.5 ft-kips Check the capacity based on the limit state of lateral-torsional buckling. The capacity is 35.4 kips, based on buckling about the y axis, with an effective length of 14 feet. From AISC Equation J8-2, Pp = 0.85 fc′ A1 A2 ≤ 1.7 fc′ A1 A1 For fcPp ≥ Pu , ⎡ 18(18) ⎤ 0.65 ⎢ 0.85(3) A1 ≥ 349.6 A1 ⎣⎦ ⎣ A1 ≥ 137.3 in.2 Check upper limit: fc1.7f c′ A1 = 0.65(1.7)(3)(137.3) = 455 kips > 349.6 kips Also, the plate must be at least as large as the column, so bf d = 10.0(10.0) = 100 in.2 < 137.3 in.2 (OK) For B = N = 13 in., A1 provided = 13(13) = 169 in.2 The dimensions of the cantilever strips are N − 0.95d 13 − 0.95(10) = = 1.75 in. This is consistent with the provisions of the International Building Code (ICC, 2009). No longitudinal reinforcement is used. Using these directions and relative magnitudes as a guide and bearing in mind that forces add by the parallelogram law, we can conclude that the lower right-hand fastener will have the largest resultant force. For the gross section, Ft = 0.6 Fy = 0.6(36) = 21.6 ksi The corresponding allowable load is Ft Ag = 21.6(2.41) = 52.06 kips For the net section, Ft = 0.5Fu = 0.5(58) = 29 ksi The corresponding allowable load is Ft Ae = 29(1.514) = 43.91 kips ANSWER 3.3 Because 43.91 kips < 52.06 kips, fracture of the net section controls, and the allowable strength for the two angles is 2 × 43.91 = 87.8 kips. 2 2 Smallest net area = 3.31 in.2 FIGURE 3.18 4 @ 2″ a 1 1⁄2″ b 3″ c 1 1⁄2″ C6 × 13 d e When staggered holes are present in shapes other than angles, and the holes are in different elements of the cross section, the shape can still be visualized as a plate, even if it is an I-shape. The location of the neutral axis can be found by applying the principle of moments with the axis of moments at the top of the slab. FIGURE P3.8-2 3.8-3 Compute the factored joint loads for the truss of Problem 3.8-2 for the following conditions. They are presented here in a slightly different format. An = Ag − Aholes = 5.86 − 2 (43 + 81)(85) = 4.766 in. Five separate beams are represented on this graph of load versus central deflection. E (10.3) 680 Chapter 10 Plate Girders The minimum intermittent fillet weld will likely be adequate (Salmon and Johnson, 1996). If an end panel is being considered, or if intermediate stiffeners are not used, the shear strength can be found from AISC Equation G2-1, which gives the strength in the absence of a tension field. Part 6 of the Manual, “Design of Members Subject to Combined Loading,” contains tables based on design aids developed by Aminmansour. There is a construction load of 20 psf, a partition load of 20 psf, and a live load of 120 psf. The characteristics of steel that are of the most interest to structural engineers can be examined by plotting the results of a tensile test. 732 References Chesson, Jr., Eugene, Faustino, Norberto L., and Munse, William H. 2 2 2 2 a= The allowable flexural strength is M n Cy 384(10.72) = = = 2465 in.-kips = 205 ft-kips > 187 ft-kips Ωb Ωb 1.67 (OK) Check shear: Va = wa L 1.664(30) = = 25.0 kips 2 2 From the Zx table, Vn = 70.5 kips > 25.0 kips Ωv ANSWER (OK) Use a W16 × 26. 2 2 2. 29, 000(375) 384 EI x 384 (N..G.) Solve the deflection equation for the required moment of inertia: I required = 5wL L4 5(0.700兾12)(30 × 12)4 = = 440 in.4 384 EΔ required 384(29, 000)(1.0) Part 3 of the Manual contains selection tables for both Ix and Iy. From the Ix table, try a W18 × 35: I x = 510 in.4 > 440 in.2 (OK) Mn = 166 ft-kips > 122 ft-kips Ωb Vn = 106 kips > 16.2 kips Ωv ANSWER (OK) (OK) Use a W18 × 35. 0.0661M 9.1-5 304 ft-kips 9.2-1 a. For the member to be concentrically loaded at its ends, the resultant resisting force furnished by the connection must also act along this axis. These computations are shown in Table 4.3, which is an expanded version of Table 4.2. The moment of inertia about the x-axis is Ix = 1193 in.4 164 Chapter 4 TABLE 4.3 Compression Members A y Ay I¯ d I¯ + Ad 2 Plate W 1.500 19.10 0.1875 9.575 0.2813 182.9 0.01758 1070 8.706 0.6820 113.7 1079 Σ 20.60 Component 183.2 For the vertical axis, Iy = 1 ⎛ 3⎞ 3 (4) + 54.8 = 56.80 in.4 12 ⎜⎝ 8 ⎟⎠ Since I y < I x , the y-axis controls. In this book we record intermediate values to three or four digits (usually four), depending on the circumstances, and record final results to three digits. In an actual design situation, the spacing and edge distances would be selected once the required number of bolts was determined, and then the bearing strength could be checked. 8.7 Column Stiffeners and Other Reinforcement 547 The minimum width is b≥ bb t w 6.02 0.360 − = − = 1.83 in. The maximum axial compressive force will occur in member DE (and in the adjacent member, to the right of the center of the span) and can be found by considering the equilibrium of a free body of the portion of the truss to the left of section a–a: ∑ M I = (19.2 − 2.4)(30) − 4.8(10 + 20) + FDE (4) = 0 FDE = −90 kips (compression) Design for an axial load of 90 kips and a bending moment of 3.0 ft-kips. 1.05 6.6-3 a. From Figure A.1b, the moment causing first yield can be written as M y = Fy S x and 718 M p Fy Z x Z x = = M y Fy S x Sx A.1 FIGURE A.1 FIGURE A.2 Introduction 719 720 Appendix Plastic Analysis and Design This ratio is a constant for a given cross-sectional shape and is called the shape factor. ∑ A 65.00 Since this is less than 5 inches (the thickness of the slab) the neutral axis lies within the slab. Continue the process until a shape is found that gives an interaction equation result less than 1.0 and close to 1.0 (greater than 0.9). The steel and concrete strengths are Fy = 50 ksi and fc⬘ = 4 ksi. We cover beams in Chapter 5.) For I shapes, the projecting flange is considered to be an unstiffened element, and its width can be taken as half of the full nominal width. Another drawback is the requirement for a structural frame analysis that incorporates the partial joint restraint. Several approximate methods have been proposed to account for the effects of staggered holes. t = 3⁄8″ 2L5 × 5 × 5⁄16 D = 20 k L = 50 k FIGURE P7.11-5 7.11-6 Design a welded connection for the conditions of Problem 7.4-3. Design the girder cross section. 21.26 ⎣⎦ The total loss in area is 2(b − be)t = 2(7.652 − 6.477)(0.116) = 0.2726 in.2 and the reduced area is Ae = 2.70 − 0.2726 = 2.427 in.2 4.4 Local Stability The reduction factor is Qa = Ae 2.427 = = 0.8989 Ag 2.70 Q = QsQa = 1.0(0.8989) = 0.8989 Compute the local buckling strength. There are two possible failure mechanisms for this beam. The relationship between the nominal strength, Mn, and the unbraced length is shown in Figure 5.13. I would appreciate learning of any errors that you may find. From mid-depth of the beam to the inside face of the compression flange b. 2″ 3″ 3″ 2″ t = 3⁄8″ 2″ 3″ 3″ 2″ 8″ t = 3⁄8″ 10 k FIGURE P8.2-9 8.2-10 A plate is connected to the web of a channel as shown in Figure P8.2-10. Compression buckling of the column web must be investigated when the loads are delivered to both column flanges. The given load is a service load. (See Figure 8.48.) 8 548 Chapter 8 Eccentric Connections FIGURE 8.48 The required weld size is therefore D= 3 46.5 = 2.16 sixteenths < in. Total number = (OK) 30(12) = 20 18 For N1 = 20兾2 = 10, ΣQn = 10(17.23) = 172.3 kips. The critical load is sometimes referred to as the Euler load or the Euler buckling load. The moment of inertia of the cross section can be found by using the parallel axis theorem, and the results of the calculations are summarized in Table 5.1. Component Flange Flange Web Sum I¯ A d I¯ + Ad 2 0.6667 0.6667 72 8 8 — 6.5 6.5 — 338.7 338.7 72.0 749.4 The elastic section modulus is S = I 749.4 749.4 = = = 107 in.3 c 1 + (12 / 2) 7 and the yield moment is My = Fy S = 50(107) = 5350 in.-kips = 446 ft-kips 5.2 ANSWER Bending Stress and the Plastic Moment 195 S = 107 in.3 and My = 446 ft-kips. Figure 8.50 illustrates two categories of the beam-to-column connection: the simple (or shear only) connection (Type PR construction) and the rigid (moment-resisting) connection (Type FR construction). The total loss in area is therefore 2(b − be)t = 2(7.652 − 4.784)(0.116) = 0.6654 in.2 and the reduced area is Ae = 2.70 − 0.6654 = 2.035 in.2 The reduction factor is Qa = Ae 2.035 = = 0.7537 Ag 2.70 Q = QsQa = 1.0(0.7537) = 0.7537 Compute the local buckling strength: 4.71 29, 000 E = 4.71 = 136.2 0.7537(46) QFy KL = 105.3 < 136.2 r ∴ Use AISC Equation E7-2 QFy 0.7537(46) ⎛ ⎞ ⎛ ⎞ Fcr = Q⎢ 0.658 Fe ⎥ Fy = 0.7537⎢ 0.658 25.81 ⎥ 46 = 19.76 ksi ⎝ ⎠ ⎝ ⎠ Pn = Fcr Ag = 19.76(2.70) = 53.35 kips Since this is less than the flexural buckling strength of 58.91 kips, local buckling controls. Next, determine whether this shape is compact, noncompact, or slender: bf = 6.97 2t f 0.38 (from Part 1 of the Manual) 29, 000 E = 0.38 = 9.15 > 6.97 50 Fy ∴ The flange is compactt. For a W14 × 68, it is given as 13.3 in.) For a doubly-symmetric I shape, c = 1.0. From AISC Equation F2-6, Lr = 1.95rts E 0.7 Fy 2 ( ) 0.7 Fy Jc ⎛ Jc ⎞ + ⎜ + 6.76 ⎟ ⎝ S x ho ⎠ S x ho E 2 3.01(1.0) Jc = = 0.002201 S x ho 103(13.28) Lr = 1.95(22.799) 29, 000 ⎡ 0.7(50) ⎤ 0.002201 + (0.002201)2 + 6.76 ⎢ 0.7(50) ⎣ 29, 000 ⎦⎥ = 351.3 in. The lightest W14 that is a possibility is a W14 × 53, with p = 2.96 × 10−3 and bx = 3.50 × 10−3. Use a 3⁄16-inch fillet weld for the stiffener-to-column web welds. Bending Strength of Compact Shapes 5.5-1 The beam shown in Figure P5.5-1 is a W14 × 61 of A992 steel and has continuous lateral support. Use the basic weld strength for both the longitudinal and transverse welds. Sufficient steel anchors are provided for full composite action. FIGURE 3.25 D = 35k L = 70k⁄ 72 Chapter 3 LRFD SOLUTION Tension Members The factored load is Pu = 1.2 D + 1.6 L = 1.2(35) + 1.6(70) = 154 kips Requireed Ag = Pu 154 = = 4.75 in.2 ft Fy 0.90(36) Required Ae = Pu 154 = = 3.54 in.2 F 0 . The bay size, such as 30 ft × 40 ft, is a measure of the geometry of a building. This moment can be caused by lateral loads or by unbalanced gravity loads. 2010. North American Specification for the Design of Cold-Formed Steel Structural Members. Although the cross section is reduced during loading (the Poisson effect), the original cross-sectional area is used to compute all stresses. In some situations, the use of shoring will enable a smaller steel shape to be used. • Minimum slab thickness above the top of the deck = 2 inches. 2 2 7兾8 d = 3.469 in. Yes: For end panel, Vu = 95.4 kips < f vVn = 98.2 kips; For second panel, Vu = 74.0 kips < f vVn = 352 kips; For middle panel, Vu = 51.2 kips < f vVn = 54.7 kips b. 6 Fcr ≥ Ag ≥ Pa Ag Pa 0 . The length is 8 feet. The net moment is of concern, so if beams are connected to both sides of the column, the algebraic sum of the moments induces this web shear. For full composite behavior, the compressive force in the concrete at ultimate (equal to the horizontal shear at the interface between the concrete and steel) will be the smaller of As Fy = 7.68(50) = 384 kips or 0.85 fc′Ac = 0.85(4)(90)(3.5) = 1071 kips Use C = V ′ = 384 kips. To determine the web thickness, first examine the limiting values of h兾tw. Multiply the working loads (service loads) by the load factor to obtain the failure loads. (The gage length is the distance between two marks placed along the length of the specimen.) a. The concrete strength is f c′ = 3000 psi. The spacing of trusses in the roof system is 15 feet. Solving for Ru , we get Ru = 489.9 kips Check the shear capacity of the base metal (the angle controls). End-loaded welds are longitudinal welds at the end of an axially loaded member. If the factored column shear adjacent to the connection is Vu = 0, and Pu 兾Py = 0.6, determine whether panel zone reinforcement is required. This curve is typical of a class of steel known as ductile, or mild, steel. The connection in Figure 8.31 is the same as the one in Figure 8.30, except that the beam flanges are welded to the column flange. Load combinations for ASD are also given in ASCE 7. Excessive deflection is usually an indication of a very flexible beam, which can lead to problems with vibrations. Ponding is one deflection problem that does affect the safety of a structure. American Iron and Steel Institute. The minimum edge distance in any direction is 11⁄8 in. Bolt diameters typically range from 1⁄2 inch to 11⁄2 inches in 1⁄8-inch increments. The most common type of compression member occurring in buildings and bridges is the column, a vertical member whose primary function is to support vertical loads. Strength of Filled Composite Columns The compressive strength of concrete-filled hollow sections is based on the limit state of flexural buckling, just as the strength of encased sections is. Recording too many significant digits is misleading and can imply an unrealistic degree of precision. At a distance of 36 inches from the left end, the shear is ⎛ 36 ⎞ Vu = 223.4 − 4.34 ⎜ ⎟ = 210.4 kips ⎝ 12 ⎠ Required φvVn 210.4 = = 10.86 ksi Aw 19.38 Tension-field action can be used outside the end panels, so the curves of AISC Table 3-16b will be used. 2 2 From Table 3-19 in the Manual, by interpolation, Mn = 158 ft-kips > 143 ft-kips Ωb (OK) Adjust Ma for the beam weight: wa = 1.272 + 0.026 = 1.298 kips兾ft 1 1 M a = wa L2 = (1.298)(30)2 = 146 ft-kips < 158 ft-kips 8 8 (OK) Check shear: From the Zx table, Vn 兾Ωv = 70.9 kips Va = wa L 1.298(30) = = 19.5 kips < 70.9 kips 2 2 (OK) Before the concrete cures, wD = 309.4 + 26 = 335.4 lb兾ft wL = 110.0 lb兾ft wa = wD + wL = 0.3354 + 0.1100 = 0.4454 kips兾ft Ma = 1 1 wa L2 = (0.4454)(30)2 = 50.1 ft-kips 8 8 From Table 3-19, Mn M p = = 100 ft-kips > 50.1 ft-kips Ωb Ωb (OK) For the design of the studs and the deflection check, see the LRFD solution. The two 1-inch cantilever strips are labeled m and n in Figure 5.41. The proportional limit, elastic limit, and the upper and lower yield points are all very close to one another and are treated as a single point called the yield point, defined by the stress Fy. The other point of interest to the structural engineer is the maximum value of stress that can be attained, called the ultimate tensile strength, Fu. The shape of this curve is typical of mild structural steels, which are different from one another primarily in the values of Fy and Fu. The ratio of stress to strain within the elastic range, denoted E and called Young’s modulus, or modulus of elasticity, is the same for all structural steels and has a value of 29,000,000 psi (pounds per square inch) or 29,000 ksi (kips per square inch). In addition to its own weight, the beam must support a service live load of 5 kips/ foot. In some cases, you must continue the search to be certain that you have found the lightest shape. The 8-kip load is a service load, composed of 25% dead load and 75% live load. Span length 60(12) = = 72 in. If there are weld segments of different lengths in the direction of the load, use the average length. These tables give load capacities in pounds per foot of length for various standard joists. For convenience, the weight of a reinforced concrete floor slab is usually expressed in pounds per square foot of floor surface. Design in Structural Steel. Clearly, the web will completely yield long before the flanges begin to yield. Earthquake loads are another special category and need to be considered only in those geographic locations where there is a reasonable probability of occurrence. “Shear Strength of Stud Connectors in Lightweight and Normal-Weight Concrete.” Engineering Journal, AISC 8 (no. Use A572 Grade 50 steel. 3): 77–89. 3950 in.4 b. Sometimes space limitations, such as a limit on dimension a in Figure 3.14a, necessitate using more than one line. Sidesway web buckling should be checked when the compression flange is not restrained against movement relative to the tension flange. The members are oriented so that bending is about the strong axis. Pn = FcrAg = 21.16(2.70) = 57.13 kips ∴ Local buckling controls LRFD SOLUTION Design strength = fcPn = 0.90(57.13) = 51.4 kips ASD SOLUTION Allowable strength Pn 57.13 = = 34.2 kips Ω 1.67 (Allowable stress = 0.6Fcr = 0.6(21.16) = 12.7 ksi) 4.5 TABLES FOR COMPRESSION MEMBERS The Manual contains many useful tables for analysis and design. Use of a prescribed drift index enables the designer to determine the final value of B2 at the outset. Use factored loads. PD = 135 k PL = 415 k MD = 90 ft-k ML = 270 ft-k 20′ MD = 30 ft-k ML = 90 ft-k FIGURE P6.8-5 6.8-6 The member shown in Figure P6.8-6 is part of a braced frame. For convenience, the equations for critical buckling load will be written as Pcr = π 2 EA (KL兾r)2 or Pcr = π 2 Et A (KL兾r)2 (4.6a/4.6b) where KL is the effective length, and K is the effective length factor. (Values shown in the table have been rounded to three decimal places for presentation purposes.) From Equation 8.4, P(r0 + e) = ∑ rR P= ∑ rR 23.1120 = = 2.78951 kips r0 + e 1.57104 + 6.71429 where e has been expressed to six significant figures for consistency. At the top of the steel, fst = Myt I tr 1.5 *The ACI Building Code (ACI, 2008) gives the value of Ec as wc (33) fc′ , where fc′ is in pounds per square inch. The flexural strength is computed in the usual way, based on Chapter F of the Specification (Chapter 5 of this book). This type of member will be used to illustrate the requirements for this category of built-up members. Vu = wu L 1.931(30) = = 29.0 kips < 106 kips 2 2 (OK) Before the concrete cures, wD = 309.4 + 26 = 335.4 lb兾ft wL = 110.0 lb兾ft wu = 1.2 wD + 1.6 wL = 1.2(0.3354) + 1.6(0.110) = 0.5785 kipps兾ft Mu = 1 wu L2 = 1 (0.5785)(30)2 = 65.1 ft-kips 8 8 φb M n = φb M p = 151 ft-kips > 65.1 ft-kips (OK) The live-load deflection will be checked after the stud anchors have been selected, because the plastic neutral axis location may change, and this would affect the lower-bound moment of inertia. As was illustrated in Figure 6.5, the maximum secondary moment from the sidesway is always at the end. Let 0.9279 DL (1.5) = Fyst Ast Ωst where L = (b − clip) × 2 sides × 2 stiffeners and the factor of 1.5 is used because the load is perpendicular to the axis of the weld. Here the equal end moments are caused by the sidesway (from the horizontal load). Deflections: Before the concrete has cured, wD = wslab + wbeam = 0.500 + 0.026 = 0.526 kips兾 ft Δ1 = 5wD L4 5(0.526兾12)(30 × 12)4 = = 1.098 in. ST9): 1459–68. ANSWER The shear strength is inadequate. For design, the tables can be entered with the required strength, and a combination of steel shape and ΣQn can be selected. If the applied load transmitted by the beam flange or flange plate exceeds the available strength for any of the limit states considered, column reinforcement must be used. > 0.678 in. For ASD, the safety factor is Ω = 1.50. 9.1 Introduction 597 FIGURE 9.4 At the bottom of the steel, fsb = Myb I tr where M = applied bending moment Itr = moment of inertia about the neutral axis (same as the centroidal axis for this homogeneous section) yt = distance from the neutral axis to the top of the steel yb = distance from the neutral axis to the bottom of the steel The stress in the concrete may be computed in the same way, but because the material under consideration is steel, the result must be divided by n (see Equation 9.1) so that Maximum fc = My nI tr where y– is the distance from the neutral axis to the top of the concrete. Use a weld sufficient to develop the yield strength of the stiffener. The vertical member BD is subjected to couples transferred from each beam, but for the symmetrical frame shown, they are equal and opposite, thereby canceling each other. (1972) and Kulak and Timler (1984) and closely parallels the method developed for eccentric bolted connections by Crawford and Kulak (1971). 248 Chapter 5 Beams Beams with large holes in their webs will require special treatment and are beyond the scope of this book. Adequate: Ma = 12,400 ft-kips < Mn兾Ωb = 13,900 ft-kips 8.6-3 a. Column base plates can be categorized as large or small, where small plates are those whose dimensions are approximately the same as the column dimensions. Part 6. 5.13-2 Use Figure 5.35 and select an open-web steel joist for the following floor system. Adequate: Ma = 100 ft-kips < Mn兾Ω b = 135 ft-kips, Va = 75 kips < Vn兾Ω v = 87.5 kips 5.12-1 a. Compute Cb. Do not include the beam weight in the loading. Do not use the column load tables. ANSWER a. Shear: These bolts are in double shear. The elastic section modulus is Sx = Ix 40, 570 = = 1248 in.3 c 32.5 From AISC Equation F5-10, the tension flange strength based on yielding is M n = Fy Sxt = 36(1248) = 44, 930 in.-kips = 3744 ft-kips The compression flange strength is given by AISC Equation F5-7: Mn = RpgFcr Sxc where the critical stress Fcr is based on either flange local buckling or yielding. Ae = An = Ag − Ahole ⎛ 7 1⎞ = (1 × 3.5) − ⎜ + ⎟ (1) = 2.5 in.2 > 2.414 in.2 ⎝ 8 8⎠ (OK) Check the slenderness ratio: 3.5(1)3 = 0.2917 in.4 12 A = 1(3.5) = 3.5 in.2 Imin = From I = Ar 2, we obtain I min 0.2917 = = 0.2887 in.2 A 3.5 L 5.75(12) Maximum = = 239 < 300 r 0.2887 rmin = ANSWER (OK) Use a PL 1 × 31⁄2. Lightweight concrete is used (unit weight = 115 pcf), and the 28-day compressive strength is 4 ksi. From Equation 9.4, the estimated beam weight is w= 3.4 Mu 3.4(275 × 12) = = 22.7 lb兾ft d a 16 f b Fy ⎛ + t− ⎞ 0.90(50) ⎛ + 3.5 − 0.5⎞ ⎝ 2 ⎝ 2 ⎠ 2⎠ Try a W16 ∂ 26. The ends have bearing-type supports and are not framed. Determine the required number of bolts and show their location on a sketch. Lateral support is provided only at the ends. 30k 4 3 W14 × 109 15′ 15′ FIGURE P5.15-2 5.15-3 The beam shown in Figure P5.15-3 is a W18 × 76 of A992 steel and has lateral support only at the ends. A value may be assumed and verified after a shape is selected, or the weight may be ignored initially and checked after a shape has been selected. As the load is gradually increased, yielding begins at the supports when the bending moment reaches My = FySx. Further increase in the load will cause the simultaneous formation of plastic hinges at each end at a moment of Mp = FyZx. At this level of loading, the structure is still stable, the beam having been rendered statically determinate by the formation of the two plastic hinges. An Introduction to The Design and Behavior of Bolted Joints. > 1.833 kips兾in. Use 0.85 times the basic weld strength for the longitudinal welds and 1.5 times the basic weld strength for the transverse weld. ⎝ 16 ⎠ Ω From Equation 7.38, the base metal shear rupture strength per unit length is Rn ⎛ 9⎞ = 0.3Fu t = 0.3(58) ⎜ ⎟ = 9.79 kips兾in. All contribute to reducing the effectiveness of the member, but shear lag is the most important. The stress in an axially loaded tension member is given by f = P A where P is the magnitude of the load and A is the cross-sectional area (the area normal to the load). > 2.33 in.; Le = 1.5 in. The total bearing strength is 2(44.0t) + 6(78.3t) = 121.2, t = 0.217 in. If the resistance factor f is 0.90, what is the required nominal strength? Assume that all moments are uniform over the length of the beam. For the bearing-type connection with threads in the shear plane, assume that tension controls: Fnt′ = 1.3Fnt − Fnt frv ≤ Fnt φ Fnv = 1.3(90) − 90 fr v ≤ 90 0.75(54) = 117 − 2.222 frv ≤ 90 φ Fnt′ = 0.75(117 − 2.222 frv) ≤ 0.75(90) = 87.75 − 1.667 frv ≤ 67.5 Let φ Fnt′ = 150 ΣAb and frv = 75 ΣAb where ΣAb is the total bolt area. The change in the force N is then obtained from Equation 7.4 as ΔN = A f E f6 f L f (7.6) where Lf is the flange thickness. Building Officials and Code Administrators International Inc. 50k 50k W16 × 31 1′ 1′ 6′ FIGURE P5.8-3 5.8-4 The cantilever beam shown in Figure P5.8-4 is a W10 × 77 of A992 steel. From Table 3-23, “Shears, Moments and Deflections,” in Part 3 of the Manual, the fixed-end moment for each top-chord member is M = M nt = PL 2.4(10) = = 3.0 ft-kips 8 8 These end moments and the corresponding reactions are shown in Figure 6.29. CHAPTER 6 Beam–Columns 6.1 DEFINITION While many structural members can be treated as axially loaded columns or as beams with only flexural loading, most beams and columns are subjected to some degree of both bending and axial load. 1968. The member will be connected through one leg with 1-inch diameter bolts in two lines. EXAMPLE 6.5 The member shown in Figure 6.14 is a W12 × 65 of A242 steel. 10.5 SHEAR STRENGTH The shear strength of a plate girder is a function of the depth-to-thickness ratio of the web and the spacing of any intermediate stiffeners that may be present. For fRn ≥ Ru ,) (1.5 ⎡ 4 b ⎛ t w ⎞ ⎤ EFy t f f(0.40)t w2 ⎢1 + − 0.2 ⎥ ⎥ ≥ Ru ⎝ t f ⎠ ⎦⎥ d tw ⎣⎢ ⎦)( ) (4 b 0.430 ⎡ − 0.2 0.75(0.40)(0.430)2 ⎢1 + 21.1 0.685 ⎣ 1.5 ⎤ 29, 000(50)(0.685) ≥ 99.73 ⎥ 0.430 ⎦ This results in the requirement b ≥ 3.00 in. Design of Welded Structures. For the inner bolts, with a spacing of 3 inches, the bearing strength from Manual Table 7-4 is frn = 78.3t kips兾bolt For the edge bolts, use Table 7-5 with an edge distance of 11⁄4 inches. Other components of some grades of steel include copper, manganese, nickel, chromium, molybdenum, and silicon. I. Approximately 47 ksi c. Analyze the truss with these total joint loads acting. Deflection after composite behavior is attained, with partitions but no live load: Δ1 + Δ3 = 0.6670 + 0.08596 = 0.753 in. The flange plates and the web plate are all shop welded to the column flange and field bolted to the beam. At the supports, the members are welded to a rectangular plate that is bolted to a concrete footing. For the commonly used grades of steel, only two electrodes need be considered: Use E70XX electrodes with steels that have a yield stress less than 60 ksi. In this book, we limit our coverage to hot-rolled I shapes. To avoid results that are even less precise, however, it is reasonable to assume that the given parameters of a problem, such as the yield stress, are exact and then decide on the degree of precision required in subsequent calculations. EXAMPLE 4.17 The column shown in Figure 4.19 is fabricated by welding a 3⁄8-inch by 4-inch cover plate to the flange of a W18 × 65. Assume that V = 0 and Pr兾Pc = 0.4. The connection is not near the end of the column. From the Ix table, try a W18 × 35: Ix = 510 in.4 > 440 in.4 (OK) fb Mn = 249 ft-kips > 177 ft-kips (OK) fvVn = 159 kips > 23.1 kips (OK) ANSWER ASD SOLUTION Use a W18 × 35. The x-axis (the axis of no symmetry) is subject only to flexural buckling. Use Kx = 0.9. The load and moments are 30% dead load and 70% live load. MORE ON EFFECTIVE LENGTH We introduced the concept of effective length in Section 4.2, “Column Theory.” All compression members are treated as pin-ended regardless of the actual end conditions but with an effective length KL that may differ from the actual length. 2 Ft 0.6 Fy 0.6(36) P 105 = 3.62 in. Joint rigidity does introduce some bending moment into the members, but it is usually small and considered to be a secondary effect. Floor loads, both live and dead, are supported by the floor slab. In most cases, the structure will be made up of plane frames, so Pstory and Pe story are for the columns within a story of the frame, and the lateral loads H are the lateral loads acting on the frame at and above the story. Clip at a 45° angle for 5⁄8 inch.) 554 Chapter 8 Eccentric Connections Stiffener-to-column web welds: The size required for strength, in sixteenths of an inch, is D= force resisted by stiffener 0.9279 L The force to be resisted by the stiffener = 89.78 − 58.68 = 31.10 kips. Since the ends are fixed, they are also points of lateral support. The steel is A572 Grade 50 and the concrete strength is f c′ = 4 ksi. Let AE cos 2θ = 1 ⎛ 2 Pr ⎞ = 1.500 L f ⎜⎝ Lb ⎟⎠ A= 1.500 L E cos 2θ = 1.500(581.6) 29, 000 cos 2 (21.8) = 0.0349 in.2 The stiffness requirement controls. The bolt size, spacing, and edge distances here are the same as for the shear connection; therefore, the bearing strength will be the same. Assume that the load is at a distance of at least half the beam depth from the support and design a bearing plate. Built-up shapes can also be created by attaching two or more standard rolled shapes to each other. From AISC B4, Table B4.1b, the web of a doubly symmetric I-shaped section is noncompact if 3.76 E h E < ≤ 5.70 Fy t w Fy and the web is slender if h E > 5.70 tw Fy For singly symmetric I-shaped sections, the web is noncompact if hc E hp Fy Mp ⎛ ⎞⎜ 0.54 M − 0.09⎟⎠ y 2 5.70 tw Fy where hc = twice the distance from the elastic neutral axis (the centroidal axis) to the inside face of the compression flange. When the entire cross section becomes plastic at enough locations, “plastic hinges” will form at those locations, creating a collapse mechanism. 2 2 From Equation 5.13, t= 2.22 Ru n 2 = B b Fy 2.22(99.73)(3.810)2 = 1.22 in. From the Zx table, L p = 15.1 ft and Lr = 42.5 ft L b = 45 ft > Lr ∴ Failure is by elastic LTB. “A New Approach for Design of Steel Beam-Columns.” Engineering Journal, AISC 37 (no. 1992. Noncompact shapes: The nominal strength Mn will be the smaller value computed from the limit states of flange local buckling (FLB) and web local buckling (WLB). In practice, however, the strength of most double angles and tees can be found in the column load tables. The connection was designed to have two vertical rows of four bolts, but one bolt was inadvertently omitted. Table 3-16a in Part 3 of the Manual will be used to obtain the required size of the end panel. If the tensile load is applied at a distance less than 10tf from the end of the column, reduce the strength given by AISC Equation J10-1 by 50%. “Steel—Structural, Reinforcing, Pressure Vessel, Railway.” Annual Book of ASTM Standards. This phenomenon can occur if the flanges are not adequately restrained against movement relative to one another by stiffeners or lateral bracing. 8.8 3. The load to be carried by the longitudinal welds is 129.6 – 8(8.352) = 62.78 kips so the required length of the longitudinal welds is 62.78 = 6.63 in. Check compression. (If the web is noncompact, AISC F5 can still be used, but it will be conservative.) 3. one eighth of the span length, 2. From Equation 8.7, the required stiffener area is Ast = Pbf − fRn min 134.7 − 88.20 = = 1..44 in.2 0.90(36) fst Fyst The stiffener dimensions will be selected on the basis of the criteria given in AISC Section J10.8, and the area of the resulting cross section will then be checked. fRn = 0.85 (4.176)(4 + 4) + 1.5(4.176)(4) = 53.5 kips The larger value controls. Maximum size = − 2 16 16 Minimum weld size = LRFD SOLUTION (AISC Table J2.4) (AISC J2.2b) Compute the required design strength. Use Cb = 1.67. “A Case for a Single Stiffness Reduction Factor in the 2010 AISC Specification.” Engineering Journal, AISC 47 (no. 311 kips b. 16 Rn = 1.2 c tFu ≤ 2.4 dtFu 1 1.2 c tFu = 1.2(1.688) ⎛⎜ ⎞⎟ (58) = 58.74 kips ⎝ 2⎠ Upper limit (the upper limit is independent of c and is the same for all bolts): 2.4dtFu = 52.20 kips < 58.74 kips ⬘ Use Rn = 52.20 kips兾bolt. With this modification, the load capacity of compression members is a function of only the slenderness ratio and modulus of elasticity. Most engineers would probably choose the last alternative. Any realistic problem in plastic analysis or design will certainly include this type of loading. Points on the curves corresponding to Lp are indicated by a solid circle; Lr is represented by an open circle. For relative bracing, the required strength is Prb = 0.004Pr (AISC Equation A-6-1) where Prb = lateral shear force resistance to be provided by the brace Pr = vertical load to be stabilized by the brace In a frame, more than one column will be stabilized by a brace. The individual strength at a given bolt location is the minimum of the bearing and shear strengths at that location. The concrete in the ribs—that is, below the top of the deck—is neglected in determining section properties when the ribs are perpendicular to the beam [AISC I3.2c(2)]. The Manual does not contain design aids for concrete-encased steel shapes. However, the ends of stiff columns connected to flexible beams can more freely rotate and approach the pinned condition, giving relatively large values of G and K. 7.9-1 a. The other is to use the “odd-add” rule, in which we leave the last digit to be retained unchanged if it is an even number, and add 1 if it is an odd number, making it even. It is laterally supported at A and B. Note that there is no distinction between LRFD and ASD when designing steel anchors for full composite behavior. 640 Chapter 9 Composite Construction TABLE 9.8 Shape W14 × 26 W14 × 22 PNA Location ΣQn fbMn 6 3 135 241 230 230 Since 66 in. 9.3 Effective Flange Width 607 The third criterion will apply to edge beams only, so for interior beams, the full effective width will be the smaller of one fourth the span length or the center-tocenter spacing of the beams (assuming that the beams are evenly spaced). What is the percent increase in strength? It is not known whether the bolt threads will be in shear. If the bolt is pretensioned, however, a large part of the applied load is used to relieve the compression, or clamping forces, on the connected parts. Dr.ir. (If the compressive load is applied to one flange only, check web crippling [AISC Equation J10-4 or J10-5]. Computation of the corresponding available strength has been discussed in Chapter 4, “Compression Members.” As the name implies, an effective length factor, K, must be determined. The required compressive strength is Pr = Pa = Pnt + B2 Pt = 280 + 0 = 280 kips (B2 = 0 for a braced frame) From the column load tables, for KL = 1.0 × 14 = 14 feet, the axial compressive strength of a W12 × 65 is Pn = 456 kips Ωc 280 Pa = = 0.6140 > 0.2 Pn 兾Ωc 456 FIGURE 6.12 ∴ Use Equation 6.5 (AISC Equation H1-1a). ASD Procedure. A 2(1)(6) ft = fr = ft2 + fv2 = (2.969)2 + (1.250)2 = 3.221 kips兾 in. Design all welds. 181 kips, Combination 5 Combination 5 Chapter 3: Tension Members 3.2-1 a. At this point, all components of the girder have been sized. The pinned-end condition requires that the member be restrained from lateral translation, but not rotation, at the ends. The combinations to be investigated are therefore Combination 2: Combination 4: Combination 6: 1.2D + 1.6L 1.2D + 1.0W + 0.5L 0.9D ± 1.0W Combination (6) will be investigated first. The girders and their loads are supported by the columns. • partition load = 15 psf, live load = 100 psf • maximum permissible live-load deflection = L兾360 (use a lower-bound moment of inertia) Use steel with a yield stress of Fy = 50 ksi and lightweight concrete weighing 115 pcf with fc′ = 4 ksi. Figure 9.26 illustrates these two types. The column is of A992 steel and the angles and plate are A36. ⎝ 2⎠ From Equation 7.36, the shear rupture strength is ⎛ 1⎞ fRn = 0.45Fu t = 0.45(58) ⎜ ⎟ = 13.1 kips兾in. Therefore, for tension members LRFD requires slightly more computation. The beam-toangle part of the connection is also eccentric, but the load is in the plane of shear, so there is no tension. To the following reviewers, I give my thanks for their thoughtful and constructive comments: Leonard D. 2 ⎛ 7⎞ ⎤ ⎡ Anv = 0.300 ⎢11 − 3.5 ⎥ ⎥ ⎥ = 2.381 in. SOLUTION From the dimensions and properties tables in Part 1 of the Manual, the web width-tothickness ratio of a W14 × 90 is h = 25.9 tw 5.8 Shear Strength 221 and the web area is Aw = dtw = 14.0(0.440) = 6.160 in.2 E 29, 000 = 2.24 = 54.0 Fy 50 2.24 Since h E < 2.24 tw Fy the strength is governed by shear yielding of the web and Cv = 1.0. (As pointed out in the Specification User Note, this will be the case for most W shapes with Fy ≤ 50 ksi.) The nominal shear strength is Vn = 0.6Fy AwCv = 0.6(50)(6.160)(1.0) = 184.8 kips LRFD SOLUTION Determine the resistance factor fv. For the braced condition, the bending factor is M1 ⎞ 15 ⎞ Cm = 0.6 − 0.4 ⎛⎜ ⎟ = 0.6 − 0.4 ⎛⎜ ⎟ = 0.2667 ⎝ M 2 ⎟⎠ ⎝ 18 ⎟⎠ (Pe1 is independent of the loading) Pe1 = 3767 kips Cm Cm = 1 − (α Pr 兾Pe1) 1 − α [(Pnt + Pt)兾Pe1] 0.2667 = = 0.281 < 1.0 ∴ Use B1 = 1.0. 1 − 1.60[(85 + 33.6)]兾3767 B1 = For the unbraced condition, the amplification factor for sidesway, B2, must be computed. Column BC, Pu = 600 kips. A method for estimating the shrinkage deflection can be found in the Commentary to the Specification. To determine whether bearing stiffeners are needed, check the web resistance to concentrated loads (web yielding, web crippling, and web sidesway buckling). There is a construction load of 20 psf and a live load of 250 psf. The AISC Specification addresses this in Section 16, “Load Transfer.” Although there is some chemical bond between the concrete and the steel, it cannot be relied on for full load transfer in an encased composite member and is not permitted by the Specification. To determine the required plate thickness, we use Equation 5.12 with the following changes: Instead of n, use the larger of m and n from Figure 5.41 and call it . From AISC Table J3.4, the minimum edge distance is 1 inch. Flexural buckling. The straight line in the graph is called the beam line, or load line. (We also covered this limit state in Chapter 5.) Bearing stiffeners are required to prevent sidesway web buckling only under a limited number of circumstances. 9.8-3 Use LRFD and design a beam to the following specifications: • span length = 35 ft • beam spacing = 10 ft • 2-in. Matching these diameters with the minimum lengths prescribed by AISC, we get the common stud sizes of 1⁄2 × 2, 5⁄8 × 21⁄2, 3⁄4 × 3, 7⁄8 × 31⁄2, and 1 × 4 (but longer studs may be used). For unshored construction, this larger moment of inertia is available only after the concrete slab has cured. If there is a combination of differences, such as different thicknesses, edge distances, and grades of steel, both the tension member and the gusset plate should be checked. SOLUTION Ec = wc1.5 fc′ = (145)1.5 4 = 3492 ksi Es 29,000 n= = = 8.3 ∴ Use n = 8. The shear center is always located on an axis of symmetry; thus the shear center will be at the centroid of a cross section with two axes of symmetry. From AISC Equation J3-4, the slip-critical strength is Rn = mDuhfTbns From AISC Table J3.1, the prescribed tension for a 7⁄8-inch-diameter Group A bolt is Tb = 39 kips If we assume Class A surfaces, the slip coefficient is m = 0.30, and for four bolts, Rn = mDuhfTb ns × 4 = 0.30(1.13)(1.0)(39)(1) × 4 = 52.88 kips fRn = 1.0(61.70) = 61.70 kips Since there is a tensile load on the bolts, the slip-critical strength must be reduced by a factor of ksc = 1 − Tu 72 =1− = 0.5916 Du Tb nb 1.13(39)(4) The reduced strength is therefore ksc(52.88) = 0.5916(52.88) = 31.3 kips < 54 kips ANSWER ASD SOLUTION The connection is inadequate as a slip-critical connection. This is less than the shear strength of two welds (using two welds for each plate), so use a weld strength of 6.750 kips兾in. Carbon: 0.26% (maximum) Phosphorous: 0.04% (maximum) Sulfur: 0.05% (maximum) These percentages are approximate, the exact values depending on the form of the finished steel product. LRFD SOLUTION From Example 8.10, the flange force can be conservatively taken as Pbf = H = 121.2 kips Check local flange bending with AISC Equation J10-1: fRn = f (6.25Fyf t 2f) = 0.90[6.25(50)(0.780)2] = 171 kipps > 121.2 kips (OK) Check local web yielding with AISC Equation J10-2: φRn = φ [Fyw t w (5k + b)] { } 5 = 1.0 50(0.485) ⎡5(1.38) + ⎤ = 182 kips > 121.2 kips ⎣⎢ 8 ⎦⎥ (OK) Check web crippling with AISC Equation J10-4: fRn = f 0.80 t w2 1.5 ⎡ b ⎞ ⎛ t w ⎞ ⎤ EFyw t f ⎛ ⎢1 + 3 ⎜⎝ ⎟⎠ ⎥ ⎥ tw d ⎝ t f ⎠ ⎦⎥ ⎣⎢ 1.5 ⎡ 5兾8 ⎞ ⎛ 0.485 ⎞ ⎤ 29, 000(50)(0.780) = 0.75(0.80)(0.485)2 ⎢1 + 3 ⎛⎜ ⎝ 14.2 ⎟⎠ ⎜⎝ 0.780 ⎟⎠ ⎥⎦ 0.485 ⎣ = 229 kips > 121.2 kips (OK) ANSWER Column web reinforcement is not required. There is one exception for welded plates, however. For the usual case of a compression member with no supports between its ends, n = 1 and the Euler equation is written as Pcr = π 2 EI L2 (4.3) It is convenient to rewrite Equation 4.3 as Pcr = π 2 EI π 2 EAr 2 π 2 EA = = (L兾r)2 L2 L2 where A is the cross-sectional area and r is the radius of gyration with respect to the axis of buckling. 5.2-2 An unsymmetrical flexural member consists of a 3 × 22 top flange, a 3 × 16 bottom flange, and a 1⁄2 × 66 web. Try a W10 × 60. 70.9 kips b. Without regard to load or resistance factors, investigate this member for stability. All steel is A572 Grade 50. Figure P6.7-2b shows the wind load effects obtained from a first-order analysis. C8 × 11.5 FIGURE P3.2-7 Effective area 3.3-1 Determine the effective area Ae for each case shown in Figure P3.3-1. Although there is no limit on the dead load deflection in this example, this deflection may be needed if the beam is to be cambered. Compute the moment amplification factor. 394 Chapter 7 7.5 Simple Connections INSTALLATION OF HIGH-STRENGTH BOLTS In certain cases, high-strength bolts are installed to such a degree of tightness that they are subjected to extremely large tensile forces. If this shape is not satisfactory, try a larger one. The 30-kip load can be located anywhere between the two rods. If P is greater than F and slippage occurs, shear and bearing will then exist and will affect the capacity of the connection. Divide this force by Qn , the shear capacity of a single stud, to obtain Nl, which, in most cases, is half the total number of studs required. The shapes are categorized as steel pipe, round HSS, and square and rectangular HSS. In this appendix, we consider only continuous beams. It is connected to a 3⁄8-inch-thick gusset plate (also A36 steel) with 3⁄4-inch-diameter bolts. History of Strength of Materials. When it is not known whether the threads are in the plane of shear, assume that they are and use the lower shear strength. Failure is assumed to occur by rupture (fracture) on the shear area (subject to an upper limit) and rupture on the tension area. 1984. Maximum diameter = 2.5t f = 2.5(0.345) = 0.863 in. The loads are 40% live load and 60% dead load. Mn = Mp = Fy Z (AISC Equation F7-1) (Because of the high torsional resistance of closed cross-sectional shapes, lateral-torsional buckling of HSS need not be considered, even for rectangular shapes bent about the strong axis.) ii. To determine which of the three cases governs, compute the compressive resultant as the smallest of 1. Use the effective length KL to select p, and use the unbraced length Lb to select bx (the constant by determines the weak axis bending strength, so it is independent of the unbraced length). The frames are connected by the beams in the north-south direction, completing the framework for the building. Cross-sectional requirements for these stiffeners, called intermediate stiffeners, are minimal because their primary purpose is to provide stiffness rather than resist directly applied loads. Otherwise, the shape is slender. The load and moments are computed from service loads, and bending is about the x axis (the end shears are not shown). Determine whether the column behavior is elastic or inelastic. In such connections, illustrated in Figure 7.3a, b, and c, each fastener or each unit length of weld will resist an equal amount of FIGURE 7.3 (a) (b) Section (c) (d) (e) (f) 380 Chapter 7 Simple Connections force.* The load capacity of the connection can then be found by multiplying the capacity of each fastener or inch of weld by the total number of fasteners or the total length of weld. If rectangular components are used, Px L f1x = and f1 y = Py L where Px and Py are the x and y components of the applied load. This condition corresponds to the loading of Figure 5.49a. For this reason, bearing strength, as well as spacing and edge-distance requirements, which also are independent of the type of fastener, will be considered before bolt shear and tensile strength. Composite columns are covered in Chapter 9 of this textbook. The connection will be with two 9-in.-long longitudinal welds. Ae = AnU = 4.536(0.60) = 2.722 in.2 Pn = Fu Ae = 58(2.722) = 157.9 kips φt Pn = 0.75(157.9) = 118 kips (controls) FIGURE 7.16 406 Chapter 7 Simple Connections The member capacity is therefore adequate with two gage lines of bolts. This procedure requires the strains in the fictitious steel to be the same as those in the concrete it replaces. 3.3 Effective Area FIGURE 3.10 U = 0.60 Single and double angles U = 0.80 Single and double angles W10 × 19 bf 2 = 0.394 < 3 d U = 0.85 W8 × 24 bf 2 = 0.820 > 3 d U = 0.90 WT5 × 22.5 bf = 0.794 > d U = 0.90 FIGURE 3.11 2 (for parent shape) 3 W shape U = 0.70 55 56 Chapter 3 Tension Members FIGURE 3.12 The length of the connection is = 3 + 3 = 6 in. Compute the required plate thickness: s= 1 1 bp g = 7(3.5) = 2.475 in. • The following bolt pitch is recommended: db + 1⁄2 in. For this reason, the user note in AISC D1 suggests a maximum slenderness ratio of 300. A possible failure mode resulting from excessive bearing is shear tear-out at the end of a connected element, as shown in Figure 7.7a. The procedure is straightforward, and involves comparing the width-to-thickness ratio with a limiting value and then computing Qs from an expression that is a function of the width-to-thickness ratio, Fy, and E. The maximum stress will occur at the extreme fiber, where y is maximum. The lap joint illustrated in Figure 7.33a is made with fillet welds, which are defined as those placed in a corner formed by two parts in contact. The beams are spaced at 12 feet, and the span length is 40 feet. For a兾h ≤ 1.5, ⎛ h⎞ ⎜⎝ t ⎟⎠ w = 12.0 max E Fy (AISC Equation F13-3) 696 Chapter 10 Plate Girders For a > 1.5, h ⎛ h⎞ ⎜⎝ t ⎟⎠ w = max 0.40 E Fy (AISC Equation F13-4) Once h and tw have been selected, determine whether the web width-tothickness ratio qualifies this member as a slender-web flexural member. A shape can be selected 6.8 Design of Beam–Columns 357 by using the method given in Part 6 of the Manual. FIGURE 4.12 142 Chapter 4 LRFD SOLUTION Compression Members Factored load = Pu = 1.2(200) + 1.6(200) = 560 kips Assume that the weak direction controls and enter the column load tables with KL = 9 feet. From AISC Equation J4-1, Rn = Fy Ag fRn = 0.90 Fy Ag Required Ag = H fRn 121.2 = = = 3.741 in.2 0.90 Fy 0.90 Fy 0.90(36) 8.6 Moment-Resisting Connections 529 From AISC Equation J4-2, Rn = Fu Ae fRn = 0.75Fu Ae Required Ae = 121.2 fRn H = = 2.786 in.2 = 0.75Fu 0.75Fu 0.75(58) Try a plate width of wg = 6.5 in. Although there is no yield point, one needs to be defined. As with model building codes, design specifications are written in a legal format by nonprofit organizations. Group A: ASTM A325, F1852, A352, A354 Grade BC, and A449. The analysis of each mechanism is accomplished by application of the principle of virtual work. 364 Chapter 6 Beam–Columns 250 k 240 ft-k W12 × 106 Kx = Ky = 1.0 14′ 240 ft-k 250 k FIGURE P6.2-1 6.2-2 How much service live load, in kips per foot, can be supported? This equation was derived by Jacob Bernoulli and independently by Euler, who specialized it for the column buckling problem (Timoshenko, 1953). Assume that the nominal bearing strength will be 2.4dtFu and answer the following questions: a. Use A992 steel and select a W-shape. Check the spacing and edge distance requirements transverse to the load. Larson, Jay W. In cases such as this one, only the thinner component need be checked. From Table 4-22 with KL兾r = 54.55, Fcr = 24.09 ksi Ωc Pn Fcr = Ag = 24.09(17.0) = 410 kips Ωc Ωc ANSWER FIGURE 4.11 Allowable strength = 410 kips. The computations for the neutral axis location and the moment of inertia are summarized in Table 9.5. FIGURE 9.15 A c = 10.3 in. 0.85 f c′b 0.85(4)(72) The moment arm is y= d a 12.2 2.982 +t− = +4− = 8.609 in. This alternative will definitely save weight, but here also, fabrication costs must be considered. For the tension member and the holes nearest the edge of the member, h = d + 13兾16 h = 1.25 − = 0.8438 in. 5.11 FLOOR AND ROOF FRAMING SYSTEMS When a distributed load acts on an area such as a floor in a building, certain portions of that load are supported by various components of the floor system. From Equation 7.35, the shear yield strength per unit length is φ Rn = 0.6 Fy t = 0.6(36) (14) = 5.400 kips兾in. Determine whether a W12 × 79 of A992 steel is adequate. Part 11. Beam flange force: Mu 173.2 × 12 Ffu = = = 120.3 kips d − t f b 17.7 − 0.425 Ff u 120.3 = = 60.2 kips 2 2 568 Chapter 8 Eccentric Connections The shear yield strength of the end plate is f(0.6)Fy tpbp = 0.90(0.6)(36)(1)(7) = 136 kips > 60.2 kips (OK) Shear rupture strength of end plate: 1⎞ ⎤ ⎛ ⎛ 7 1⎞ ⎤ ⎡ ⎡ An = t p ⎢ bp − 2 ⎜ d b + ⎟ ⎥ = (1) ⎢ 7 − 2 ⎜ + ⎟ ⎥ = 5.000 in.2 ⎝ ⎝ 8 8⎠ ⎦ 8⎠ ⎦ ⎣ ⎣ f (0.6) Fu An = 0.75(0.6)(58)(5.000) = 131 kips > 60.2 kips (OK) Check bolt shear. Although there are exceptions, the ribs of the deck are usually oriented perpendicular to floor beams and parallel to supporting girders. ASD SOLUTION Try a 3⁄16-in. In the computation of the net area for line abcde in Example 3.6, An = Ag – ∑ t × (d or d ′) (3)2 ⎤ ⎡ = 0.75(16) - 0.75(1.125) − 0.75 ⎢1.1225 – × 2 = 10.1 in.2 4(5) ⎥⎦ ⎣ As each fastener resists an equal share of the load (an assumption used in the design of simple connections; see Chapter 7), different potential failure lines may be subjected to different loads. For each fastener, the horizontal and vertical components of force resulting from direct shear are pcx = Px Py and pcy = n n where Px and Py are the x- and y-components of the total connection load, P, as shown in Figure 8.4. The horizontal and vertical components caused by the eccentricity can be found as follows. 1989a. For any material, the critical buckling stress can be plotted as a function of slenderness, as shown in Figure 4.6. The tangent modulus curve is tangent to the Euler curve at the point corresponding to the proportional limit of the material. Problems 285 PD = 2.7k PL = 5.4k wD = 0.5k/ft wL = 1.0k/ft a d b 10′ c 15′ 10′ FIGURE P5.5-11 5.5-12 A W24 × 76 of A992 steel is used as a simply supported beam with a span length of 48 feet. In general, a noncompact beam may fail by lateral-torsional buckling, flange local buckling, or web local buckling. The Uniform Building Code has been essentially the only one used west of the Mississippi, the Standard Building Code has been used in the southeastern states, and the BOCA National Building Code has been used in the northeastern part of the country. Before the concrete cures, the load and moment are wa = wD + wL = 541.3 + 180 = 721.3 lb兾ft Ma = 1 1 wa L2 = (0.7213)(30)2 = 81.2 ft-kips 8 8 From the Zx table, Mn M p = = 166 ft-kips > 81.2 ft-kips Ωb Ωb (OK) After the concrete cures, the load and moment are wa = wD + wL = 541.3 + 1305 = 1846 lb兾ft Ma = 1 (1.846)(30)2 = 208 ft-kips 8 The allowable strength of the composite section is M n 536.9 = = 322 ft-kips > 208 ft-kips Ωb 1.67 (OK) Check shear: Va = wa L 1.846(30) = = 27.7 kips 2 2 From the Zx table, Vn = 106 kips > 27.7 kips Ωv ANSWER (OK) The beam complies with the AISC Specification. The unified approach continues with the 2010 Specification and the 14th edition of the Steel Construction Manual. Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Application. ⎝ 4⎠ N1 = Maximum longitudinal spacing = 8t ≤ 36 in. The element with the smallest Δm兾r is the critical element, that is, the one that reaches its ultimate capacity first. The total depth of the girder will be 86 inches, and the unbraced length is 25 feet. From Example 4.2, K = 1.0, so KL = 1.0(20) = 20 ft For a W14 × 74, Fy = 50 ksi and KL = 20 ft, fcPn = 495 kips ASD SOLUTION a. Since that time, high-strength bolting has rapidly gained in popularity, and today the widespread use of high-strength bolts has rendered the rivet obsolete in civil engineering structures. For this condition to occur, however, member CD must transmit the load P1 + P2 from C to D. 1966. The capacity of a 1.5-inch length of a pair of welds is 6.750(1.5) = 10.13 kips To determine the spacing, let 10.13 VuQ = s Ix 10.7 Design 709 where s is the center-to-center spacing of the welds in inches and s= 10.13 10.13 = = 3.21 in. For these reasons, we limit further consideration to flexural-torsional buckling of shapes with one axis of symmetry. “Strength of Plate Girders in Shear.” Journal of the Structural Division, ASCE 87 (no. EXAMPLE 8.1 Determine the critical fastener force in the bracket connection shown in Figure 8.5. SOLUTION The centroid of the fastener group can be found by using a horizontal axis through the lower row and applying the principle of moments: y= FIGURE 8.5 2(5) + 2(8) + 2(11) = 6 in. Not only can each load effect have a different load factor but also the value of the load factor for a particular load effect will depend on the combination of loads under consideration. The superimposed loads consist of a 20-psf construction load, a partition load of 20 psf, and a live load of 125 psf. In addition to the weight of the slab, there is a 20 psf partition load and a live load of 125 psf (light manufacturing). = 384 EI s 384(29,000)(510) The construction load is w = 180 lb兾ft and Δ2 = 5wL4 5(0.180 兾12)(30 × 12)4 = = 0.2218 in. The controlling load combination is the one that produces the largest factored load. Other useful constants that are tabulated include Lp and Lr (which is particularly tedious to compute). Solutions manual to cmos Mixed-Signal. Part (a) of the figure shows one of the rigid frames comprising the building. 178 Chapter 4 4.7-5 Compression Members Use A992 steel and select a W shape. 0.75(0) ⎛ 1.988 ⎞ ⎤ 6α b ′ ⎤ ⎡ ⎡ = 20 ⎢1 + Bc = T ⎢1 + ⎥ ⎥ = 20 kips ⎢⎣ (1 + 0.75 × 0) ⎝ 3.469 ⎠ ⎥⎦ ⎣ (1 + 6α) a ′ ⎦ The prying force is q = Bc – T = 20 – 20 = 0 kips This indicates that the tee section is not flexible enough to produce prying action. (In order not to obscure the combined loading features of this example, prying action has not been included in the analysis.) b. (Although the limiting width-to-thickness ratio is labeled b/t in the table, that is a generic notation, and it applies to h兾t as well.) Because this cross-sectional element is a stiffened element, Qs = 1.0, and Qa must be computed from AISC Section E7.2. The shape is a rectangular section of uniform thickness, with b E ≥ 1.40 , t f So AISC E7.2 (b) applies, where f= Pn Ae and Ae is the reduced effective area. FIGURE P3.6-4 3.6-5 Choose a pipe to be used as a tension member to resist a service dead load of 10 kips and a service live load of 25 kips. In the United States, LRFD has been an accepted method of design for reinforced concrete for years and is the primary method authorized in the American Concrete Institute’s Building Code, where it is known as strength design (ACI, 2008). Use Group A bearing-type bolts and E70 electrodes. TABLE 5.4 Deflection Limits Max. Check the shear capacity of the base metal (the angle controls): Applied direct shear = fv = 1.250 kips兾in. For the common case of a simply supported, uniformly loaded beam such as that in Figure 5.22, the maximum vertical deflection is Δ = 5 wL4 384 EI Deflection formulas for a variety of beams and loading conditions can be found in Part 3, “Design of Flexural Members,” of the Manual. The capacity per inch for two ⁄16-inch fillet welds per stiffener plate is 3 1.392 × 3 × 2 = 8.352 kips兾in. This cross section is illustrated in Figure 10.4. The compressive strength should always be based on the radius of gyration about an axis in the plane of the web, as instability about the other principal axis is prevented by the web itself. The member has been designed to resist a service dead load of 40 kips and a service live load of 80 kips. ILB = 927 in.4 EXAMPLE 9.8 Compute deflections for the beam in Example 9.4. SOLUTION Selected data from Example 9.4: W18 × 35, A992 steel 926.9 in.4 9.6 Deflections 623 Slab thickness t = 4.5 in. For a compact shape, this represents the strength corresponding to yielding (reaching the plastic moment Mp). 9.4-3 How many 3⁄4-inch × 3-inch studs are required for the beam in Problem 9.1-1 for full composite behavior? It is connected to a 5⁄16-inch-thick gusset plate with 3⁄4-inch-diameter, Group A bolts. For FLB, ⎛ b M n = M p − (M p − Fy S) ⎜ 3.57 ⎝ tf Fy ⎞ − 4.0 ⎟ ≤ M p ⎠ E (AISC Equation F7-2) For WLB, ⎛ h M n = M p − (M p − Fy S x) ⎜ 0.305 ⎝ tw Fy ⎞ − 0.738 ⎟ ≤ M p ⎠ E (AISC Equation F7-5) c. The nominal slip resistance of a bolt is given by Rn = m DuhfTbns (AISC Equation J3-4) where m = mean slip coefficient (coefficient of static friction) = 0.30 for Class A surfaces Du = ratio of mean actual bolt pretension to the specified minimum pretension. From Chapter 5, when Cb is not equal to 1.0, the design bending strength is adjusted as Modified fbMnx = unmodified φb M nx × Cb ≤ φb Mpx To modify bx to account for Cb, let bx = 8 8 ≤ 9(φ b M nx × Cb) 9 (φb M px) (6.11) So the modified value of bx (before checking the upper limit) is the tabular value of bx divided by Cb. Because fbMpx is the value of fbMnx when Lb = 0, inequality 6.11 will be satisfied when bx ≥ bx for Lb = 0 Cb Therefore, Modified bx = bx ≥ bx for Lb = 0 Cb 6.8 Design of Beam–Columns 345 or ⎛ bx ⎞ Modified bx = max ⎜ , bx for Lb = 0 ⎟ ⎝ Cb ⎠ Note that, this modification is independent of the design approach used, whether LRFD or ASD. This shape is sometimes referred to as a split-tee. Unless they also serve as bearing stiffeners, intermediate stiffeners are not required to bear against the tension flange, so their length can be somewhat less than the web depth h, and fabrication problems associated with close fit can be avoided. From Example 4.2, K = 1.0, so KL = 1.0(20) = 20 ft From the column load tables, for a W14 × 74 with Fy = 50 ksi and KL = 20 ft, Pn = 329 kips Ωc 132 Chapter 4 Compression Members The values from Table 4-22 (Manual) are based on flexural buckling and AISC Equations E3-2 and E3-3. EXAMPLE 7.7 The C8 × 18.75 shown in Figure 7.15 has been selected to resist a service dead load of 18 kips and a service live load of 54 kips. The effective length with respect to the x-axis is 25 feet 6 inches, the effective length with respect to the y-axis is 20 feet, and the effective length with respect to the z-axis is 20 feet. Steel producers who provide A36 steel must certify that it meets the ASTM standard. For fc′ = 4 ksi, the modulus of elasticity of the concrete is Ec = wc1.5 fc′ = 1451.5 4 = 3492 ksi From AISC Equation I3-3, the shear strength of one stud is Qn = 0.5 Asa fc′Ec ≤ Rg R p Asa Fu = 0.5(0.4418) 4(34922) = 26.11 kips Rg Rp Asa Fu = 1.0(0.6)(0.4418)(65) = 17.23 kips < 26.11 kips ∴ Use Qn = 17.23 kips. An open-web steel joist can be selected with the aid of the standard load tables (SJI, 2005). In such cases, fatigue of the fasteners can become critical if the connection is allowed to slip with each reversal, and a slip-critical connection is advisable. The results are typical in that the requirements are minimal. Thus the minimum moment of inertia and radius of gyration of the cross section should ordinarily be used in Equations 4.3 and 4.4. EXAMPLE 4.1 A W12 × 50 is used as a column to support an axial compressive load of 145 kips. 2 (1 24 . Superimposed loading consists of a 20 psf partition load and a 100 psf live load. A992 steel is used, and f c′ = 4 ksi. The required axial compressive strength is given by Pr = Pnt + B2Pt (AISC Equation A-8-2) where Pnt = axial load corresponding to the braced condition Pt = axial load corresponding to the sidesway condition We cover the evaluation of B1 and B2 in the following sections. Ω The stiffener yield strength of 9.0 kips兾in. PD = 120 k, PL = 240 k 30 k MD = 15 ft-k ML = 40 ft-k 130 ft-k MD = 18 ft-k ML = 48 ft-k 130 ft-k 16′ (a) Gravity (b) Wind FIGURE P6.7-2 Design of Beam–Columns 6.8-1 Use Fy = 50 ksi and select the lightest W12-shape for the beam–column shown in Figure P6.8-1. For ASD, the allowable flexural strength must at least equal the applied moment: Mp ≥ Ma Ωb Fy t 2⁄ 4 Ra n 2 ≥ 1.67 2 B b t≥ 3.34 Ra n 2 B b Fy (5.14) where Ra is the service-load beam reaction. The length is 16 feet. Tables for Composite Beam Analysis and Design 9.8-1 Use the composite beam tables to solve Problem 9.7-3. 4.40 bolts required b. Equations 7.18 and 7.19 are the same as those given in Part 9 of the Manual. For the plate, 5 tFu = ⎛⎜ ⎞⎟ (58) = 36.25 kips兾in. It provides for all structural materials normally used in bridges, including steel, reinforced concrete, and timber (AASHTO, 2010). A more comprehensive treatment of the subject is given by Lothars (1972). Because Ef approximately equals Eb (Bickford, 1981), and Af is much larger than Ab, A f E f6 f Ab Ebδ b L f Lb and therefore Δ N ΔT The ratio of ΔT to ΔN is in the range of 0.05 to 0.1 (Kulak, Fisher, and Struik, 1987). Determine whether the flexural strength of this beam is adequate. π (7兾8)2 = 0.6013 in.2 4 Rn = Fnv Ab = 54(0.6013) = 32.47 kips Ab = For the bearing strength, use a hole diameter of h=d+ 1 7 1 15 = + = in. The spacing and edge distance to be considered, denoted s and e , are illustrated in Figure 7.9. Summary of Bearing Strength, Spacing, and Edge-Distance Requirements (Standard Holes) a. 240 240 ΔL = ANSWER 5 wL L4 5 (4.5兾12)(30 × 12)4 = = 1.19 in. The applied load consists of 9 kips dead load and 15 kips live load. For the support conditions shown, the boundary conditions are At x = 0, y = 0 and at x = L, y = 0. For each framing plan investigated, the individual components must be designed. The weight of the beam is part of the dead load but is unknown at this point. or hr + 1.5 = 1.5 + 1.5 = 3 in. The advantage of a partially-restrained connection is that it can equalize the negative and positive moments within a span. Use fc′ = 4 ksi. The deck is 11⁄2 inches deep, and the total depth from bottom of deck to top of slab is 41⁄2 inches. Assume that the slab provides continuous lateral support. D = 40k L = 80k L5 × 5 × 5⁄8 t = 3⁄8″ FIGURE P8.4-17 8.4-18 Same as Problem 8.4-17, but use ASD. The service-load moment to be transferred is 180 ft-kips, consisting of 45 ft-kips of dead-load moment and 135 ft-kips of live-load moment. Although there is a linear elastic portion and a distinct tensile strength, there is no well-defined yield point or yield plateau. For Group A bolts with the threads in the plane of shear (type N), the nominal shear strength is Fnv = 54 ksi, and Rn =Fnv Ab = 54(0.4418) = 23.86 kips 392 Chapter 7 Simple Connections As in Part a, this value is smaller than the bearing strength at each hole, so the total nominal strength for the connection is Rn = 23.86 + 23.86 = 47.72 kips ANSWER For LRFD, the design strength is f Rn = 0.75(47.72) = 35.8 kips. The weld is a 1⁄8-inch fillet weld with E70 electrodes. 2.23 in. If not, the unbraced length Lb must be taken into account, and lateral-torsional buckling may control the flexural strength. The x-component of pm is pmx = y Md y Md My y pm = = = d d ∑ d 2 d ∑(x 2 + y2) ∑(x 2 + y2) Similarly, pmy = FIGURE 8.4 Mx ∑(x 2 + y2) 8.2 Eccentric Bolted Connections: Shear Only 481 and the total fastener force is p = (∑ px)2 + (∑ py)2 where ∑ px = pcx + pmx ∑ py = pcy + pmy If P, the load applied to the connection, is a factored load, then force p on the fastener is the factored load to be resisted in shear and bearing—that is, the required design strength. The given loads are factored, and the uniformly distributed load includes a conservative estimate of the girder weight. Kulak, G. Coverage of beams in the AISC Specification is spread over two chapters: Chapter F, “Design of Members for Flexure,” and Chapter G, “Design of Members for Shear.” Several categories of beams are covered in the Specification; in this book, we cover the most common cases in the present chapter, and we cover a special case, plate girders, in Chapter 10. From the design of the flange-to-web welds, the base metal shear strength is 6.750 kips兾in. A992 steel is used for the beam and column, and A36 steel is used for the angles. Since bf = 0.345 in., t⬘ is less than bf , and the PNA is in the flange, as assumed. The value of eo, which FIGURE 5.46 5.15 Biaxial Bending 267 locates the shear center for channel shapes, is tabulated in the Manual. In Example 4.2, ry < rx , and there is excess strength in the x-direction. Section G2.1 of the AISC Specification covers both beams with stiffened webs and beams with unstiffened webs. 0.732 < L兾360 (OK) 9.7-1 a. FIGURE 7.5 382 Chapter 7 Simple Connections FIGURE 7.6 Thus the bearing stress would be computed as fp = P兾(dt), where P is the force applied to the fastener, d is the fastener diameter, and t is the thickness of the part subjected to the bearing. For the connection, Rn = 2.784 kips兾in. Circular rods and rolled angle shapes are frequently used. The allowable strength for ASD is 3.6 Rn 82.51 = = 41.3 kips. Figure 6.20 shows the calculated axial load and moments for combinations 2, 5, and 6a. Determine dimension b so that web yielding and web crippling are prevented. “A Fresh Look at Bolted EndPlate Behavior and Design.” Engineering Journal, AISC 15 (no. Figure 9.2 shows a slab with formed steel deck and the ribs perpendicular to the beam axis. From Example 3.12, Pa = 105 kips Required rmin = 0.600 in. (normal-weight concrete weighs approximately 145 lb兾ft3) fc′ = 28-day compressive strength of concrete (kips兾in.2) The AISC Specification also gives a metric version of the equation for Ec. Equation 9.1 can be interpreted as follows: n square inches of concrete are required to resist the same force as one square inch of steel. For lightweight concrete, a unit weight of 115 pounds per cubic foot can be used in lieu of more specific data. 8.6-4 Design a three-plate moment connection of the type shown in Problem 8.6-2 for the connection of a W18 × 35 beam to a W14 × 99 column for the following conditions: The service dead-load moment is 42 ft-kips, the service live-load moment is 104 ftkips, the service dead-load beam reaction is 8 kips, and the service live-load beam reaction is 21 kips. Maximum stud diameter = 2.5t f = 2.5(0.345) = 0.8625 in. 7 (5 .49)2 ⎣ ⎦ Fey + Fez = 323.0 + 80.06 = 403.1 ksi Fe = ⎛ ⎝ = Fey + Fez ⎞ ⎡ 4 Fey Fez H ⎤ 1− 1− 2 H ⎠ ⎢⎣ (Feyy + Fez)2 ⎥⎦ 403.1 ⎡ 4(323.0)(80.06)(0.937) ⎤ 1− 1− ⎥ = 78.46 ksi 2(0.937) ⎢⎣ (403.1)2 ⎦ (H is tabulated) Since Kx L E < 4.71 = 133.7 rx Fy use AISC Equation E3-2: Fcr = 0.658(Fy兾Fe)Fy = 0.658(36兾78.46)(36) = 29.71 ksi The nominal strength is Pn = Fcr Ag = 29.71(14.7) = 436.7 kips The flexural buckling strength controls, and the nominal strength is 113.5 kips. A36 steel is used for both the tension member and the gusset plate. 71.5 kips 7.6-1 a. One of the LRFD tables is reproduced in Figure 5.35. ⎝ 2⎠ Ω From Equation 7.38, the shear rupture strength is Rn ⎛ 1⎞ = 0.3Fu t = 0.3(5 8) ⎜ ⎟ = 8.70 kips兾in. A second analysis is performed in which the frame is assumed to be unbraced. Similarly, the weld shown in Figure 7.3c must resist shearing forces. If the lower right-hand corner is selected, My 942.6(6) = = 6.181 ksi 915.0 J Mx 942.6(8 − 2.286) = = = 5.886 ksi 915.0 J f2 x = f2 y fv = (6.181)2 + (2.143 + 5.886)2 = 10.13 ksi = 10.13 kips兾 in. 16 2 0.9279 ANSWER Use a 1⁄2-inch fillet weld, E70 electrode. 10.7-6 Design flange-to-web welds for the girder of Problem 10.4-5. However, there is an excess of flexural strength, so partial composite action will probably be adequate. For efficiency, the nominal strength for each limit state will be computed before specializing the solution for LRFD and ASD. Because of this, two amplification factors must be used. Intermediate Stiffeners Plate girders can be designed without intermediate stiffeners. and Matuska, J. For most of the W-shapes in the column load tables, rx 兾ry ranges between 1.6 and 1.8, but it is as high as 3.1 for some shapes. 48.7 kips 7.4-3 a. Two of the axes are parallel to the web and 0.80bf apart, and two axes are parallel to the flanges and 0.95d apart. Because of symmetry, the centroid of the connection is at middepth. (M ay ⎞ 8 ⎛ M ax Pa 8 90 20 + ⎜ + = 0.4535 + + ⎜ Pn 兾Ωc 9 ⎝ M nx 兾Ωb M ny 兾Ωb ⎠ 9 237 107) = 0.957 < 1.0 (OK)) ANSWER 6.7 The W12 × 65 is satisfactory. Minimum spacing and edge distance: In any direction, both in the line of force and transverse to the line of force, s ≥ 2 2⁄3 d (preferably 3d) e ≥ value from AISC Table J3.4 For single- and double-angle shapes, the usual gage distances given in Table 1-7A in Part 1 of the Manual (see Section 3.6) may be used in lieu of these minimums. As shown in Figure 5.13, the boundary between elastic and inelastic behavior 5.5 Bending Strength of Compact Shapes 205 will be for an unbraced length of Lr, which is the value of Lb obtained from AISC Equation F2-4 when Fcr is set equal to 0.7Fy with Cb = 1.0. The following equation results: Lr = 1.95rts E 0.7 Fy 2 0.7 Fy ⎞ Jc ⎛ Jc ⎞ + ⎜ + 6.76 ⎛⎜ ⎟ ⎝ E ⎟⎠ ⎝ S x h0 ⎠ S x h0 2 (AISC Equation F2-6) As with columns, inelastic buckling of beams is more complicated than elastic buckling, and empirical formulas are often used. For the inner holes, conservatively use the spacing of 2 2⁄3 db found in Manual Table 7-4. Figure 8.50 shows three versions of the rigid end plate connection: an unstiffened four-bolt connection, a stiffened four-bolt connection, and a stiffened eight-bolt connection. Column MN, Pu = 1000 kips. 5.16 Bending Strength of Various Shapes 279 c. Usually the category of shape (W, WT, etc.) will have been decided upon in advance. The effective lengths with respect to the two principle axes are KxL = 36 feet and Ky L = 12 feet. 4.40 bolts required 7.9-5 a. Falls Church, VA. If the parallel-axis theorem is used and the polar moment of inertia of each circular area about its own centroid is neglected, J for the total area can be approximated as J = ∑ Ad 2 = A ∑ d 2 FIGURE 8.3 480 Chapter 8 Eccentric Connections provided all fasteners have the same area, A. Upset ends are relatively expensive, however, and in most cases unnecessary. As a consequence of this condition, the maximum primary and secondary moments are usually additive and there is no need for the factor Cm; in effect, Cm = 1.0. Even when there is a reduction, it will be slight and can be neglected. The maximum live load deflection must not exceed 1兾360 of the span length. Try 21⁄2 inches. The lateral load produces Mt moments. The required axial compressive strength is Pr = Pu = Pnt + B2Pt = 300 + 0 = 300 kips and the axial compressive design strength from the column load tables is fcPn = 662 kips 6.6 Members in Braced Frames 325 Determine which interaction formula to use: 300 Pu = = 0.4532 > 0.2 ∴ Use Equation 6.3 (AISC C Equation H1-1a). (In most cases, when the higher strength corresponding to threads not in shear is used, some limit TABLE 7.1 Fastener A307 Group A threads in plane of shear threads not in plane of shear Group B threads in plane of shear threads not in plane of shear Nominal Shear Strength Rn = Fnv Ab 27Ab 54Ab 68Ab 68Ab 84Ab 390 Chapter 7 Simple Connections state other than bolt shear will control the joint design.) For LRFD, the resistance factor is 0.75, and the design strength is fRn = 0.75Fnv Ab For ASD, the safety factor is 2.00, and the allowable strength is Rn Fnv Ab = Ω 2.00 EXAMPLE 7.2 Determine the strength of the connection shown in Figure 7.11, based on bearing and shear, for the following bolts: a. The load at which buckling occurs is a function of slenderness, and for very slender members this load could be quite small. wu = 1.2(0.4375 + 0.026) + 1.6(0.200) = 0.8762 kips兾ft Mu = 1 (0.8762)(30)2 = 98.6 ft-kips 8 9.5 Design 617 From the Zx table, fb Mn = fb Mp = 166 ft-kips > 98.6 ft-kips (OK) After the concrete cures and composite behavior has been achieved, wD = wslab + wbeam = 0.4375 + 0.026 = 0.4635 kips兾ft wu = 1.2 wD + 1.6 wL = 1.2(0.4635) + 1.6(1.200) = 2.476 kips兾ft Mu = 1 1 wu L2 = (2.476)(30)2 = 279 ft-kips 8 8 Before computing the design strength of the composite section, we must first determine the effective slab width. Furthermore, the most commonly used of these shapes, the double angle, is a built-up shape, and we postpone consideration of it until Section 4.9. For singly symmetrical shapes, the flexural-torsional buckling stress Fe is found from AISC Equation E4-5. Since there will be a stiffener at each concentrated load, there is no need to investigate the resistance of the web to these loads. They are not covered here. From Figure 10.17, the moment of inertia about the axis of bending is Ix = (5兾16) (62)3 + 2(1.5) (18)(31.75)2 = 60, 640 in.4 12 and the elastic section modulus is I 60 , 640 Sx = x = = 1866 in.3 c 32.5 An examination of AISC Equations F5-7 and F5-10 shows that for a girder with a symmetrical cross section, the flexural strength will never be controlled by tensionflange yielding; therefore only compression-flange buckling will be investigated. It is written and kept current by an AISC committee comprising structural engineering practitioners, educators, steel producers, and fabricators. 2″ 3″ 2″ 65k 6″ 2″ 4″ 3″ 2″ FIGURE P8.2-3 8.2-4 The fasteners in the connection of Figure P8.2-4 are placed at the usual gage distance (see Figure 3.24). 4(5) The second condition will give the smallest net area: ANSWER An = twn = 0.75(13.52) = 10.1 in.2 Equation 3.2 can be used directly when staggered holes are present. Load combination (4) will be more critical than (3), so combination (3) can be FIGURE 6.17 56k 6.7 Members in Unbraced Frames 331 eliminated. 4.4 Local Stability 125 AISC Specification Section E7.1 gives the procedure for calculating Qs for slender unstiffened elements. With the exception of the KCS joists, all open-web steel joists are designed as simply supported trusses with uniformly distributed loads on the top chord. (b兾t and h兾t ratios for HSS are given in the Manual in Part 1, “Dimensions and Properties.”) 5.16 FIGURE 5.52 Bending Strength of Various Shapes 277 b h t t b. Alternatively, the bending moments and beam reactions can be found by treating the top chord as a continuous beam with supports at the panel points. The maximum tensile stress can be computed from the flexure formula ft = Mc I where I is the moment of inertia about the centroidal axis of the area consisting of the total throat area of

the weld, and c is the distance from the centroidal axis to the farthest point on the tension side. Economy usually means minimum weight—that is, the minimum amount of steel. To facilitate the connection of beams to other beams so that the top flanges are at the same elevation, a short length of the top flange of one of the beams may be cut away, or coped. Problems Metal deck: Built-up roof: Purlins: Snow: Truss weight: 107 4 psf of roof surface 12 psf of roof surface 3 psf of roof surface (estimated) 20 psf of horizontal projection 5 psf of horizontal projection (estimated) FIGURE P3.8-4 3.8-5 Use A36 steel and design sag rods for the truss of Problem 3.8-4. Figure 6.18 shows the axial loads and bending moments calculated for combinations (2) and (4). The deflection itself can cause problems if elements attached to the beam can be damaged by small distortions. FIGURE 4.4 4.2 Column Theory 113 The solution to the differential equation is therefore n π x⎞ y = B sin⎛ ⎝ L ⎠ and the coefficient B is indeterminate. or 3 in. The closest required spacing of 3 inches. 6.9 Trusses with Top-Chord Loads between Joints 361 FIGURE 6.29 Bending is about the x axis, and the member is braced against sidesway, so Mnt = 3.0 ft-kips, M t = 0 Because there is a transverse load on the member, use Cm = 1.0 (the Commentary approach will not be used here). If so, use A36 steel and determine the required dimensions. Use a 3⁄16-inch fillet weld on each side of the plate. That is, for a function f (x), +∞ ∫−∞ f (x) dx = 1.0 which means that the probability that one of the sample values or events will occur is 1.0. The probability of one of the events between a and b in Figure 2.2 equals the area under the curve between a and b, or b ∫a f (x) dx When a theoretical probability density function is used, the following notation is conventional: m = mean s = standard deviation FIGURE 2.2 32 Chapter 2 Concepts in Structural Steel Design The probabilistic basis of the load and resistance factors used by AISC is presented in the ASCE structural journal and is summarized here (Ravindra and Galambos, 1978). As a general rule, most of the moment transfer is through the beam FIGURE 8.37 524 Chapter 8 Eccentric Connections flanges, and most of the moment capacity is developed there. 7.3 BEARING STRENGTH, SPACING, AND EDGE-DISTANCE REQUIREMENTS Bearing strength is independent of the type of fastener because the stress under consideration is on the part being connected rather than on the fastener. In each case, Cb will be large enough so that Mp 鼻Ωb will control; therefore, Mn 鼻Ωb = 237 ft-kips. The 20-kip load is a service load, with a live load-to-dead load ratio of 2.0. Use an elastic analysis and assume that the shear strength of the base metal is adequate. The yield stress is a function of the flange thickness, which for this shape is 0.720 in. 1.09 b. or Use b = 90 in. Much better agreement is obtained if the force Bc is shifted toward the stem of the tee by an amount d鼻2, where d is the bolt diameter. 10.4-3 Determine the nominal flexural strength of the following welded shape: The flanges are 7⁄8 inch × 12 inches, the web is 3⁄8 inch × 60 inches, and the member is simply supported, uniformly loaded, and has a span length of 40 feet. As another example, consider the addition of the following numbers, both accurate to four significant figures: 36,000 + 1.240 = 36,001.24 The result should be recorded as 36,000 (four significant figures). studs. The uniform load is a service dead load that includes the beam weight, and the concentrated load is a service live load. The factor of safety between first yielding and the fully plastic state can be expressed in terms of the section moduli. FIGURE 7.44 * If end returns are used in certain types of connections, AISC Commentary J2.2b places restrictions on their length. For the transverse weld, use Rn = 1.5 × 4.640 = 6.960 kips鼻in. At failure, the bolts are assumed to have reached their ultimate tensile stress and have a strength of Pt = Ft A b where Ft is the ultimate tensile stress of the bolt and Ab is the bolt area. The lightest W14 that is a possibility is a W14 × 53, with p = 4.45 × 10−3 and bx = 5.26 × 10−3. Case 1: For hot-rolled I shapes with h E ≤ 2.24 tw Fy The limit state is shear yielding, and (AISC Equation G2-2) Cv = 1.0 fv = 1.00 Ωv = 1.50 Most W shapes with Fy ≤ 50 ksi fall into this category (see User Note in AISC G2.1[a]). and K = 0.65. Prior to the 2005 Specification and Manual of the American Institute of Steel Construction, load and resistance factor design (LRFD) was covered by the 1999 AISC Specification and LRFD Manual of Steel Construction, Third Edition. 0.85 fc′b 0.85(4)(90) y= d a 15.7 1.255 + 4.75 − = 11.97 in. The area required for strength is A = F Fy 鼻Ω ⎛ 2P ⎞ 2. (< 36 in. In the direct analysis method, member stiffnesses are reduced, and an effective length factor of K = l is used both in the analysis and in computing the available strength from AISC Chapter 4. LRFD SOLUTION Compute the strong axis bending moments. The total length required is 39.6 kips = 7.11 in. What is the maximum service load Pa that can be supported? 16 From AISC J2.2b, Maximum size = 1 1 7 − = in. Extended End-Plate Moment Connections. Therefore, use of the theoretical values in these two cases is conservative. 1964. Determine the distance y from the top of the shape to the horizontal plastic neutral axis. In each combination, one of the effects is considered to be at its “lifetime maximum” value and the others at their “arbitrary point in time” values. • Material on the moment amplification method has been expanded. Example 3.11 illustrates that once the required area has been determined, the procedure is the same for both LRFD and ASD. Figure C-D3.2 shows some special cases for x−, including channels and I-shaped members connected through their webs. This part contains details on standard hot-rolled shapes, pipe, and hollow structural sections, including all necessary cross-sectional dimensions and properties such as area and moment of inertia. This is not a disadvantage; in fact, it will be the most economical alternative. If the requirements are especially severe, then a built-up section, such as one of those shown in Figure 1.8, may be needed. Steel Design Guide Series, No. 16. For the case shown, the bearing load would be on the left side of each hole. 6.7-2 A W14 × 82 of A992 steel, 16 feet long, is used as a column in an unbraced frame. For more unusual situations, 5.9 225 Deflection FIGURE 5.22 standard analytical methods such as the method of virtual work may be used. For rupture, the nominal strength is Rn = 0.6Fu Anv (AISC Equation J4-4) where 0.6Fu = shear ultimate stress Anv = net area of the shear surface = tL (for material adjacent to a weld) For LRFD, f = 0.75, and for ASD, Ω = 2.00. 84.3 kips 2-3 a. Account for the beam weight after a selection has been made. 9.7-2 A composite floor system uses formed steel deck of the type shown in Figure P9.7-2. There are two lines of bolts, so the connected leg must be at least 5 inches long (see the usual gages for angles in Figure 3.24). 0.707 Fnw 0.707 Fnw 0.7077(0.6 × 70) 2 502 Chapter 8 Eccentric Connections Alternatively, for E70 electrodes, Rn鼻Ω = 0.9279 kips鼻in. × 2 in., placed as shown in Figure 9.11. High-alloy or specialty steels: similar in composition to the low-alloy steels but with a higher percentage of the components added to iron and carbon. (This number also satisfies Equation 4.14.) ANSWER ASD SOLUTION Use 2L 31⁄2 × 3 × 1⁄4 SLBB with three intermediate connectors within the 14-foot length. 2 2 1 1 n′ = db f = 10.0(10.0) = 2.5 in. Let I x = I web + I flanges ≈ 1 1 t w h 3 + 2 A f y 2 ≈ t w h 3 + 2 A f (hh 鼻2)2 12 12 (10.4) where Af = cross-sectional area of one flange y = distance from the elastic neutral axis to the centroid of the flange The contribution of the moment of inertia of each flange about its own centroidal axis was neglected in Equation 10.4. The section modulus can be estimated as Sx = I x t w h 3鼻12 2 A f (h鼻2)2 t w h 2 ≈ + = + Af h c h鼻 2 h鼻 2 6 If we assume that compression flange buckling will control the design, we can find the required section modulus from AISC Equation F5-7: M n = Rpg Fcr S xc Required S xc = M n req Rpg Fcr where Mn req is the required nominal moment strength. Based on this one data point, what is the modulus of elasticity? Σy 12.85 The maximum permissible live-load deflection is L 30(12) = = 1.00 in. Other shapes are sometimes used as flexural members, however, and this section provides a summary of some of the relevant AISC provisions. λ= bf 18 = = 6.0 2t f 2(1.5) λ p = 0.38 E 29, 000 = 0.38 = 10.79 Fy 36 Since l < lp, there is no flange local buckling. The plates are notched, or clipped, at the inside top and bottom corners so as to FIGURE 10.3 10.2 General Considerations 669 avoid the flange-to-web welds. For a given material, the load capacity is a function of the slenderness ratio only. Both components are of A242 steel. If the slenderness ratio makes this member an inelastic column, what is the stiffness reduction factor, tb? All essential computations are included, however. The shear and tensile strengths for bearing-type bolts are based on test results (Chesson et al., 1965) that can be represented by the elliptical interaction curve shown in Figure 7.31. Lst 19.7 552 Chapter 8 Eccentric Connections Check the base metal shear strength of the column web. This relationship is illustrated in Figure 5.19. The allowable stress approach will be used. No slip is permitted. When the reactions are added to the loads that are directly applied to the joints, the loading condition shown in Figure 6.29 is obtained. Design the steel anchors: a. FIGURE 5.44 LRFD SOLUTION The factored load is Pu = 1.2D + 1.6L = 1.2(98) + 1.6(145) = 349.6 kips 264 Chapter 5 Beams Compute the required bearing area. 5 kips 694 Chapter 10 Plate Girders This is the same as the required Vn from the LRFD solution. All of the connections were made by riveting, so there was no way to attach the flange directly to the web, and additional cross-sectional elements were introduced for the specific purpose of transmitting the load from one component to the other. There are some limiting values for the effective area: For bolted splice plates, Ae = An ≤ 0.85Ag. This limit is given in a user note and is from a requirement in Chapter J of the Specification “Design of Connections.” For open cross-sectional shapes (such as W, M, S, C, HP, WT, and ST) and (angles), the value of U need not be less than the ratio of the connected element gross area to the total gross area. In the direction perpendicular to the frame, Ky = 1.0. There are no transverse loads on the member. Create a table of stress and strain values. 180k 20⬘ FIGURE P4.8-3 4.8-4 Select an American Standard Channel for the compression member shown in Figure P4.8-4. 451 ft-kips b. The load is a service load consisting of 33% dead load and 67% live load. 2 2 3 Rn = 1.2 c tFu = 1.2(1.094) ⎛⎜ ⎞⎟ (58) = 28.55 kips ⎝8⎠ 3 3 Upper limit = 2.4 dtFu = 2.4 ⎛⎜ ⎞⎟ ⎛⎜ ⎞⎟ (58) ⎝ 4⎠ ⎝ 8⎠ c = e − = 39.15 kips > 28.55 kips ∴ Usee Rn = 28.55 kips for this bolt. The total deflection must not exceed L鼻240. This type of failure is caused by a combination of flexural buckling and torsional buckling. Thus the block shear strength will be a limiting value of the reaction. The total strength will consist of the strength prior to buckling plus the postbuckling strength deriving from tensionfield action. The top number is the total load capacity in pounds per foot. For example, using LRFD and U.S. Customary units, a 52G9N10.5F is 52 inches deep, provides for 9 equal joist spaces on the top chord, and will support 10.5 kips of factored load at each joist location. For all but the simplest of structures, a computerized frame analysis is required to obtain the bending moments and axial loads. In addition, the riveting operation was noisy and somewhat dangerous because of the practice of tossing the heated rivet from the point of heating to the point of installation. This requirement is given in Chapter J (“Connections, Joints, and Fasteners”), Section J7 (Figure 3.38c): f = 0.75, Ω = 2.00, Pn = 1.8Fy Apb (AISC Equation J7-1) 4. Burgett, Lewis B. Of particular interest is the material on combined axial compression and flexure, which is the subject of Chapter 6 of this textbook, “Beam–Columns.” Parts 7 through 15 cover connections: Part 7. The amplification factor B1 was assumed to be equal to 1.0 for purposes of selecting a trial shape; B1 could then be evaluated for this trial shape. Because of the length constraint, however, try a slightly larger weld. Deflections: The computation of deflections is the same for LRFD and ASD. In hot-rolled I-shaped beams, these two conditions will be met if the steel yield stress is no greater than 65 ksi and the shape is compact—that is, if bf E ≤ 0.38 2t f Fy and h E ≤ 3.76 tw Fy To prevent lateral buckling, the value of Lb, the unbraced length of the compression flange, is limited to Lpd adjacent to plastic hinge locations. Assume that the threads are in the plane of shear and that slip of the connection is permissible. It is permissible to use a cross-sectional shape that does not satisfy the width-tothickness ratio requirements, but such a member may not be permitted to carry as large a load as one that does satisfy the requirements. Solving for td , we get td = Rn 鼻Ω 89.78 − 53.35 = = 0.341 in. A36 steel 11⁄2″ 3″ 3″ 11⁄2″ 2 @ 11⁄2″ t = 3⁄8″ t = 1⁄2″ t = 3⁄8″ PD = 35k PL = 86k FIGURE P7.6-2 7.6-3 A WT 7 × 19 of A572 Grade 50 steel is used as a tension member. The “target” values of b shown in Table 2.1, selected and used in computing both load and resistance factors for the AISC Specification, were based on the recommendations of Ravindra and Galambos (1978), who also showed that φ = Rm −0.55βVR e Rn 34 Chapter 2 TABLE 2.1 Target Values of b Concepts in Structural Steel Design Loading Condition Type of Component D + (L or S) D+L+W Members Connections 3.0 4.5 2.5 4.5 D+L+E 1.75 4.5 where Rm = mean value of the resistance R Rn = nominal or theoretical resistance VR = coefficient of variation of R 2.6 STEEL CONSTRUCTION MANUAL Anyone engaged in structural steel design in the United States must have access to AISC’s Steel Construction Manual (AISC, 2011a). A36 steel is used for the plates, and A992 steel is used for the beam and column. Use a composite section that consists of the steel shape and the reinforcing steel, subject to the following conditions: a. 5.14 Beam Bearing Plates and Column Base Plates 259 Determine the value of b required to prevent web crippling. Clip the inside corners to avoid the fillets at the column flange-to-web intersection. Ultimate Strength Analysis The foregoing procedure is relatively easy to apply but is inaccurate—on the conservative side. This textbook was written under the assumption that you would have access to the Manual at all times. A secondary consideration in the design of tension members is slenderness. From the Zx table, a W18 × 71 is seen to be a compact shape (no footnote to indicate otherwise). In regions where seismic forces are large, the connection shown in Figure 8.37a requires special design procedures (FEMA, 2000). Dead load deflection can also be accounted for by pouring a variable depth slab with a level top surface, the variable depth being a consequence of the deflection of the beam (this is referred to as ponding of the concrete). In those examples, the LRFD solution is given first. As discussed in Chapter 4, shapes with slender compression elements are identified with a footnote (footnote c). The web is compact for all shapes in the Manual for Fy ≤ 65 ksi; therefore, a W14 × 68 is compact for Fy = 50 ksi. A possible treatment at the peak or ridge is shown in Figure 3.31a. The concrete has a 28-day compressive strength of 4 ksi and the total slab thickness is 4 inches. Once the concrete has cured, the temporary shoring can be removed, and the weight of the slab, as well as any additional loads, will be carried by the composite beam. Figure 5.10 further illustrates the effects of local and lateral-torsional buckling. The strength corresponding to each of these three limit states must be computed, and the smallest value will control. But the design strength cannot exceed fbMp , which is only 398 ft-kips (obtained from the chart), FIGURE 5.30 5.10 Design 237 so the actual design strength must be taken as fb Mn = fbMp = 398 ft-kips < Mu = 406.1 ft-kips (N.G.) For the next trial shape, move up in the charts to the next solid curve and try W18 × 55. Shear on the effective area (Figure 3.38b): fsf = 0.75, Ωsf = 2.00, Pn = 0.6Fu Asf (AISC Equation D5-2) 3. Pa = D + L = 100 + 300 = 400 kips Try Fcr = 33 ksi (an arbitrary choice of two-thirds Fy): Required Ag = Pa 400 = = 20.2 in.2 0.6 Fcr 0.6(33) Try a W18 × 71: Ag = 20.9 in.2 > 20.2 in.2 KL 26 × 12 = = 183.5 < 200 rmin 1.70 (OK) (OK) 4.6 Fe = 4.71 Since Design 137 p 2E p 2 (29, 000) = = 8.5 ksi (KL 鼻r)2 (183.5)2 E 29, 000 = 4.71 = 113 Fy 50 KL E > 4.71 , AISC Equation E3-3 applies. 7.6-5 A plate 1⁄2 × 61⁄2 of A36 steel is used as a tension member as shown in Figure P7.6-5. The AISC Specification does not require end returns.∗ Small welds are generally cheaper than large welds. Assume that the member will be connected to a 3⁄8-inch-thick gusset plate with a single line of five 7⁄8-inch diameter bolts. Referring to Figure 10.14, the moment of inertia about an axis in the plane of the web is I = ∑ (I + Ad 2) = 2 ⎡ 0.75(7.5)3 4.5(3鼻8)3 7.5 3鼻8 ⎞ ⎤ 4 +4⎢ + 7.5(0.75) ⎛ + ⎥ = 454.3 in. We will use the approach of Design Guide 4 in this book and consider only the four-bolt unstiffened end-plate connection. Most of the AISC Specification provisions for welds have been taken from the Structural Welding Code of the American Welding Society (AWS, 2010). By interpolation, Mn = 318 ft-kips Ωb which compares favorably with Example 9.2. Allowable strength = 318 ft-kips ANSWER EXAMPLE 9.12 Use the Tables in Part 3 of the Manual and select a W-shape of A992 steel and steel headed stud anchors for the following conditions: The beam spacing is 5 feet 6 inches, and the span length is 30 feet. Figure 10.15 shows the loading, shear, and bending moment diagrams for the service loads (verification of the values is left as an exercise for the reader). Design Examples, v.14.0. www.aisc.org. The strategy will be to select a bolt size for trial, determine the number required, and then try a different size if the number is too large or too small. fcPn = 1190 kips a. 1. Having established this objective, the engineer must decide how to do it safely, which is where different approaches to design come into play. In this equation, y is defined as the axis of symmetry (regardless of the orientation of the member), and flexural-torsional buckling will take place only about this axis (flexural buckling about this axis will not occur). Guide to Stability Design Criteria for Metal Structures, 6th ed. Steel anchors. In this book, we assume that that is the case, and we will use tb = 1.0. Keep in mind that in this book, we do not perform any structural analyses; we only use the results of the analyses. These aids are not reproduced in this book, because one objective is to encourage familiarity with the Manual. 1961. r Fy Fcr = 0.877 Fe = 0.877(21.42) = 18.79 ksi f c Pn = f c Fcr Ag = 0.90(18.79)(38.3) = 648 kips > 600 kips (OK.) This shape is not slender (there is no footnote in the dimensions and properties table to indicate that it is), so local buckling does not have to be investigated. The Structural Tee is produced by splitting an I-shaped member at middepth. Problems 2″ 3″ 3″ 3″ 465 2″ t = 5⁄8″ FIGURE P7.6-1 7.6-2 Determine the total number of 7⁄8-inch, Group A bearing-type bolts required for the tension splice in Figure P7.6-2. New York: Marcel Dekker. Joist girders are designed to support open-web steel joists. Superimposed on the actual distribution is the average stress in the web, V鼻Aw, which does not differ much from the maximum web stress. The computation of Qa for slender stiffened elements is given in AISC E7.2 and is slightly more complicated than the procedure for unstiffened elements. This is because for noncompact shapes, the tablulated values of fbMp and Mp鼻Ωb are based on flange local buckling and not the plastic moment (see Section 5.6). Equivalently, if a probability density function of R鼻Q, the factor of safety, is used, survival is represented by values of R鼻Q greater than 1.0. The corresponding probability of failure is the probability that R鼻Q is less than 1; that is, ⎡⎛ R ⎞ ⎤ PF = P ⎢⎜ ⎟ < 1 ⎥ ⎣⎝ Q ⎠ ⎦ Taking the natural logarithm of both sides of the inequality, we have ⎡ ⎛ R⎞ ⎤ ⎡ ⎛ R⎞ ⎤ PF = P ⎢ ln⎜ ⎟ < ln 1 ⎥ = P ⎢ ln⎜ ⎟ < 0 ⎥ ⎝ ⎠ ⎝ ⎠ Q Q ⎣ ⎦ ⎣ ⎦ The frequency distribution curve of ln(R鼻Q) is shown in Figure 2.3. The standardized form of the variable ln(R鼻Q) can be defined as ⎛ R⎞ ⎡ ⎛ R⎞ ⎤ ln⎜ ⎟ − ⎢ ln⎜ ⎟ ⎥ ⎝ Q ⎠ ⎣ ⎝ Q ⎠ ⎦m U= s ln(R鼻Q) FIGURE 2.3 2.5 Probabilistic Basis of Load and Resistance Factors 33 where ⎛ R⎞ ⎡ ⎛ R⎞ ⎤ ln⎜ ⎟ ⎢ ln⎜ ⎟ ⎥ = the mean value of ln ⎜⎝ Q ⎟⎠ ⎣ ⎝ Q ⎠ ⎦m ⎛ R⎞ s ln(RQ) = standard deviation of ln ⎜ ⎟ ⎝ Q⎠ This transformation converts the abscissa U to multiples of standard deviations and places the mean of U at U = 0. The load P is a service live load. From symmetry, the left vertical reaction is VL = wu L 20.8(30) = = 312 kips 2 2 The middle of this unbraced length is at 15⁄2 = 7.5 feet from the left end. An economical structure requires an efficient use of materials and construction labor. Any reinforcement (buildup outside the hypotenuse of the triangle) or penetration is neglected. 2 2 From Equation 5.14, t≥ ANSWER 3.34 Ra n 2 = B b Fy 3.34(71.24)(3.810)2 = 1.27 in. The factored loads are the loads that bring the structure or member to its limit. From Figure 5.15b, Cb is 1.30 for the load and lateral support conditions of this beam. ⎝ 16 ⎠ Use a center-to-center spacing of 41⁄2 inches, resulting in a clear spacing of 4.5 – 1.5 = 3 in. Other important considerations include serviceability (how well the structure performs in terms of appearance and deflection) and economy. 2 2 8.8 h1 = d − d b Req′d = End Plate Connections 567 t fb 0.425 − t f b − p fi = 17.7 − − 0.425 − 2 = 15.06 in. Determine the axial compressive design strength of column AB. Use A992 steel and select a W14 shape for the columns. Failure of the tee is assumed to occur when plastic hinges form at section a–a, the face of the stem of the tee, and at b–b, the bolt line, thereby creating a beam mechanism. (We covered block shear in Chapters 3 and 5, and it is described in AISC J4.3.) Depending on the type of connection and loading, connection fittings such as gusset plates and framing angles may require an analysis for shear, tension, bending, or block shear. AISC J3.8 requires that shear and bearing be checked in slip-critical connections. The Specification deals with combined loading primarily through the use of interaction formulas, which account for the relative importance of each load effect in relation to the strength corresponding to that effect. If there is no tension-field action, the second term in Equation 10.2 is omitted, resulting in Vn = 0.6 Fy AwCv (AISC Equation G2-1) Solution of AISC Equations G2-1 (without tension field) and G3-2 (with tension field) is facilitated by curves given in Part 3 of the Manual. Noncompact shapes are identified in the Zx table by an “f” footnote (this same identification is used in the dimensions and properties tables). Furthermore, the manner in which the computations are grouped will influence the final result. Lightweight concrete with a unit weight of 115 pcf is used • construction load = 20 psf • partition load = 20 psf • live load = 225 psf • Fy = 50 ksi and f c′ = 4 ksi 662 Chapter 9 Composite Construction A cross section of the formed steel deck is shown in Figure P9.8-9. and the strength of the transverse weld is 1.5(3.712) = 5.568 kips鼻in. In Example 5.12, the value of Cb is the same (to three significant figures) for both the factored and the unfactored moments. B. Although the load predicted by this equation is actually a lower bound on the true value of the critical load, the difference is slight (Bleich, 1952). To determine the required weld size, let 0.9279D = 1.530, D = 1.65 sixteenths of an inch The minimum weld size from AISC Table J2.4 is 3⁄16 inch, based on the thinner connected part (the shear plate). To ensure stability of the top chord, the floor or roof deck must be attached in such a way that continuous lateral support is provided. Bringing our world enriched living with advanced it solutions. The loading history of the beam, from working load to collapse load, is traced in Figure A.4a–d. Estimate the proportional limit. For the transverse weld, use fRn = 1.5 × 6.960 = 10.44 kips鼻in. As shown in Figure 5.43, this plate segment is assumed to be fixed at the Chapter 5 Beams bf FIGURE 5.43 2 bf d Free d Simply supported Fixed 262 Simply supported (a) Area of Plate Considered (b) Approximated Size and Edge Conditions web, simply supported at the flanges, and free at the other edge. 2 2 2 Mu 2(173.2 × 12) = = 0.753 in. If the force in a web member is small, single angles can be used, although doing so eliminates the plane of symmetry from the truss and causes the web member to be eccentrically loaded. This requires a knowledge of the stress in the effective area, so iteration is required. These organizations have merged into the International Code Council, and the new code has replaced the three regional codes. Use A992 steel and the following load data (wind is not considered in this example): Purlins: Snow: Metal deck: Roofing: Insulation: FIGURE 3.34 M8 × 6.5 20 psf of horizontal projection 2 psf 4 psf 3 psf 3.8 SOLUTION Tension Members in Roof Trusses 85 Calculate loads: Snow = 20(40)(20) = 16,000 lb Dead load (exclusive of purlins) = Deck Roof Insulation Total Total dead load = 9(40)(20) = 7200 lb Total purlin weight = 6.5(20)(9) = 1170 lb 2 psf 4 _3___ 9 psf Estimate the truss weight as 10% of the other loads: 0.10(16,000 + 7200 + 1170) = 2437 lb Loads at an interior joint are 7200 2437 + + 6.5(20) = 1335 lb 8 8 16,000 S= = 2000 lb 8 D= At an exterior joint, the tributary roof area is half of that at an interior joint. I can be contacted at William T. Use Fy = 46 ksi. From Table 8-8, the nominal strength of the connection is given by Rn = CC1D For ASD, Rn = Pa Ω so ΩPa = CC1D and the required value of D is D= 2.00(40) ΩPa = = 3.99 sixteenths CC1 1.67(1.0)(12) The required weld size is, therefore, 3.99 = 0.249 in. The reliability index is a function of both the load effect Q and the resistance R. Compute the moments acting before and after the concrete cures. For tension members designed by LRFD, the requirement is Pu ≤ ft Pn or ft Pn ≥ Pu where Pu is the sum of the factored loads. 4 4 As a conservative simplification, let l = 1.0, giving = max(m, n, ln′) = max(1.75, 2.5, 2.5) = 2.5 in. 2): 41–72. The approximate secondorder analysis method of AISC Appendix 8 can be used, making this a direct analysis method. Twenty 7⁄8-inch, Group A, Type N, bearing-type bolts are used: eight at each flange, and four in the web. Use E80XX electrodes with steels that have a yield stress of 60 ksi or 65 ksi. The bearing strength based on this spacing is 3 rn = 48.6t = 48.6 ⎛⎜ ⎞⎟ = 18.2 kips鼻bolt ⎝ 8⎠ Ω For the holes nearest the edge of the plate, the bearing strength from Table 7-5 is 3 rn = 27.2t = 27.2 ⎛⎜ ⎞⎟ = 10.2 kips鼻bolt ⎝ Ω 8⎠ Using the minimum of shear and bearing strengths for each bolt location, the total connection strength is Rn = 2(edge bolt strength) + 4(inner bolt strenggth) Ω = 2(10.2) + 4(16.2) = 85.2 kips > 72 kips (OK) The tentative connection design is shown in Figure 7.17 and will now be checked for block shear in the gusset plate (the geometry of the failure block in the channel is identical, but the gusset plate is thinner). For ASD, the available strength of the connection is Rn鼻Ω = 36.51鼻2.00 = 18.3 kips. The stress then remains constant, even though the strain continues to increase. ANSWER The maximum stress in the steel is 21.1 ksi tension, and the maximum stress in the concrete is 0.651 ksi compression. Using AISC notation gives λ= b b f 鼻2 b f = = 2t f t tf where bf and tf are the width and thickness of the flange. Lincoln Arc Welding Foundation. Use the 0.2% offset method to determine the yield stress. Other dead load is 5 psf. The relationship between shear strength and the web width-to-thickness ratio is analogous to that between flexural strength and the width-to-thickness ratio (for FLB) and between flexural strength and the unbraced length (for LTB). As far as floor loads are concerned, this transmission of loads is accomplished as follows: 1. The horizontal and vertical components of force in each bolt resulting from the concentric load are pcx = 22.36 44.72 = 2.795 kips ← and pcy = = 5.590 kiips ↓ 8 8 8.2 Eccentric Bolted Connections: Shear Only 483 For the couple, ∑ (x 2 + y 2) = 8(2.75)2 + 2[(6)2 + (1)2 + (2)2 + (5)2] = 192.5 in.2 480.7(6) My pmx = = = 14.98 kips ← 192.5 ∑(x 2 + y2) Mx 480.7(2.75) = = 6.867 kips ↓ pmy = 192.5 ∑(x 2 + y2) ∑ px = 2.795 + 14.98 = 17.78 kips ← ∑ py = 5.590 + 6.867 = 12.46 kips ↓ p = (17.78)2 + (12.46)2 = 21.7 kips (see Figure 8.6c) ANSWER The critical fastener force is 21.7 kips. 3): 161–9. per stiffener. Concrete strength is fc′ = 4 ksi, and A992 steel is to be used. Adequate: Required size = 1.46 sixteenths of an inch a. 5 sp @ 4″ 21⁄2″ 7.9-5 21⁄2″ 1′-1″ D = 55k L = 145k FIGURE P7.9-5 Fillet Welds 7.11-1 Determine the maximum service load that can be applied if the live load-to-dead load ratio is 3.0. Investigate all limit states. A single bolt and a portion of the connected parts will be isolated and examined both before and after loading. A service load of 400 kips, with equal parts of dead and live load, must be supported. 2 16 16 516 Chapter 8 Eccentric Connections Try w = 1⁄4 inch. There is a construction load of 20 psf and a live load of 175 psf. The shear rupture stress is taken as 60% of the tensile ultimate FIGURE 3.22 Plate tension member d f e g Block shear in gusset plate h j i k Block shear in tension member 66 Chapter 3 Tension Members stress, so the nominal strength in shear is 0.6Fu Anv and the nominal strength in tension is Fu Ant , where Anv = net area along the shear surface or surfaces Ant = net area along the tension surface This gives a nominal strength of Rn = 0.6Fu Anv + Fu Ant (3.3) The AISC Specification uses Equation 3.3 for angles and gusset plates, but for certain types of coped beam connections (to be covered in Chapter 5), the second term is reduced to account for nonuniform tensile stress. Compute the lower-bound moment of inertia, and determine the deflection for a service live load of 1 kip鼻ft. Depending on the span length, open-web joists may be more economical than rolled shapes, although there are no general guidelines for making this determination. 0.85 fc′b 0.85(4)(90) 630 Chapter 9 Composite Construction The moment arm of the internal resisting couple is y= d a 15.7 1.255 +t− = + 4.75 − = 11.97 in. If the bending moment and axial force are from a second-order analysis, you can go straight to the interaction equations from AISC Specification Chapter H. For three fasteners in the direction of loading, U = 0.60. 2003. Thus member BD is subjected only to axial compression arising from the vertical loads. Check to see which axis controls the axial compressive strength. 7.4-5 a. a For ≤ 1.5, h E ⎛ h⎞ = 12.0 ⎜⎝ ⎟⎠ t w max Fy For (AISC Equation F13-3) a > 1.5, h 0.40 E ⎛ h⎞ = ⎜⎝ ⎟⎠ t w max Fy (AISC Equation F13-4) where a is the clear distance between stiffeners. For KL鼻r = 96.77 and Fy = 50 ksi, fcFcr = 22.67 ksi fcPn = fcFcr Ag = 22.67(21.8) = 494 kips b. 5.15-5(a), the beam is satisfactory: Result of interaction equation = 0.588. For joint A, Ginelastic = tb × Gelastic = 0.9877(1.52) = 1.50 For joint B, Ginelastic = 0.9877(1.36) = 1.34 ANSWER ASD SOLUTION From the alignment chart, Kx = 1.43. A36 is a ductile steel, with an elongation as defined by Equation 1.1 of 20% based on an undeformed original length of 8 inches. For ASD, Ω = 1.50. (OK) The cross-sectional area of one stud is Asa = π (0.5)2 = 0.1963 in.2 4 If we assume normal-weight concrete, the modulus of elasticity of the concrete is Ec = wc1.5 fc′ = (145)1.5 4 = 3492 ksi From AISC Equation I8-1, the nominal shear strength of one stud is Qn = 0.5 Asa fc′Ec ≤ Rg R p Asa Fu = 0.5(0.1963) 4(3492) = 11.660 kips Rg Rp Asa Fu = 1.0(0.75)(0.1963)(65) = 9.570 kips < 11.60 kips ∴ Use Qn = 9.570 kips. Munse, W. For line abdf, the net area is An = Ag – ∑ t w × (d or d ′) = 6.8 0– 0.5(1.0) × 2 = 5.8 0 in.2 For line abceg, (1.5)2 ⎤ ⎡ 2 An = 6.8 0– 0.5(1.0) – 0.5 ⎢1.0 – ⎥ – 0.5(11.0) = 5.413 in. The design value is the smallest of these values. Compute the plastic section modulus Z with respect to the minor principal axis. Determine whether this member satisfies the appropriate AISC Specification interaction equation. Determine the maximum stresses in the steel and concrete resulting from a positive bending moment of 160 ft-kips. Balance the welds. Compute the total deflection (there is no limit to be checked). 2): 33–35. FIGURE 1.6 14 Chapter 1 Introduction The American Standard, or S-shape, is similar to the W-shape in having two parallel flanges, a single web, and two axes of symmetry. The loads and moments are unfactored and are 25% dead load and 75% live load. The limiting value of h鼻tw is a function of the aspect ratio, a鼻h, of the girder panels, which is the ratio of intermediate stiffener spacing to web depth (see Figure 10.6). Use a single angle for the tension member. The dead load deflection is ΔD = 5 wD L4 5 (0.500鼻12)(30 × 12)4 = = 0.616 in. Deflection is a serviceability limit state, not one of strength, so deflections should always be computed with service loads. The allowable strength of the connection is 80.52 = 40.3 kips > 36 kips 2.00 (OK) The connection is satisfactory by ultimate strength analysis. The length is 12 feet. The AISC specification requires that the roof system have sufficient stiffness to prevent ponding, and it prescribes limits on stiffness parameters in Appendix 2, “Design for Ponding.” 5.10 DESIGN Beam design entails the selection of a cross-sectional shape that will have enough strength and that will meet serviceability requirements. We consider only I-shaped welded plate girders in this chapter. From Table 6-1, p = 3.16 × 10–3, bx = 4.58 × 10–3, and rx 鼻ry = 2.64. < 1.5 in. The ninth edition was followed by editions one through three of the LRFD-based manuals. The AISC Specification covers shear strength in Chapter G, “Design of Members for Shear.” In that coverage, the constants kv and Cv are used. The beams are spaced at 8 feet, and the Problems 661 span length is 36 feet. “Tests on Eccentrically Loaded Fillet Welds.” Department of Civil Engineering, University of Alberta, Edmonton. A comprehensive building design example is also included. Deflections caused by loads applied before the concrete cures must be computed with the moment of inertia of the steel shape. The maximum permissible live-load deflection is L鼻360. Use the slope of the best-fit line to estimate the modulus of elasticity. Standard weld sizes are specified in increments of 1⁄16 inch. The nominal tensile strength is then Rn = F′nt Ab (AISC Equation J3-2) Equation 7.26 will now be presented in the two design formats. The eccentricity is ex = 8 + 1.5 = 9.5 in. The average shearing stress in this case will be fv = P P = A π d 2鼻4 where P is the load acting on an individual fastener, A is the cross-sectional area of the fastener, and d is its diameter. Problems 3 bolts 103 3 bolts FIGURE P3.6-3 3.6-4 Select an S shape for the tension member shown in Figure P3.6-4. bf = 6.28 2t f 0.38 (from Part 1 of the Manual) 29, 000 E = 9.15 > 6.28 ∴ The flange is compact. FIGURE 7.35 FIGURE 7.36 7.11 Fillet Welds 443 Of the two major types of welds, fillet welds are the most common and are considered here in some detail. The numbers on the reference line, from left to right, are the weld size and length. PL 1⁄2 × 6 2 PL 5⁄16 × 3 P P 3″ FIGURE P7.11-4 7.11-5 Design a welded connection. The total load is Pa = D + L = 12 + 23 = 35 kips From the column load tables, select 2L 31⁄4 × 3 × 1⁄4 SLBB, weighing 10.8 lb鼻ft. If a test specimen is subjected to an axial load P, as shown in Figure 1.3a, the stress and strain can be computed as follows: ΔL P and ε = A L f = where f A e L ΔL = = = = = axial tensile stress cross-sectional area axial strain length of specimen change in length 1.5 Structural Steel 9 FIGURE 1.3 If the load is increased in increments from zero to the point of fracture, and stress and strain are computed at each step, a stress–strain curve such as the one shown in Figure 1.3b can be plotted. If the symbol for the type of weld is below the reference line, the weld is on the arrow side of the joint, that is, the part of the joint that the arrow is touching. From Figure 8.40, the force at the interface between the beam flange and the plate is H= M 140(12) = = 80.77 kips d 20.8 where d is the depth of the beam. Part 12 of the Manual, “Design of Fully Restrained (FR) Moment Connections,” gives guidance for extended end-plate moment connections, but the actual design 560 Chapter 8 Eccentric Connections procedures, along with examples, can be found in AISC Design Guide 4 (Murray and Sumner, 2003) and AISC Design Guide 16 (Murray and Shoemaker, 2002). Figure 5.18 shows the shearing stress distribution for a W shape. A 13.17 KL Kh 0.75(62) = = = 10.24 < 25 r r 4.539 ∴ Pn = Fy Ag = 36(13.17) = 474.1 kips φ Pn = 0.90(474.1) = 427 kips > 223.4 kips FIGURE 10.22 (OK) 4 708 Chapter 10 Plate Girders Since the load at midspan is smaller than the reaction, use the same stiffener at midspan. A 2(32 + 0.75) fr = (0.007582 R)2 + (0.01527 R)2 = 0.01705 R kips鼻 in. In Figure 5.49b, the twisting moment Pe is resisted by a couple consisting 5.15 Biaxial Bending 271 FIGURE 5.48 FIGURE 5.49 of equal forces acting at each flange. Use of this nomogram would result in an effective length factor somewhat less than 1.0, and some savings could be realized.* As with any design aid, the alignment charts should be used only under the conditions for which they were derived. Connections by transverse welds alone are not common. An advantage of this product is its versatility, since almost any conceivable cross-sectional shape can easily be formed. Tables 8-4 through 8-11 give available strength coefficients for various common combinations of horizontal and vertical weld segments based on an ultimate strength analysis. From Equation 5.22, Muy 12.40 1.034 Mux + = + = 0.705 < 1.0 (OK) f b Mnx fb Mny 42.9 4.968鼻 2 The shear is Vu = 0.4410(15) = 3.31 kips 2 From the uniform load tables, fvVn = 46.7 kips > 3.31 kips ANSWER ASD SOLUTION (OK) Use a C10 × 15.3. For dead load plus a roof live load, load combination 3 will control: qa = qD + qLr = 42 psf The width of roof surface tributary to each purlin is 15 10 = 7.906 ft 2 3 Then Purlin load = 42(7.906) = 332.1 lb鼻ft 3 (332.1) = 315.1 lb鼻ft Normal component = 10 1 (332.1) = 105.0 lb鼻ft Parallel component = 10 5.15 Biaxial Bending 275 and M ax = 1 2 1 wL = (0.3151)(15)2 = 8.862 ft-kips 8 8 With sag rods placed at the midpoint of each purlin, the purlins are two-span continuous beams with respect to weak axis bending. 340.6 a > 1.5, h 0.40 E 0.40(29, 000) ⎛ h⎞ = = = 322.2 ⎜⎝ ⎟⎠ t w max Fy 36 For min t w = 62 = 0.192 in. Dividing it by the cross-sectional area gives the buckling stress: Fcr = p 2 Et (KL鼻r)2 The rotational stiffness of a column in this state would be proportional to Et Ic 鼻Lc , and the appropriate value of G for use in the alignment chart is Ginelastic = ∑ Et I c 鼻Lc Et = Gelastic ∑ EI g 鼻Lg E Because Et is less than E, Ginelastic is less than Gelastic, and the effective length factor K will be reduced, resulting in a more economical design. The use of Manual Table 7-2 for this value is intuitive and is similar to Table 7-1 for shear strength. This behavior is analogous to bending about the x-axis of the double-angle shape. The floor beams are supported by girders, which in turn are supported by the columns. Shoring is not used, and the applied bending moments are as follows: from the beam weight, Mbeam = 13 ft-kips; from the slab weight, Mslab = 77 ft-kips; and from the live load, ML = 150 ft-kips. The compression flange has lateral support at the ends and at the points of application of the concentrated loads. > 6.756 kips鼻in. This chapter is devoted to simple connections. To determine the length of the stiffeners, first compute the distance between the stiffener-to-web weld and the web-to-flange weld (see Figure 10.9): 5 ⎞ Minimum distance = 4t w = 4 ⎛ = 1.25 in. Next, there is a length factor of 0.90 for connections no longer than 38 inches (for longer connections, this factor is reduced to 0.75). 466 Chapter 7 7.6-4 Simple Connections a. Try bp = 7 in. The tips of the flanges, for example, cool faster than the junction of the flange and the web. For the free body shown in Figure 3.36b, ∑ M E = 13.40(20) − 1.732(20) − 3.335(15 + 10 + 5) − 4 FIJ = 0 FI J = 33.33 kips For the gross section, Ft = 0.6Fy = 0.6(36) = 21.6 ksi Required Ag = FIJ 33.33 = = 1.54 in.2 21.6 Ft 88 Chapter 3 Tension Members For the net section, Ft = 0.5Fu = 0.5(58) = 29.0 ksi Required Ae = FIJ 33.33 = = 1.15 in.2 Ft 29.0 Try an MT6 × 5.4: Ag = 1.59 in.2 > 1.54 in.2 (OK) 1.86 U = 1− x = 1− = 0.7933 9 Ae = AgU = 1.59(0.7933) = 1.26 in.2 > 1.15 in.2 (OK) Assuming that the bottom chord is braced at the panel points, we get L 5(12) = = 106 < 300 r 0.566 ANSWER 3.9 (OK) Use an MT6 × 5.4. PIN-CONNECTED MEMBERS When a member is to be pin-connected, a hole is made in both the member and the parts to which it is connected and a pin is placed through the holes. The total applied load is Pa = D + L = 62.5 + 125 = 188 kips From the column load tables, the choices are as follows: W8: There are no W8s with Pn 鼻Ωc ≥ 188 kips. 288 Chapter 5 Beams 90k wD = 150 lb/ft 1′ 10′ FIGURE P5.8-4 Design 5.10-1 Use A992 steel and select a W shape for the following beam: • Simply supported with a span length of 25 feet • Continuous lateral support • Service dead load = 1.0 kips鼻ft • The service live load consists of a 35-kip concentrated load at the center of the span There is no limit on the deflection. Depending on the value of the axial load P, the amplified moment can be either larger or smaller than the end moment. AISC B4.3b specifies that any gage line crossing the heel of the angle be reduced by an amount that equals the angle thickness. Design of Fully Restrained (FR) Moment Connections. 19.94 A Check the unbraced length. The tension member is A572 Grade 50 steel and the gusset plate is A36. Allowable compressive strength = 262 kips. The gross area of one flange is Afg = tf bf = 0.535(6.53) = 3.494 in.2 The effective hole diameter is 3 1 7 + = in. This hinge is similar to an ordinary hinge except that the plastic hinge will have some moment resistance, much like a “rusty” hinge. ANSWER Use 8 inches total, 4 inches on each side. American Institute of Steel Construction (AISC): This specification provides for the design of structural steel buildings and their connections. 2010b. They are rarely used today. 44.0 kips 5.5-9 a. Knowledge of statically indeterminate structural analysis is not a prerequisite for the use of this book. Cambering will make the fit even more critical and is not recommended. Shortt Senior Acquisitions Editor: Randall Adams Senior Developmental Editor: Hilda Gowans Editorial Assistant: Tanya Altieri © 2013, 2007 Cengage Learning ALL RIGHTS RESERVED. If the member has slender elements, the local buckling strength must be computed using the provisions of AISC E7. Upper-bound theorem (kinematic theorem): The load that corresponds to an assumed mechanism must be greater than or equal to the collapse load. 322.2 Try a 5⁄16 ∂ 62 web plate. New York: HarperCollins. When columns are placed in a rectangular grid, the region between four columns is called a bay. 384 EI LB 384(29, 000)(1316) The following is a summary of the deflections: Immediate deflection, before composite behavior is attained: Δ1 + Δ2 = 0.6670 + 0.2218 = 0.889 in. “A Technical Note: Derivation of the LRFD Column Design Equations.” Engineering Journal, AISC 38 (no. We define the distance from the top of the steel shape to the centroid of the concrete as Y2, where Y2 = t − a 1.0 = 4.5 − = 4.0 in. The method used to calculate bolt tension when the load is not in the plane of the connection has been modified to be consistent with the approach used in the Manual. EXAMPLE 3.10 Compute the block shear strength of the tension member shown in Figure 3.23. Since h E < 2.24 , tw Fy Ωv = 1.50 and the allowable shear strength is Vn 184.8 = = 123 kips Ωv 1.50 222 Chapter 5 Beams From Example 5.6, the total service load is wa = wD + wL = 0.400 + 1.000 = 1.4 kips鼻ft The maximum shear is Va = wa L 1.4(45) = = 31.5 kips < 123 kips (OK) 2 2 Alternately, a solution in terms of stress can be done. There are two broad categories of failure: failure of the fastener and failure of the parts being connected. The requirements for proportioning the stiffeners for the no-tension-field case are different from those for the tension-field case. (For negative moment, this would be the four bolts on the bottom side. As far as equilibrium is concerned, the force can be moved to the shear center provided that a couple is added. This 5.11 Floor and Roof Framing Systems 239 task is greatly simplified by the moment of inertia selection tables in Part 3 of the Manual. Asa = π (0.75)2 鼻4 = 0.4418 in.2 Qn = 0.5 Asa fc′Ec ≤ Rg R p Asa Fu For one stud per rib, Rg = 1.0. In this book, we always conservatively use Rp = 0.6 with steel deck. The required axial compressive strength is Pa = Pnt + B2Pt = 200 + 0 = 200 kips 6.7 Members in Unbraced Frames 327 For KL = 1.0(15) = 15 feet, the allowable compressive strength from the column load tables is Pn = 441 kips Ωc Check the interaction formula: 200 Pa = = 0.4535 > 0.2 Pn 鼻Ωc 441 ∴ Use Equation 6.5 (A AISC Equation H1-1a). 398 Chapter 7 Simple Connections Shear strength: For one bolt, π(3鼻4)2 = 0.4418 in.2 4 Rn = Fnv Ab = 54(0.4418) = 23.86 kips鼻bolt Ab = Slip-critical strength: Because no slippage is permitted, this connection is classified as slip-critical. If tension-field action is permitted, ⎛ ⎞ 1 − Cv Vn = 0.6 Fy Aw ⎜ Cv + ⎟ ⎝ 1.15 1 + (a鼻h)2 ⎠ b. From Equation 7.14, the total bolt force, including prying, is 0.75(0) ⎛ 1.988 ⎞ ⎤ δα b ′ ⎤ ⎡ ⎡ = 30 ⎢1 + Bc = T ⎢1 + ⎥ ⎥ = 30 kips ⎢ ⎣ (1 + 0.75 × 0) ⎝ 3.469 ⎠ ⎦ ⎥ ⎦ ⎣ (1 + δα) a ′ ⎦ The prying force is q = Bc − T = 30 − 30 = 0 kips This indicates that the tee section is not flexible enough to produce prying action. Thus there are two maxima: maximum compressive stress in the top fiber and maximum tensile stress in the bottom fiber. From Equation 7.35, the shear yield strength per unit length (considering a web thickness on each side of the stiffener pair) is fRn = 0.6Fytw × 2 = 0.6(50)(0.360)(2) = 21.6 kips鼻in. The uniform load consists of a 4 kip鼻ft live load. ⎝ 16 ⎠⎟ The base metal shear strength is therefore 6.75 kips鼻in., which is greater than the required strength of 4.588 kips鼻in. 2 For the assumed weld configuration in Figure 8.27, 2(1)(6)3 6 = 36 in.4 , c = = 3 in. Structural Stability Research Council, Task Group 20. Any curve to the right is for a beam with exactly the required moment capacity, although for a larger unbraced length. Title: Structural Steel Design 4th Edition Solution Manual Keywords: Structural Steel Design 4th Edition Solution Manual Created Date: 9/5/2014 12:49:57 PM. 108鼻 2 9.5 Design 613 FIGURE 9.11 Either arrangement is satisfactory; either spacing will be between the lower and upper limits. The angle shapes are available in either equal-leg or unequal-leg versions. 2nd ed. The most common of these devices is a washer with protrusions on its surface. Use Equation 3.1 for U. This member is laterally braced at its ends, and bending is about the x-axis. The beams are spaced at 6 feet, and the span length is 25 feet. Show a sketch with a possible layout. Check the slip-critical load against the reduced slip-critical strength. Moreover, web crippling and web yielding are not factors in column base plates design. For a tee shape, it is located at the intersection of the centerlines of the flange and the stem. p fFt (h0 + h1) p (0.75)(90)(19.49 + 15.06) Try db = 7⁄8 inch. Up to this point, the number of steps is the same. Hybrid girders, in which the steel in the flanges is of a higher strength than that in the web or webs, are sometimes used. Use LRFD and select a W shape of A992 steel for the purlins. Since shear yielding controls (Cv = 1.0) and Ωv = 1.50, the allowable shear stress is Fv = 0.4Fy = 0.4(50) = 20 ksi The required shear strength (stress) is fa = ANSWER Va 31.5 = = 5.11 ksi < 20 ksi (OK) Aw 6.160 The required shear strength is less than the available shear strength, so the beam is satisfactory. The axial load and end moments obtained from a first-order analysis of the gravity loads (dead load and live load) are shown in Figure 6.17a. A practical approach to achieving an economical design is to prepare several alternatives and compare their costs, using estimates of material and fabricating costs. The “odd-add” rule tends to average out the rounding process when many numerical operations are involved, as in statistical methods, but that is not the case in most structural design problems. The allowable tensile strength of one bolt is B= Rn Ft Ab 90.0(0.6013) = = = 27.06 kips Ω Ω 2.00 The total applied load is D + L = 20 + 60 = 80 kips and the number of bolts required (without considering prying action) is 80鼻27.06 = 2.96. The determination of the stiffener spacing can be facilitated by the use of the design curves in Part 3 of the Manual; we illustrate this technique in Example 10.2. ANSWER The shear strength is inadequate. With this arrangement, the beam is conveniently held in position so that the flanges can be field welded to the column. The other difference for columns in end-plate connections is in the column web yielding strength. 4): 136–142. Figure 8.51 shows the pitch of the outer bolts (pfo) and the pitch of the inner bolts (pfi). For those interested in delving deeper, the Guide to Stability Design Criteria for Metal Structures (Ziemian, 2010) is a good starting point, and Buckling Strength of Metal Structures (Bleich, 1952) and Theory of Elastic Stability (Timoshenko and Gere, 1961) will provide the fundamentals of stability theory. PL 1⁄2 × 8 FIGURE P3.2-2 3.2-3 A C12 × 30 is connected with 1-in. 2-2 A column is subjected to the following loads: dead load = 26 kips, occupancy live load = 15 kips, roof live load = 5 kips, snow load = 8 kips, rain load = 5 kips, and wind load = 8 kips. Equation 4.2 can then be written as y ″ + Pcr y = 0 EI where the prime denotes differentiation with respect to x. An alternative approach is to use the thickness of the slab above the deck plus half the height of the rib as the thickness of concrete in computing the weight of the slab. The local flange bending strength of 88.20 kips is the smallest of the three limit states. flange and a 1-in. Determine the nominal strength based on the net area. 10.2 General Considerations 667 Before considering specific AISC Specification requirements, we need to examine, in a very general way, the peculiarities of plate girders as opposed to ordinary rolled beams. This part contains the AISC Specification and Commentary, a specification for high-strength bolts (RCSC, 2009), and the AISC Code of Standard Practice (AISC, 2010b). (OK) Re quired tf = 428 Chapter 7 Simple Connections Determine the prying force (this is not required). Problems 293 Holes in Beams 5.12-1 A W16 × 40 of A992 steel has two holes in each flange for 3⁄4-inch diameter bolts. For r • The weld connecting the stiffener to the web should have the capacity to transfer the unbalanced force. Although this building is subjected to wind, the resulting forces on the structure are resisted by members other than this particular column. Part (b) of Example 8.2 was worked with the aid of standard spreadsheet software. Compact shapes: The strength will be the smaller value for the limit states of yielding and LTB. The value of m will depend on FIGURE 7.12 7.5 Installation of High-Strength Bolts 395 the surface condition of the steel—for example, whether it is painted or whether rust is present. Filler plates are covered in more detail in the Commentary to AISC Section J3.8. In this book, we do not use fillers in any connections, so we will always use hf = 1.0. The resistance factor for LRFD and the safety factor for ASD are different for different types of bolt holes. If they are, use A36 steel and specify the required dimensions. The required shear strength is Va = wa L 4.584(30) = = 68.8 kips 2 2 From the Zx table, the available shear strength is Vn = 227 kips > 68.8 kips Ωv (OK) Check deflection. FIGURE 3.24 g1 g2 g Usual Gages for Angles (inches) Leg g g1 g2 8 7 6 5 4 31⁄2 3 21⁄2 2 13⁄4 11⁄2 13⁄8 11⁄4 1 41⁄2 3 3 4 21⁄2 3 31⁄2 21⁄4 21⁄2 3 2 13⁄4 21⁄2 2 13⁄4 13⁄8 11⁄8 1 78 78 34 58⁄⁄⁄ EXAMPLE 3.12 Select an unequal-leg angle tension member 15 feet long to resist a service dead load of 35 kips and a service live load of 70 kips. SOLUTION In keeping with the notation adopted in Example A.1, we will use Mp in the solution and substitute fb Mp in the final step. The connection is through the flange of the tee and is a bearing-type connection. For example, part of the slab and its load will be supported directly by the girders, but most of it will be carried by the floor beams. 2 2 2 2 The nominal strength is the moment of the couple, or Mn = Cy = Ty = 530(12.05) = 6387 in.-kips = 532.2 ft-kips ANSWER For LRFD, the design strength is fb Mn = 0.90(532.2) = 479 ft-kips. 2 2 Try 41⁄2 inches. FIGURE 8.14 LRFD SOLUTION Determine the shear and bearing strengths. Buckling will take place as soon as the load reaches the value given by Equation 4.3, and the column will become unstable about the principal axis corresponding to the largest slenderness ratio. 20′ W18 × 97 FIGURE P6.2-2 Members in Braced Frames 6.6-1 Compute the moment amplification factor B1 for the member of Problem 6.2-1. The member is part of a braced frame. The Steel Construction Manual, in Part 10, “Design of Simple Shear Connections,” gives guidelines for achieving this flexibility. For welds, it is measured from one end of the weld to the other. Use edge distances of 11⁄4 inches, spacings of 3 inches, and determine the minimum plate thickness required for bearing. The effective length is 24 feet. The presence of holes in a member will influence the stress at a cross section through the hole or holes. If the absolute value of a is equal to or greater than 1.0, plastic hinges have formed at both a–a and b–b, and the controlling limit state is flexural failure of the tee flange. Philadelphia. When the largest resultant is determined, the fastener size is selected so as to resist this force. Recall that these are the factors used for the fracture—or rupture—limit state, and block shear is a rupture limit state. The maximum permissible live load deflection is L 鼻180. 5.5-7 A W18 × 60 is used for a beam with an unbraced length of 25 feet. Select the flange and web dimensions so that intermediate stiffeners will be required. This was determined by Kulak, Fisher, and Struik (1987) and is demonstrated here. Use A992 steel and select a W-shape for AB. Both LRFD and ASD solutions are presented in a side-by-side format. At a distance x from the left end and at the neutral axis of the cross section, the state of stress is as shown in Figure 5.17d. These provisions are based on the work of Yura (2001). If l ≤ lr, the shape is nonslender. Web doubler plates are welded to the column web to increase its thickness. From Table 6-1, try a W10 ∂ 60, with p = 2.84 × 10−3 and bx = 5.27 × 10−3. Width-to-thickness parameters (see Figure 5.52): i. From Manual Table 7-1, the shear strength of one bolt is rn鼻Ω = 11.9 kips鼻bolt. Assume Class A surfaces and that threads are in the plane of shear. Therefore, for the gusset plate (the thinner part), the strength is frn = 78.3t = 78.3(3鼻8) = 29.4 kips鼻bolt Edge bolts: From Table 7-5, only two edge distances are given: 11⁄4 inches and 2 inches. From the LRFD solution, Rn = 384.3 kips Since 384.3 kips > 360.6 kips, block shear in the plate controls. A 9′ B 12 @ 9′ = 108′ FIGURE P1.5-2 1.5-3 During a tensile test of a specimen of unknown material, an increase in length of 6.792 × 10–3 inches within the gage length was recorded at a load of 5000 lb. Is this member satisfactory? The form of the resulting equation for Pcr will also be the same. Slip-critical allowable strength, one slip plane, Rn 鼻Ω (kips) 3.16 . The “M” is for “miscellaneous.” The M-shape has two parallel flanges and a web, but it does not fit exactly into either the W or S categories. The inner member is a PL 1⁄2 × 6 and each outer member is a PL 5⁄16 × 3. The same AISC equations for Pn (I2-2 and I2-3) are used, but the value of Pno depends on whether the hollow section is compact, noncompact, or slender. A572 Grade 50 steel is used. For courses in Structural Steel Design and Structural Loads An Instructor’s Solutions Manual and presentation slides are available for download for this text. This is the same rationale that is used for compression members, where the net area is not considered. Figure 5.31(c) shows a typical bay of the floor framing system. • Maximum rib height h r = 3 inches. 0.340 kips, b. ⎝ 8⎠ Ω The base metal shear strength is therefore 5.400 kips鼻in., and the weld strength of 4.640 kips鼻in. What total length of 1⁄4-inch fillet weld, E70XX electrode, is required? Compute the lower-bound moment of inertia. Consider the simple beam of Figure 5.17. The load being resisted (the beam reaction) is vertical, so the horizontal edge distance need only conform to the clearance requirements of AISC Table J3.4. For a 3⁄4-inch bolt, the minimum edge distance is 1 inch. Problems 171 Problems AISC Requirements 4.3-1 Use AISC Equation E3-2 or E3-3 and determine the nominal axial compressive strength for the following cases: a. Dead loads are those that are permanent, including the weight of the structure itself, which is sometimes called the self-weight. The ends will be connected by welding completely around the circumference of the pipe. If the symbol is above the line, the weld is on the other side of the joint, which may or may not be hidden from view in the drawing. Beam flange-to-plate weld design: The flange force is Ffu = 120.3 kips 8.8 End Plate Connections 569 AISC Design Guide 4 recommends that the minimum design flange force should be 60% of the flange yield strength: Min. Once the connected parts separate, however, any increase in external load will be resisted entirely by a corresponding increase in bolt tension. Alternative 2: Use a Diagonal Stiffener. Overall buckling is illustrated in Figure 5.9a. ASTM A325 (from Group A) and A490 (from Group B) are the traditional highstrength bolts and are covered in the Specification for Structural Joints Using HighStrength Bolts (RCSC, 2009), which is the basis for the AISC provisions for high-strength bolts. For simply supported trusses loaded as shown—a typical loading condition—each of the top chord members will be in compression, and the bottom chord members will be in tension. 1957. The bolt spacing and edge distance are the same as for the plate. Krishnamurthy, N. Partial penetration groove welds can be made from one or both sides, with or without edge preparation (Figure 7.35b). The shear areas are 4 8 8 Agv = t w (2 + 3 + 3 + 3) = 0.300(11) = 3.300 in. The moment of inertia I and the effective length factor K1 are for the axis of bending, and K1 = 1.0 unless a more accurate value is computed (AISC C3). 291 292 Chapter 5 5.11-4 Beams Select a W shape for the following conditions: Beam spacing = 10 ft Span length = 20 ft Slab and deck weight = 43 psf Partition load = 20 psf Ceiling weight = 5 psf Flooring weight = 2 psf Live load = 160 psf Fy = 50 ksi The maximum live load deflection cannot exceed L鼻360. The shear center is that point through which the loads must act if there is to be no twisting, or torsion, of the beam. The AISC Specification covers flexural members with slender webs in Section F5, “Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent About Their Major Axis.” This is the category usually thought of as plate girders. Two sets of curves are available, one for W shapes with Fy = 50 ksi and one for C and MC shapes with Fy = 36 ksi. If sag rods are used, they are designed to support the component of roof loads parallel to the roof. Mn = 139 ft-kips > 120 ft-kips (OK) Ωb 5.10 Design 235 Account for the beam weight: 1 M a = 6(20) + (0.053)(24)2 = 124 ft-kips < 139 ft--kips 8 (OK) The required shear strength is Va = 20 + 0.053(24) = 20.6 kips 2 From the Zx table (or the uniform load table), Vn = 83.2 kips > 20.6 kips (OK) Ωv Since deflections are computed with service loads, the deflection check is the same for both LRFD and ASD. For eccentric connections not included in the tables, the elastic method, which is conservative, may be used. 2011b. A value of zero gives the trivial case of no load. This connection is sometimes referred to as a shear connection, because it can transmit shear but virtually no moment. Note that the total number of studs is not affected by this requirement. From Equation 3.1, U =1− x 0.746 =1− = 0.9171 9 This is larger than the value of 0.80 that was initially used, so the strength based on fracture will be larger than originally calculated, and the strength will be governed by yielding as assumed. Recall that these factors are different for some hot-rolled shapes. Shear Strength AISC I4.2 conservatively requires that all shear be resisted by the web of the steel shape, as provided for in Chapter G of the Specification (Chapter 5 of this book). Basler, K. For this reason, the slip-critical strength should not be reduced in this type of connection. In many cases, the connections are not designed by the same engineer who designs the rest of the structure, but by someone associated with the steel fabricator who furnishes the material for the project. EXAMPLE 5.8 Determine the maximum reaction, based on block shear, that can be resisted by the beam shown in Figure 5.21. Both documents have been revised to reflect current research and practice, but the format remains the same. D = 84 kips L = 66 kips 2″ 7.9-1 WT6 × 29 2″ W12 × 120 3 @ 3″ 30° FIGURE P7.9-1 7.9-2 A structural tee bracket is attached to a column flange with six bolts as shown in Figure P7.9-2. These distances will now be used to check the bearing strength of the connection. This type of failure is caused by twisting about the longitudinal axis of the member. 81.0 kips Chapter 4: Compression Members 4.3-1 a. If the total deflection after the concrete has cured exceeds L鼻240, select another steel shape using either LRFD or ASD. Adequate: For shear, 18.5 kips < 21.2 kips; For tension, 7.93 kips < 15.1 kips 737 8.3-3 a. • New material on the shear strength of bolts incorporates the updated strength values given in the Specification. Electrodes should be selected to match the base metal. 98 Chapter 3 Tension Members 2″ 31⁄2″ 3″ 31⁄2″ 3″ 3″ 3″ 31⁄2″ W16 × 40 FIGURE P3.4-4 3.4-5 The tension member shown in Figure P3.4-5 is an L6 × 31⁄2 × 5⁄16. 5.15-5(b), the beam is unsatisfactory: Result of interaction equation is 1.13. The ratio of stress area to nominal area varies but has a lower bound of approximately 0.75. The beams are spaced at 8 feet, and the span length is 35 feet. Example 4.15 illustrates the computation of r0 and H. Composite columns are being used again in some structures after a period of disuse; we cover them later in this chapter. Use load and resistance factor design. 8.4 Eccentric Welded Connections: Shear Only 497 FIGURE 8.16 An eccentric load in the plane of the weld subjects the weld to both direct shear and torsional shear. Since the line of action of the load acts through the center of gravity of the connection, each fastener can be assumed to take an equal share of each component. ANSWER ASD SOLUTION Use the design shown in Figure 9.13. From AISC Equation E3-4, p 2E KL = r Fe For KL E ≤ 4.71 , r Fy p2E E ≤ 4.71 Fe Fy Fy ≤ 2.25 Fe The complete AISC Specification for compressive strength is as follows: When KL E ≤ 4.71 r Fy or Fy ≤ 2.25, Fe Fcr = (0.658 Fy鼻Fe) Fy When KL E > 4.71 r Fy Fcr = 0.877 Fe or (AISC Equation E3-2) Fy > 2.25, Fe (AISC Equation E3-3) In this book, we will usually use the limit on KL鼻r, as expressed in Equations 4.10 and 4.11. From AISC Equation F2-4, Cb p 2 E Jc 2 1 + 0.078 S x ho (Lb 鼻rts) Fcr = = 1.0p 2 (29, 000) (30 × 12 2.799) 2 1 + 0.078 ⎛ Lb ⎞ ⎜ ⎝ ⎟⎠ rts 2 (3.01(1.0) 30 × 12 103(13.28) 2.799) 2 = 33.90 ksi From AISC Equation F2-3, Mn = Fcr Sx = 33.90(103) = 3492 in.-kips = 291.0 ft-kips < Mp = 479.2 ft-kips LRFD SOLUTION fb Mn = 0.90(291.0) = 262 ft-kips ASD SOLUTION Mn 鼻Ωb = 0.6Mn = 0.6(291.0) = 175 ft-kips If the moment within the unbraced length Lb is uniform (constant), there is no moment gradient and Cb = 1.0. If there is a moment gradient, the value of Cb is given by Cb = 12.5 M max 2.5 M max + 3 M A + 4 M B + 3 MC (AISC Equation F1-1) where Mmax = absolute value of the maximum moment within the unbraced length (including the end points of the unbraced length) MA = absolute value of the moment at the quarter point of the unbraced length MB = absolute value of the moment at the midpoint of the unbraced length MC = absolute value of the moment at the three-quarter point of the unbraced length AISC Equation F1-1 is valid for doubly-symmetric members and for singly-symmetric members in single curvature. Determine the required size of bearing stiffeners at B, C, and D (assume that there will be a framed connection at each end of the girder). The gross area of one flange is Afg = tf bf = 0.810(7.64) = 6.188 in.2 The effective hole diameter is dh = 1 + 1 1 = 1 in. AISC J1.7 permits this eccentricity to be neglected in statically loaded members. The area of concrete to be used is Ac = C 258.5 = = 5.17 in.2 Fy 50 and the distance from the top of the steel shape to the centroid of the concrete is Y2 = t − a 0.8448 = 4.75 − = 4.328 in. The three-plate connection shown in Figure 8.37c does not have this handicap, and it has the additional advantage of being completely field bolted. V., and Ravindra, M. This metal deck also contributes to the strength of the slab, the design of which we do not consider here. Use ASTM A992 steel. If there is also a transverse weld at the end, then Ae = Ag. Assuming the latter, we have Pn FA 58(1鼻2)(8) = u e = = 116.0 kips Ω t 2.00 2.00 Design for a load of 86.23 kips and use E70 electrodes. The width-to-thickness parameter is λ = h tw where h is the distance between the roots of the flanges, and tw is the web thickness. Assuming full composite action and the PNA in the slab (i.e., steel controlling, the most common case for full composite action), we can write the nominal strength (refer to Figure 9.12) as Mn = Ty = As Fy y LRFD Procedure. A circle at the bend in the reference line is an instruction to weld all around the joint. Flexural strength: To determine the flexural strength, the elastic section modulus will be needed. 4.6-7 For the conditions shown in Figure P4.6-7, use LRFD and do the following. ANSWER Use a W12 × 53. 2000a. The tee and bolts are adequate: Required tf = 0.888 in. If the angle between the direction of the load and the axis of the weld is denoted q (see Figure 7.38), the nominal fillet weld strength is Fnw = 0.60 FEXX (1.0 + 0.50 sin1.5 θ) (AISC Equation J2-5) Table 7.2 shows the strength for several values of q. Pa = D + L = 18 + 52 = 70.0 kips For yielding, Ft = 0.6Fy = 0.6(36) = 21.6 ksi, and Required Ag = Pa 70 = = 3.24 in.2 Ft 21.6 (OK) 70 Chapter 3 Tension Members For fracture, Ft = 0.5Fu = 0.5(58) = 29.0 ksi, and Required Ae = Pa 70 = = 2.414 in.2 Ft 29.0 (The rest of the design procedure is the same as for LRFD. 4.9 BUILT-UP MEMBERS If the cross-sectional properties of a built-up compression member are known, its analysis is the same as for any other compression member, provided the component parts of the cross section are properly connected. These tables, which we will refer to as the “column load tables,” give the available strengths of selected shapes, both fcPn for LRFD and Pn鼻Ωc for ASD, as a function of the effective length KL. Figure 5.46b shows the deflected position of two different beams when loads are applied through the shear center and when they are not. The applied moment is Ma = Pae = 60(2.75) = 165 in.-kips From Equation 8.6, the resisting moment is M = nrt d = 4rt(3 − 1.5 + 3 + 3 − 1.5) = 24rt Equating the resisting and applied moments, we get 24rt = 165 or rt = 6.875 kips The shearing stress is frv = 7.5 = 16.98 ksi 0.4418 496 Chapter 8 Eccentric Connections and from AISC Equation J3-3b, the nominal tensile stress is ΩFnt frv ≤ Fnt Fnv 2.00(90) (16.98) = 60.40 ksi < 90 ksi = 1.3(90) − 54 Fnt′ = 1.3Fnt − The allowable tensile strength is Rn Fnt′ Ab 60.40(0.4418) = = = 13.3 kips > 6.875 kiips Ω Ω 2.00 ANSWER (OK) The connection is satisfactory. From the summation of moments at b–b in Figure 7.28b, Tb – Ma–a = qa (7.9) From Figure 7.28c, Mb–b = qa (7.10) Finally, equilibrium of forces requires that Bc = T + q (7.11) These three equilibrium equations can be combined to obtain a single equation for the total bolt force, which includes the effects of prying action. It must support a service dead load of 4.2 kips鼻ft (including the self-weight of the girder) and a service live load of 5 kips鼻ft on a simple span of 75 feet. Although other configurations are possible, these three are commonly used. Moment-rotation curves for three different connections are shown in Figure 8.33. There are two shear-failure planes: 3 Agv = 2 × (3 + 1.5) = 3.375 in.2 8 Since there are 1.5 hole diameters per horizontal line of bolts, ⎛ 7⎞ ⎤ 3⎡ Anv = 2 × ⎢3 + 1.5 − 1.5 ⎜ ⎟ ⎥ = 2.391 in.2 ⎝ 8⎠ ⎦ 8⎣ For the tension area, 3⎛ 7⎞ Ant = ⎜ 3 − ⎟ = 0.7969 in.2 8⎝ 8⎠ Since the block shear will occur in a gusset plate, Ubs = 1.0. From AISC Equation J4-5, Rn = 0.6 Fu Anv + U bs Fu Ant = 0.6(58)(2.391) + 1.0(58)((0.7969) = 129.4 kips with an upper limit of 0.6Fy Agv + UbsFu Ant = 0.6(36)(3.375) + 1.0(58)(0.7969) = 119.1 kips The nominal block shear strength is therefore 119.1 kips. 10.4-2 Determine the nominal flexural strength of the following welded shape: The flanges are 1 inch × 10 inches, the web is 3⁄8 inch × 45 inches, and the member is simply supported, uniformly loaded, and has continuous lateral support. An expression for this factor will now be developed. After the concrete cures, Partition load = 20(5.5) = 110.0 lb鼻ft Live load: 150(5.5) = 825.0 lb鼻ft Ceiling: 5(5.5) = 27.5 lb鼻ft LRFD SOLUTION Loads to be carried by the composite section: wD = wslab + wceil = 309.4 + 27.5 = 336.9 lb鼻ft wL = 110 + 825 = 935 lb鼻ft wu = 1.2 wD + 1.6 wL = 1.2(0.3 3 6)9+ 1.6(0.935) = 1.900 kips鼻ft Mu = 1 1 wu L2 = (1.900)(30)2 = 214 ft-kips 8 8 Assume a = 2 in.: Y2 = t − a 2 = 4.5 − = 3.5 in. • A discussion of frame analysis methods has been added. Use AISC Equation J10-9 to find the required plate thickness. stud, where there are two studs per rib. If allowable strength design is used, determine the required load capacity (required strength) to be used in the design of the column. 826 in.4 b. EXAMPLE 8.8 An L6 × 4 × 1⁄2 is used in a seated beam connection, as shown in Figure 8.26. ANSWER Use a 3⁄8-inch-thick doubler plate. 4): 137–9. The safety factor is Ω = 2.00. For one stud, Qn = 0.5 Asa fc′Ec ≤ Rg R p Asa Fu (AISC Equation I8-1) where Asa = cross-sectional area of stud (in.2) fc′ = 28-day compressive strength of the concrete (ksi) Ec = modulus of elasticity of the concrete (ksi) Rg = 1.0 for solid slabs (no formed steel deck) Rp = 0.75 for solid slabs Fu = minimum tensile strength of stud (ksi) When a formed steel deck is used, Rg and Rp depend on the deck properties. Select a W16-shape with Fy = 50 ksi. This determination usually involves apportioning a share of the total load to each frame. Problems 65ft-k 2 bolts W8 × 31 591 W14 × 22 25k 3⁄4″-diameter, Group A bolts Bolts are pretensioned. 3.6 Design of Tension Members 75 Using a shear lag factor U of 0.80, the actual effective net area is computed as follows: An = Ag − Aholes = 4.93 − 2 (43 + 81)(85) = 3.836 in. If load types not present in this example (E, Lr , S, and R) are omitted, the load combinations can be summarized as Combination 1: Combination 2: Combination 3: Combination 4: Combination 5: Combination 6: 1.4D 1.2D + 1.6L 1.2D + (0.5L or 0.5W) 1.2D + 1.0W + 0.5L 1.2D + 0.5L 0.9D ± 1.0W The dead load is less than eight times the live load, so combination (1) can be ruled out. Figure 7.25c shows the forces acting on a freebody diagram of the segment of the structural tee flange and the corresponding segment of the bolt. From Table 3-22c, “Continuous Beams,” the maximum moment in a two-span continuous beam with equal spans is at the interior support and is given by M = 0.125w2 where w = uniform load intensity = span length The maximum moment about the y axis is therefore Muy = 0.125(0.1470)(15鼻2)2 = 1.034 ft-kips To select a trial shape, use the beam design charts and choose a shape with a relatively large margin of strength with respect to major axis bending. For both cases, check for Ffu ≤ fRn 2 For shear yielding, f = 0.90 Rn = (0.6Fy)Ag Ag = tp bp For shear rupture, f = 0.75 Rn = (0.6 Fu)An An = tp [bp − 2(db + 1鼻8)] Increase tp if necessary. Assume that the tensile strength of the member is adequate, and determine the available strength of the welded connection. This form of instability is called lateral-torsional buckling (LTB). 1.99 bolts; use two. The minimum length is 4d = 4(3⁄4) = 3 in. For local buckling (see user notes in AISC F9), b ⎛ M n = Fy Sc ⎜ 2.43 − 1.72 ⎛ ⎞ ⎝ ⎝ t⎠ Fy ⎞ ≤ 1.5 M y E ⎟⎠ (AISC Equation F10-7) V. EXAMPLE 5.6 A simply supported beam with a span length of 45 feet is laterally supported at its ends and is subjected to the following service loads: Dead load = 400 lb鼻ft (including the weight of the beam) Live load = 1000 lb鼻ft If Fy = 50 ksi, is a W14 × 90 adequate? In either case, the tendency for columns to sway is

blocked within a given panel, FIGURE 4.16 150 Chapter 4 Compression Members or bay, for the full height of the frame. This concept is essentially the same as that used for the fasteners. The gusset plate is the thinner of the connected parts and controls. 140k 7″ L7 × 4 × 3⁄4 6″ FIGURE P8.4-8 8.4-9 An L6 × 6 × 3⁄8 is attached to a 3⁄8-in.-thick gusset plate with E70 fillet welds. For these reasons, all loading conditions in this book will be treated as static, and fatigue will not be considered. Satisfactory: Result of interaction equation = 0.810 b. LRFD SOLUTION Factored load shear = 1.2[0.25(50)] + 1.6[0.75(50)] = 75 kips Factored load tension = 1.2[0.25(100)] + 1.6[0.75(100)] = 150 kips a. LRFD SOLUTION The design strength is fb Mn = 0.90(5320) = 4788 in.-kips = 399 ft-kips The factored load and moment are wu = 1.2 wD + 1.6 wL = 1.2(0.400) + 1.6(1.000) = 2.080 kips兾ft Mu = ANSWER ASD SOLUTION 1 1 wu L2 = (2.080)(45)2 = 527 ft-kips > 399 ft-kips (N.G.) 8 8 Since Mu > fbMn, the beam does not have adequate moment strength. Compute everything with the equations in Chapter F of the AISC Specification. The resulting relationship between the resistance factor f and the safety factor Ω, as expressed in Equation 2.8, can be derived as follows. hf = filler factor Tb = minimum fastener tension from AISC Table J3.1 ns = number of slip planes (shear planes) A Class A surface is one with clean mill scale (mill scale is an iron oxide that forms on the steel when it is produced). If both sides of Equation 2.7 are divided by area (in the case of axial load) or section modulus (in the case of bending moment), then the relationship becomes f≤F where f = applied stress F = allowable stress This formulation is called allowable stress design. 33.0 kips a. A peak value, the upper yield point, is quickly reached after that, followed by a leveling off at the lower yield point. Figure 3.7 illustrates x− for various types of connections. ⎝ 4⎠ The shear rupture strength is ⎛ 1⎞ φ Rn = φ (0.6 Fu t) = 0.75(0.6)(58) ⎜ ⎟ = 6.525 kiips兾in. EXAMPLE 3.16 Select a structural tee for the bottom chord of the Warren roof truss shown in Figure 3.34. The loads are service loads, consisting of 30% dead load and 70% live load. When the stress is at its limit, the shear load is the nominal strength, given by Rn = Fnv Ab where Fnv = nominal shear strength (expressed as a stress) Ab = cross-sectional area of the unthreaded part of the bolt (also known as the nominal bolt area or nominal body area) High-strength bolts are available in two groups, defined by the strength of the bolts in those groups. The upper limit is λr = 1.49 E Fy Stiffened and unstiffened elements of various cross-sectional shapes are illustrated in Figure 4.9. The appropriate compression member limit, lr , from AISC B4.1 is given for each case. General Design Considerations. They are easily computed, however, if x0 and y0 are known. The components can be welded together and can be designed to have FIGURE 1.8 W-shape with cover plates Plate girders Double angle Double channel 16 Chapter 1 Introduction exactly the properties needed. and b 2.59 = = 0.12 < 0.2 d 21.1 (OK)) 1.5 [AISC Equation J10-5(a)] ⎤ 29, 000(50)(0.685) ≥ 99.73 ⎥ 0.430 ⎦ 258 Chapter 5 Beams Try b = 6 in. It is covered in Appendix 7. Column Stiffeners and Other Reinforcement 8.7-1 A service-load moment of 118 ft-kips, 30% dead load and 70% live load, is applied to the connection of Problem 8.6-2. Based on the strength of the 10 angle-to-column bolts, determine the following: a. This uneven cooling induces stresses that remain permanently. We focus on the analysis and design of individual members and connections, rather than complete structures. This part of the Manual also presents other guidelines. The connection in Figure 8.37a (the same type of connection as in Figure 8.31) typifies this concept. Ω 558 Chapter 8 Eccentric Connections For the stiffener shear yield strength, Rn = 0.4 Fy t × 2 stiffeners = 0.4(36)(3兾4)(2) = 21.6 kips兾in. Satisfactory: Before concrete has cured, Ma = 34.1 ft-kips < 50.1 ft-kips; after concrete has cured, Ma = 90.3 ft-kips < 112 ft-kips; Va = 14.5 kips < 52.8 kips 9.4-1 a. = 10.77 ft Fy 36 Lr = π rt E 29, 000 = π (4.138) = 441.0 in. For Lb = 12 ft, the design strength from the chart is 335 ft-kips for Cb = 1. Determine the plastic moment Mp about the horizontal plastic neutral axis. 1 M a = 270 + (0.055)(24)2 = 274 ft-kips < 280 ft-kiips (OK) 8 The maximum shear is Va = 9 + 3.055(24) = 41.2 kips 2 From the Zx table (or the uniform load table) Vn = 141 kips > 41.2 kips (OK) Ωv ANSWER Use a W18 × 55. wa = wslab + wbeam + wconst = 0.4375 + 0.026 + 0.200 = 0.66635 lb兾ft Ma = 1 1 wa L2 = (0.6635)(30)2 = 74.6 ft-kips 8 8 From the Zx table, Mn M p = = 110 ft-kips > 74.6 ft-kips Ωb Ωb (OK) After the concrete cures and composite behavior has been achieved, the load and moment are wa = wslab + wbeam + wpart + wL = 0.4375 + 0.026 + 0.200 + 1.000 = 1.664 kips兾ft 1 M a = (1.664)(30)2 = 187 ft-kips 8 The effective slab width for a typical interior beam is the smaller of Span 30 × 12 = = 90 in. If lateral support is provided at 12-foot intervals, what is the maximum service live load, in kips兾ft, that can be supported? If the load is applied at a distance less than d兾2 from the end of the column, ⎡ b ⎞ ⎛ t w ⎞ ⎛ d ⎟⎠ ⎜⎝ t f ⎟⎠ Rn = 0.40 t w2 ⎢1 + 3 ⎛⎜ ⎢⎣ 1.5 ⎤ EFyw t f ⎥ tw ⎥⎦ ⎛ for b ≤ 0.2⎞⎟ ⎝⎜ ⎟⎠ d (AISC Equation J10-5a) or 1.5 ⎡ 4 b ⎛ t w ⎞ ⎤ EFyw t f Rn = 0.40 t w2 ⎢1 + ⎛⎜ − 0.2⎞⎟ ⎜ ⎟ ⎥ tf ⎞ ⎥ tw ⎢⎣ ⎝ d ⎠ ⎛ for b > 0.2⎞⎟ ⎝⎜ ⎟⎠ d (AISC Equation J10-5b) For LRFD, f = 0.75. “Design of Hybrid Steel Beams.” Journal of the Structural Division, ASCE 94 (no. Expressing the bending moment as a function of the section modulus and stress gives M = fb Sx AISC F5 gives the nominal flexural strength based on tension flange yielding as Mn = Fy Sxt (AISC Equation F5-10) where Sxt = elastic section modulus referred to the tension side. The tables in Part 7 of the Manual will be used. There is a 20 psf construction load, a 20 psf partition load, a ceiling load of 8 psf, and a live load of 100 psf. The trusses are spaced at 25 feet on centers and support W6 × 12 purlins at the joints and midway between the joints. x 1.67 ⎞ U = 1− ⎛ ⎞ = 1− ⎛ = 0.6964 ⎝ ⎠ ⎝ 5.5 ⎠ ANSWER Ae = AgU = 5.77(0.6964) = 4.02 in.2 3.4 Staggered Fasteners 57 FIGURE 3.13 3.4 STAGGERED FASTENERS If a tension member connection is made with bolts, the net area will be maximized if the fasteners are placed in a single line. Do not use any of the design aids in the Manual. All structural steel is A992. 20k 2″ 4″ 2″ 10″ FIGURE P8.5-1 8.5-2 Determine the maximum load in the weld in kips per inch of length. 2 4.328″ 15.7″ 4.75″ –y 634 Chapter 9 TABLE 9.7 Composite Construction A y Ay – I d – I + Ad 2 Concrete W16 × 26 5.17 7.68 20.03 7.85 103.6 60.3 — 301 7.28 4.90 274.0 485.4 Sum 12.85 Component y = 759.4 in.4 163.9 ΣAy 163.9 = = 12.75 in. In addition, the theoretical strength of the member is reduced by the application of a resistance factor. 9.6-3 8.4-7 4.37 kips兾in. Although the eccentricities in this type of connection are small and can sometimes be neglected, they do exist and are used here for illustration. Determine whether the flexural strength is adequate. 8.4-15 Use the elastic method and design a welded connection for an L6 × 6 × 5⁄16 of A36 steel connected to a 3⁄8-inch-thick gusset plate, also of A36 steel. 6″ 6″ 6″ 34″ FIGURE P10.6-2 Design 10.7-1 A plate girder must be designed to resist a service-load bending moment of 2400 ft-kips consisting of 25% dead load and 75% live load. d. 8.7 Column Stiffeners and Other Reinforcement 539 FIGURE 8.44 If the connection is near the end of the column (i.e., if the compressive force is applied within a distance d兾2 from the end), the strength given by AISC Equation J10-8 should be reduced by half. When h b fc or h >6 b ft where Aw = area of the web Afc = area of the compression flange Aft = area of the tension flange bfc = width of the compression flange bft = width of the tension flange Summary Computation of the nominal shear strength can be condensed to the following procedure: 1. the design strength for LRFD. L., Fisher, J. M1 is the end moment that is smaller in absolute value, M2 is the larger, and the ratio is positive for members bent in reverse curvature and negative for single-curvature bending (Figure 6.9). A992 steel is used for all components. The following equation is used by AISC: ⎛ Lb − L p ⎞ ⎤ ⎡ M n = Cb ⎢ M p − (M p − 0.7 Fy S x) ⎥ ≤ Mp ⎝ Lr − L p ⎠ ⎦ ⎣ (AISC Equation F2-2) where the 0.7Fy Sx term is the yield moment adjusted for residual stress, and L p = 1.76ry E Fy (AISC Equation F2-5) Summary of Nominal Flexural Strength The nominal bending strength for compact I and C-shaped sections can be summarized as follows: For Lb ≤ Lp, Mn = Mp (AISC Equation F2-1) For Lp < Lb ≤ Lr, ⎛ Lb − L p ⎞ ⎤ ⎡ M n = Cb ⎢ M p − (M p − 0.7 Fy S x) ⎥ ≤ Mp ⎝ Lr − L p ⎠ ⎦ ⎣ (AISC Equation F2-2) For Lb > Lr, Mn = Fcr Sx ≤ Mp (AISC Equation F2-3) where Fcr = Cb p 2 E Jc ⎛ Lb ⎞ 1 + 0.078 ⎜ ⎟ S x h0 ⎝ rts ⎠ (Lb 兾rts)2 2 (AISC Equation F2-4) 206 Chapter 5 Beams EXAMPLE 5.4 Determine the flexural strength of a W14 × 68 of A242 steel subject to a. Since 5.90 × 10–3 > 3.92 × 10–3, use bx = 5.90 × 10–3. To provide for variation in the beam depth, the distance between flange plates is made larger than the nominal depth of the beam, usually by about 3⁄8 inch. The span length is 55 feet, the service live load is 2.0 kips兾ft, and the dead load is 0.225 kips兾ft, including the weight of the girder. Only the thicknesses are different, so the gusset plate will control. Problems 373 P = 92 k Mntx = 160 ft-k Mnty = 24 ft-k Kx = 0.8 Ky = 1.0 16′ Mntx = 214 ft-k Mnty = 31 ft-k FIGURE P6.8-6 6.8-7 Use A992 steel and select a W shape for the loads and moments shown in Figure P6.8-7. This same procedure also is used for determining the shear strength of hot-rolled shapes with unstiffened webs (see Chapter 5). Corresponding to the two most common values of resistance factors in LRFD are the following values of the safety factor Ω in ASD: For limit states involving yielding 2.4 Safety Factors and Load Combinations for ASD 27 or compression buckling, Ω = 1.67.* For limit states involving rupture, Ω = 2.00. Reston, VA. ANSWER Allowable strength = 28.0 kips. required strength per inch (kip ps兾in.) 4.419 Use two stiffeners, 3⁄4 × 3, one on each side of the web, and 3⁄16-inch × 11⁄2-inch intermittent fillet welds spaced at 31⁄2 inches on center, on each side of each diagonal stiffener. Panel AB, however, has no such anchorage on the left side. First-order analysis method Except in very simple cases, computer software is used for the analysis. Unlike a column, 5.3 Stability 197 FIGURE 5.9 Cross frame Diaphragm (c) however, the compression portion of the cross section is restrained by the tension portion, and the outward deflection (flexural buckling) is accompanied by twisting (torsion). It is essentially a one-worker job made possible by an automatic tool that allows the operator to position the stud and weld it to the beam in one operation. If the curves are not used, the intermediate stiffener spacing can be determined as follows: a. 6.6 Members in Braced Frames 311 Mt = maximum moment caused by sidesway (the subscript t is for “lateral translation”). Determine the controlling load combination for LRFD and the corresponding factored load. There is also some notational change: Fe becomes Fcr , Fey becomes Fcry , and Fez becomes Fcrz. hp = h for girders with equal flanges). With C = 384.0 kips, the depth of the compressive stress distribution in the concrete is a= FIGURE 9.18 C 384.0 = = 1.255 in. Only a very small rotation is necessary in order for a connection to be treated as pinned. 4.4 FIGURE 4.9 t b Local Stability b tw d t h t —— ≤ 0.56 √ E⁄Fy —— b⁄t E w ≤ 1.49 √⁄Fy b⁄t b ——⁄t ≤ 0.56 √ E⁄Fy d⁄t —— ≤ 0.75 √ E⁄Fy b b b t t h tw b ——⁄t ≤ 0.56 √ E⁄Fy b⁄t —— ≤ 0.45 √ E⁄Fy ——⁄t ≤ 0.56 √ E⁄Fy —— b⁄t E w ≤ 1.49 √⁄Fy b b t b h t tw ——⁄t ≤ 1.40 √ E⁄Fy —— b⁄t E w ≤ 1.40 √⁄Fy b D b⁄t —— ≤ 1.40 √ E⁄Fy t⁄t ≤ 0.11 E⁄Fy D EXAMPLE 4.3 Investigate the column of Example 4.2 for local stability. 2010a. Bending is about the major axis. Two possibilities are summarized in Table 9.8. The W14 × 22 is the lighter shape, but because ΣQn is larger, it will require more steel anchors. This matches the thickness range corresponding to footnote l in Table 2-4; therefore, Fy = 50 ksi. In many situations, however, it is good practice to limit the slenderness of tension members. The member length is 16 feet, and the effective length factors are Kx = 1.0 (nonsway), Kx = 2.0 (sway), and Ky = 1.0. Factored loads and moments based on first-order analyses are Pu = 75 kips, Mnt = 270 ft-kips, and Mt = 30 ft-kips. Elastic analysis. For example, if the web is compact and the flange is noncompact, the shape is classified as noncompact. If a rolled shape is to be used, however, the area to be deducted cannot be predicted in advance because the member’s thickness at the location of the holes is not known. Then, D= φst Fyst Ast 0.90(36)(1.88) = = 3.07 sixteenths 1.392 L (1.5) 1.392 [(3 − 5兾88) × 2 × 2] (1.5) Minimum size = ANSWER 3 in. 5.5-9 The beam shown in Figure P5.5-9 is a W36 × 210. Are the member and its connection satisfactory? Assume b兾d ≤ 0.2 and use AISC Equation J10-5a: ( ) ( )( ) 1.5 ⎡ Rn 1 b ⎛ t w ⎞ ⎤ EFy t f ≥ Ra = (0.40)t w2 ⎢1 + 3 ⎥ Ω Ω d ⎝ t f ⎠ ⎦ tw ⎣ 1 b 0.430 ⎡ (0.40)(0.430)2 ⎢1 + 3 2.00 21.1 0.685 ⎣ 1.5 ⎤ 29, 000(50)(0.685) ≥ 71.24 ⎥ 0.430 ⎦ b ≥ 3.78 in. It also requires a second-order or approximate second-order analysis. Standard upset ends with threads will actually have more net area in the threaded portion than in the unthreaded part. Because of symmetry, Sxt = Sxc = Sx The computations for Ix , the moment of inertia about the strong axis, are summarized in Table 10.1. The moment of inertia of each flange about its centroidal axis was neglected because it is small relative to the other terms. ⎝ 16 ⎠ The base metal shear strength is therefore 13.5 kips兾in. Steel Design Solution Manual Read/Download Tricia's Compilation for 'solutions manual structural steel design 5th edition' triciajoy.com/subject/solutions+manual+structural+steel+design+5th+. If the frame is actually unbraced, there is an additional component of the secondary moment, shown in Figure 6.4b, that is caused by sidesway. The reduced slipcritical strength is kscfRn where ksc = 1 − Tu 150 4.741 =1− =1− Du Tb nb 1.13(28)nb where Tb = 28 kips (from AISC Table J3.1). They belong to one of two broad categories: dead load and live load. 2 ⎝ 8⎠ ⎦ ⎣ 224 Chapter 5 Beams The net tension area is 1 ⎛ 7⎞ ⎤ ⎡ Ant = 0.300 ⎢1.25 − ⎜ ⎟ ⎥ = 0.2438 in.2 2 ⎝ 8⎠ ⎦ ⎣ Since the block shear will occur in a coped beam with a single line of bolts, Ubs = 1.0. From AISC Equation J4-5, Rn = 0.6Fu Anv + Ubs Fu Ant = 0.6(65)(2.381) + 1.0(65)(0.2438) = 108.7 kips with an upper limit of 0.6Fy Agv + Ubs Fu Ant = 0.6(65)(3.300) + 1.0(65)(0.2438) = 144.5 kips The nominal block shear strength is therefore 108.7 kips. This is usually the minor principal axis— the one with the smallest radius of gyration. We can conservatively use 11⁄4 inches, and if the strength is not adequate, we can manually compute the bearing strength. fc Pn 662 (Muy ⎞ 8 ⎛ Mux 8 135 30 Pu + + = 0.4532 + + fc Pn 9 ⎝ fb M nx fb M ny ⎠ 9 356 160.8 = 0.956 < 1.0 ANSWER ASD SOLUTION) (OK) The W12 × 65 is satisfactory. American Society for Testing and Materials. The actual distribution is difficult to determine, but it can be approximated quite easily. The maximum live-load deflection cannot exceed L兾360 (use a lower-bound moment of inertia). Try a W16 × 31: Mn = 135 ft-kips > 118 ft-kips Ωb (OK) (A W14 × 30 has an allowable moment strength of exactly 118 ft-kips, but the beam weight has not yet been accounted for.) Account for the beam weight: 1 M a = 118 + (0.031)(30)2 = 122 ft-kips < 135 ft-kips (OK) 8 244 Chapter 5 Beams The required shear strength is Va = wa L (1.05 + 0.031)(30) = = 16.2 kips 2 2 From the Zx table, the available shear strength is Vn = 87.5 kips > 16.2 kips Ωv (OK) Check deflection. AISC 45 (no. Select a W10 of A992 steel. All loads are compression except for the wind load, which can be either tension or compression. The nominal strength for this limit state is given by ⎡ ⎛ Lb − L p ⎞ ⎤ M n = Cb ⎢ M p − (M p − 0.7 Fy S x ⎥ ≤ Mp ⎝ Lr − L p ⎠ ⎦⎥ ⎣ (AISC Equation F2-2) where M p = Fy Z x = 50(146) = 7300 in.-kips 10 − 6 ⎞ ⎤ = 6460 in.-kips M n = 1.0 ⎡ 7300 − (7300 − 0.7 × 50 × 127) ⎛ 19.6 − 6 ⎠ ⎦⎥ ⎣ Check to see if the flange holes need to be accounted for. ⎝ 16 ⎠ Use two welds per stiffener for a total of four. 22.6 kips a. Required spacing of welds = ANSWER strength of weld group (kips) 16.7 = = 3.78 in. ILB = 6950 in.4, ΔL = 0.381 in. A free-body diagram of one ridge purlin illustrates this effect, as shown in Figure 3.31b. From Example 4.2, KL兾r = 96.77 and Fy = 50 ksi. The strength of components at connections, including connecting elements such as gusset plates and elements of the members, are covered in AISC J4, “Affected Elements of Members and Connecting Elements.” Shear strength is covered in Section J4.2, where the limit states of shear yielding and shear rupture are given. ∴ Use 4 in. Load Combination 2: Pnt = 454 kips, Mnt = 104.8 ft-kips, and Mt = 0 (because of symmetry, there are no sidesway moments). This eccentricity consists of the initial crookedness, y0, plus additional deflection, y, resulting from bending. 2 2 Rn = 1.2 c tFu ≤ 2.4 dtFu c = e − 1 1.2c tFu = 1.2(0.8438) ⎛⎜ ⎟⎞ (58) = 29.36 kips ⎝ 2⎠ Check upper limit: 3 1 2.4 dtFu = 2.4 ⎛ ⎞ ⎛ ⎞ (58) = 52.20 kips ⎝ 4⎠⎝ 2⎠ 29.36 kips < 52.20 kips ∴ Use Rn = 29.36 kips/bolt. The end shears are not shown. So that we can use the values in Manual Table 7-5, try 11⁄4 inches. 608 Chapter 9 Composite Construction FIGURE 9.9 Since this member is an interior beam, the third criterion is not applicable. > 0.52 in. If the result is consistent with the assumption, then the PNA is in the top flange. Although wind is present most of the time, wind loads of the magnitude considered in design are infrequent and are not considered to be fatigue loads. Compute the thickness needed to satisfy the gross area requirement. The AWS Code should be used for criteria not covered in the AISC Specification. The Specification covers other surfaces, but in this book we conservatively use Class A surfaces, which are assigned the smallest slip coefficient. Member BC is a W12 × 50. (It cannot be used in the end panels.) Referring to the conditions of AISC G3.1, a a. pPu + bx Mux + by Muy = (2.01 × 10−3)(300) + (3.14 × 10−3)(100) + 0 = 0.917 < 1.0 (OK) Verify that Equation 6.9 is the correct one: Pu = pPu = (2.01 × 10 −3)(300) = 0.603 > 0.2 ∴ Equation 6.9 controls, as assumed. When loads are applied to both flanges, web compression buckling must be checked (AISC J10.5). Determine dimension B from a consideration of bearing strength. One assumption made in the usual methods of structural analysis is that deformations are very small, which means that only a slight rotation of the support is needed to qualify it as a pinned connection. For LRFD, f = 1.00. (total) b. (The upper limit of fbMpx does not control.) a. At all stages of loading, the sum of the absolute values of the maximum negative and positive moments is wL2兾8. The minimum spacing is 22⁄3 d = 2.667(3兾4) = 2.0 in. FIGURE 3.26 3.7 Threaded Rods and Cables 77 The effective cross-sectional area in the threaded portion of a rod is called the stress area and is a function of the unthreaded diameter and the number of threads per inch. Compute the weak axis bending moments. The nominal strength can then be computed as Pn = Fcr Ag (AISC Equation E4-1) 4.8 Torsional and Flexural-Torsional Buckling 159 where 4 Fcry Fcrz H ⎛ Fcry + Fcrz ⎞ ⎡ Fcr = ⎢ ⎥1 − 1 − ⎝ 2H ⎠ ⎢ (Fcry + Fcrz) 2 ⎣ ⎤ ⎥ ⎦ (AISC Equation E4-2) All other terms from Section E4(b) remain unchanged. The required strength is Pa = 500 kips Starting with the smallest ratio of 1.2, we compute the approximate value Kx L 16 = = 13.3 rmx 兾rmy 1.2 An HSS12 × 10 × 1⁄2, weighing 69.3 lb兾ft, is a possibility. (We cover this later in this chapter.) In this book, we will use the designations Group A and Group B. 2(4.733) The second option requires shorter longitudinal welds. Because the bearing strength will be the same for parts a, b, and c, it will be calculated first. The external and internal virtual work in this case are 2 Pu (15θ) = M pθ + M p (2θ) + M pθ and Pu = 2 Mp 15 This second possibility requires the smaller load and is therefore the correct mechanism. In a beam connection, any moment transfer takes place mostly through the flanges, in the form of a couple. 16 (AISC Table J2.4) Try a 3⁄16-in. The weld strength therefore controls. 9.8-4 Same as Problem 9.8-3, but use ASD. From the LRFD solution, for one 3⁄4-inch × 3-inch stud every other rib, Mn = 343.3 in.-kips The allowable moment strength is therefore M n 343.3 = = 206 ft-kips > 205 ft-kips Ωb 1.67 ANSWER 9.8 (OK) Use the steel anchors shown in Figure 9.20. > 1.5 in. The 20-kip service loads are 30% dead load and 70% live load. From the dimensions and properties tables in Part 1 of the Manual, rmin = 0.746 in. It covers two approaches: “thick plate” theory, in which thick plates and small diameter bolts result, and “thin plate” theory, with thin plates and large bolts. Is the member adequate? If the plate covers the full area of the support, the nominal strength is Pp = 0.85 fc′A1 (AISC Equation J8-1) If the plate does not cover the full area of the support, Pp = 0.85 fc′A1 A2 ≤ 1.7 fc′A1 A1 (AISC Equation J8-2) where fc′ = 28-day compressive strength of the concrete A1 = bearing area A2 = full area of the support If area A2 is not concentric with A1, then A2 should be taken as the largest concentric area that is geometrically similar to A1, as illustrated in Figure 5.38. For this reason, try the W14 ∂ 26. For the edge bolts, rn兾Ω = 29.4t kips兾bolt, and for the inner bolts, rn兾Ω = 52.2t kips兾bolt. This installation produces the snug-tight condition referred to earlier in the discussion of the turn-ofthe-nut method. This book is intended for junior-and senior-level engineering students, although some of the later chapters can be used in a combination undergraduate/graduate course. Compare the flexural and the flexural-torsional buckling strengths. For shear yielding, from AISC Equation J4-3, Rn = 0.60Fy Ag = 0.60(36)(8.5t) = 183.6t fRn = 1.00(183.6t) = 183.6t For shear rupture, the net area is ⎛ 3 1⎞ ⎤ ⎡ Anv = ⎢8.5 − 3 ⎜ + ⎟ ⎥ t = 5.875t ⎝ 4 8⎠ ⎦ ⎣ From AISC Equation J4-4, Rn = 0.6Fu Anv = 0.6(58)(5.875t) = 204.5t fRn = 0.75(204.5t) = 153.4t Shear rupture controls. The frame analysis was a first-order analysis with reduced member stiffnesses, so the moment amplification method with Kx = 1.0 can be used. The load factors and load combinations in this standard are based on extensive statistical studies and are prescribed by most building codes. In addition to their use in bridges and cable roof systems, they are also used in hoists and derricks, as guy lines for towers, and as longitudinal bracing in metal building systems. The lowest PNA location, level 7, corresponds to the recommended lower limit of ΣQn = 0.25As Fy (Hansell, et al., 1978). Do not include the beam weight in the loading. L = 15 ft b. W10 × 33 40k 65 ft-k 10′ FIGURE P6.6-5 40k 65 ft-k 366 Chapter 6 6.6-6 Beam–Columns The beam–column in Figure P6.6-6 is a member of a braced frame. Compute the allowable strength: b = beam spacing or span length ÷ 4 = 5.5(12) = 66 in. 330 Chapter 6 Beam–Columns EXAMPLE 6.6 A W12 × 65 of A992 steel, 15 feet long, is to be investigated for use as a column in an unbraced frame. 9.4-4 How many 7⁄8-inch × 31⁄2-inch studs are required for the beam in Problem 9.1-2 for full composite behavior? An attempt has been made to strike a balance between the theoretical and the practical. AISC C2.3(3) permits the use of tb = 1.0 if a small additional notional load is included. Equation 5.9 can be written more generally as follows: Fu Afn < Yt Fy Afg (5.10) where Yt = 1.0 for Fy 兾Fu ≤ 0.8 = 1.1 for Fy 兾Fu > 0.8 Note that, for A992 steel, the preferred steel for W shapes, the maximum value of Fy 兾Fu is 0.85. 8.4-19 a. Design Considerations for Welds. r Fy Fcr = 0.658 (Fy兾Fe) Fy = 0.658(50 兾 24.36) (50) = 21.18 ksi The nominal strength is Pn = Fcr Ag = 21.18(20.60) = 436.3 kips LRFD SOLUTION The design strength is ASD SOLUTION From Equation 4.7, the allowable stress is fcPn = 0.90(436.3) = 393 kips Fa = 0.6Fcr = 0.6(21.18) = 12.71 ksi The allowable strength is Fa Ag = 12.71(20.60) = 262 kips ANSWER Design compressive strength = 393 kips. 3″ D = 30k L = 65k 2″ WT6 × 29 W12 × 120 3 @ 3″ 2″ FIGURE P8.3-2 W18 × 106 578 Chapter 8 8.3-3 Eccentric Connections Check the adequacy of the bolts for the given service loads. where t is the thickness of the slab. When LRFD was introduced into the AISC Specification in 1986, the load factors were determined in such a way as to give the same results for LRFD and ASD when the loads consisted of dead load and a live load equal to three times the dead load. Starting with a value of KL, the strength was obtained by a procedure similar to the following: • KL was divided by ry to obtain KL兾ry. If P is a service load, then p will be the required allowable strength of the fastener. For the four bolts, Rn = 2Rn (inner bolts) + 2Rn (outer bolts) 7. The bearing strength for these bolts is f rn = 49.4t = 49.4(0.560) = 27.7 kips兾bolt Since the shear strength is less than the bearing strength of any bolt, the shear strength controls. Although no width-to-thickness ratio limit is given in the Specification for bearing stiffeners, the requirement of AISC Equation G3-3 for intermediate stiffeners can be used as a guide in proportioning bearing stiffeners: ⎛ b ⎞ ≤ 0.56 E ⎝ t ⎠ st Fyst Bearing-stiffener analysis is illustrated in Example 10.1, Part c. 8k 12″ 3⁄16 2L5 × 5 × 1⁄2 E70 electrodes 5″ Section FIGURE P8.4-4 8.4-5 Use E70 electrodes and determine the required weld size. For local web yielding, a pair of transverse stiffeners or a doubler plate should be used. From Table 7-1, the shear strength is frn = 17.9 kips兾bolt For the inner bolts with a spacing of 3 inches, the bearing strength from Table 7-4 is f rn = 87.8t = 87.8(0.560) = 49.2 kips兾bolt 494 Chapter 8 Eccentric Connections For the edge bolts, use Table 7-5 and a conservative edge distance of 11⁄4 inches. ∴ Use 60 in. The framed beam connection shown in Figure 8.1a is a typical eccentric connection. In a continuous beam, the points of inflection are also points of zero moment and, in general, 2N1 anchors will be required for each span. The equation of this curve can be expressed in a general way as ⎛ required tensile strength ⎞ ⎛ required shear strength ⎞ ⎜⎝ available tensile sttrength ⎟⎠ + ⎜⎝ available shear strength ⎟⎠ = 1.0 2 2 (7.22) where the strengths can be expressed as forces or stresses and in either LRFD or ASD format. A242 steel is used for the tension member and A36 steel is used for the gusset plate. To approximate these two effects, two amplification factors, B1 and B2, are used for the two types of moments. The procedure for design is as follows: 1. This includes the following: • The new provisions of the Specification and Manual are covered in detail. “Selection of a ‘Trial’ Column Section.” Engineering Journal, AISC 10 (no. The effective length method of analysis is covered in Appendix 7. International Conference of Building Officials. For LRFD, the design strength f b Mn is tabulated. Compression members are also used in trusses and as components of bracing systems. For I-shaped members, M1⬘ ⎞ ⎤ ⎛ E ⎞ ⎡ L pd = ⎢ 0.12 + 0.076 ⎛ ry ⎝ M 2 ⎠ ⎥⎦ ⎜⎝ Fy ⎟⎠ ⎣ (AISC Equation A-1-5) To define M1⬘, we first need to define the variables M1, M2 , and Mmid. 1). Even so, other advantages may make composite construction attractive. The effective slab width is 72 inches. Yes: 728 kips < 759 kips b. All beams are W16 × 40, and all columns are W12 × 58. The design curves in Part 3 of the Manual can be used for this purpose. The second possibility corresponds to the concrete controlling, and the PNA will be in the steel (Figure 9.7b or c). M. There will be four bolts in each line. Use A36 steel and design for the following loads. If an analysis is being done for a member not in the column load tables, then any element slenderness must be accounted for. This limit does not apply to cables, and the user note explicitly excludes rods. FIGURE 8.51 tp de pfo pfi pfi pfo de bp g 8.8 End Plate Connections 561 • All of the shear is resisted by the compression-side bolts. ANSWER (OK) Use 3⁄16-in. If l > lr, the shape is slender. In a rolled shape, this section will be at the toe of the fillet, a distance k from the outside face of the flange (this dimension is tabulated in the dimensions and properties tables in the Manual). Check the shear capacity of the base metal (the angle controls): Applied direct shear = fv = 1.833 kips兾in. The Commentary to Section J1.7 states that for fatigue loading conditions the welds should be balanced. To find the required weld size, equate fr to the weld capacity per inch of length: fr = 0.9279D 3.221 = 0.9279D, D = 3.471 where D is the weld size in sixteenths of an inch for E70 electrodes. > 8.425 in. Check for Ffa Rn ≤ 2 Ω where Ffa = Ma d − t fb For shear yielding, Ω = 1.67 Rn = (0.6Fy)Ag Ag = tpbp For shear rupture, Ω = 2.00 Rn = (0.6Fu)An An = tp [bp – 2(db + 1兾8)] 6. 5.11-5 Select an A992 W shape for beam AB of the floor system shown in Figure P5.11-5. Bracing must be designed for both strength and stiffness. This can be accomplished with either of two types of stability bracing: lateral bracing, illustrated schematically in Figure 5.9b, and torsional bracing, represented in Figure 5.9c. Assume that Ae = An. a. 2 Because the length of the connection is not known, Equation 3.1 cannot be used to compute the shear lag factor U. From the dimensions and properties tables in Part 1 of the Manual, rmin = 0.859 in. Compute the radius of gyration about each of the principal axes. 5. Although the 3-inch center-to-center spacing can be used for the entire length of the girder, an increased spacing can be used where the shear is less than the maximum of 223.4 kips. ANSWER ASD SOLUTION Use a W14 × 145. LRFD SOLUTION FIGURE 7.14 Shear: These bolts are in double shear. Use Figure 5.35 and select a K-series joist. In a braced frame, the member ends do not undergo translation, so only the P-d moments are present. Ωb 1.67 When full composite behavior exists, the plastic neutral axis will normally be within the slab, as in Example 9.2. Analysis for the case of the plastic neutral axis located within the steel section will be deferred until partial composite action has been covered. The determination of the stiffener spacing can be facilitated by the use of the design curves in Part 3 of the Manual. An ultimate strength approach, based on the relationships in AISC J2.4b, is presented in Part 8 of the Manual and is summarized here. In any case, the spacing must not exceed half of the smaller column dimension. Revise if necessary. Compute the nominal block shear strength. Welded shapes usually fall into the category classified as plate girders. The resisting moment furnished by these bolt forces is computed as the resultant of the tension bolt forces times a moment arm equal to the distance between the centroid of the tension bolt areas and the centroid of the “compression” bolt areas, or M = nrt d FIGURE 8.13 (8.6) 8.3 Eccentric Bolted Connections: Shear Plus Tension 493 where M = resisting moment rt = bolt force n = number of bolts above the neutral axis d = distance between the centroid of the tension bolts and the centroid of the “compression” bolts When the resisting moment is equated to the applied moment, the resulting equation can be solved for the unknown bolt tensile force rt . This graph shows the relationship between moment transferred at a joint and the corresponding relative rotation of the connection. For this type of braced frame, Kx should be taken as 1.0. 4.7 More on Effective Length 151 column given by Equation 4.6b. 4.9 FIGURE 4.19 Built-Up Members 163 y x 65 TABLE 4.2 SOLUTION A y Ay Plate W 1.500 19.10 0.1875 9.575 0.2813 182.9 Σ 20.60 Component 183.2 With the addition of the cover plate, the shape is slightly unsymmetrical, but the flexural-torsional effects will be negligible. (The exact notation and form of the AISC Equations are covered in Chapter 6.) For LRFD, Equation 5.21 becomes Muy Mux + ≤ 1.0 φb Mnx φbMny (5.22) where Mux = factored-load moment about the x axis Mnx = nominal moment strength for x-axis bending Muy = factored-load moment about the y axis Mny = nominal moment strength for the y axis For ASD, M ay M ax + ≤ 1.0 M nx 兾Ωb M ny 兾Ωb (5.23) where Max = service load moment about the x axis May = service load moment about the y axis Weak-Axis Bending Strength To this point, the strength of I-shaped cross sections bent about the weak axis has not been considered. American Society of Civil Engineers. Although no building code or other standard used in the United States contains a limit on the drift index, values of 1⁄500 to 1⁄200 are commonly used (Ad Hoc Committee on Serviceability, 1986). Most hollow structural sections available in the United States today are produced by cold-forming and welding (Sherman, 1997). A 2(32.75) ANSWER The maximum service load reaction = 327 kips. Sometimes, however, the roof deck cannot span the distance between joints, and intermediate purlins may be needed. The load to be resisted is a service dead load of 31 kips and a service live load of 31 kips. 4.6-3 Select a square HSS (Fy = 46 ksi). Since one foot of length has a volume of As 兾144 cubic feet and structural steel weights 490 pounds per cubic foot, w= As (490) = 3.4 As lb兾ft (for As in square inches) 144 From Equation 9.3, the estimated weight per foot is, therefore, w= 3.4 Mu lb兾ft fb Fy ⎛ d + t − a ⎞ ⎝ 2 2⎠ (9.4) where Mu is in in.-kips; Fy is in ksi; and d, t, and a are in inches. governs. Also recommended is Detailing for Steel Construction (AISC, 2009), which is intended for detailers but also contains information useful for designers. LRFD SOLUTION All the load combinations given in ASCE 7 involve dead load, and except for the first one, all combinations also involve either live load or wind load or both. When a beam must frame into the web of a column rather than its flange (for example, in a space frame), the connection shown in Figure 8.37b can be used. The central problem of all member design, including tension member design, is to find a cross section for which the required strength does not exceed the available strength. The details are given in AISC I6 and will not be covered here. Which load combination controls? 2.3 LOAD FACTORS, RESISTANCE FACTORS, AND LOAD COMBINATIONS FOR LRFD Equation 2.4 can be written more precisely as ΣgiQi ≤ fR n (2.5) where Qi = a load effect (a force or a moment) gi = a load factor Rn = the nominal resistance, or strength, of the component under consideration f = resistance factor The factored resistance fRn is called the design strength. The connections shown in Figure 7.3d and f are of this type. The service live load is 65 psf. Use A992 steel and select a W shape. At failure, the fastener group rotates about an instantaneous center (IC). We discuss hot-rolled shapes in the present chapter of this book and built-up shapes in Chapter 10, “Plate Girders.” The AISC provisions for hot-rolled shapes are covered in Section G2.1. Before covering the AISC provisions for shear strength, we will first review some basic concepts from mechanics of materials. ST9): 1443–57. The design of the welds connecting the doubler plate to the column will depend on the exact configuration of the plate, including whether the plate extends beyond the transverse stiffeners. Part (b) shows what would be seen if a horizontal section were cut through the building above one of the floors and the lower portion viewed from above. (OK) 16 16 4 Design the welds. The eccentricity e is 10 + 8 − 2.286 = 15.71 in., and the torsional moment is M = Pe = 40(15.71) = 628.4 in.-kips If the moment of inertia of each horizontal weld about its own centroidal axis is neglected, the moment of inertia of the total weld area about its horizontal centroidal axis is Ix = 1 (1)(12)3 + 2(8)(6)2 = 720.0 in.4 12 8.4 Eccentric Welded Connections: Shear Only 501 Similarly, ⎡1 ⎤ I y = 2 ⎢ (1)(8)3 + 8(4 − 2.286)2 ⎥ + 12(2.286)2 = 195.0 in.4 ⎣ 12 ⎦ and J = Ix + Iy = 720.0 + 195.0 = 915.0 in.4 Figure 8.18 shows the directions of both components of stress at each corner of the connection. Check block shear. Mu = Pue = 22(2.375) = 52.25 ft-kips M c 52.25(3) ft = u = = 4.354 kips兾in. and Huzzard, Robert K. • total slab thickness = 51⁄2 in. For a plate connection, if the welds are along the sides only, Ae = AgU. Assume that the beam has full lateral support during construction and that there is a 20 psf construction load. The moment at each of these locations will equal Mp , the plastic moment capacity of a length of flange tributary to one bolt. Required bolt diameter: h0 = d − t fb 0.425 + p fo = 17.7 − + 2 = 19.49 in. Use the column load tables. In cases such as this one, the secondary moment PΔ will be very small, and Mt can usually be neglected. Can this member resist a service dead load of 180 kips and a service live load of 320 kips? LRFD Specification for Steel Hollow Structural Sections. 5.8-3 The beam shown in Figure P5.8-3 is a W16 × 31 of A992 steel and has continuous lateral support. Segui The University of Memphis Australia • Brazil • Japan • Korea • Mexico • Singapore • Spain • United Kingdom • United States This is an electronic version of the print textbook. Ultimate Strength Analysis Eccentric welded shear connections may be safely designed by elastic methods, but the factor of safety will be larger than necessary and will vary from connection to connection (Butler, Pal, and Kulak, 1972). It is connected to a 3⁄8-inch-thick gusset plate with 7⁄8-inch-diameter bolts. SOLUTION The collapse loads are obtained by multiplying the service loads by the appropriate load factors. Each member is oriented so that its web is in the plane of the frame. For full composite behavior, the compressive force in the concrete at ultimate (equal to the horizontal shear at the interface between the concrete and steel) will be the smaller of AsFy = 7.68(50) = 384.0 kips or 0.85f c′ Ac = 0.85(4)[90(4.75 − 1.5)] = 994.5 kips Only the concrete above the top of the deck has been accounted for. EXAMPLE 8.2 The bracket connection shown in Figure 8.8 must support an eccentric load consisting of 9 kips of dead load and 27 kips of live load. The design strength is based on yielding of the gross section. These connections illustrate the two basic categories of eccentric connections: those causing only shear in the fasteners or welds and those causing both shear and tension. Although this objective can usually be accomplished by a design that requires a minimum amount of material, savings can often be realized by using more material if it results in a simpler, more easily constructed project. 5″ 2L5 × 31⁄2 × 5⁄16 LLBB 5″ t = 3⁄8″ FIGURE P7.11-2 7.11-3 Determine the maximum service load, P, that can be applied if the live load-to-dead load ratio is 2.5. Each component is a PL 5⁄8 × 7 of A242 steel. minimum 1.392(15.5) 16 Try w = 3⁄16 in. 6.2 Interaction Formulas 303 FIGURE 6.2 For an unbraced length Lb = 17 ft, fb Mn = 197 ft-kips For the end conditions and loading of this problem, Cb = 1.32 (see Figure 5.15c). If the member is one with an unsymmetrical cross section, such as the double-angle section in Figure 8.20, a symmetrical placement of welds or bolts will result in an eccentrically loaded connection, with a couple of Te, as shown in Figure 8.20b. For LRFD, fc Fcr Ag ≥ Pu Ag ≥ Pu fc Fcr For ASD, 0 . This will be shear on the gross area of the plate (Sections a–a and b–b in Figure 8.53) and shear on the net area (Sections c–c and d–d). The service dead-load moment is 20 ft-kips, and the service liveload moment is 48 ft-kips. ANSWER ASD SOLUTION Use the weld shown in Figure 8.23. Satisfactory: Result of interaction equation = 0.776 b. For those connections not covered by the tables, the more conservative elastic method may be used. Combination 1: Combination 2: Combination 3: Combination 4: Combination 5: D = 109 kips. Enter the tables with KL = 10.34 feet. Stiffeners are added in this example. (Note that the coefficient C can be interpreted as the ratio of total connection load to maximum fastener force.) What is the percent difference from the value obtained by the elastic analysis? To maintain the back-to-back separation of the angles along the length, fillers (spacers) of the same thickness as the gusset plate are placed between the angles at equal intervals. Any cross-sectional configuration may be used, because for any given material, the only determinant of the strength of a tension member is the cross-sectional area. Allowable stress design has been the primary method used for structural steel buildings since the first AISC Specification was issued in 1923, although plastic design was made part of the Specification in 1963. 4.6 DESIGN The selection of an economical rolled shape to resist a given compressive load is simple with the aid of the column load tables. An example of the designation of an S-shape is “S18 × 70,” with the S indicating the type of shape, and the two numbers giving the depth in inches and the weight in pounds per foot. LRFD SOLUTION The factored load is Pu = 1.2D + 1.6L = 1.2(12) + 1.6(23) = 51.2 kips From the column load tables, select 2L 31⁄2 × 3 × 1⁄4 SLBB, weighing 10.8 lb兾ft. Although we used a bolted connection here for illustration, welded connections can be similarly categorized as either shear only or shear plus tension. LRFD SOLUTION The factored load is Pu = 1.2D + 1.6L = 1.2(35.5) + 1.6(142) = 269.8 kips Enter Table 4-21 in Part 4 of the Manual with Pu 269.8 = = 27.79 ksi Ag 9.71 and obtain the stiffness reduction factor tb = 0.9877 by interpolation. The selection of an “off-the-shelf ” item will almost always be the most economical choice, even if it means using slightly more material. Part 17. Fillet welds can also be used in a tee joint, as shown in Figure 7.33b. Tie rod at the ridge: 46.6 ⎞ P = 3.203⎛ = 3.317 kips ⎝ 45 ⎠ Required Ab = ANSWER 3.317 = 0.1525 in.2 21.75 Use a 5⁄8-inch-diameter threaded rod (Ab = 0.3068 in.2) for the tie rod at the ridge. 19.76 ⎥⎦ The total loss in area is 2(b − be)t = 2(7.652 − 6.65)(0.116) = 0.2325 in.2 and the reduced area is Ae = 2.70 − 0.2325 = 2.468 in.2 The reduction factor is Qa = Ae 2.468 = = 0.9141 Ag 2.70 Q = QsQa = 1.0(0.9141) = 0.9141 Compute the local buckling strength. ANSWER Use the stud anchors shown in Figure 9.20. BLOCK SHEAR For certain connection configurations, a segment or “block” of material at the end of the member can tear out. This includes most (but not all) W-shapes. Many fabricators, however, prefer 2 inches or 21⁄2 inches for all bolt sizes. Account for the beam weight: wa = wD + wL = 0.084 + 4.5 = 4.584 kips兾ft Ma = 1 1 wa L2 = (4.584)(30)2 = 515.7 ft-kips 8 8 The required plastic section modulus is Zx = Ωb M a 1.67(515.7 × 12) = = 207 in.3 < 224 in.3 Fy 50 (OK) Instead of searching for the required section modulus, the search could be based on the required value of Mp 兾Ωb, which is also tabulated. If this force is taken as the horizontal component of an axial compressive force P in the stiffener, P cos θ = 54.51 kips where ⎛d ⎞ ⎛ 17.9 ⎞ q = tan −1 ⎜ b ⎟ = tan −1 ⎜ ⎟ = 65.26° ⎝ dc ⎠ ⎝ 8.25 ⎠ P= 54.51 = 130.3 kips cos(65.26°) Since the stiffener is connected along its length, we will treat it as a compression member whose effective length KL is zero. h h 63 = = = 3.94 < 6 b fc b ft 16 (OK) (OK) Therefore tension-field action can be used. If we begin at the point of buckling, then from Figure 4.3 the bending moment is Pcr y. 3.5 Block Shear 65 FIGURE 3.21 Shear a b Tension c For the case illustrated, the shaded block would tend to fail by shear along the longitudinal section ab and by tension on the transverse section bc. If the floor is a slab of uniform thickness, the weight can be calculated as follows. Modify bx to account for Cb. bx 4.58 × 10 −3 = = 2.74 × 10 −3 Cb 1.67 This is less than bx for Lb = 0, which is 3.69 × 10−3, so use bx = 3.69 × 10−3. Structural shapes of this type are created by bending thin material such as sheet steel or plate into the desired shape without heating. The steel yield stress is Fy = 50 ksi, and Grade 60 648 Chapter 9 Composite Construction FIGURE 9.27 reinforcing bars are used. The differences in the two approaches are mostly conceptual, and there is very little difference in the computations. The edge distance for the angle is the same as the edge distance for the gusset plate and the angle is thinner than the gusset plate, so the angle thickness of 1⁄4 inch will be used. Bolt lines FIGURE P3.6-1 3.6-2 Use A36 steel and select a double-angle tension member to resist a service dead load of 20 kips and a service live load of 60 kips. The 28-day compressive strength of the concrete is f c′ = 4 ksi. Cleveland, OH: The James F. concrete column as shown in Figure 9.27. From AISC Equation J10-3, the nominal strength for this limit state is Rn = Fy tw(2.5k + b) 5.14 Beam Bearing Plates and Column Base Plates 257 For fRn ≥ Ru , 1.0(50)(0.430)[2.5(1.19) + b] ≥ 99.73 resulting in the requirement b ≥ 1.66 in. FIGURE 9.14 A c = 6.700 in.2 4.5″ Y2 15.9″ TABLE 9.4 A y Ay – I d – I + Ad 2 Concrete W16 × 31 6.700 9.130 19.90 7.95 133.30 72.58 — 375 6.89 5.06 318.1in.4 608.8in.4 Sum 15.83 Component y = ANSWER –y Σ Ay ΣA = 205.9 15.83 205.9 = 13.01 in. From Equation 7.20, α= 1 ⎛ 4Tb ′ 1 ⎛ 4(30)(1.988) ⎞ ⎞ − 1 ⎟ = − 1 ⎟ = −0.07657 ⎜ 2 ⎟ ⎠ 0.75 ⎝ 0.90(44)(1.04)2 (65) ⎠ δ ⎝ φb pt f Fu Since a must be between 0 and 1 inclusive, use a = 0. Design a welded connection, balanced to eliminate eccentricity, that will resist the full tensile capacity of the member. In Example 4.3, the width-to-thickness ratios bf 兾2tf and h兾tw were computed. One pair of legs was attached to the web, and the other pair attached to the flange, as shown in Figure 10.1b. The total combined width will be 2(8) + 5⁄16 (the web thickness) = 16.31 inches, or slightly less than the flange width of 18 inches. The flexural-torsional buckling strengths are based on the procedure of AISC E4(a). 176 Chapter 4 Compression Members D = 122k L = 242k 15⬘ FIGURE P4.6-4 4.6-5 Select an HP-shape for the conditions of Problem 4.6-4. If a variable cross section was desired, one or more cover plates of different lengths were riveted to the flanges. For the connection of the shear plate to the column flange, the required strength per inch is Va 26 = = 3.059 kips兾in. No factors are applied to Qn (neither a resistance factor for LRFD nor a safety factor for ASD); the overall flexural resistance factor or safety factor accounts for all strength uncertainties. SOLUTION Compute the nominal strength of the composite section. 5.2-3 A built-up tee-shape (Figure P5.2-3) consists of a 7⁄8-in. The nominal bearing strength specified in AISC J8 is the same as that given in the American Concrete Institute’s Building Code (ACI, 2008) and may be 254 Chapter 5 Beams FIGURE 5.38 used if no other building code requirements are in effect. Determine the strength of the connection. Compute the flexural design strength, fb Mn. c. In this book, we follow the first practice. Prying Action In most connections in which fasteners are subjected to tension forces, the flexibility of the connected parts can lead to deformations that increase the tension applied to the fasteners. Therefore, for an unbraced length Lb = 10 ft, Lp < Lb < Lr 5.12 Holes in Beams 247 and the beam is subject to inelastic lateral-torsional buckling. Use the trial-and-error approach covered in Section 4.6. 1. 8.4-17 A single-angle tension member is connected to a gusset plate as shown in Figure P8.4-17. If both lateral loads and unsymmetrical gravity loads are present, the AJR force should be added to the actual lateral loads when M t is computed. EXAMPLE 5.17 A W10 × 49 is used as a column and is supported by a concrete pier as shown in Figure 5.44. Compute the nominal shear strength of an interior panel with a stiffener spacing of 200 inches. For ASD, Ω = 2.00. This distribution is accounted for by a factor, Cm , applied to the amplification factor given by Expression 6.7. The amplification FIGURE 6.7 6.6 Members in Braced Frames 313 factor given by Expression 6.7 was derived for the worst case, so Cm will never be greater than 1.0. The final form of the amplification factor is B1 = Cm ≥1 1 − (a Pr 兾Pe1) (AISC Equation A-8-3) where Pr = required unamplified axial compressive strength (Pnt + Pt) = Pu for LRFD = Pa for ASD a = 1.00 for LRFD = 1.60 for ASD Pe1 = p 2 EI * (K1 L)2 (AISC Equation A-8-5) EI* = flexural rigidity In the direct analysis method, EI* is a reduced stiffness obtained as EI* = 0.8tbEI (6.8) where tb = a stiffness reduction factor = 1.0 when α Pr ≤ 0.5 Py Pr ⎞ α Pr ⎛ Pr ⎞ ⎛ > 0.5 = 4 ⎜ α ⎟ ⎜ 1 − α ⎟ when ⎝ Py ⎠ ⎝ ⎠ Py Py (AISC Equation C2-2a) (AISC Equation C2-2b) This stiffness reduction factor is the same one used in Chapter 4 in connection with the alignment charts for inelastic columns. Design for a drift index of 1⁄400 based on service wind load. AISC Equation G2-5 is based on elastic stability theory, and AISC Equation G2-4 is an empirical equation for the inelastic region, providing a transition between the limit states of web yielding and elastic web buckling. Because of the support conditions normal to the frame, Ky can be taken as 1.0. According to the AISC Specification, the effective length factor, K, should be determined by a “sidesway buckling analysis” (Chapters E, C, and Appendix 7). > 10.13 kips兾in. The trusses are spaced at 25 feet and are subjected to the following loads: Purlins: Snow: Metal deck: Roofing: Insulation: W6 × 8.5, located at joints and midway between joints 20 psf of horizontal roof surface projection 2 psf 4 psf 3 psf Problems 375 a. D + S = 4342 + 8100 = 12,440 lb The component parallel to the roof is 12 ⎞ T = 12.44⎛ = 3.203 kips ⎝ 46.6 ⎠ The allowable tensile stress is Ft = 0.375Fu = 0.375(58) = 21.75 ksi. 6.8-9 The single-story unbraced frame shown in Figure P6.8-9 is subjected to dead load, roof live load, and wind. All design aids in the Manual give values for both allowable strength design (ASD) and load and resistance factor design (LRFD). Similarly, a connection designed as pinned (simply supported; no moment) would actually be capable of transmitting a moment of about 20% of the fixed-end moment, with a rotation of approximately 80% of the simple support rotation. When Pr ≤ 0.4Pc , Rn = 0.60Fy dc tw (AISC Equation J10-9) When Pr > 0.4Pc , P⎞ ⎛ Rn = 0.60 Fy dc t w ⎜ 1.4 − r ⎟ ⎝ Pc ⎠ (AISC Equation J10-10) where Pr = required axial strength of the column = factored axial load for LRFD = service axial load for ASD Pc = yield strength of the column = Py = Fy Ag for LRFD = 0.6Py = 0.6Fy Ag for ASD Ag = gross cross-sectional area of column Fy = yield stress of column dc = total column depth tw = column web thickness For LRFD, the design shear strength is fRn , where f = 0.90. Provide a complete sketch showing all information needed for the fabrication of the connection. When the bolt is tightened, the protrusions are compressed in proportion to the tension in the bolt. Ω The capacity of the weld across the end of the angle is P3 = 6.960(5) = 34.80 kips Summing moments about an axis along the bottom, we get ∑ ML 2 5 = 80.85(3.26) − 34.80 ⎛⎜ ⎟⎞ − P1 (5) = 0, P1 = 35.31 kips ⎝ 2⎠ ∑ F = 80.85 − 35.31 − 34.80 − P2 = 0, P2 = 10.74 kips 35.31 P1 = = 8.95 in. The nominal strength is Rn = 6.25Fy f t 2f (AISC Equation J10-1) where Fy f = yield stress of the column flange t f = thicknesss of the column flange For LRFD, the design strength is fRn , where f = 0.90. Different grades of structural steel are identified by the designation assigned them by the American Society for Testing and Materials (ASTM). Investigate a typical interior beam for compliance with the AISC Specification if no temporary shores are used. Lothars, J. However, a continuous linear stress distribution as shown in Part c of the figure is valid only if the beam is assumed to be homogeneous. P 8″ W14 × 159 WT7 × 41 16″ 5⁄16 A992 steel E70 FIGURE P8.5-5 Moment-Resisting Connections 8.6-1 A W18 × 50 beam is connected to a W14 × 99 column. If we initially assume that AISC Equation H1-1a governs, then Pr 8 ⎛ Mrx Mry ⎞ + + ≤ 1.0 Pc 9 ⎝ Mcx Mcy ⎠ (AISC Equation H1-1a) This can be written as ⎛ 8 ⎞ ⎛ 1⎞ ⎛ 8 ⎞ ⎝ P ⎠ Pr + ⎝ 9 M ⎠ Mrx + ⎝ 9 M ⎠ Mry ≤ 1.0 ⎝ c cx cy ⎠ or pPr + bxMrx + by Mry ≤ 1.0 (6.9) where p= 1 Pc bx = 8 9 Mcx by = 8 9 Mcy If AISC Equation H1-1b controls (that is, Pr兾Pc < 0.2, or equivalently, pPr < 0.2), then use Pr ⎛ Mrx Mry ⎞ + + ≤ 1.0 2 Pc ⎝ Mcx Mcy ⎠ (AISC Equation H1-1b) or 9 0.5 pPr + (bx Mrx + by Mry) ≤ 1.0 8 (6.10) Table 6-1 in Part 6 of the Manual gives values of p, bx , and by for various W shapes, in both LRFD and ASD format. Determine the required plate thickness. Because of the thinness of the cross-sectional elements, the problem of instability (discussed in Chapters 4 and 5) is a particularly important factor in the design of cold-formed steel structures. Assume one stud at each beam location. EXAMPLE 4.7 Select the lightest W-shape that can resist a service dead load of 62.5 kips and a service live load of 125 kips. We always use the design dimension in calculations.) Use AISC Equation J10-5 to determine the value of b required to prevent web crippling. To analyze this case, we first determine whether the PNA is in the top flange or in the web. The current version, the fourteenth edition, also covers both ASD and LRFD. This connection, in either bolted or welded form, is commonly used to connect beams to columns. Hence the principal axes and the corresponding radii of gyration about these axes must be determined. 5.11-3 Select a W shape for the following conditions: Beam spacing = 12 ft Span length = 25 ft Slab and deck combination weight = 51 psf Partition load = 20 psf Miscellaneous dead load = 10 psf Live load = 80 psf Fy = 50 ksi The maximum live load deflection cannot exceed L兾360. Modern reinforcing bars are deformed—that is, the surface of the bar has protrusions that help the steel grip or bond with the concrete. The factored load is Pu = 1.2D + 1.6L = 1.2(20) + 1.6(40) = 88 kips and the shear 兾bearing load per bolt is 88兾8 = 11 kips. (A member with these properties is said to have sufficient strength and would be at the verge of failure when subjected to the factored loads.) 3. Therefore, the allowable strength is Pn = Fa Ag = 15.07(21.8) = 329 kips Ωc b. 384 EI 29,000(291) 384 (N.G.) Solving the deflection equation for the required moment of inertia yields I required = 5wL L4 5(0.700兾12)(30 × 12)4 = = 440 in.4 384 EΔ required 384(29,000)(1.0) Part 3 of the Manual contains selection tables for both Ix and Iy. These tables are organized in the same way as the Zx table, so selection of the lightest shape with sufficient moment of inertia is simple. EXAMPLE 6.7 Select a W shape of A992 steel for the beam–column of Figure 6.22. ⎝ 2⎠ The base metal shear strength is therefore 10.8 kips兾in. 1.4 DESIGN SPECIFICATIONS In contrast to building codes, design specifications give more specific guidance for the design of structural members and their connections. > p fi ∴ Use the originall value of p fi = 2.0 in. Effective Length 4.7-1 A W18 × 97 with Fy = 60 ksi is used as a compression member. For a positive service load moment of 135 ft-kips, compute the stress at the top of the steel (indicate whether tension or compression), the stress at the bottom of the steel, and the stress at the top of the concrete. Equation 2.5 can also be written in the form Ru ≤ fRn where (2.6) Ru = required strength = sum of factored load effects (forces or moments) Section B2 of the AISC Specification says to use the load factors and load combinations prescribed by the governing building code. Bolt tension: 14.6 kips < 19.9 kips; shear: 6.25 kips < 11.9 kips; bearing: 6.25 kips < 25.5 kips Chapter 9: Composite Construction 9.1-1 a. This part includes a brief overview of various specifications (including a detailed discussion of the AISC Specification), codes and standards, some fundamental design and fabrication principles, and a discussion of the proper selection of materials. 2 2 2 M n = Cy = 515(12.51) = 6443 in.-kips = 536.9 ft-kips a= LRFD SOLUTION Before the concrete cures, the factored load and moment are wu = 1.2 wD + 1.6 wL = 1.2(541.3) + 1.6(180) = 937.6 lb兾ft 1 1 Mu = wu L2 = (0.9376)(30)2 = 106 ft-kips 8 8 From the Zx table, fb Mn = fb Mp = 249 ft-kips > 106 ft-kips (OK) After the concrete cures, the factored load and moment are wu = 1.2 wD + 1.6 wL = 1.2(541.3) + 1.6(1305) = 2738 lb兾ft Mu = 1 (2.738)(30)2 = 308 ft-kips 8 The design strength of the composite section is fb Mn = 0.90(536.9) = 483 ft-kips > 308 ft-kips (OK) 9.3 Effective Flange Width Check shear: wu L 2.738(30) = = 41.1 kips 2 2 Vu = From the Zx table, fvVn = 159 kips > 41.1 kips ANSWER ASD SOLUTION (OK) The beam complies with the AISC Specification. r Fy Fcr = 0.877 Fe = 0.877(21.27) = 18.65 ksi fc Pn = fc Fcr Ag = 0.90(18.65) (35.1) = 589 kips < 600 kips (N..G.) This is very close, so try the next larger size. The weight of the slab, along with the loads it supports, is supported by the floor beams. The probability of an event having a value falling between a and b in Figure 2.1 equals the area of the diagram between a and b. Assume that the components are connected in such a way that the member is fully effective and compute the strength based on flexural buckling. < 8.352 kips兾in. The stiffeners are clipped 1⁄2 inch to clear the flange-to-web welds. The thin plate approach requires the incorporation of prying action, whereas the thick plate theory does not. Sometimes a built-up shape must be used because none of the standard rolled shapes are large enough; that is, the cross section does not have enough area or moment of inertia. This method is represented schematically in Figure 6.27. Assume continuous lateral support and select a shape of A992 steel. That is, Ru = Ra Ω f 1.2 D + 1.6 L = (D + L)Ω f or 1.2 D + 1.6(3D) = (D + 3D)Ω f Ω= 1.5 f 30 Chapter 2 2.5 Concepts in Structural Steel Design PROBABILISTIC BASIS OF LOAD AND RESISTANCE FACTORS Both the load and the resistance factors specified by AISC are based on probabilistic concepts. If the absolute value of a, obtained from Equation 7.15, is less than 1.0, the moment at the bolt line is less than the moment 7.8 High-Strength Bolts in Tension 423 at the face of the stem, indicating that the beam mechanism has not formed and the controlling limit state will be tensile failure of the bolt. It is connected to a gusset plate with 3⁄4-inch-diameter bolts through the long legs. This type of instability is a localized buckling or wrinkling at an isolated location. FIGURE 5.24 LRFD SOLUTION Ignore the beam weight initially then check for its effect after a selection is made. Solve for P from Equation 8.4. Substitute r0 and P into Equations 8.3 and 8.5. If these equations are satisfied within an acceptable tolerance, the analysis is complete. ASCE 104 (no. The member will be 12 feet long and will be connected to a 3⁄8-inch-thick gusset plate. Pa = D + L = 15 + 45 = 60 kips a. Using the design thickness in strength computations is a conservative way to account for tolerances in the manufacturing process. Determine the effective length factors with respect to each axis for member AB. ⎝ 16 ⎠ 500 Chapter 8 Eccentric Connections From Equation 7.36, the base-metal shear rupture strength per unit length is ⎛ 9⎞ φ Rn = 0.45Fu t = 0.45(58) ⎜ ⎟ = 14.7 kips兾in. In buildings, vertical compression members such as these are referred to as columns. The reason is that the spacing of the studs is limited by the spacing of the ribs, and the exact number of required studs cannot always be used. Flexural members with noncompact webs are covered in Section F4, “Other I-shaped Members with Compact or Noncompact Webs Bent About Their Major Axis.” This section deals with both doubly and singly symmetric sections. The unmodified slenderness ratio is ⎛ KL ⎞ = KL = 16(12) = 154.8 ⎝ r ⎠ 0 ry 1.24 The spacing of the connectors is a = 16(12) = 48 in. Roof trusses normally are spaced uniformly along the length of the building and are tied together by longitudinal beams called purlins and by x-bracing. From Table 6-1, for Lb = 20 ft, bx = 9.45 × 10−3 bx 9.45 × 10 −3 = = 5.66 × 10 −3 Cb 1.67 For Lb = 0, bx = 6.25 × 10−3 ANSWER Since 6.25 × 10−3 > 5.66 × 10−3, use bx = 6.25 × 10–3. The bending moment at a distance x from the left end is ⎛ x⎞ M = 60.48 x − 4.44 x ⎜ ⎟ = 60.48 x − 2.22 x 2 ⎝ 2⎠ (for x ≤ 12 ft) For x = 3 ft, MA = 60.48(3) − 2.22(3)2 = 161.5 ft-kips For x = 6 ft, MB = 60.48(6) − 2.22(6)2 = 283.0 ft-kips For x = 9 ft, MC = 60.48(9) − 2.22(9)2 = 364.5 ft-kips For x = 12 ft, Mmax = Mu = 60.48(12) − 2.22(12)2 = 406.1 ft-kips Cb = = 2.5 M max 12.5 M max + 3 M A + 4 M B + 3MC 12.5(406.1) = 1.36 2.5(406.1) + 3(161.5) + 4(283.0) + 3(364.5) Enter the charts with an unbraced length Lb = 12 ft and a bending moment of Mu 406.1 = = 299 ft-kips 1.36 Cb Try W21 × 48: fb Mn = 311 ft-kips (for Cb = 1) Since Cb = 1.36, the actual design strength is 1.36(311) = 423 ft-kips. From Figure 10.11, 3 Iy = 1 1 ⎛ 3⎞ (1)(16)3 + (10.5) ⎜ ⎟ = 341.4 in.4 ⎝ 8⎠ 12 12 ⎛ 3⎞ A = 16(1) + 10.5 ⎜ ⎟ = 19.94 in.2 ⎝ 8⎠ rt = Iy 341.4 = = 4.138 in. > 4 in. 372 kips 3.4-3 a. “Ponding of Concrete Deck Floors.” Engineering Journal, AISC 23 (no. D = 100k L = 300k 12′ FIGURE P4.6-3 4.6-4 Select a steel pipe. M ay ⎞ 134 8 ⎛ 72.25 Pa 8 ⎛ M ax ⎞ + + = + + 0 ⎟ = 0.922 < 1.0 (OK) ⎝ ⎠ Pn 兾Ωc 9 ⎝ M nx 兾Ωb M ny 兾Ωb ⎠ 270 9 ⎝ 151 ANSWER This member satisfies the AISC Specification. In some cases, the ASD solution is abbreviated but complete and independent of the LRFD solution. No: Ma = 1080 ft-kips > Mn兾Ωb = 943 ft-kips 5.5-5 a. Instead of applying this reduction directly to the bolt area, a factor of 0.80 is applied to Fnv. A nonuniform case is illustrated in the Commentary to the Specification. p= 8 = 4 in. The inside faces of the flanges are sloping, just as with the American Standard shape. Specifications and Codes. For some cross-sectional configurations, the member will fail by twisting (torsional buckling) or by a combination of twisting and bending (flexural-torsional buckling). C 384 = = 1.255 in. The loads are service loads with a live-load–to–dead-load ratio of 3.0. The uniform load of 4 kips兾ft includes the weight of the girder. In each of the connections discussed, the vertical welds on the column flange are loaded as shown in Figure 8.25. The 5-inch spacing can be used when 10.13I x 10.13 (60,640) VuQ 10.13 = or Vu = = = 143.3 kips 857.2(5) Ix s Qs Refer to Figure 10.18 and let x be the distance from the left support, giving Vu = 223.4 – 4.34x = 143.3 kips x = 18.46 ft The 13.5-inch spacing can be used when Vu = 10.13I x 10.13 (60,640) = = 53.08 kipss Qs 857.2(13.5) 710 Chapter 10 Plate Girders FIGURE 10.23 Figure 10.18 shows that the shear never gets this small, so the maximum spacing never controls. These steels are higher in strength than the plain carbon steels and also have some special quality, such as resistance to corrosion. Determine the required bolt diameter. For the most favorable condition of both ends fixed against rotation and translation, K = 0.5. Values of K for these and other cases can be determined with the aid of Table C-A-7.1 in the Commentary to AISC Specification Appendix 7. Single-shear Slip-critical design Single-shear design strength, one slip allowable strength,fRn plane, fRn strength, Rn 兾 Ω (kips) (kips) (kips) 7.07 . For an explanation, refer to Figure 6.3, which shows a beam–column with an axial load and a transverse uniform load. Structural Steel Design. Compute the nominal flexural strength for an unbraced length of 10 feet. Then, 15 = 2.063 in. 4.71 E 29, 000 = 4.71 = 123.7 QFy 0.9141(46) KL = 105.3 < 123.7 r ∴ Use AISC Equation E7-2 QFy ⎞ ⎛ Fcr = Q ⎜ 0.658 Fe ⎟ Fy ⎟ ⎜ ⎟⎠ 0.9141(46) ⎞ ⎛ = 0.9141 ⎜ 0.658 25.81 ⎟ 46 = 21.26 ksi ≠ 19.76 ksi (the assumed value) ⎝ ⎠ Try f = 21.26 ksi: be = 1.92(0.116) 29, 000 ⎡ 0.38 ⎢1 − 21.26 ⎢⎣ (66.0) 29, 000 ⎤ ⎥ = 6.477 in. Threaded Rods and Cables 3.7-1 Select a threaded rod to resist a service dead load of 43 kips and a service live load of 4 kips. CL 30′-0″ W14 × 82 FIGURE P8.3-9 8.3-10 Same as Problem 8.3-9, but use ASD. However, the value obtained from Equation 3.1 is more accurate. Ω(2 M a) p Ft (h0 + h1) db = where Ω = 2.00 4. a ′ = a + = 3.031 + 2 2 b′ = b − 1 7 1 = + = 1 in. Bending is assumed to take place about axes at middepth of the plate near the edges of the column flanges. ⎝ 16 ⎠ Ω The base metal shear strength is therefore 8.10 kips兾in. 4): 109–115. a C A D B F G H E FIJ I a 13.40k (b) Member IJ is the bottom chord member with the largest force. ⎝ 8⎠ The weld strength of 4.176 kips兾in. SOLUTION Strong-axis buckling will control when K L 16 Kx L > K y L; that is, whenever rmx 兾rmy < x = =2 rmx 兾rmy KyL 8 An inspection of Table 4-14 shows that rmx兾rmy is always < 2, therefore x-axis buckling always controls. The applied load is a service load. The resulting axial load in the top-chord member is the axial compressive load to be used in the design. Flush and Extended Multiple Row Moment End Plate Connections. 10⬘ C12 × 20.7 FIGURE P4.8-2 Problems 4.8-3 183 Select a WT section for the compression member shown in Figure P4.8-3. Beam CL Problems 581 Eccentric Welded Connections: Shear Only 8.4-1 Use an elastic analysis and determine the maximum load in the weld (in kips per inch of length). With this approach, known as the method of inequalities, the correct mechanism can be determined directly by linear programming techniques. 6.6 MEMBERS IN BRACED FRAMES The amplification factor given by Expression 6.7 was derived for a member braced against sidesway—that is, one whose ends cannot translate with respect to each other. With some notational changes, the nominal moment strength is Mn = Fcr Sx FIGURE 5.12 FIGURE 5.13 204 Chapter 5 Beams where Fcr is the elastic buckling stress and is given by Fcr = π L bS x 2 ⎛ pE⎞ EI y GJ + ⎜ ⎟ I yCw, ksi ⎝ Lb ⎠ (5.5) where Lb = unbraced length (in.) Iy = moment of inertia about the weak axis of the cross section (in.4) G = shear modulus of structural steel = 11,200 ksi J = torsional constant (in.4) Cw = warping constant (in.6) (The constants G, J, and Cw were defined in Chapter 4 in the discussion of torsional and lateral-torsional buckling of columns.) Equation 5.5 is valid as long as the bending moment within the unbraced length is uniform (nonuniform moment is accounted for with a factor Cb, which is explained later). FIGURE 7.41 Minimum Length The minimum permissible length of a fillet weld is four times its size. Working loads are also known as service loads. For a rectangular cross section of depth d and width b, the error for d兾b = 2 is approximately 3%. Values of fcFcr in Table 4-22 are given only for integer values of KL兾r; for decimal values, KL兾r may be rounded up or linear interpolation may be used. Assume that the controlling load combination causes no moment in AB. Members designed by plastic theory would reach the point of failure under the factored loads but are safe under actual working loads. For LRFD, Mt will be a factored load moment, and for ASD, it will be a service load moment. The general procedure is to assume a shape and then compute its strength. Compression flange buckling can take the form of vertical buckling into the web or flange local buckling (FLB), or it can be caused by lateral-torsional buckling (LTB). ⎝ 8⎠ The weld strength of 2.784 kips兾in. This will be addressed presently. E70XX electrodes are used. 3.712 23.23 − 8 = 7.62 in. Tension members are frequently connected at their ends with bolts, as illustrated in Figure 3.2. The tension member shown, a 1⁄2 × 8 plate, is connected to a gusset plate, which is a connection element whose purpose is to transfer the load from the member to a support or to another member. We will consider compression to be positive. Although a complete treatment of probability theory is beyond the scope of this book, we present a brief summary of the basic concepts here. The Specification allows a simplifying assumption, however, so iteration can be avoided. The threads are not in the shear planes. Rods and cables are frequently used in suspended roof systems and as hangers or suspension members in bridges. ⎝ 16 ⎠ From Equation 7.36, the base metal shear rupture strength per unit length is 5 ⎞ φ Rn = 0.45Fu t = 0.45(58) ⎛ = 8.156 kips兾in. Table 4.1 shows that the Manual does not give the constants r0 and H for tees, although they are given on the Companion CD. There are no intermediate stiffeners, and A36 steel is used throughout. Check local stability (check the width-to-thickness ratios). AISC Steel Design Guide Series No. 2, Chicago: AISC. Determine the design strength. To determine the area of steel that will resist the same force as the concrete, divide the concrete area by n. First, the ultimate shear stress is taken as 0.625 times the ultimate tensile stress (Fisher et al., 1978). (The upper limit of Mpx兾Ωb does not control.) 346 Chapter 6 Beam–Columns EXAMPLE 6.9 A structural member in a braced frame must support the following service loads and moments: an axial compressive dead load of 25 kips and a live load of 75 kips; a dead load moment of 12.5 ft-kips about the strong axis and a live load moment of 37.5 ft-kips about the strong axis; a dead load moment of 5 ft-kips about the weak axis and a live load moment of 15 ft-kips about the weak axis. The angles are attached to a 3⁄8-inch-thick gusset plate. Width-to-thickness parameters: Use the limiting ratios for single angles from Table B4.1b: λ= b t λ p = 0.54 E Fy λr = 0.91 E Fy where b is the leg length and t is the thickness. 4.9-5 25,600 kips 4.9-7 b. If there are no transverse loads acting on the member, ⎛M ⎞ Cm = 0.6 − 0.4 ⎜ 1 ⎟ ⎝ M2 ⎠ (AISC Equation A-8-4) M1兾M2 is a ratio of the bending moments at the ends of the member. For the inner holes and a spacing of 3 inches, Manual Table 7-4 gives 1 1 φrn = 65.3t = 65.3 ⎛⎜ + ⎞⎟ = 32.7 kips兾bolt ⎝ 4 4⎠ For the holes nearest the edge of the plate, we will conservatively use an edge distance of 11⁄4 inches to find the bearing strength from Manual Table 7-5. Compute the available strength for LRFD. 5.11-7 Same as Problem 5.11-6, but let all the loads on the girder act as a uniform load (be sure to include the weight of the beams). 174 Chapter 4 Compression Members 10⬘ HSS 6.625 × 0.250 ASTM A500 Grade B (Fy = 42 ksi) FIGURE P4.3-8 Local Stability 4.4-1 An HSS10 × 8 × 3⁄16 is used as a compression member with one end pinned and the other end fixed against rotation but free to translate. These equations are based on experimental and theoretical studies that account for the effects of residual stresses and an initial out-of-straightness of L兾1500, where L is the member length. The prying force has shifted to the tip of the flange and it is at its maximum value. In keeping with the objective of providing a basic textbook, a large number of assigned problems are given at the end of each chapter. PL = 30k wD = wL = 3.5k/ft 1.0k/ft A D B 9′ C 9′ 18′ 36′ FIGURE P5.5-16 Bending Strength of Noncompact Shapes 5.6-1 A W21 × 48 is used as a simply supported, uniformly loaded beam with a span length of 50 feet and continuous lateral support. Design the steel anchors. Without a Tension Field. A W8 × 58 has an interpolated strength of fcPn = 596 kips > 560 kips (OK) Next, investigate the W10 shapes. In general, the magnitude of a live load is not as well defined as that of a dead load, and it usually must be estimated. Blodgett, O. Another source of error is the assumption that the strength of the weld is independent of the direction of the applied load. Building Code Requirements for Structural Concrete and Commentary (ACI 318-08). paper industry, the metalworking and steel industries, and for solid/liquid separation in the municipal and industrial sectors. by Gertude Arnold 128. Compute the design compressive strength for LRFD and the allowable compressive strength for ASD. Use Group A bolts and A572 Grade 50 steel for both the tension member and the gusset plate. A formula that will give the required area (or, alternatively, the required weight per foot of length) can be developed if a beam depth is assumed. pPu + bx Mux + by Muy = (3.00 × 10−3)(150) + (4.32 × 10−3)(75) + (11.7 × 10−3)(30) = 1.13 > 1.0 (N.G.) ANSWER ASD SOLUTION Use a W10 × 49. 65.0 kips Chapter 8: Eccentric Connections 8.2-1 26.5 kips 8.2-3 43.7 kips 8.2-5 11.5 kips 8.2-7 a. The bearing strength for these bolts is rn = 29.4 t kips兾bolt Ω To find the required plate thickness, equate the total bearing strength to the applied load: 29.4t + 2(52.2t) = 26, t = 0.194 in. Note that compression members have different criteria than flexural members, so a shape could be compact for flexure but slender for compression. From Figure 6.24b, d ≈ Δcosq where Δ is the lateral displacement. fc Pn 405 ( ) Muy ⎞ 200.4 8 107.1 8 ⎛ Mux Pu + ⎟ + = + + 0 = 0.915 < 1.0 (OK) f c Pn 9 ⎝ fb M nx fb M ny ⎠ ⎠ 405 9 226.5 ANSWER This member satisfies the AISC Specification. If bearing stiffeners are used, the following design procedure can be used. (determined experimentally) As with the elastic analysis, it is more convenient to work with rectangular components of forces, or Ry = x y R and Rx = R r r where x and y are the horizontal and vertical distances from the instantaneous center to the fastener. 60 Chapter 3 Tension Members FIGURE 3.16 EXAMPLE 3.7 An angle with staggered fasteners in each leg is shown in Figure 3.17. In some of the examples, particularly in the later chapters, only an LRFD solution is given. (. If a tension member is being connected, tension on both the gross area and effective net area must be investigated. Thus the plate is treated as a cantilever of span length n = (B – 2k)兾2 and a width of b. Plates

In general, U = 1.0 for plates, since the cross section has only one element and it is connected. Use A992 steel and select a W10 shape. The design or analysis of composite beams with formed steel deck is essentially the same as with slabs of uniform thickness, with the following exceptions. The moment at this location is Mmid = −Mp + V L (7.5) − wu 7.5)2糞2 = − 1170 + 312(7.5) − 20.8(7.5)2糞2 = 585 ft-kips 724 Appendix Plastic Analysis and Design Because this is a positive value, Mmid causes compression in the same flange as M2, and Mmid is treated as positive in subsequent computations. For this plate girder, the bearing stiffeners will serve as intermediate stiffeners, and a 12(12) ≈ = 2.286 h 63 (This ratio is approximate because a is not exactly 12 feet. design problems in which there is an element of trial and error in the solution procedure or for which there is more than one acceptable answer b. for bolt diameters up to 1 inch and db + 3⁄4 in. Finally, a flag placed at the bend in the reference line indicates a field weld. ii. From Table 3-19, the allowable strength for this composite beam is 3 Mn = 166 ft-kips > 146 ft-kips Ωb ANSWER (OK) Use a W14 × 26 and 20 studs, 3⁄4 × 3, one every third rib. Note that some other limit state that has not been checked, such as tension on the net area of the member, may govern the strength of the connection of Example 7.2. EXAMPLE 7.3 A plate 3⁄8 × 6 is used as a tension member to resist a service dead load of 12 kips and a service live load of 33 kips. For LRFD, the analysis results for the controlling factored load combination are Pnt = 300 kips, Mntx = 135 kips, and Mnty = 30 ft-kips. The holes are for 3⁄4-inch-diameter bolts. From Equation 5.23, M ay M ax 8.862 0.7383 + = + = 0.757 < 1.0 M nx 糞Ωb M ny 糞Ωb 28.6 3.300糞2 (OK) 276 Chapter 5 Beams The maximum shear is Va = 0.3151(15) = 2.36 kips 2 From the uniform load tables, Vn = 31.0 kips > 2.36 kips Ωv ANSWER 5.16 (OK) Use a C10 × 15.3. BENDING STRENGTH OF VARIOUS SHAPES W, S, M, and C shapes are the most commonly used hot-rolled shapes for beams, and their bending strength has been covered in the preceding sections. R. That is, ΣRy in Table 8.1 is the same as C.) To obtain a safe connection load, this constant must be multiplied by the available strength of the particular fastener used. Determine whether the beam is adequate for ASD. Determine the required weld size by the ultimate strength method (use the tables in Part 8 of the Manual). The table also gives the approximate weight in pounds per foot of length. 3.13 in.2 3.3-3 158 kips 3.3-5 a. Both longitudinal and transverse welds will be used. To determine the required weld size, let 1.392D = 2.294, D = 1.65 sixteenths of an inch The minimum weld size from AISC Table J2.4 is 3⁄16 inch, based on the thinner connected part (the shear plate). The remaining details are the same as for a bolted connection. High-strength Bolting for Structural Joints. Built-up shapes, either from plates, rolled shapes, or a combination of plates and rolled shapes, are sometimes used when large loads must be resisted. Lateral-torsional buckling: If Lb ≤ Lp , there is no LTB If Lp < Lb ≤ Lr , there is inelastic LTB, and ⎛ Lb − L p ⎞ ⎤ ⎡ Mn = Cb ⎢ Mp − (Mp − 0.7 Fy S x) ⎜ ⎥ ≤ Mp ⎝ Lr − L p ⎟⎠ ⎦ ⎣ 216 Chapter 5 Beams If Lb > Lr , there is elastic LTB, and Mn = Fcr Sx ≤ Mp where Fcr = 5.8 Cb p 2 E Jc ⎛ Lb ⎞ 1 + 0.078 ⎜ ⎟ S x h0 ⎝ rts ⎠ (Lb 糞rts)2 2 SHEAR STRENGTH Beam shear strength is covered in Chapter G of the AISC Specification, “Design of Members for Shear.” Both hot-rolled shapes and welded built-up shapes are covered. Two formulas are given in the Specification: one for small axial load and one for large axial load. The curved line in the graph is the moment-curvature relation for the connection that is used with the beam. Camber is a curvature in the opposite direction of the dead load deflection curve and can be accomplished by bending the beam, with or without heat. From AISC Equation F4-12, aw = 63 (3糞8) hc t w = = 1.477 < 10 b fc t fc 16(1.0) 10.6 Bearing Stiffeners 685 From AISC Equation F5-6, Rpg = 1 − aw 1200 + 300 aw = 1− ⎛ hc E⎞ ⎟ − 5.7 ⎟ ≤ 1.0 Fy ⎟⎠ ⎝ tw ⎛ 1.447 29, 000 ⎞ ⎜ 168 − 5.7 ⎟ = 0.9945 1200 + 300(1.447) ⎝ 36 ⎠ ⎟ From AISC Equation F5-7, the nominal flexural strength for the compression flange is Mn = RpgFcr Sxc = 0.9945(36)(1248) = 44,680 in.-kips = 3723 ft-kips To check lateral-torsional buckling we need the slenderness ratio rt. I-shaped cross sections with thin flanges or webs are susceptible to this phenomenon, and their use should be avoided whenever possible. Problems Bearing Strength, Spacing, and Edge-Distance Requirements 7.3-1 The tension member is a PL 1⁄2 × 6. Qn = 0.5(0.4418) 4(3492) = 26.11 kips Rg R p Asa Fu = 1.0(0.6)(0.4418)(65) = 17.23 kips < 26.11 kips ∴ Use Qn = 17.23 kips. EXAMPLE 9.4 A composite floor system consists of W18 × 35 steel beams spaced at 9 feet and supporting a 4.5-inch-thick reinforced concrete slab. In 1947, the Research Council of Riveted and Bolted Structural Joints was formed, and its first specification was issued in 1951. Disque, Robert O. The maximum resultant stress can be found by adding these two components vectorially: fr = fv2 + ft2 For units of kips and inches, this stress will be in kips per square inch. B., Galambos, T. Find the total bolt force from Equation 7.14. For a group of four welds, the allowable strength is 4(0.9279DL) = 4(0.9279)(3)(1.5) = 16.7 kips The required weld capacity per inch of stiffener length is P 87.05 = = 4.419 kips糞in. Neglect flexural-torsional buckling and compute the available strength for Kx L = Ky L = 30 feet. The shear consists of a 13-kip service dead load and a 34-kip service live load. 0.992 kips糞ft 5.6-3 5360 ft-kips 5.8-1 764 kips 5.8-3 a. For a homogeneous material, the neutral axis coincides with the centroidal axis. KxL Approximate rmx 糞 rmy rmx 糞 rmy 1.2 1.3 1.4 1.5 1.6 1.8 13.3 12.3 11.4 10.7 10.0 8.9 (ft) Shape Actual rmx 糞 rmy 1 HSS12 × 10 × ⁄2 5 HSS14 × 10 × ⁄16 5 HSS12 × 8 × ⁄8 5 HSS20 × 12 × ⁄8 3 HSS20 × 12 × ⁄8 5 ⁄ HSS16 × 8 × 16 1.17 1.34 1.41 1.55 1.56 1.82 Pn 糞Ωc (kips) Weight (lb糞ft) 565 518 546 > 1170 > 900 504 69.3 48.9 76.3 127.0 78.5 48.9 Problems 655 The lightest weight is 48.9 lb糞ft, a property of both the HSS14 × 10 × 5⁄16 and the HSS16 × 8 × 5⁄16, but the HSS14 × 10 × 5⁄16 has a larger allowable strength. But if intermediate values are rounded, recorded, and used in subsequent computations, then a consistent number of significant figures will not have been used. The requirements for stiffeners when a tension field is not present are given in AISC G2.2. The required moment of inertia of a pair of stiffeners about an axis through the web is I st ≥ bt w3 j (AISC Equation G2-7) where j= 2.5 − 2 ≥ 0.5 (a糞h)2 (AISC Equation G2-8) b = smaller of a and h. Try a W12 × 53 (Pn糞Ωc = 407 kips for KL = 9 ft): Kx L 18 = = 8 .53 ft < 9 ft rx 糞 ry 2.11 ∴ K y L controls for this shape, and Pn 糞Ω c = 407 kips. For standard holes (the only type considered in this book), these factors are f = 1.00 and Ω = 1.50 Although slip-critical connections are designed to not slip, if slip does occur because of an overload, the bolts must be capable of resisting shear and bearing. Use AISC Equation E3-2 or E3-3. For x-axis bending, Max = MDx + MLx = 48 + 144 = 192 ft-kips For y-axis bending, May = MDy + MLy = 6 + 18 = 24 ft-kips Check interaction equation 5.23: M ay M ax 192 24 + = + = 0.979 < 1.0 (OK) M nx 糞Ωb M ny 糞Ωb 548.6糞1.67 101.7糞1.67 (Note that Mnx糞Ωb can be obtained from the beam design charts.) ANSWER The W21 × 68 is satisfactory. 45 k 5 k/ft 12′ FIGURE P6.8-3 372 Chapter 6 6.8-4 Beam–Columns A member in a braced frame supports an axial compressive load and end moments that cause bending about both axes of the member. The design strength for plate flexural yielding is fbFytp2Yp where fb = 0.90 tp = end plate thickness Yp = yield line mechanism parameter (8.13) 8.8 End Plate Connections 563 For a four-bolt unstiffened extended end plate, the yield-line mechanism parameter is given in AISC Design Guide 4 as Yp = bp 2 ⎛ 1 ⎞ 1⎤ 2 ⎡ ⎛ 1 1⎞ + ⎜ + h0 ⎟ − ⎥ + [h1 (p fi + s)] ⎢ h1 ⎜ ⎝ p fo ⎟⎠ 2 ⎦ g ⎣ ⎝ p fi s ⎠ (8.14) where s= 1 bp g 2 (if pf i > s, use pfi = s) To ensure thick-plate behavior (no prying action), use 90% of the strength given by Equation 8.13 to match the moment strength provided by the bolts (fMn from Equation 8.12). They are used in floor and roof systems in a wide variety of structures. 2 0.90 Fy 0.90(50) and the required effective net area is Ae ≥ Pu 48.0 = = 0.985 in. 1 φrn = 45.7t = 45.7 ⎛⎜ ⎞⎟ = 11.4 kips糞bolt ⎝ 4⎠ Using the minimum of shear and bearing strengths for each bolt location, the connection strength based on shear and bearing is φ Rn = edge bolt strength + 3(inner bolt strength)) = 11.4 + 3(24.4) = 84.6 kips > 48.0 kips (OK) Now check block shear. The load capacity of K-series joists must be verified by the manufacturer by testing. The reason is that the small slenderness ratio for members of this type causes a large buckling stress (from Equation 4.4). If an odd number of bolts had been required and two rows had been used, the symmetry would not exist and the connection would be eccentric. 2): 55–64. First, we will use the basic electrode strength of 0.6FEXX for both the longitudinal and transverse welds (this corresponds to fRn = 6.960 kips糞in.) The capacity of the weld across the end of the angle is P3 = 6.960(5) = 34.80 kips Summing moments about an axis along the bottom, we get ∑ ML 2 5 = 121.5(3.26) − 34.80 ⎛ ⎞ − P1 (5) = 0, P1 = 61.82 kipps ⎝ 2⎠ ∑ F = 121.5 − 61.82 − 34.80 − P2 = 0, P2 = 24.88 kips P1 61.82 = = 8.88 in. (equal to the beam flange width). The equation of the sloping line is given by ⎛ ft ⎞ ⎛ fv ⎞ ⎟ ⎜ F ⎟⎠ + ⎜ ⎝ F ⎟⎠ = 1.3 t v (7.24) To avoid going above the line, ⎛ ft ⎞ ⎛ fv ⎞ ⎟ ⎜ F ⎟⎠ + ⎜ ⎝ F ⎟⎠ ≤ 1.3 t v If Equation 7.24 is solved for the required tensile strength ft, we obtain, for a given fv, ft = 1.3Ft − fv Ft Fv (7.25) Let Available strength = Φ × nominal strength or Nominal strength = available strength Φ where Φ = φ for LRFD = 1 for ASD Ω If ft is viewed as the available tensile strength in the presence of shear, then from Equation 7.25, the corresponding nominal strength is ft F F = 1.3 t − t fv Φ Φ ΦFv 7.9 Combined Shear and Tension in Fasteners 431 or Fnt′ = 1.3Fnt − Fnt f Fv v Fnt′ = 1.3Fnt − Fnt frv ΦFnv or where F′nt Fnt Fnv frv (7.26) = nominal tensile stress in the presence of shear = nominal tensile stress in the absence of shear = nominal shear stress in the absence of tension = required shear stress Note that F′nt must not exceed Fnt, and frv must not exceed Fnv. The column loads accumulate from the top story to the foundation. The area of the bar at section a–a is (1⁄2)(8) = 4 in.2, but the area at section b–b is only 4 – (2)(1⁄2)(7⁄8) = 3.13 in.2 41 42 Chapter 3 Tension Members FIGURE 3.1 FIGURE 3.2 8″ 7⁄8″-diameter 1⁄2″ holes b b a a Bar 8 × 1⁄2 Section a–a 7⁄8″ Section b–b and will be more highly stressed. Because the applied shear force V will usually be variable, the spacing of intermittent welds, if used, can also vary. 1952. bolts 3 Weld (d) (c) 2″ 2″ 2″ 2″ PL5⁄8 × 6 7⁄8-in.-diam. 8t = 8(4.5) = 36 in. Steel, an alloy of primarily iron and carbon, with fewer impurities and less carbon than cast iron, was first used in heavy construction in the nineteenth century. Engesser’s tangent modulus theory had its detractors, who pointed out several inconsistencies. Slip-critical connection. 7.8 High-Strength Bolts in Tension 425 Although the prying analysis presented here is for tee sections, with a slight modification it can be used for double angles. Myrtle Beach, S.C.: Steel Joist Institute, 2005. Type FR connections (rigid connections), however, should be stiff enough so that relative rotation of the connected members is kept to a minimum. For stems in compression, Mn = Mp = Fy Zx ≤ My (AISC Equation F9-3) Since Sx < Zx , the yield moment My will control. 7.4 LRFD SOLUTION Shear Strength 393 The factored load is Pu = 1.2D + 1.6L = 1.2(12) + 1.6(33) = 67.2 kips Compute the capacity of one bolt. Figure 8.34 shows the moment-rotation curve for a beam connection and includes the moment-rotation relationship for the beam as well as the connection. As shown in Figure 8.55b, a larger area will be available when the load is transmitted through the additional thickness of the end plate. 10(6)(36) Use a PL 11⁄2 × 6 × 10. In addition, the girder must support column loads at B, C, and D consisting of 115 kips dead load and 170 kips live load at each location. 2″ 8″ 4 @ 3″ 3 6″ 4 105k FIGURE P8.3-5 6″ 2″ Problems 8.3-6 579 The flange of a portion of a WT6 × 20 is used as a bracket and is attached to the flange of a W14 × 61 column as shown in Figure P8.3-6. The general procedure is to assume the location of the instantaneous center, then determine if the corresponding value of P satisfies the equilibrium equations. Reduce the computed moment of inertia by 25%. This is not necessary, however, because these ratios are tabulated in the dimensions and properties table. The flange-to-web welds must resist the horizontal shear at the interface between the two components. 4.176 ⎛ 3⎞ Minimum length = 4 w = 4 ⎜ ⎟ = 0.75 in. For simplicity and without loss of generality, we consider only this case. Even with this assumption, however, the computations for even the most trivial problems are overwhelming, and computer assistance is needed. 1.5 STRUCTURAL STEEL The earliest use of iron, the chief component of steel, was for small tools, in approximately 4000 B.C. (Murphy, 1957). 3.8 Tension Members in Roof Trusses 81 FIGURE 3.30 The extra amount of material in question is insignificant, and the use of the same size for each segment eliminates the possibility of a mix-up during construction. and Gere, James M. (The calculations are not shown here, but for a = 36 in., the computed value of fvVn is 231 kips, which is greater than the required value of 223.4 kips.) Determine the intermediate stiffener spacings needed for shear strength outside the end panels. Although A307 bolts are adequate for many applications, they are rarely used today. Try two plates 5⁄16 × 3 × 0′- 41⁄2″. Deflections 9.6-1 Compute the following deflections for the beam in Problem 9.2-1. and for the longitudinal welds, fRn = 0.85 × 6.960 = 5.916 kips糞in. (Because the applied moment is caused by gravity loads and is not reversible, the stiffeners opposite the beam compression flange can be fitted to bear on the column flange and need not be welded, but this option is not exercised here.) Check the column web for shear. The weld strength is fRn = 1.392D × 11.70 × 1.5 where D is the weld size in sixteenths of an inch, and the factor of 1.5 accounts for the direction of the load on the weld. Details are given in Part 8 of the Manual, “Design Considerations for Welds,” so they are not fully covered here. Specify the number of intermediate connectors. ANSWER Use 108 studs, 1⁄2 in. As an approximation, each flange can be considered to resist each of these forces independently. For line abde, wn = 16 – 2(1.125) = 13.75 in. Asa = (OK) πd π (0.5) = = 0.1963 in.2 4 4 2 2 For normal weight concrete, Ec = w1.5 fc′ = (145)1.5 4 = 3492 ksi From AISC Equation I3-3, Qn = 0.5 Asa fc′Ec ≤ Rg R p Asa Fu = 0.5(0.1963) 4(3492) = 11.660 kips Rg Rp Asa Fu = 1.0(0.75)(0.1963)(65) = 9.570 kips < 11.60 kips ∴ Use Qn = 9.570 kips. Tide, R. Support conditions in the direction perpendicular to the plane of the frame are such that Ky = 1.0. The beams are W16 × 57, and the columns are W10 × 100. For permission to use material from this text or product, submit all requests online at www.cengage.com/permissions. For a shear lag factor U of 0.80, An = Ag − Aholes = 5.80 − 2 (43 + 81)(12) = 4.925 in. Consequently, it is not reasonable to record this number to more than four significant figures. Satisfactory: 0.720 < 1.00 b. Required d = 0.705 in. Lateral support is provided at the ends and at the load points. AISC Section H1 incorporates a comparable ratio for axial loads and gives two interaction formulas, one for small axial loads and one for large axial loads (we discuss the reason for this in Chapter 6). ANSWER bx 9.45 × 10 −3 = = 8.29 × 10 −3 > 6.25 × 10 −3 Cb 1.14 Use bx = 8.29 × 10–3. Try a width that brings the edge of the stiffener near the edge of the flange and a thickness that satisfies AISC Equation G3-3: ⎛ b ⎞ ≤ 0.56 E ⎝ t ⎠ st Fyst b. FIGURE 4.6 116 Chapter 4 Compression Members Effective Length Both the Euler and tangent modulus equations are based on the following assumptions: 1. This redistribution will change the relationship between all forces, and the bolt tension will increase from B0 to B. These are the areas above and below the plastic neutral axis, which is not necessarily the same as the elastic neutral axis. The direct shearing stress is the same for all segments of the weld and is equal to f1 y = 40 40 = = 1.429 ksi 8 + 12 + 8 28 To locate the centroid of the weld shear area, use the principle of moments with summation of moments about the y axis. Sag rods can be located at the midpoint, the third points, or at more frequent intervals along the purlins, depending on the amount of support needed. Table 8.2 in the high-strength bolt specification (RCSC, 2009) gives the required nut rotation for various bolt sizes in terms of the ratio of length to diameter. 18 b. The tabulated unsymmetrical shapes—the structural tees and the single and double angles—require special consideration and are covered in Section 4.8. EXAMPLE 4.6 A compression member is subjected to service loads of 165 kips dead load and 535 kips live load. Grade 60 reinforcing bars are used. The first category, laterally supported compact beams, is quite common and is the simplest case. This result implies a probability of 1.0 that an event will fall within the boundaries of the diagram. New York: McGraw-Hill. The reduced slip-critical strength is ksc Rn Ω where ksc = 1 − 1.5Ta 1.5(100) 4.741 =1− =1− Du Tb nb 1.13(28)nb nb where Tb = 28 kips (from AISC Table J3.1). Use four 7⁄8-inch-diameter Group A bolts. PL = 125k wD = 0.5 k/ft 25′ 25′ 50′ FIGURE P10.7-3 10.7-4 A plate girder must be designed for the conditions shown in Figure P10.7-4. 1979. This factor is a function of the ductility of the material, fastener spacing, stress concentrations at holes, fabrication procedure, and a phenomenon known as shear lag. Thus, the mechanism requiring the largest moment capacity is the correct one. The effect on a composite beam is to increase the curvature and hence the vertical deflection. If the connected parts are considered as separate free bodies, the forces consist of the bolt tension T0 and the normal clamping force N0, shown here as uniformly distributed. If such an assumption is made, the shearing stress in each fastener can be found from the torsion formula fv = Md J (8.1) where d = distance from the centroid of the area to the point where the stress is being computed J = polar moment of inertia of the area about the centroid and the stress fv is perpendicular to d. The allowable stress based on yielding is Ft = 0.6Fy = 0.6(36) = 21.6 ksi and the corresponding allowable strength is Ft Ag = 21.6(14.7) = 318 kips. ANSWER ASD SOLUTION The live-load deflection is satisfactory. This will take you to the product page where these resources can be found. Use A36 steel. Check for compactness. HP-shapes are designated in the same manner as the W-shape; for example, HP14 × 117. The computations for the neutral axis location and the moment of inertia are summarized in Table 9.7. The deflection caused by the live load is Δ3 = FIGURE 9.21 5wL L4 5(1.200糞12)(30 × 12)4 = = 0.9931 384 EI LB 384(29, 000)(759.4) A c = 5.17 in. Another category of steel products for structural applications is cold-formed steel. Compute the deflections that occur before and after the concrete has cured. In general, a trial-and-error approach must be used, and a compressive stress–strain curve such as the one shown in Figure 4.5 must be used to determine Et for trial values of Pcr. Adequate: For shear, 10 kips < 16.2 kips; For tension, 16.7 kips < 18.5 kips 8.3-5 a. Compute the available block shear strength of the gusset plate for both LRFD and ASD. 0.994 kips糞ft b. Problems 3″ 583 2″ 18k 4″ 6″ 1 ⁄2″ E70XX 2″ Fillet welds FIGURE P8.4-6 8.4-7 Use an elastic analysis and compute the extra load in the weld (in kips per inch of length) caused by the eccentricity. ( ) Muy ⎞ 8 104.8 Pu 8 ⎛ Mux + + = 0.6858 + + 0 = 0.947 < 1.0 (OK) fc Pn 9 ⎜⎝ fb M nx fb M ny ⎟⎠ 9 356 Load combination 4: Pnt = 212 kips, Mnt = 47.6 kips, Pt = 56 kips, and Mt = 132 ftkips. G., Slutter, R. Both forces are transmitted to the column web, with compression being more critical because of the stability problem. Use LRFD and design a welded connection for the bracket shown in Figure P8.4-19. The service gravity loads are shown in Figure 6.25a, and the service wind load (including an uplift, or suction, on the roof) is shown in Figure 6.25b. The length is 17.5 ft. 3.6 Design of Tension Members 69 EXAMPLE 3.11 A tension member with a length of 5 feet 9 inches must resist a service dead load of 18 kips and a service live load of 52 kips. The angle connecting the top flange provides torsional stability to the beam at its end and does not assist in supporting the reaction. 2 Ae = AnU = 4.925(0.80) = 3.94 in.2 ft Pn = ft Fu Ae = 0.75(58)(3.9 4) = 171 > 154 kips ANSWER ASD SOLUTION (OK) Use an L8 × 4 × 1⁄2, connected through the 8-inch leg. In addition, users of the structure may view large deflections negatively and wrongly assume that the structure is unsafe. This way, the slab weight can be combined with other loads similarly expressed. Figure 7.36 shows the plug or slot weld, which sometimes is used when more weld is needed than length of edge is available. Select a W18 of A992 steel. Because slabs on formed metal deck are usually lightly reinforced (sometimes welded wire mesh, rather than reinforcing bars, is used), adding 5 pcf for reinforcement may seem excessive, but the deck itself can weigh between 2 and 3 psf. The uniform load is a service load consisting of 50% dead load and 50% live load. 7. It may be attached to the beam web instead of the top flange. “Eccentrically Loaded Bolted Connections.” Journal of the Structural Division, ASCE 97 (no. 7.11 Fillet Welds 453 Minimum Size The minimum size permitted is a function of the thickness of the thinner connected part and is given in AISC Table J2.4. This requirement is taken directly from the American Welding Society Structural Welding Code (AWS, 2010). Welds are weak in shear and are usually assumed to fail in shear, regardless of the direction of loading. The ASD solution procedure is essentially the same as for LRFD, and the same trial values of Fcr will be used here. at the end of the gusset plate = 8 in. For example, instead of referring to an ASTM A325 bolt, we will call it a Group A bolt. Effective Flange Width 9.3-1 A fully composite floor system consists of W12 × 16 floor beams supporting a 4-inch-thick reinforced concrete floor slab. The AISC requirements for stability bracing are given in Appendix 6 of the Specification, “Stability Bracing for Columns and Beams.” Although we are considering frame stability here, it is the columns (beam–columns) that are being braced. In this book, we treat all weld segments as line segments, which we assume to be the same length as the edge of the connected part that they are adjacent to. Exceptions are listed in AISC J2. The upper limit is λr = 0.56 E Fy The webs of I shapes are stiffened elements, and the stiffened width is the distance between the roots of the flanges. 3⁄8″ a b PL D = 14k L = 25k 2L4 × 4 × 1⁄4 Section FIGURE P7.4-3 7.4-4 The splice plates shown in Figure P7.4-4 are 1⁄4-inch-thick. There is no minimum spacing given in the Specification, but the AISC publication, “Detailing for Steel Construction,” (AISC, 2009) states that intermittent welds are more economical than continuous welds only if the center-to-center spacing is more than twice the length of the weld. 280k 47ft-k 47 3.5′ 49 A 3.5′ 14′ W12 × 65 51 B 3.5′ 53 C 3.5′ 55 55ft-k Moment(ft-k) 318 Chapter 6 Beam–Columns In the plane of bending, Pe1 = p 2 EI (K1L)2 = p 2 EI x (K x L)2 = p 2 (29, 000)(533) (11.0 × 14 × 12)2 ⎛M ⎞ Cm = 0.6 − 0.4 ⎜ 1 ⎟ = 0.6 − 0.4 ⎝ M2 ⎠ B1 = = 5405 kips ⎛ 47 ⎞ ⎜⎝ − 55 ⎟⎠ = 0.9418 Cm 0.9418 Cm = = = 1.027 1 − (a Pr 糞Pe1) 1 − (1.60 Pa 糞Pe1) 1 − (1.60 × 280糞5405) Next, from the Beam Design Charts with Cb = 1.0 and Lb = 14 feet, the moment strength is Mn = 230 ft-kips Ωb For the actual value of Cb , refer to the moment diagram of Figure 6.12: 12.5 M max 2.5 M max + 3 M A + 4 M B + 3 MC 12.5(55) = 1.062 = 2.5(55) + 3(49) + 4(51) + 3(53) Cb = ∴ But Mn = Cb (230) = 1.062(230) = 244.3 ft-kips Ωb Mp Mn = 237 ft-kips (from the charts) < 244.3 ft-kips, so use = 237 ft-kips. Some web members will be in tension and others will be in compression. From Figure 9.9, C 515 = = 1.683 in. No additional stiffeners will be needed, since the maximum factored load shear outside of the end panels is less than the design strength of 237 kips. The other bolts listed in Groups A and B have the same strengths, but have special distinguishing characteristics. For Lb = 12 ft, the allowable strength for Cb = 1 is 223 ft-kips. Sidesway web buckling would not be an applicable limit state because the girder has continuous lateral support (making the unbraced length Lb = 0 and (h糞tw)糞(Lb糞bf) > 2.3). Bending is about the strong axis, and Kx = Ky = 1.0. Use A992 steel and select a W10 shape. Let Rn from Equations 2.6 and 2.7 be the same when L = 3D. one half of the beam center-to-center spacing, or 3. Built-up shapes consisting of plates or a combination of plates and shapes are addressed in AISC Section J3.5 of Chapter J (“Design of Connections”). Stiffener-to-column flange welds: Since the beam tension flange force is an issue (for both column flange bending and web yielding), the stiffener must be welded to 8.7 Column Stiffeners and Other Reinforcement 549 the column flange. For an x-y coordinate system with the origin at the center of the lower left bolt (Figure 8.9), 2(3) + 2(6) + 1(9) = 3.857 in. If the member is connected with longitudinal welds on each side with no transverse weld (as in Figure 3.9), the following values apply: For ≥ 2w U = 1.0 For 1.5w ≤ < 2w, U = 0.87 For w ≤ < 1.5w, U = 0.75 3. (upper limit is 36 in.) The number of studs required between the end of the beam and midspan is N1 = V′ 515 = = 53.8 Qn 9.570 Use a minimum of 54 for half the beam, or 108 total. The design strength for LRFD. Equating the weld strength to the required shear strength, we get 1.392D × 8.425 × 2 = 33.6, D = 1.43 sixteenths (w = 1糞8 in.) From AISC Table J2.4, the minimum weld size is 3⁄16 in. Adequate: Before concrete cures, Ma = 98.8 ft-kips < 166 ftkips After concrete cures, Ma = 270 ft-kips < 323 ft-kips 9.3-1 a. The corresponding composite cross section is shown in Figure 9.25b. The strength of a partial penetration groove weld will depend on the amount of penetration; once that has been determined, the design procedure will be essentially the same as that for a fillet weld. Compute the required allowable strength. Table 6-1 in Part 6 of the Manual is for the design of W shapes only, and there are no corresponding tables for structural tees. SOLUTION The appropriate strategy here is to find the lightest shape for each nominal depth in the column load tables and then choose the lightest overall. However, that will also increase the length of the fastener and could subject it to bending. Clip at a 45⬚ angle for 5⁄8 inch. ECCENTRIC WELDED CONNECTIONS: SHEAR PLUS TENSION Many eccentric connections, particularly beam-to-column connections, place the welds in tension as well as shear. Butler, L. Then 1 1 w L2 = (0.481)(30)2 = 54.11 ft-kips 8 D 8 1 M L = (0.550)(30)2 = 61.88 ft-kips 8 MD = LRFD SOLUTION The dead load is less than 8 times the live load, so load combination 2 controls: Mu = 1.2MD + 1.6ML = 1.2(54.11) + 1.6(61.88) = 164 ft-kips. < 8.62 in. In Figure 5.3b, yielding has just begun. 5.568 23.28 − 8 = 7.64 in. If a significant portion of the live load is considered to be sustained, then creep should be accounted for. The framed beam connection, which is very common, subjects the vertical angleto-column welds to the same type of load as the seated beam connection. Assume that B1 = 1.0. PD = 54k PL = 147k FIGURE 6.22 MD = 18 ft-k ML = 49 ft-k 16′ MD = 18 ft-k ML = 49 ft-k LRFD SOLUTION The factored axial load is Pnt = Pu = 1.2PD + 1.6PL = 1.2(54) + 1.6(147) = 300 kips (There is no amplification of axial loads in members braced against sidesway.) The factored moment at each end is Mntx = 1.2MD + 1.6ML = 1.2(18) + 1.6(49) = 100 ft-kips Since B1 = 1.0, the factored load bending moment is Mux = B1Mntx = 1.0(100) = 100 ft-kips The effective length for compression and the unbraced length for bending are the same: KL = Lb = 16 ft 342 Chapter 6 Beam–Columns The bending moment is uniform over the unbraced length, so Cb = 1.0. Try a W10 shape. Square and Rectangular HSS and Box-Shaped Members (AISC F7): a. This unified behavior is possible only if horizontal slippage between the two components is prevented. Use LRFD and the alignment chart to check the adequacy of the following columns for Fy = 50 ksi. Assume that Ae = Ag and compute the following. The difference in effort between the two methods involves the load side of the relationships. A separate analysis must be made for buckling about the other axis. Compute the nominal strength in bearing. The capacity of the connection is reached when the ultimate strength of the fastener farthest from the IC is reached. Assume a shear lag factor of U = 0.80. 100 Chapter 3 Tension Members 7″ HSS 6 × 6 × 1⁄2 t = 5⁄8″ FIGURE P3.5-2 3.5-3 A WT8 × 13 of A992 steel is used as a tension member. Stiffeners are placed at each end and at 4 feet, 16 feet, and 28 feet from each end. 4.9-2 Verify the values of y2, rx , and ry given in Part 1 of the Manual for the combination shape consisting of an S12 × 31.8 with a C8 × 11.5 cap channel. The composite curve, called a column strength curve, completely describes the strength of any column of a given material. It is connected with 11⁄8-inch-diameter, Group A slip-critical bolts to a 3⁄8-inch-thick gusset plate. When structural tee-shapes are used as chord members in welded trusses, the web angles can usually be welded to the stem of the tee. Problems 9.6-5 659 For the beam of Problem 9.4-2, a. (Neither Design Guide 4 nor Design Guide 16 lists this assumption.) • Beam web-to-end plate welds near the tension bolts should be designed to develop the yield stress of the beam web unless the required moment is less than 60% of the beam strength. Chegg.com Application of Structural Steel Design Solutions Manual. For Equation 4.1 to be valid, the member must be elastic, and its ends must be free to rotate but not translate laterally. Carter, Charles J., 1999. LRFD SOLUTION Pu = 1.2D + 1.6L = 1.2(100) + 1.6(300) = 600 kips Try Fcr = 33 ksi (an arbitrary choice of two-thirds Fy): Required Ag = Pu 600 = = 20.2 in.2 φc Fcr 0.90(33) Try a W18 × 71: Ag = 20.9 in.2 > 20.2 in.2 (OK) KL 26 × 12 = = 183.5 < 200 rmin 1.70 Fe = 4.71 Since (OK) p 2E p 2 (29, 000) = 8.5 ksi 2 = (KL糞r) (183.5)2 E 29, 000 = 4.71 = 113 Fy 50 KL E > 4.71 , AISC Equation E3-3 applies. For ASD, the allowable strength is Rn 47.72 = = 23.9 kips. B1x = 1.60 Pa 1.60(100) 1− 1− 1922 Pe1x Pe1x = Pe1 y = B1 y = π 2 (0.8 EI y) π 2 (0.8)(29, 000)(93.4) = = 660 kipps (15 × 12)2 (K y L)2 Cmy 0.6 = = 0.792 < 1.0 1.60 Pa 1.60(100) 1− 1− Pe1 y 660 ∴ B1 y = 1.0 as assumed. If two outer blocks are considered (Figure 8.41b), the total tension width is 2(1.5) = 3.0 in. The only load in addition to the beam weight is a uniform live load. The tensile stress is nonuniform when some rotation of the block is required for failure to occur. A unified building code, the International Building Code (International Code Council, 2009), has been developed to eliminate some of the inconsistencies among the three national building codes. Product Information: Title: Solution Manual for Structural Steel Design A Practice Oriented Approach by Aghayere, ISBN-10: 0132340186, ISBN-13: 978. Max. Figure 8.41 shows the bolt layout and two possible block shear failure modes. The maximum value of the prying force, q, will be reached when only the corners of the flange remain in contact with the other connected part. The line of action of the load passes through the center of gravity of the connection. LRFD Equations Weld shear strength: fRn = 0.75(0.707wLFnw) (7.27) Base-metal shear strength: (7.28) fRn = min[1.0(0.6Fy tL), 0.75(0.6Fu tL)] It is frequently more convenient to work with the strength per unit length; in which case, L = 1, and Equation 7.27 and 7.28 become 7.11 Fillet Welds 447 Weld shear strength: fRn = 0.75(0.707wFnw) for a one-inch length (7.29) Base-metal shear strength: fRn = min[1.0(0.6Fy t), 0.75(0.6Fut)] for a one-inch length (7.30) ASD Equations Weld shear strength: Rn 0.707wLFnw = Ω 2.00 Base-metal shear strength: 0.6 Fy tL 0.6 Fu tL ⎤ Rn = min ⎡ , ⎢ ⎣ 1.50 Ω 2.00 ⎥⎦ (7.31) (7.32) If the strength per unit length is used, L = 1 and Equations 7.31 and 7.32 become Weld shear strength: Rn 0.707wFnw = for a one-inch length Ω 2.00 Base-metal shear strength: (7.33) 0.6 Fy t 0.6 Fu t ⎤ Rn for a one-inch length = min ⎡ , ⎢ ⎣ 1.50 Ω 2.00 ⎥⎦ (7.34) EXAMPLE 7.12 A plate used as a tension member is connected to a gusset plate, as shown in Figure 7.40. back-to-back, Fy = 36 ksi; KL is 20 feet for all axes, and there are two intermediate connectors. In addition, cold-working will increase the yield point of the steel, and under certain conditions it may be accounted for in design (AISI, 2007). 8.2-11 8.3-1 a. Part 16. We use Table 4.2 to keep track of the computations. Problems 7.11-10 475 Design a tension member and its connection for the following conditions: • The tension member will be an American Standard Channel. Part 2 also lists the load combinations we discussed in Sections 2.3 and 2.4. Part 3. Thus they are in a category that lies somewhere between cases (a) and (d) in Table C-A-7.1 of the Commentary, and K is between 0.5 and 1.0. A value of 1.0 is therefore always conservative for members of braced frames and is the value prescribed by AISC Appendix 7.2.3(a) unless an analysis is made. Note that the subscript 1 corresponds to the braced condition and the subscript 2 corresponds to the unbraced condition. If the connection has a plane of symmetry, the centroid of the shear area of the fasteners or welds may be used as the reference point, and the perpendicular distance from the line of action of the load to the centroid is called the eccentricity. Two categories are covered: relative and nodal. 1.5-2 The strain in member AB was measured to be 8.9 × 10–4. The required strength is Ru = 1.2D + 1.6L = 1.2(15) + 1.6(45) = 90.0 kips < 91.7 kips ASD SOLUTION The allowable strength is (OK) Rn 122.3 = = 61.2 kips. The column is supported by a 12-in. CHAPTER 1 Introduction 1.1 STRUCTURAL DESIGN The structural design of buildings, whether of structural steel or reinforced concrete, requires the determination of the overall proportions and dimensions of the supporting framework and the selection of the cross sections of individual members. Only that part of the concrete used in resisting the bending moment is considered to be effective. This secondary moment has a maximum value of PΔ, which represents an amplification of the end moment. In addition, most calculators, spreadsheet programs, and other software use the first method, and our results will be consistent with those tools; therefore, we will round up when the first digit dropped is a 5 with no digits following. Assume a beam setback of 1⁄2 inch. Determine the locations of the intermediate stiffeners, but do not proportion them. All W, M, and S shapes tabulated in the Manual have compact webs (for flexure) for Fy ≤ 50 ksi, so the first condition will govern for all composite beams except those with built-up steel shapes. Kx = Ky = 1.0. FIGURE 6.13 LRFD SOLUTION The factored axial load is Pu = 1.6(28) = 44.8 kips The factored transverse loads and bending moment are Qu = 1.6(28) = 44.8 kips wu = 1.2(0.035) = 0.042 kips糞ft Mu = 44.8(10) 0.042(10)2 + = 112.5 ft-kips 4 8 This member is braced against sidesway, so Mt = 0. “Semi-Rigid Frame Design Methods for Practicing Engineers.” Engineering Journal, AISC 36 (no. Conservatively assume that the full area of the support is used and determine B from a consideration of bearing strength. Chicago. Before any analysis, however, a decision must be made on the primary building material to be used; it will usually be reinforced concrete, structural steel, or both. This allowable value is obtained by dividing the nominal, or theoretical, strength by a factor of safety. In Figure 9.16, the ribs are shown perpendicular to the beam. All bending moments are about the strong axis. Ae = AnU = 4.536(0.60) = 2.722 in.2 Pn = Fu Ae = 58(2.722) = 157.9 kips Pn 157.9 = = 79.0 kips Ωt 2.00 (controls) The member capacity is therefore adequate with two gage lines of bolts. • There is a new emphasis on the use of the lower-bound moment of inertia in computing deflections. = 29.28 ft Since Lp < Lb < Lr , ⎛ Lb − L p ⎞ ⎤ ⎡ M n = Cb ⎢ M p − (M p − 0.7 Fy S x) ⎜ ⎥ ≤ Mp ⎝ Lr − L p ⎟⎠ ⎦ ⎣ ⎛ 20 − 8.692 ⎞ ⎤ ⎡ = 1.0 ⎢5, 750 − (5, 750 − 0.7 × 50 × 103) ⎜ ⎝ 29.28 − 8.692 ⎟⎠ ⎥⎦ ⎣ = 4572 in.-kips = 381.0 ft--kips < M p = 479.2 ft.-kips LRFD SOLUTION The design strength is ASD SOLUTION The allowable moment strength is fb Mn = 0.90(381.0) = 343 ft-kips Mn = 0.6 M n = 0.6(381.0) = 229 ft-kips Ωb 2 208 Chapter 5 Beams c. Tension Flange Yielding From Chapter 5, the maximum bending stress in a flexural member bent about its strong axis is fb = M Sx where Sx is the elastic section modulus about the strong axis. What is the available axial compressive strength? If a substantial amount of axial load is also present, the member is referred to as a beam–column (beam–columns are considered in Chapter 6). Other types of live load are often treated as separate categories, such as hydrostatic pressure and soil pressure, but the cases we have enumerated are the ones ordinarily encountered in the design of structural steel building frames and their members. Satisfactory: Before concrete has cured, Mu = 44.6 ft-kips < 75.4 ft-kips; after concrete has cured, Mu = 135 ft-kips < 168 ft-kips; Vu = 21.5 kips < 79.1 kips b. The minimum plate width is g + 2Le = 3.50 + 2(1.125) = 5.75 in. “Design of Lightly Loaded Steel Column Base Plates.” Engineering Journal, AISC 20 (no. 16 not d + 1⁄8 inch (although if d + 1⁄8 were used, the slight error would be on the conservative side). For rigid frames, software for second-order analysis is available. Although a is defined as the clear spacing, we will treat it conservatively as a center-to-center spacing and place the first intermediate stiffener at 54 inches from the end of the girder. (controls) This thickness is also greater than that required for bearing, so it will be the minimum acceptable thickness. FIGURE 1.1 1.1 Structural Design 5 FIGURE 1.2 For the loading shown in Figure 1.2b, the frame will deform as indicated by the dashed line (drawn to a greatly exaggerated scale). The connection is at a distance from the end of the column that is more than the depth of the column. Use LRFD and determine whether this member is adequate. Geschwindner, L.F. 2010. Safe designs can be achieved with beams corresponding to any of these curves, but curves 1 and 2 represent inefficient use of material. The following data for a W21 × 68 are obtained from the Zx table. Chapter 1 D B G F Facc A E f f 1 1 cr C 2 Solutions Manual • Instructor's Solution Manual to Accompany Mechanical Engineering Design 1-6 (Chapter 1-6 N 4 Solutions Manual • Instructor's Solution Manual to Accompany Mechanical are: UNS G10350 HR steel: 42 Solutions Manual • Instructor's Solution Manual. 7.9 SOLUTION Combined Shear and Tension in Fasteners (The following values are used in both the LRFD and ASD solutions.) Compute the nominal bearing strength (flange of tee controls). Since this concept is not covered until Chapter 5 of this book, we will use x− for channels as shown in Case 2 of Specification Table D3.1 and in Figure 3.7b of this book. The stiffeners resist the vertical component of the diagonal compression, and the flanges resist the horizontal component. Beginning with the smallest shapes, the first one found that will work is a W8 × 58 with a design strength of 634 kips. A 15′ B 18′ 20′ FIGURE P4.7-10 4.7-11 The frame shown in Figure P4.7-11 is unbraced against sidesway. No numerical examples are given in this section, but Example 6.11 includes the computation of the flexural strength of a structural tee-shape. (OK) 16 16 16 This connection is one-sided, so full-depth stiffeners are not required. Consider the beam reactions to act as point loads on the girder. Loads act through the shear center, producing moments about the x and y axes. However, many isolated one-story columns can be realistically treated as axially loaded compression members. 30,000 ksi 1.5-5 b. For a wind direction that would produce uplift, 0.6D – 0.6W = 0.6(85) – 0.6(56) = 17.4 kips The positive result means that the net load is compressive, and we need not consider this load combination further. These hinges will form sequentially, although it is not always necessary to know the sequence. This will constitute most of the moment capacity of the beam, but a small amount of restraint will also be provided by the plate connection. live load defl. The length of the connection cannot exceed 8 inches, and all welding must be done on the near side. D + 0.75L ± 0.75(0.7E) + 0.75S. Its purpose is to show that the number of studs is compatible with the spacing requirements. (The gage length is the distance between two marks on the specimen. A comparison of the tensile properties of A36, A572 Grade 50, and A992 is given in Table 1.1. 1.6 STANDARD CROSS-SECTIONAL SHAPES In the design process outlined earlier, one of the objectives—and the primary emphasis of this book—is the selection of the appropriate cross sections for the individual members of the structure being designed. 2): 54–59. 2 2 Try a plate 5⁄16 × 3: 5 ⎞ Ast = 3 ⎛⎜ ⎟ × 2 stiffeners = 1.88 in.2 > 1.44 in.2 ⎝ 16 ⎟⎠ (OK) Check for tst ≥ b糞16: b 3 5 = = 0.188 in. Use LRFD. The beam is a W27 × 94 with Fy = 50 ksi, and the concrete strength is fc′ = 3 ksi. If A992 steel is used, is a W16 × 31 adequate? Doing so is relatively simple. r Fy Fcr = 0.658 (Fy糞Fe) Fy = 0.658(36糞19.39) (36) = 16.55 ksi The nominal strength is Pn = Fcr Ag = 16.55(7.50) = 124.1 kips 168 Chapter 4 Compression Members To determine the flexural-torsional buckling strength for the y-axis, use the modified slenderness ratio, which is based on the spacing of the connectors. 94.9 kips 8.4-1 3.99 kips糞in. If load and resistance factor design is used, determine the factored load (required strength) to be used in the design of the column. The procedure is illustrated in Example 4.17. 9.1-6 Compute the nominal flexural strength of the composite beam in Problem 9.1-3. With ASD, the nominal strengths are divided by load factors. The eccentricity of the load with respect to the welds at the column flange will therefore be the distance from this center of gravity to the column flange. Use Cm = 0.6 and Cb = 1.67. pPr + bx Mrx + by Mry = pPu + bx Mux + by Muy = (1.89 × 10 −3)(300) + (3.51 × 10 −3)(100) + 0 = 0.918 < 1.0 (OK) To be sure that we have found the lightest W10, try the next lighter one, a W10 × 54, with p = 2.12 × 10−3 and bx = 3.97 × 10−3 From Equation 6.9, pPu + bx Mux + byMuy = (2.12 × 10−3)(300) + (3.97 × 10−3)(100) + 0 = 1.03 > 1.0 (N.G.) Try a W12 shape. FIGURE 7.48 E70 1⁄4 7″ (OK) 460 Chapter 7 ASD SOLUTION Simple Connections The allowable strength of the member based on its gross area is Fy Ag 36(1糞2)(8) Pn = = 86.23 kips Ω t 1.67 1.67 Compute the allowable strength based on the effective area. Such a function is illustrated in Figure 2.2. Probability functions are designed so that the total area under the curve is unity. This was a practical alternative when the primary means of fireproofing structural steel was to encase it in concrete; the rationale was that if the concrete was there, we might as well account for its contribution to the strength of the beam. FIGURE 5.29 236 Chapter 5 Beams SOLUTION Neglect the beam weight and check it later. A total of eight bolts will be required for the connection. Lanham, MD. All values available for angles are for A36 steel. Required size = 3.18 sixteenths of an inch b. If the member is stockier, as shown in Figure 4.1b, a larger load will be required to bring the member to the point of instability. A connection almost always weakens the member, and the measure of its influence is called the joint efficiency. 1987. Myb (160 × 12)(16.70) fsb = = = 21.0 ksi (tension) I tr 1530 The stress at the top of the concrete is fc = My (160 × 12)(4.205) = = 0.660 ksi nI tr 8(1530) If the concrete is assumed to have no tensile strength, the concrete below the neutral axis should be discounted. 126 kips, b. • Superimposed service dead load = 0.5 kips糞ft (does not include the girder weight). 8.7 Column Stiffeners and Other Reinforcement 543 EXAMPLE 8.11 Determine whether stiffeners or other column web reinforcement is required for the connection of Example 8.10. The required size in sixteenths is therefore 10.13 = 7.3 sixteenths 1.392 ANSWER ASD SOLUTION use 8 1 in. Determine the effective length factor Kx for column BC. The service dead load is 48 kips and the service live load is 72 kips. The tensile design strength. If column stiffeners are needed, use A36 steel and specify the required dimensions. The structural steel is A992, and the concrete strength is fc′ = 4 ksi. The gusset plate bearing strength is therefore frn = 44.0t = 44.0(3糞8) = 16.5 kips糞bolt Even with a conservative estimate of the bearing strength, the slip-critical strength controls. 5.4-2 Repeat Problem 5.4-1 for a W6 × 15. Further permissions questions can be emailed to Copyeditor: Shelly Gerger-Knechtl Proofreader: Pamela Ehn Library of Congress Control Number: 2011934128 Indexer: Shelly Gerger-Knechtl ISBN-13: 978-1-111-57600-4 ISBN-10: 1-111-57600-9 Compositor: MPS Limited, a Macmillan Company Senior Art Director: Michelle Kunkler Internal Designer: Carmela Periera Cover Designer: Andrew Adams Cover Image: © Remi Cauzid/Shutterstock; sack/iStockphoto; vm/iStockphoto; Lefthome/iStockphoto Rights Acquisitions Specialist: Deanna Ettinger Text and Image Permissions Researcher: Kristiina Paul Cengage Learning 200 First Stamford Place, Suite 400 Stamford, CT 06902 USA Cengage Learning is a leading provider of customized learning solutions with office locations around the globe, including Singapore, the United Kingdom, Australia, Mexico, Brazil, and Japan. The effective lengths with respect to the x, y, and z axes are each 13 feet. Depending on its design, the formwork for the concrete slab may or may not provide lateral support for the steel beam. For example, the initial tension in a 5⁄8-inch-diameter, Group A bolt can be as high as 19 kips. These tables show the number of intermediate connectors required for the given y-axis flexural-torsional buckling strength. total ANSWER Use the connection detail as shown in Figure 7.18. 8.9 CONCLUDING REMARKS In this chapter we emphasized the design and analysis of bolts and welds rather than connection fittings, such as framing angles and beam seats. • The nominal axial strength is based on the provisions of AISC J4.4, “Strength of Elements in Compression,” which are as follows: For KL ≤ 25, r Pn = Fy Ag (AISC Equation J4-6) This is the “squash load” for the stiffener—that is, the load that causes compression yielding, with no buckling. If the applied moment never changes direction, the stiffener 8.7 Column Stiffeners and Other Reinforcement 537 FIGURE 8.43 resisting the compressive load (the bottom stiffener in this illustration) can be fitted to bear on the flange and need not be welded to it. Bjorhovde, R., Galambos, T. The top surface of the pier is 18 inches by 18 inches. W10: W10 × 54, Pn = 188 kips Ωc 134 Chapter 4 Compression Members W12: W12 × 58, Pn = 194 kips Ωc W14: W14 × 61, Pn = 195 kips Ωc Note that the strength is not proportional to the weight (which is a function of the cross-sectional area). Required spacing of welds = ANSWER ASD SOLUTION 25.06 strength of weld group (kips) = = 3.79 in. Rounding up, try B = 10 in. Figure 6.5 shows a simply supported member with an axial load and an initial out-of-straightness. The load of 2.78951 kips just computed is the failure load for the connection and is based on the critical fastener reaching its ultimate load capacity. The value of Qn is found to be 17.2 kips.) Number of studs: V′ 135 = = 7.84 ∴ Use 8. Effective length method 3. Minimum transverse spacing is 4d = 4(0.5) = 2 in. at the end of the channel + 2 spaces at 23⁄4 in. This will result in the smallest block shear strength. In LRFD, the loads are factored before adding. 10k 6k A 6k B C D hinge 14′ 14′ 28′ 14′ 14′ 28′ FIGURE P5.5-5 14′ 14′ 28′ 284 Chapter 5 Beams 5.5-6 A W16 × 26 of A992 steel has an unbraced length of 8 feet. The design of the steel anchors is the same for both LRFD and ASD. Design a welded connection. Use the stiffness reduction factor if applicable. For the interior bearing stiffeners, first compute the bearing strength. In other words, use a hole diameter of h=d+ 1 in. The required strength of the connection is Pa = D + L = 30 + 25 = 55 kips reqquired strength 55 Number of bolts required = = = 3.31 bolts strength per bolt 16.6 404 Chapter 7 Simple Connections Try four bolts, two per line, on each side of the splice. The loads shown are service loads, consisting of equal parts dead load and live load. Additional stiffeners may be required at points of concentrated loads for the purpose of protecting the web from the direct compressive load. Building codes do not give design procedures, but they do specify the design requirements and constraints that must be satisfied. Several examples of commonly used moment connections are illustrated in Figure 8.37. ⎣ ⎦ The total loss in area is 2(b − be)t = 2(7.652 − 6.489)(0.116) = 0.2698 in.2 and the reduced area is Ae = 2.70 − 0.2698 = 2.430 in.2 The reduction factor is Qa = Ae 2.430 = = 0.9000 Ag 2.70 Q = QsQa = 1.0(0.9000) = 0.9000 Compute the local buckling strength. Detailed rules for proportioning eyebars are given in AISC D6 and are not repeated here. From AISC J4.4, for compression elements with KL糞r < 25, the nominal strength is Pn = Fy Ag and for ASD, Ω = 1.67, so Pn 36 Ag = = 21.56 Ag Ω 1.67 Equating this available strength to the required strength, we obtain the required area of stiffener: 21.56Ag = 87.05 Ag = 4.04 in.2 8.7 Column Stiffeners and Other Reinforcement 557 Try two stiffeners, 3⁄4 × 3, one on each side of the web: 3 Ast = 2(3) ⎛⎜ ⎞⎟ = 4.50 in.2 > 4.04 in.2 required ⎝ 4⎠ (OK) Check for tst ≥ b糞16: b 5 3 = = 0.188 in. PNA location 6 corresponds to a ΣQn midway between ΣQn for level 5 and ΣQn for level 7. At the point of impending buckling, the web loses its ability to support the diagonal compression, and this stress is shifted to the transverse stiffeners and the flanges. As in other chapters dealing with local buckling, shapes are classified as compact, noncompact, or slender: If l ≤ lp, the shape is compact. 0.31E Fy 278 Chapter 5 Beams b. 698 Chapter 10 Plate Girders EXAMPLE 10.2 Use LRFD and design a simply supported plate girder to span 60 feet and support the service loads shown in Figure 10.16a. Maximum distance = 6t w = 6 ⎛ ⎞ ⎝ 16 ⎠ If we assume a flange-to-web weld size of 5⁄16 inch and 1.25 inches between welds, the approximate length of the stiffener is h – weld size – 1.25 = 62 – 0.3125 – 1.25 = 60.44 in. Curves 2 and 3 correspond to beams that can be loaded past first yield but not far enough for the formation of a plastic hinge and the resulting plastic collapse. From Equation 7.35, the shear yield strength of the angle leg is ⎛ 1⎞ fRn = 0.6 Fy t = 0.6(36) ⎜ ⎟ = 10.8 kips糞in. For 0.08 E Lb d 1.9 E < 2 ≤ , Fy Fy t ⎡ ⎛ L d ⎞ Fy ⎤ M n = Cb ⎢1.52 − 0.274 ⎜ b2 ⎟ ⎥ M y ≤ M p ⎝ t ⎠ E⎦ ⎣ For (AISC Equation F11-2) Lb d 1.9 E > , Fy t2 Mn = Fcr Sx ≤ Mp (AISC Equation F11-3) where Fcr = 1.9 ECb Lb d糞t 2 (AISC Equation F11-4) t = width of bar (dimension parallel to axis of bending) d = depth of bar b. The total gravity load, including an estimated purlin weight, is 42 psf of roof surface, with a live-load- to-dead-load ratio of 1.0. Assuming that this is the critical loading condition, use A36 steel and select a channel shape for the purlins. Anton van Beek the elastic regime. The eccentric load P can be replaced with the same load acting at the centroid plus the couple, M = Pe, where e is the eccentricity. Compute deflections. rx = 5.17 in., ry = 4.66 in. 5.5-13 The beam shown in Figure P5.5-13 is laterally braced only at the ends. The floor beams, which fill in the panels defined by the columns, are sometimes called filler beams. Although a connection designed as rigid has a theoretical moment FIGURE 8.35 Moment FEM ≈ 0.90 FEM ≈ 0.20 FEM Rotation 522 Chapter 8 Eccentric Connections capacity equal to the fixed-end moment (FEM) of the beam, it will actually have a moment resistance of about 90% of the fixed-end moment. The allowable strength for ASD. An additional requirement for filled composite members is that the steel crosssectional area must be at least 1% of the total area (the same requirement as for encased sections). “Column Web Compression Strength at End-Plate Connections.” Engineering Journal, AISC 21 (no. Although slenderness is critical to the strength of a compression member, it is inconsequential for a tension member. This is discussed in Chapter 6, “Beam–Columns.” EXAMPLE 4.13 A W10 × 54 of A992 steel is used as a column. S = 176 in.3, My = 733 ft-kips 5.2-3 305 ft-kips 5.4-1 Noncompact 5.5-1 a. If we assume that the upper limit in AISC Equation J4-5 does not control, the required design strength is φ Rn = 0.75(0.6 Fu Anv + U bs Fu Ant) = 0.75[0.6(58) Anv + 1.0(58)(0.7500)] = 108 kips Required Anv = 2.888 in.2 3 Anv = [s + s + 1.25 − 2.5(1.0)](2) = 2.888 in.2 8 ∴ Use s = 2 3 4 in. Design Guide 3, 2nd Edition. The values of strength in these tables are not based on flexural-torsional buckling theory, but on the provisions of AISC E5. Maximum Size Along the edge of a part less than 1⁄4 inch thick, the maximum fillet weld size is equal to the thickness of the part. If the load on the floor is uniformly distributed, we can express the uniform load on beam EF as a force per unit length (for example, pounds per linear foot [plf]) by multiplying the floor load in force per unit area (for example, pounds per 5.11 Floor and Roof Framing Systems 241 square foot [psf]) by the tributary width s. Since this analysis was based on a unit bolt ultimate strength, we can obtain the true connection capacity by multiplying by the actual bolt ultimate strength. SOLUTION Compute the net area: An = Ag − ∑ t × (d or d ′) Effective hole diameter = 3 1 7 + = 4 8 8 For line ad, 7 An = 14.7 − 4⎛ ⎞ (0.622) = 12.52 in.2 ⎝ 8⎠ For line abcd, the gage distance for use in the s2糞4g term is g t 3.5 0.550 + g1 − w = + 2.75 − = 4.225 in. Prepare a detailed sketch of your recommended connection. Elastic Analysis The load on the bracket shown in Figure 8.16a may be considered to act in the plane of the weld—that is, the plane of the throat. Appropriate limits for deflection can usually be found from the governing building code, expressed as a fraction of the span length L, such as L糞360. Show your design on a sketch. Select a W-shape. The corresponding bending moments are MA = 156(15) − 40(3) − 4(15)2糞2 = 1770 ft-kips MB = 156(18) − 40(6) − 4(18)2糞2 = 1920 ft-kips MC = 156(21) − 40(9) − 4(21)2糞2 = 2034 ft-kips From AISC Equation F1-1, 12.5 M max 2.5 M max + 3 M A + 4 M B + 3 M C 12.5(2112) = = 1.083 2.5(2112) + 3(1770) + 4(1920) + 3(2034) Cb = FIGURE 10.15 10.6 Bearing Stiffeners 693 This value of Cb is the same as that obtained in the LRFD solution (this is typically the case), so the critical stress for lateral-torsional buckling is the same as for the LRFD solution. As a simplification, l can always be conservatively taken as 1.0 (Thornton, 1990b). Open-web joists are available in standard depths and load capacities from various manufacturers. It hardly makes sense to perform computations with 12 significant figures and record the answer to that degree of precision when the yield stress is known only to the nearest 10 kips per square inch (two significant figures). 5.6 BENDING STRENGTH OF NONCOMPACT SHAPES As previously noted, most standard W, M, S, and C shapes are compact. 2.4 SAFETY FACTORS AND LOAD COMBINATIONS FOR ASD For allowable strength design, the relationship between loads and strength (Equation 2.1) can be expressed as Ra ≤ Rn Ω (2.7) where Ra = required strength Rn = nominal strength (same as for LRFD) Ω = safety factor Rn糞Ω = allowable strength The required strength Ra is the sum of the service loads or load effects. Building up sections is an effective way of strengthening an existing structure that is being rehabilitated or modified for some use other than the one for which it was designed. Compute this as illustrated in Example 9.7. 9.6-2 Compute the following deflections for the beam in Problem 9.2-2. The previously undefined terms in these three equations are defined as Cw = warping constant (in.6) Kz = effective length factor for torsional buckling, which is based on the amount of end restraint against twisting about the longitudinal axis G = shear modulus (ksi) = 11,200 ksi for structural steel J = torsional constant (equal to the polar moment of inertia only for circular cross sections) (in.4) Fex = Fey = p 2E (K x L 糞rx)2 p 2E (K y L糞ry)2 (AISC Equation E4-7) (AISC Equation E4-8) where y is the axis of symmetry for singly symmetrical shapes. Neglect the weight of the beam and determine whether the beam is adequate. Englewood Cliffs, NJ: Prentice-Hall, Inc. Determine whether tension-field action can be used. In each case shown in Figure 9.7, we can find the nominal moment capacity by computing the moment of the couple formed by the compressive and tensile resultants. and that the plastic section modulus is ⎛ A ⎞ a = 11(11.09) = 122 in.3 ⎝ 2⎠ The plastic moment is Mp = Fy Z = 50(122) = 6100 in.-kips = 508 ft-kips ANSWER Z = 122 in.3 and Mp = 508 ft-kips. In this type of construction, one end of the connection of the truss to the walls usually can be considered as pinned and the other as roller-supported. For Cb = 1.32, the design strength is fb Mn = Cb × 197 = 1.32(197) = 260 ft-kips This moment is larger than fb Mp = 226.5 ft-kips (also obtained from the beam design charts), so the design moment must be limited to fb Mp. Therefore, fb Mn = 226.5 ft-kips Factored loads: Pu = 1.2PD + 1.6PL = 1.2(35) + 1.6(99) = 200.4 kips Qu = 1.2QD + 1.6QL = 1.2(5) + 1.6(12) = 25.2 kips The maximum bending moment occurs at midheight, so Mu = 25.2(17) = 107.1 ft-kips 4 Determine which interaction equation controls: 200.4 Pu = = 0.4948 > 0.2 ∴ Use Equation 6.3 (AIISC Eq. H1-1a). In this example, the shape that comes closest to meeting the section modulus requirement is a W21 × 93, with Zx = 221 in.3, but the lightest one is a W24 × 84, with Zx = 224 in.3. Because section modulus is not directly proportional to area, it is possible to have more section modulus with less area, and hence less weight. (Muy ⎞ 0.1251 114.2 Pu ⎛ Mux + + = + +0 2fc Pn ⎜⎝ f b M nx f b M ny ⎟⎠ 2 130 = 0.941 < 1.0 (OK) ANSWER ASD SOLUTION A W8 × 35 is adequate. Solve for kv from AISC Equation G2-4 or G2-5. Approximately 16,000 ksi d. 300 300 Try an L31⁄2 ⅆ 21⁄2 ⅆ 1⁄4: Ag = 1.45 in.2 > 1.07 in.2 (OK) rmin = rz = 0.541 in. Values of Cb for other cases can be found in Part 3 of the Manual, “Design of Flexural Members.” For unbraced cantilever beams, AISC specifies a value of Cb of 1.0. A value of 1.0 is always conservative, regardless of beam configuration or loading, but in some cases it may be excessively conservative. In Figure 5.3c, the yielding has progressed into the web, and in Figure 5.3d the entire cross section has yielded. For yielding, the nominal strength is Rn = 0.6Fy Agv (AISC Equation J4-3) 446 Chapter 7 Simple Connections FIGURE 7.39 L A B A B Gusset plate thickness = t (b) (a) L t Shear area (c) where 0.6Fy = shear yield stress Agv = gross area of the shear surface = tL For LRFD, f = 1.00, and for ASD, Ω = 1.50. If this slight approximation is made, the load will be resisted by the area of weld shown in Figure 8.16b. The 80-kip load is a service load, consisting of 20 kips dead load and 60 kips live load. From the column load tables for KL = 1.0(26) = 26 ft, a W14 × 145 has a design strength of 1230 kips. < 1.20 in. 106 Chapter 3 Tension Members Snow: 20 psf of horizontal projection Roofing: 12 psf MC8 × 8.5 purlins Truss weight: 1000 lb (estimated) a. The values of the constants are based on the assumption that weak axis buckling controls the axial compressive strength and that Cb = 1.0. 3. The Manual uses a color-coding scheme for these values: ASD allowable strength values (Rn糞Ω) are shown as black numbers on a green background, and LRFD design strength values (fRn) are shown as blue numbers on a white background. This relationship is reflected in the provisions of the AISC Specification. For full composite behavior, the compressive force in the concrete at ultimate (equal to the horizontal shear at the interface between the concrete and steel) will be the smaller of As Fy = 7.68(50) = 384 kips 620 Chapter 9 Composite Construction or 0.85 f c′ Ac = 0.85(4) (3.5 × 90) = 1071 kips Use C = 384 kips. 20′ HSS12 × 8 × 3⁄16 FIGURE P9.10-1 Problems 9.10-2 663 The composite compression member shown in Figure P9.10-2 has lateral support at midheight in the weak direction only. Thanks also to Adam Belew and Ali Hajihashemi for their assistance in the preparation of the solutions manual. (OK) ANSWER ASD SOLUTION Use a 1⁄4-inch fillet weld, E70XX electrodes. Determine the lightest W14. 8 8 8 4 Line abe: An = Ag − t w d = 3.82 − 0.437 (43) = 3.49 in. Web crippling or compression buckling of the web. AISC defines kv, which is a plate-buckling coefficient, in Section G2 as follows: 5 (a糞h)2 a = 5 if > 3 h kv = 5 + a ⎡ 260 ⎤ > h ⎢ ⎜ (h糞t w) ⎟ ⎥ = 5 if (AISC Equation G2-6) 2 = 5 in unstiffened webs with h < 260 tw For Cv, which can be defined as the ratio of the critical web shear stress to the web shear yield stress, If h kE ≤ 1.10 v , tw Fy Cv = 1.0 If 1.10 kv E h kE < ≤ 1.37 v , Fy tw Fy Cv = If (AISC Equation G2-3) 1.10 kv E糞Fy h糞t w (AISC Equation G2-4) h kE > 1.37 v , tw Fy Cv = 1.51kv E (h糞t w)2 Fy (AISC Equation G2-5) 676 Chapter 10 Plate Girders Whether the shear strength is based on web shear yielding or web shear buckling depends on the web width-to-thickness ratio h糞tw. It can be treated as a simple support. “Load and Resistance Factor Design for Steel.” Journal of the Structural Division. The compressive force in the concrete is C = 335 kips, and the depth of the stress block is a = 1.0 inch. This places the bolts at the “workable gage” location in the beam flange (see Part 1 of the Manual). Tables 3-16a and 3-16b present curves that relate the variables of these two equations for steel with a yield stress of 36 ksi, and Tables 3-17a and 3-17b do the same for steels with a yield stress of 50 ksi. The summation on the left side of Equation 2.5 is over the total number of load effects (including, but not limited to, dead load and live load), where each load effect can be associated with a different load factor. The design strength for LRFD is fRn = 0.75(82.51) = 61.9 kips. Design of Partially Restrained Moment Connections. Obviously, this stiffener must be welded to both web and flange. Check the assumption: b 3.00 = = 0.14 < 0.2 d 21.1 (N.G.) For b糞d ≤ 0.2, ( ) 1.5 ⎡ b ⎛ t w ⎞ ⎤ EFy t f f(0.40)t w2 ⎢1 + 3 ≥ Ru ⎢ d ⎜⎝ t f ⎟⎠ ⎥ tw ⎣ ⎦ ( )(0.430 b ⎡ 0.75(0.40)(0.430)2 ⎢1 + 3 21 1 0.685 . The applied axial load is Pa = 28 kips The applied transverse loads are Qa = 28 kips and wa = 0.035 kips糞ft) 6.6 Members in Braced Frames 321 and the maximum bending moment is M nt = 28(10) 0.035(10)2 + = 70.44 ft-kips 4 8 The member is braced against end translation, so Mt = 0. This involves a minor liberalization of AISC Equation J10-2. 2011a. 5.10 Design 231 These curves will be described with reference to Figure 5.25, which shows a graph of nominal moment strength as a function of unbraced length Lb for a particular compact shape. 3⁄8″ 60k 60k FIGURE P7.4-4 464 Chapter 7 7.4-5 Simple Connections The tension member is an L6 × 31⁄2 × 5⁄16. Roof girders are W14 × 26, and floor girders are W16 × 40. Use A992 steel FIGURE 5.32 242 Chapter 5 Beams and design the floor beams. Check the block shear strength of the gusset plate. Before proceeding, some definitions are in order. For the intermediate stiffener welds: Minimum weld size = 3 in. Both Equations 9.4 (LRFD) and 9.6 (ASD) require an assumed depth and an estimate of d糞2. 1.2D + 1.6L + 0.5S = 1.2(109) + 1.6(46) + 0.5(20) = 214.4 kips Combination 3: 1.2D + 1.6(Lr or S or R) + (0.5L or 0.5W). Web yielding, web crippling, and concrete bearing strength are addressed by AISC in Chapter J, “Design of Connections.” Web Yielding Web yielding is the compressive crushing of a beam web caused by the application of a compressive force to the flange directly above or below the web. For the roof purlin shown in Figure 5.50, the load is vertical, but the axes of bending are inclined. 5.7 Summary of Moment Strength 215 In addition to the different meaning of Lp for noncompact shapes in the Zx table, the available strength values, fbMpx and Mp糞Ωb , are based on flange local buckling rather than the plastic moment. This results in a spacing of 324 = 81 in. A more rational procedure, however, will account for the degree of restraint provided by connecting members. For an unbraced length of 15糞2 = 7.5 ft, try a C10 × 15.3. For Cb = 1.0, Mnx 糞Ωb = 22.0 ft-kips. ST9): 1427–41. In practice, the combined weight of the slab and deck can usually be found in tables furnished by the deck manufacturer. Whenever a fully composite beam has excess capacity, which almost always is the case, the design can be fine-tuned by eliminating some of the studs, thereby creating a partially composite beam. M1 ⎞ Cmx = 0.6 − 0.4 ⎛ = 0.6 − 0.4(0) = 0.6 ⎝ M2 ⎠ Since a modified flexural rigidity, EI*, was used in the frame analysis, it must also be used in the computation of Pe1. For shear yielding, from AISC Equation J4-3, Rn = 0.60 Fy Ag = 0.60(36)(8.5t) = 183.6t Rn 183.6t = = 122.4t Ω 1.50 For shear rupture, the net area is ⎛ 3 1⎞ ⎤ ⎡ Anv = ⎢8.5 − 3 ⎜ + ⎟ ⎥ t = 5.875t ⎝ 4 8⎠ ⎦ ⎣ From AISC Equation J4-4, Rn = 0.6 Fu Anv = 0.6(58)(5.875t) = 204.5t Rn 204.5t = = 102.2t 2.00 Ω Shear rupture controls. • The live-load deflection cannot exceed L糞360. The guidelines and assumptions from Part 12 of the Manual can be summarized as follows (this list has been specialized for the connections covered in Design Guide 4): • Pretensioned Group A or Group B bolts, with diameter db ≤ 11⁄2 in., should be used. Determine the allowable strength. For allowable strength design, we can use the same nominal strength expressions as for LRFD. The minimum spacing is 2 2⁄3 d = 2.667(3糞4) = 2.0 in. For a simply supported, uniformly loaded beam, the maximum shear occurs at the support and is equal to the reaction. For a linear elastic material and small deformations, the distribution of bending stress will be as shown in Figure 5.2b, with the stress assumed to be uniform across the width 5.2 Bending Stress and the Plastic Moment 191 FIGURE 5.2 of the beam. The limit state can also be one of serviceability, such as a maximum acceptable deflection. “Shear Lag Effects in Steel Tension Members.” Engineering Journal, AISC 30 (no. 5.8 Shear Strength 219 Case 2: For all other doubly and singly symmetric shapes, fv = 0.90 Ωv = 1.67 and Cv is determined as follows: For h kE ≤ 1.10 v , there is no web instability, and tw Fy Cv = 1.0 (AISC Equation G2-3) (This corresponds to Equation 5.8 for shear yielding.) For 1.10 1.10 Cv = For kv E h kv E < ≤ 1.37 , inelastic web buckling can occur, and Fy tw Fy kv E Fy h糞t w (AISC Equation G2-4) h kE > 1.37 v , the limit state is elastic web buckling, and tw Fy Cv = 1.51kv E (h糞t w)2 Fy (AISC Equation G2-5) where kv = 5 This value of kv is for unstiffened webs with h糞tw < 260. For a compact shape with full lateral support, Mn = Mp = Fy Zx From Mp Ωb Fy Z x Ωb ≥ Ma, ≥ Ma Zx ≥ Ωb M a 1.67(506.3 × 12) = = 203 in.3 50 Fy The Zx table lists hot-rolled shapes normally used as beams in order of decreasing plastic section modulus. 358 Chapter 6 Beam–Columns Try a W12 × 45. I have the comprehensive instructor's solution manuals in an electronic format for the following textbooks. Use f ′c = 5 ksi. Its organizational scheme is the same as that of the Specification, so material applicable to a particular section can be easily located. For more heavily loaded base plates (the boundary between lightly loaded and heavily loaded plates is not well defined), Thornton (1990a) proposed an analysis based on two-way bending of the portion of the plate between the web and the flanges. The minimum width of the plate can be determined from a consideration of edge distances. Murray, Thomas M. This last disadvantage can be partially overcome by using shop welding instead of field welding whenever possible. Load factors are applied to the service loads, and a member is selected that will have enough strength to resist the factored loads. Although the AISC Specification uses a dual system of units, the steel construction industry is still in a period of transition. However, a large number of hollow structural shapes (HSS), double angles, and tees have slender elements. The bolts are 3⁄4 inch in diameter. 7.2 Bolted Shear Connections: Failure Modes 381 of free-body diagrams of portions of the fastener shank shows that each crosssectional area is subjected to half the total load, or, equivalently, two cross sections are effective in resisting the total load. For local flange bending, the Specification requires a pair of transverse stiffeners. For the end panels, tension-field action is not permitted, and the shear strength must be computed from AISC Equation G2-1: Vn = 0.6Fy Aw Cv = 0.6(36)(23.62)(0.2567) = 131.0 kips The design strength is fvVn = 0.90(131.0) = 118 kips The maximum factored load shear in the end panel is Vu = 234 kips > 118 kips (N.G.) Two options are available for increasing the shear strength: either decrease the web slenderness (probably by increasing its thickness) or decrease the aspect ratio of each end panel by adding an intermediate stiffener. The controlling base metal shear strength is 21.1 kips糞in., but this is greater than the weld strength of 6.614 kips糞in. Dividing this segment into four equal spaces, we get points A, B, and C located at 15 ft, 18 ft, and 21 ft from the left end of the girder. It must support a service load reaction of 5 kips dead load and 10 kips live load. Select a shape that satisfies this strength requirement. Although both the gusset plate and the tension member plate are subject to shear, we will examine the shear on the gusset plate adjacent to the weld AB. If the load is assumed to distribute itself at a slope of 1 : 2.5, as shown in Figure 5.37, the area at the support FIGURE 5.37 b b 5.14 Beam Bearing Plates and Column Base Plates 253 subject to yielding is tw(2.5k + b). Adequate: For shear, 27.6 kips < 31.8 kips; For tension, 11.8 kips < 22.9 kips. Using more than three or four significant figures is probably unrealistic in most cases, and results based on less than three may be too approximate to be of any value. AISC J10.6 also provides equations to be used when frame stability, including plastic deformation of the panel zone, is considered in the analysis. SOLUTION An = Ag − Aholes = 5.77 − ( ) 1 5 1 + (2) = 5.02 in.2 2 8 8 Only one element (one leg) of the cross section is connected, so the net area must be reduced. It is equal and opposite to the internal resisting moment corresponding to the stress distribution shown in Figure A.1c. Ωb Compute the axial compressive strength: K y L = 15 ft K x L 1.2(15) = = 10.29 ft < 15 ft ∴ Use KL = 15 ft. − 5.5) a= = 3.450 inn. 1 1 rn = 31.5t = 31.5 ⎛⎜ + ⎞⎟ = 15.8 kips糞bolt ⎝ 4 Ω 4⎠ The total strength of the connection is the sum of the minimum strengths at each bolt location (AISC J3.6 and J3.10 User Notes). 61⁄2″ D = 50k L = 25k 60° 2″ 3 sp @ 31⁄2″ 8.2-8 2″ 11⁄2″ 51⁄2″ 11⁄2″ FIGURE P8.2-8 576 Chapter 8 8.2-9 Eccentric Connections Group A bolts are used in the connection in Figure P8.2-9. If this force is considered to be the horizontal component of an axial compressive force P in the stiffener, Pcosq = 36.43 kips where 17.9 ⎞ ⎛ db ⎞ q = tan −1 ⎛ ⎞ = tan −1 ⎛⎜ ⎟ = 65.26⬚ ⎝ 8.25 ⎟⎠ ⎝ dc ⎠ 36.43 = 87.05 kips P= cos(65.26⬚) Because the stiffener will be connected along its length, we will consider it to be a compression member whose effective length is zero. Using AISC Equation J8-1, we obtain Pp 0.85 fc′A1 = ≥ Ra Ωc Ωc 0.85(3.5) A1 ≥ 71.24 2.31 A1 ≥ 55.32 in.2 The minimum value of dimension B is B= A1 55.32 = = 9.22 in. The reason is that 3⁄11 of the load will have been transferred from the member before ijfh receives any load. FIGURE 9.17 SOLUTION Compute the loads (other than the weight of the steel shape). ANSWER The strength of the connection based on the limit states investigated is fRn = 4(9.49) = 38.0 kips. As we shall see, the moment strength depends in part on the unbraced length, which is the distance between points of bracing. and Sumner. Washington D.C. 730 American Concrete Institute. In addition to the secondary moments caused by member deformation (P-d moments, shown in Figure 6.4a), additional secondary moments are present when one end of the member translates with respect to the other. This is

covered in Section 9.7. The AWS Structural Code (AWS, 2008) lists standard stud diameters of 1⁄2, 5⁄8, 3 7⁄4, ⁄8, and 1 inch. Flange: l= b t l p = 1.12 E Fy lr = 1.40 E Fy l p = 2.42 E Fy lr = 5.70 E Fy ii. ST1): 107–126. Beams are usually thought of as being oriented horizontally and subjected to vertical loads, but that is not necessarily the case. Is the connection adequate? 16 4 16 16 For the hole nearest the edge of the gusset plate, h 13冀16 = 1.5 − = 1.094 in. • span length = 30 ft • formed steel deck is used (see Figure P9.7-3) • studs are 3⁄4-inch × 3-inch, one in every third rib 6″ 21⁄2″ avg. Ω The shear strength of the weld controls. Therefore, for singly symmetrical shapes, there are two possibilities for the strength: either flexuraltorsional buckling about the y-axis (the axis of symmetry) or flexural buckling about the x-axis (Timoshenko and Gere, 1961 and Zahn and Iwankiw, 1989). Thus, Rn = 2(edge bolt strength) + 2(inner bolt strength) Ω = 2(15.8) + 2(21.8) = 75.2 kips > 55 kips (OK) ANSWER Use four bolts, two per line, on each side of the splice. 3 Agv = 2 × (2.75 + 2.75 + 1.25) = 5.063 in.2 8 3 Anv = 5.063 − (2.5 × 1.0)(2) = 3.188 in.2 8 φ Rn = 0.75(0.6 Fu Anv + U bs Fu Antt) = 0.75[0.6(58)(3.188) + 1.0(58)(0.75)] = 116 kips > 108 kips (OK) Check the upper limit: f[0.6Fy Agv + UbsFu Ant] = 0.75[0.6(36)(5.063) + 1.0(58)(0.7500)] = 115 kips < 116 kips Therefore, the upper limit controls, but the strength is still adequate. Both the second-order analysis and the approximate second-order analysis are considered direct analysis methods. The column loads are supported by the columns of the story below. The effective length KL is 26 feet. 8 8 and the net flange area is A fn = A fg − t f ∑ d h = 6.188 − 0.810(2 × 1.125) = 4.366 in.2 Fu A fn = 65(4.366) = 283.8 kips Determine Yt. For A992 steel, the maximum Fy 冀Fu ratio is 0.85. Figure 9.25a shows a typical continuous beam and the regions in which anchors would be required. When a composite beam has reached the plastic limit state, the stresses will be distributed in one of the three ways shown in Figure 9.7. The concrete stress is shown as a uniform compressive stress of 0.85fc′, extending from the top of the slab to a depth that may be equal to or less than the total slab thickness. Each graph gives the flexural strength of a standard hot-rolled shape. Shear strength: From the LRFD solution, the nominal shear strength (except for the end panels), is Vn = 263.2 kips The allowable strength is Vn 263.2 = = 157.6 kips Ωv 1.67 From Figure 10.15, the maximum shear in the middle half of the girder is 68 kips, so the shear strength is adequate where tension-field action is permitted. Computer solutions for various common configurations of eccentric welded shear connections are given in tabular form in Part 8 of the Manual. LRFD SOLUTION Temporarily ignoring the beam weight, the factored-load moment is Mu = 6(1.6 × 20) = 192 ft-kips 234 Chapter 5 Beams From the charts, with Lb = 24 ft, try W12 × 53: fb Mn = 209 ft-kips > 192 ft-kips (OK) Now, we account for the beam weight: 1 Mu = 192 + (1.2 × 0.053)(24)2 = 197 ft-kips < 209 ftt -kips (OK) 8 The shear is Vu = 1.6(20) + 1.2(0.053)(24) = 32.8 ft-kips 2 From the Zx table (or the uniform load table), fvVn = 125 kips > 32.8 kips (OK) The maximum permissible live load deflection is L 24(12) = = 1.20 in. EXAMPLE 5.10 Select a standard hot-rolled shape of A992 steel for the beam shown in Figure 5.24. In this book, we provide only a brief introduction to the standard symbols for fillet welds. If it is, provide two alternatives: (1) a doubler plate of A36 steel and (2) diagonal stiffeners of A36 steel. rx 冀ry 1.75 φc Pn = 900 kips > 840 kips (OK) Investigate W14 shapes. Adjustment for Cb Not Equal to 1.0 This procedure, suggested by Aminmansour (2009), is developed here from an LRFD point of view, but the procedure is independent of the design philosophy. The total strength is therefore Rn = 28.55 + 30.04 = 58.59 kips ANSWER For LRFD, the design strength is f Rn = 0.75(58.59) = 43.9 kips. Once the concrete has cured, composite action is possible, and all subsequently applied loads will be resisted by the composite beam. 550 Chapter 8 Eccentric Connections The design of the welds connecting the doubler plate to the column will depend on the exact configuration of the plate, including whether the plate extends beyond the transverse stiffeners. Ductility can be measured by the elongation, defined as e = L f − L0 L0 × 100 (1.1) where e = elongation (expressed as a percent) Lf = length of the specimen at fracture L0 = original length The elastic limit of the material is a stress that lies between the proportional limit and the upper yield point. (December). This layout is not necessarily the final form the arrangement will take. EXAMPLE 7.13 If the connection of Example 7.12 includes both the 4-inch-long longitudinal welds shown in Figure 7.40 and an additional 4-inch-long transverse weld at the end of the member, what is the available strength of the connection? Before the external load is applied, the normal compressive force, N0, is centered on the bolt. 5.7 SUMMARY OF MOMENT STRENGTH The procedure for computation of nominal moment strength for I and C-shaped sections bent about the x axis will now be summarized. Many connections incorporate both welds and bolts. A307 b. First-order analyses were performed with reduced member stiffnesses. Use Equation 3.1 for the shear lag factor, U. When the bending moment is uniform, the value of Cb is Cb = 12.5 M = 1.0 2.5 M + 3M + 4 M + 3M 5.5 Bending Strength of Compact Shapes 209 EXAMPLE 5.5 Determine Cb for a uniformly loaded, simply supported W shape with lateral support at its ends only. 8 8 8 d′ 1 δ = 1 − = 1 − = 0.75 4 p d′ = d + The bolt cross-sectional area will also be needed in subsequent calculations: Ab = LRFD SOLUTION π (7冀8)2 = 0.6013 in.2 4 The design strength of one bolt is B = fRn = fFt Ab = 0.75(90.0)(0.6013) = 40.59 kips The total factored load is 1.2D + 1.6L = 1.2(20) + 1.6(60) = 120 kips and the number of bolts required (without considering prying action) is 120冀40.59 = 2.96. The required number of bolts per vertical row if the factored load is 175 kips. To convert the equations, we simply replace the u subscripts with a subscripts and replace f by 1冀Ω. The live load is 80 psf, and there is a 4-inch-thick normal-weight reinforced concrete floor slab. B C 10k 20′ A D 35′ FIGURE P3.7-4 Problems 3.7-5 105 What size A36 threaded rod is required for member AB, as shown in Figure P3.7-5? ASD Φ= 1 Ω and Fnt′ = 1.3Fnt − where Ω = 2.00. The amplified moment to be used in design is computed from the loads and moments as follows (x and y subscripts are not used here; amplified moments must be computed in the following manner for each axis about which there are moments): Mr = B1Mnt + B2 M t (AISC Equation A-8-1) where Mr = required moment strength = Mu for LRFD = Ma for ASD Mnt = maximum moment assuming that no sidesway occurs, whether the frame is actually braced or not (the subscript nt is for “no translation”). From AISC Table J2.4, the minimum weld size is 3⁄16 inch. (Obviously this case will never control when live load is present.) D + L = 109 + 46 = 155 kips D + (Lr or S or R). Adequate: Ma = 141 ft-kips < Mn冀Ω b = 180 ft-kips 5.5-15 a. 2): 36–45. G., Johnson, J. Introduction: Analysis of Composite Beams 9.1-1 A W16 × 26 floor beam supports a 4-inch-thick reinforced concrete slab with an effective width b of 32 inches. ANSWER Use a 3⁄16-inch fillet weld with E70XX electrodes, with a total length of 9 inches, as shown in Figure 7.45. Christopher, John E., and Bjorhovde, Reidar 1999. Experimental evidence shows that this is not the case and that individual fasteners do not have a well-defined shear yield stress. The Plastic Methods of Structural Analysis. References ABBREVIATIONS AASHTO ACI AISC AISI AREMA ASCE ASTM AWS BOCA FEMA ICBO ICC RCSC SBCCI SJI SSRC American Association of State Highway and Transportation Officials American Concrete Institute American Institute of Steel Construction American Iron and Steel Institute American Railway Engineering and Maintenance-of-Way Association American Society of Civil Engineers American Society for Testing and Materials American Welding Society Building Officials and Code Administrators International Federal Emergency Management Agency International Conference of Building Officials International Code Council Research Council on Structural Connections Southern Building Code Congress International Steel Joist Institute Structural Stability Research Council Ad Hoc Committee on Serviceability. ∴ Use 1.5 in. Reducing the volume of weld metal will also minimize heat buildup and residual stresses. (1.5)2 ⎤ (1.5)2 ⎤ (1.5)2 ⎤⎡ ⎡ ⎡ 0 . In this manufacturing process, which 1.6 Standard Cross-Sectional Shapes 13 takes place in a mill, molten steel is taken from an electric arc furnace and poured into a continuous casting system where the steel solidifies but is never allowed to cool completely. If cover plates were added, the finished product became even more complicated. Use Kx = 0.8. a. CHAPTER 5 Beams 5.1 INTRODUCTION Beams are structural members that support transverse loads and are therefore subjected primarily to flexure, or bending. 20.60 KL 15 × 12 = = 108.4 rmin 1.661 Fe = 4.71 Since π 2E π 2 (29, 000) = 24.36 ksi = (108.4)2 (KL冀r)2 E 29, 000 = 4.71 = 113 Fy 50 KL E < 4.71 , use AISC Equation E3-2. If the connection is to be slip critical, how many 13⁄8-inch-diameter, Group A bolts are needed? The effective length factor for the fixed-pinned compression member is 0.70. ANSWER 5.3 Mp = Fy Z = 50(74.62) = 3731 in.-kips = 311 ft-kips. With relative bracing, a brace point is restrained relative to adjacent brace points. The number of steel anchors required between each support and the point of maximum positive moment will be half the total number required. b. *8K1 is no longer made as of 2010. In each of the two situations depicted in Figure 5.49, only about half the cross section is considered to be effective with respect to its y axis; therefore, when considering the strength of a single flange, use half the tabulated value of Zy for the shape. The fasteners at the top of the connection will therefore be subjected to both shear and tension. 609 610 Chapter 9 9.4 Composite Construction STEEL HEADED STUD ANCHORS As we have shown, the horizontal shear force to be transferred between the concrete and the steel is equal to the compressive force in the concrete, C. Flange local buckling: If l ≤ lp, there is no FLB If lp < l ≤ lr , the flange is noncompact, and ⎛ l − lp ⎞ M n = M p − (M p − 0.7 Fy S x) ⎜ ⎝ lr − l p ⎟⎠ b. For the inner holes, conservatively use the spacing of 22⁄3db in Manual Table 7-4. ANSWER For LRFD, the design strength is fcPn = 0.90(683.3) = 615 kips. Our strategy will be to select the lightest shape for various values of rmx 冀rmy, then choose the lightest shape overall. For this reason, some general discussion of roof trusses is in order. Investigate all possible failure modes. Furthermore, we neglect the moment of inertia of a line segment about the axis coinciding with the line. The usual configuration is a single web with two equal flanges, with all parts connected by welding. If the hole is slightly larger than the fastener and the fastener is assumed to be placed loosely in the hole, contact between the fastener and the connected part will exist over approximately half the circumference of the fastener when a load is applied. The AISC Specification furnishes little guidance other than a statement in Chapter L, “Design for Serviceability,” that deflections should not be excessive. Assume that the weak direction controls and enter the column load tables with KL = 9 feet. With a beam setback of 1⁄2 inch and edge distances of 11⁄2 inches, as shown in Figure 8.39b, the width of the plate is 0.5 + 2(1.5) = 3.5 in. The bottom chord is designed by determining the force in each member of the bottom chord and selecting a cross section to resist the largest force. 3 This result, when rounded to three significant figures, is the same as the value given in the dimensions and properties tables. In addition, the beam cross section must have a vertical axis of symmetry, and the loads must be in the longitudinal plane containing this axis. In the United States, the first structural steel railroad bridge was the Eads bridge, constructed in 1874 in St. Louis, Missouri (Tall, 1964). The intervals must be small enough that the member functions as a unit. In most beam-to-column moment connections, the members are part of a plane frame and are oriented as shown in Figure 8.37a — that is, with the webs in the plane of the frame so that bending of each member is about its major axis. For ASD, the allowable strength Mn 冀Ωb is tabulated. = 33.4 kips ANSWER ASD SOLUTION Design strength of the weld is 33.4 kips. Determine the design tensile strength for LRFD. a ⎡ 260 ⎤ ⎡ 260 ⎤ 0.5 (36冀62)2 and from AISC Equation G2-7, the required moment of inertia is I st1 = bt w3 j = min(a,h)t w3 j = min(81,62)(5冀16)3 (5.415) = 62(5冀16)3 (5.415) = 5.949 in.4 10.7 Design 705 I st 2 = moment of inertia required to develop thee buckling plus post-buckling shear strength (from AISC Equation G3-5) = h 4 ρ1st.3 ⎛ Fyw ⎞ 40 ⎝ E ⎠ 1.5 36 ⎞ ⎛ Fyw ⎞ ρst = max ⎜ , 1 ⎟ = max ⎛ ,1 =1 ⎝ 36 ⎠ ⎝ Fyst ⎠ I st 2 = (62)4 (1)1.3 ⎛ 36 ⎞ ⎟ ⎝ 40 29, 000 ⎠ 1.5 = 16.16 in.4 To the left of this stiffener, the stiffener spacing is 36 inches, and to the right, the spacing is 81 inches. The dead, live, and snow loads are gravity loads and always act downward, whereas the wind load on the roof will always act upward. Assume that V = 0 and Pr冀Pc = 0. Select a plate width bp and bolt locations relative to the beam flanges (pfi , pfo, and g). Turn-of-the-nut method. For web crippling or compressive buckling of the web, transverse stiffeners (either a single stiffener or a pair) or a web doubler plate should be used. Column AB, Pu = 750 kips. FIGURE 8.26 SOLUTION As in previous design examples, a unit throat size will be used in the calculations. 10.4-5 An 80-foot-long plate girder is fabricated from a 1⁄2-inch × 78-inch web and two 3-inch × 22-inch flanges. Problems Eccentric Bolted Connections: Shear Only 8.2-1 Use an elastic analysis to determine the maximum bolt shear force in the bracket connection of Figure P8.2-1. 2002. A992 steel is used for the beam and column, and FIGURE 8.47 546 Chapter 8 Eccentric Connections A36 is used for the plate material. This document authorized the substitution of high-strength bolts for rivets on a one-for-one basis. For each combination of span and joist, a pair of load values is given. FIGURE 7.42 Maximum Length AISC does not impose a limit on the length of welds, but for end-loaded welds, there are some restrictions. The usual selection process is to determine the number of Group A bolts needed in a connection, and if too many are required, use Group B bolts. 9.1-4 Compute the nominal flexural strength of the composite beam of Problem 9.1-1. The analysis of riveted connections is essentially the same as for connections with common bolts; only the material properties are different. L2 = 5.916 5.916 L1 = The second option results in a slight savings and will be used. wD = 0.30(3) = 0.9 kips冀ft wL = 0.70(3) = 2.1 kips冀ft LRFD SOLUTION wu = 1.2(0.9) + 1.6(2.1) = 4.44 kips冀ft Pu = 1.6(9) = 14.4 kips The factored loads and reactions are shown in Figure 5.30. This is because the required number of studs is determined by dividing a nominal strength V′ by a nominal strength Qn, and no applied loads are involved. The tables in Part 4 of the Manual may be used. This value can then be substituted into the force equilibrium equation: ∑ F = T − P1 − P2 − P3 = 0 This equation can be solved for P2, the required resisting force in the lower longitudinal weld. To prevent web crippling when the compressive load is delivered to one flange only, as in the case of an exterior column with a beam connected to one side, the applied load must not exceed the available strength. If the transverse reinforcement consists of lateral ties, the minimum amount is either #3 bars at a maximum spacing of 12 inches on center or larger bars spaced at a maximum of 16 inches on center. Required size = 4.86 sixteenths of an inch 9.6-1 a. Low-alloy steels: iron and carbon plus other components (usually less than 5%). 5.14-3 Design a base plate for a W12 × 120 column supporting a service dead load of 75 kips and a service live load of 185 kips. Determine the nominal strength based on the gross area. To specify the process used or to furnish other information, a tail can be placed at the end of the reference line and the desired notation placed beside it. Steel Anchors: Because this beam has an excess of moment strength, it will benefit from partial composite behavior. A function that satisfies both the differential equation and the boundary conditions is y = B sin px L where B is a constant. The shape is compact (no footnote to indicate otherwise) and Lp = 6.36 ft, Lr = 18.7 ft The unbraced length Lb = 12 ft, so Lp < Lb < Lr , and the controlling limit state is inelastic lateral-torsional buckling. An = Ag − Aholes = 4.75 − 2 (43 + 81)(12) = 3.875 in. 8. Let l = width-to-thickness ratio lp = upper limit for compact category lr = upper limit for noncompact category Then if l ≤ lp and the flange is continuously connected to the web, the shape is compact; if lp < l ≤ lr , the shape is noncompact; and if l > lr , the shape is slender. Ideally, alternative designs should be prepared with each. v This page intentionally left blank Contents About the Author Preface 1 1.1 1.2 1.3 1.4 1.5 1.6 2 2.1 2.2 2.3 2.4 2.5 2.6 2.7 v 3 xi Introduction 3 Structural Design 3 Loads 6 Building Codes 7 Design Specifications 7 Structural Steel 8 Standard Cross-Sectional Shapes 12 Problems 17 Concepts in Structural Steel Design Design Philosophies 21 American Institute of Steel Construction Specification 23 Load Factors, Resistance Factors, and Load Combinations for LRFD 24 Safety Factors and Load Combinations for ASD 26 Probabilistic Basis of Load and Resistance Factors 30 Steel Construction Manual 34 Design Computations and Precision 36 Problems 38 21 3.1 3.2 3.3 3.4 3.5 3.6 3.7 3.8 3.9 4 4.1 4.2 4.3 4.4 4.5 4.6 4.7 4.8 4.9 Tension Members 41 Introduction 41 Tensile Strength 42 Effective Area 50 Staggered Fasteners 57 Block Shear 64 Design of Tension Members 67 Threaded Rods and Cables 76 Tension Members in Roof Trusses 79 Pin-Connected Members 88 Problems 90 Compression Members 109 Introduction 109 Column Theory 110 AISC Requirements 117 Local Stability 121 Tables for Compression Members 130 Design 132 More on Effective Length 138 Torsional and Flexural-Torsional Buckling 155 Built-Up Members 162 Problems 171 vii viii 5 5.1 5.2 5.3 5.4 5.5 5.6 5.7 5.8 5.9 5.10 5.11 5.12 5.13 5.14 5.15 5.16 6 6.1 6.2 6.3 6.4 6.5 6.6 6.7 6.8 6.9 7 7.1 7.2 Contents Beams 189 Introduction 189 Bending Stress and the Plastic Moment 190 Stability 196 Classification of Shapes 198 Bending Strength of Compact Shapes 199 Bending Strength of Noncompact Shapes 211 Summary of Moment Strength 215 Shear Strength 216 Deflection 224 Design 226 Floor and Roof Framing Systems 239 Holes in Beams 245 Open-Web Steel Joists 248 Beam Bearing Plates and Column Base Plates 251 Biaxial Bending 265 Bending Strength of Various Shapes 276 Problems 281 Beam–Columns Introduction 377 Bolted Shear Connections: Failure Modes 380 Bearing Strength, Spacing, and Edge-Distance Requirements 382 7.4 Shear Strength 388 7.5 Installation of High-Strength Bolts 394 7.6 Slip-Critical and Bearing-Type Connections 396 7.7 Design Examples 402 7.8 High-Strength Bolts in Tension 416 7.9 Combined Shear and Tension in Fasteners 428 7.10 Welded Connections 441 7.11 Fillet Welds 443 Problems 461 8 8.1 8.2 8.3 8.4 8.5 299 8.6 8.7 8.8 8.9 Definition 299 Interaction Formulas 300 Methods of Analysis for Required Strength 305 The Moment Amplification Method 307 Braced versus Unbraced Frames 310 Members in Braced Frames 311 Members in Unbraced Frames 327 Design of Beam–Columns 339 Trusses with Top-Chord Loads Between Joints 358 Problems 363 Simple Connections 7.3 9 377 9.1 9.2 9.3 9.4 9.5 9.6 9.7 9.8 Eccentric Connections 477 Examples of Eccentric Connections 477 Eccentric Bolted Connections: Shear Only 478 Eccentric Bolted Connections: Shear Plus Tension 491 Eccentric Welded Connections: Shear Only 496 Eccentric Welded Connections: Shear Plus Tension 511 Moment-Resisting Connections 519 Column Stiffeners and Other Reinforcement 536 End Plate Connections 558 Concluding Remarks 572 Problems 572 Composite Construction 593 Introduction 593 Shored versus Unshored Construction 604 Effective Flange Width 606 Steel Headed Stud Anchors 610 Design 613 Deflections 620 Composite Beams with Formed Steel Deck 624 Tables for Composite Beam Analysis and Design 636 9.9 Continuous Beams 644 9.10 Composite Columns 645 Problems 655 Contents 10 Plate Girders 665 10.1 Introduction 665 10.2 General Considerations 667 10.3 AISC Requirements for Proportions of Plate Girders 670 10.4 Flexural Strength 672 10.5 Shear Strength 675 10.6 Bearing Stiffeners 680 10.7 Design 695 Problems 713 Appendix A.1 A.2 A.3 A.4 A.5 Plastic Analysis and Design Introduction 718 AISC Requirements 720 Analysis 721 Design 726 Concluding Remarks 729 References 730 Answers to Selected Problems Index 739 735 ix 718 This page intentionally left blank Preface Steel Design, Fifth Edition covers the fundamentals of structural steel design for buildings. Analysis and design are initially presented without the aid of tables and graphs from the Manual, followed by the introduction of these design aids. Regardless of the type of fastener, each one will receive an equal share of the shear load. From Equation 5.18, the required plate thickness is t= ANSWER ASD SOLUTION 2 Pu 2(349.6) = 2.5 = 0.893 in. “High-Strength Bolts Subjected to Tension and Shear.” Journal of the Structural Division, ASCE 91 (no. The total load is Pa = D + L =10 + 30 = 40 kips. The required plate thickness is t≥ 2 Pu 0.90 Fy BN where = max(m, n, l n ′) N − 0.95d 2 B − 0.8b f n= 2 m= l= 2 X ≤1 1+ 1− X ⎛ 4 db f ⎞ Pu X =⎜ ⎝ (d + b f)2 ⎟⎠ fcPp 1 db f 4 fc = 0.65 n′ = Pp = nominaal bearing strength from AISC Equation J8-1 or J8-2 (5.18) 5.14 Beam Bearing Plates and Column Base Plates 263 There is no need to determine whether the plate is large or small, lightly loaded, or heavily loaded. Determine the required ASD load capacity. The concentrated load is a service live load, and the uniform load is 30% dead load and 70% live load. Galambos, Theodore V., Lin, F. The actual effective net area and rupture strength are computed as follows: An = Ag − Aholes = 5.80 − 2 (43 + 81)(12) = 4.925 in. Check shear in the plate. EXAMPLE 8.12 The beam-to-column connection shown in Figure 8.47 must transfer the following gravity load moments: a service dead-load moment of 32.5 ft-kips and a service live-load moment of 97.5 ft-kips. Because of the small size required for strength, investigate the possibility of using intermittent welds. Determine which interaction equation to use: Pa = pPa = (3.35 × 10 −3)(100) = 0.335 > 0.2 Pn 冀Ωc ∴ Use Equaation 6.9. 6.8 Design of Beam–Columns 349 As a rough check (remember that B1 has not yet been computed and Cb has not been accounted for), pPa + bx Max + by May = (3.35 × 10−3)(100) + (6.54 × 10−3)(50) + (12.6 × 10−3)(20) = 0.914 < 1.0 (OK) It is likely that B1x and B1y will be equal to 1.0. In addition, the inclusion of Cb will reduce the value of bx and the result of the interaction equation, so this shape is probably satisfactory. For a positive service load moment of 150 ft-kips, compute the stress at the top of the steel (indicate whether tension or compression), the stress at the bottom of the steel, and the stress at the top of the concrete. In a bearingtype connection, slip is acceptable, and shear and bearing actually occur. Moment-rotation curves can be constructed for specific connections, and the relationship can be determined experimentally or analytically. “Flexural-Torsional Buckling and its Implications for Steel Compression Member Design.” Engineering Journal, AISC 26 (no. < 90 in., b = 66 in. 13.6% 5.12-3 a. 2 2 Figure 9.21 shows the corresponding transformed section. The structural engineer must select and evaluate the 3 4 Chapter 1 Introduction overall structural system in order to produce an efficient and economical design but cannot do so without a thorough understanding of the design of the components (the “building blocks”) of the structure. • For LRFD, the design strength is fbMn , where fb = 0.90. Thus full-depth stiffeners are required only when beams are connected to both sides of the column. The stiffeners are clipped 1 ⁄2 inch to provide clearance for the flange-to-web welds. This condition can occur in the top chord of a roof truss with purlins located between the joints. a 81 = = 1.306 < 3 h 62 b. Variables that affect economy include weight (volume of steel required) and fabricating costs. y= ∑ Ay 183.2 = = 8.893 in. 7.3 Bearing Strength, Spacing, and Edge-Distance Requirements FIGURE 7.9 385 e e s s e s e s e e Spacing and Edge-Distance Requirements To maintain clearances between bolt nuts and to provide room for wrench sockets, AISC J3.3 requires that center-to-center spacing of fasteners (in any direction) be no less than 22⁄3 d and preferably no less than 3d, where d is the fastener diameter. What is the total service load that can be supported, based on bolt shear and bearing, if the ratio of live load to dead load is 2.0? The bolt threads are in the plane of shear. This member will be connected to a 3⁄ 8-inch gusset plate with 3⁄4-inch-diameter Group A bolts. Determine the required area. One condition that usually is not satisfied is the requirement that all behavior be elastic. For the edge bolts, frn = 44.0t kips冀bolt, and for the inner bolts, frn = 78.3t kips冀bolt. AISC Specification Requirements for Shear For LRFD, the relationship between required and available strength is Vu ≤ fvVn where Vu = maximum shear based on the controlling combination of factored loads fv = resistance factor for shear For ASD, the relationship is Va ≤ Vn Ωv where Va = maximum shear based on the controlling combination of service loads Ωv = safety factor for shear As we will see, the values of the resistance factor and safety factor will depend on the web width-to-thickness ratio. Model codes are written by various nonprofit organizations in a form that can be easily adopted by a governmental unit. The holes are for 5⁄8-inch-diameter bolts. The specifications of most interest to the structural steel designer are those published by the following organizations. Bending will result, but it usually can be regarded as secondary. For one angle, the required design strength is 243.0 = 121.5 kips 2 Try a 5⁄ 16-inch fillet weld: Capacity per inch of length = fRn = 1.392D = 1.392(5) = 6.960 kips冀in. L2 = 4.640 4.640 L1 = 8.5 Eccentric Welded Connections: Shear Plus Tension 511 We will now examine the second option given in AISC J2.4c, in which we use 150% of the basic strength for the transverse weld and 85% of the basic strength for the longitudinal welds. The additional moment required to bring the beam from stage b to stage d is 10 to 20% of the yield moment, My , for W shapes. ⎞ 2⎠ Ω The direct shear to be resisted by one angle is 326.5 Ra = = 4.99 kips/in. 1′- 0″ 7″ 3″ 5″ FIGURE P9.8-8 9.8-9 Use the composite beam tables, and select a W-shape and stud anchors for the following conditions: • span length = 18′-6″ • beam spacing = 9 ft • total slab thickness = 51⁄2 in. The AISC Specification provisions for flexural strength are as follows: • For shapes with compact webs—that is, h冀t w ≤ 3.76 E冀Fy —the nominal strength Mn is obtained from the plastic stress distribution. 10.4-4 Determine the nominal flexural strength of the following welded shape: The flanges are 3⁄4 inch × 18 inches, the web is 1⁄4 inch × 52 inches, and the member is simply supported, uniformly loaded, and has a span length of 50 feet. Rn = 1.2 Lc tFu = 1.2(1.531)(0.455)(65) = 54.34 kips 7 Uppeer limit = 2.4 dtFu = 2.4 ⎛ ⎞ (0.455)(65) ⎝ 8⎠ = 62.11 kips > 54.34 kips ∴ Use Rn = 54.34 kips for this bolt. E70 electrodes, with a design strength of 1.392 kips冀inch per sixteenth of an inch in size, will be used for all welds. From C = ΣQn , a= ΣQn 135 = = 0.6016 in. This member is part of a braced frame and is subjected to the service-load axial force and bending moments shown (the end shears are not shown). = 8.692 ft Fy 50 5.5 207 Bending Strength of Compact Shapes The following terms will be needed in the computation of Lr: rts2 = I yC w 121(5380) = = 7.833 in.2 103 Sx rts = 7.833 = 2.799 in. To select a KCS joist, the engineer must compute a maximum moment and shear in the joist and enter the KCS tables with these values. FIGURE 3.37 3.9 Pin-Connected Members 89 Pin-connected members should be designed for the following limit states (see Figure 3.38). It is heavier than the lightest one found so far, so it will not be considered. ANSWER Use a W10 × 54. 0.56 E 0.56 29, 000 10.7 Design 707 FIGURE 10.21 Try two 3⁄4 ⁄ 8 stiffeners. These cases are covered in AISC Sections F4 and F5. The vertical component of the load will put the fasteners in shear, and the horizontal component will cause tension (with the possible inclusion of prying forces). The strength for Cb = 1.36 is Mp Mn = 1.36(223) = 303 ft-kips > = 280 ft-kips Ωb Ωb ∴ Mn M p = = 280 ft-kips > Ma = 270 ft-kips Ωb Ωb (OK) Account for the beam weight. EXAMPLE 4.14 A rigid unbraced frame is shown in Figure 4.17. An axial compressive load is applied and gradually increased. The usual fabrication practice is to punch or drill holes in standard locations in angle legs. The two components are connected along their length in such a way as to maintain the 3⁄8-inch separation. and Iwankiw, N. The concentrated load is a service live load. For relative bracing, the required stiffness is bbr = 1 ⎛ 2 Pr ⎞ for LRFD f ⎜⎝ Lb ⎟⎠ ⎛ 2P ⎞ = Ω ⎜ r ⎟ for ASD ⎝ Lb ⎠ (AISC Equation A-6-2) where Lb = unbraced length of the column (this will be the story height unless there are intermediate braces) f = 0.75 Ω = 2.00 EXAMPLE 6.10 A structure composed of three rigid frames must be stabilized by diagonal bracing in one of the frames. In Figure 8.35, a beam line is superimposed on the three moment-rotation curves of Figure 8.33. Determine the nominal flexural strength. The plate connecting the beam web to the column is shop welded to the column and field bolted to the beam. 9.2-2 A composite floor system consists of W12 × 22 beams and a 4-inch-thick reinforced concrete floor slab. 3″ 11⁄2″ 8″ Reaction = 35k (D = 10k, L = 25k) 3⁄4″-diameter, Group A bolts A992 steel Slip is permitted 6″ 3″ W10 × 49 WT5 × 22.5 W18 × 50 11⁄2″ FIGURE P8.3-4 8.3-5 Check the adequacy of the bolts in the connection shown in Figure P8.3-5. (Since 3⁄4-inch-diameter bolts were selected for the shear connection, try the same size here). Check this connection for compliance with the AISC Specification. The additional resistance to lateral loads can take the form of diagonal bracing or rigid shear walls, as illustrated in Figure 4.16. Smaller compression members not classified as columns are sometimes referred to as struts. The AISC Specification furnishes no guidance for gage lines crossing a “fold” when the different elements have different thicknesses. Do not check deflection. different member cross sections, but they all must have a nominal depth of 10 inches and, for a span length of 20 feet, a factored load capacity of at least 361 pounds per foot. FIGURE 8.54 1″ 13⁄4″ 11⁄2″ 7″ 31⁄2″ 13⁄4″ 5⁄16 2″ 2″ 1⁄4 1′-13⁄4″ 3⁄16 5⁄16 2′- 0 3⁄4″ 2″ 2″ 11⁄2″ Special Requirements for Columns in End-Plate Connections When end-plate connections are used, column flange bending strength is based on yield-line theory. FIGURE 5.20 5.8 Shear Strength 223 The applied load in this case will be the vertical beam reaction, so shear will occur along line AB and there will be tension along BC. The resistance factor f for each type of resistance is given by AISC in the Specification chapter dealing with that resistance, but in most cases, one of two values will be used: 0.90 for limit states involving yielding or compression buckling and 0.75 for limit states involving rupture (fracture). Steel Structures, Design and Behavior. As a consequence of the second assumption, the deformation of an individual fastener is Δ= r rmax Δ max = r rmax (0.34) where r = distance from the IC to the fastener rmax = distance to the farthest fastener Δ max = deformation of the farthest fastener at ultimate = 0.34 in. Shear in the Column Web The transfer of a large moment to a column can produce large shearing stresses in the column web within the boundaries of the connection; for example, region ABCD in Figure 8.46. The measure of this susceptibility is the width-to-thickness ratio of each cross-sectional element. 7.9 Combined Shear and Tension in Fasteners 437 EXAMPLE 7.11 A concentrically loaded connection is subjected to a service-load shear force of 50 kips and a service-load tensile force of 100 kips. From Table 6-1, with KL = L b = 15ft, p = 2.09 × 10 −3, and b x = 3.45 × 10 −3. If slip is permissible, how many bolts are required? Bearing. LRFD SOLUTION The factored load is Pu = 1.2D + 1.6L = 1.2(62.5) + 1.6(125) = 275 kips From the column load tables, the choices are as follows: W8: W10: W12: W14: There are no W8s with fc Pn ≥ 275 kips. Use the beam design charts in Part 3 of the Manual. In this book, we will use an initial trial depth of 1⁄10 to 1⁄12 of the span length. PL 3⁄ 8 × 7 FIGURE P3.2-1 3.2-2 A PL 1⁄2 × 8 tension member is connected with six 1-inch-diameter bolts, as shown in Figure P3.2-2. Eight 3⁄4-inch-diameter, Group A, fully tightened bearing-type bolts are used to attach the flange of the tee to the column flange. These distances now will be used to check the bearing strength of the 410 Chapter 7 Simple Connections connection. The composition of each is 33% dead load and 67% live load. As shown in Figure 10.21, this length is 12(5⁄16) = 3.75 in. This end condition can only be approximated in practice, so the discussion accompanying the alignment chart recommends that G be taken as 10.0. ANSWER From the alignment chart with GA = 0.95 and GB = 10.0, Kx = 1.85 for column BC. Is the strength of this system adequate? J = 4.06 in.4 Cw = 16,000 in.6 For a uniformly loaded, simply supported beam with lateral support at the ends, Cb = 1.14 (Fig. This design facilitates the connection of members meeting at a joint by permitting the use of a single gusset plate, as illustrated in Figure 3.33. 1.5″ 2.5″ 1.5″ 3″ 3″ 1.5″ 1.5″ FIGURE P7.3-2 Shear Strength 7.4-1 A C8 × 18.75 is to be used as a tension member. In some ASD solutions where there would be a lengthy duplication, the reader is referred to the LRFD solution for that portion. Verify the assumption on U to be sure that the strength of the member is governed by yielding. Use the AISC Specification equations. FIGURE 3.31 EXAMPLE 3.15 Fink trusses spaced at 20 feet on centers support W6 × 12 purlins, as shown in Figure 3.32a. The electrode is moved across the joint, and a weld bead is deposited, its size depending on the rate of travel of the electrode. SOLUTION Because of symmetry, the maximum moment is at midspan, so Mmax = M B = 1 wL2 8 Also because of symmetry, the moment at the quarter point equals the moment at the three-quarter point. < 13.5 kips冀in. 9.7 Composite Beams with Formed Steel Deck 625 FIGURE 9.16 The attachment of the deck to the beam can be considered to provide lateral support for the beam before the concrete has cured. FEMA-353, Washington, D.C. Fisher, J. In Example 3.8, all of the holes are in one element of the cross section, so this difficulty does not arise. First, we will use the basic electrode strength of 0.6FEXX for both the longitudinal and transverse welds (this corresponds to Rn冀Ω = 4.640 kips冀in.) The capacity of the weld across the end of the angle is P3 = 4.640(5) = 23.20 kips Summing moments about an axis along the bottom, we get ∑ ML 2 5 = 80.85(3.26) − 23.20 ⎛⎜ ⎞⎟ − P1 (5) = 0, P1 = 41.11 kips ⎝ 2⎠ ∑ F = 80.85 − 41.11 − 23.20 − P2 = 0, P2 = 16.54 kips 41.11 P1 = = 8.86 in. A spacing of 3 inches and edge distances of 21⁄2 inches will be used transverse to the load. If the threads are in the plane of shear, the reduction of the bolt area is accounted for by using 80% of the nominal bolt area. D., and Hajjar, J. ΣQn where As = cross-sectional area of steel shape Ac = area of concrete = tb (see Figure 9.7) ΣQn = total shear strength of the stud anchors Each possibility represents a horizontal shear force at the interface between the steel and the concrete. Although a length of weld can be loaded in any direction in shear, compression, or tension, a fillet weld is weakest in shear and is always assumed to fail in this mode. A further complication arises when electronic calculators are used. To determine the classification of the cross section, we first check the width-to-thickness ratios for the flange. Part 8. Figure 1.4 shows an idealized version of this stress–strain curve. Although other considerations, such as ease of construction, may ultimately affect the choice of member size, the process begins with the selection of the lightest cross-sectional shape that will do the job. Torque wrenches are used for this purpose. The connection must transfer the following service load effects: a dead-load moment of 35 ft-kips, a live-load moment of 105 ft-kips, a dead-load shear of 6.5 kips, and a live-load shear of 19.5 kips. The end rotation corresponding to a simple support and the actual loading is then plotted on the rotation axis (moment = 0). The plate is then considered to bend about an axis parallel to the beam span. All columns in real structures are subject to initial displacements that result from member out-of-plumbness. Foremost among these processes is submerged arc welding (SAW). A36 steel and 5⁄8-inchdiameter, Group A bolts will be used. The member shown will be connected between two plates with eight 7⁄8-in. If the maximum load reached before fracture was 14,700 lb, what is the ultimate tensile stress? As all elements of the cross section are connected, Ae = An = 11.73 in.2 For the net section, the nominal strength is Pn = Fu Ae = 58(11.73) = 680.3 kips For the gross section, Pn = Fy Ag = 36(14.7) = 529.2 kips LRFD SOLUTION The design strength based on fracture is ft Pn = 0.75(680.3) = 510 kips The design strength based on yielding is ft Pn = 0.90(529.2) = 476 kips Yielding of the gross section controls. All structural steel is A36 and the weld is a 3⁄16-inch fillet weld with E70 electrodes. What does this value represent? The allowable base metal shear yield strength is ⎛ 3⎞ 0.4 Fy t = 0.4(36) ⎜ ⎟ = 5.4 kips冀in. pmx = pmy = My ∑(x + y) Mx 2 2 ∑(x 2 + y 2) = 241.7(3.857) = 11.33 kips ← 82.29 = 241.7(1.714) = 5.034 kips ↓ 82.29 ∑ px = 11.33 kips ∑ py = 5.143 + 5.034 = 10.18 kips p = (11.33) 2 + (10.18) 2 = 15.23 kips The bolt allowable shear strength is Rn 32.47 = = 16.24 kips > 15.23 kips Ω 2.00 ANSWER (OK) The connection is satisfactory by elastic analysis. Compute the thickness needed to satisfy the net area requirement. From Table 6-1, p = 2.23 × 10−3, bx = 4.35 × 10−3, by = 8.38 × 10−3. The required size in sixteenths is, therefore, 6.756 8 1 = 7.3 sixteenths use in. 1.5-1 A 20-foot-long W8 × 67 is suspended from one end. 8.6 Moment-Resisting Connections 535 The flange plate will be designed as a tension connecting element (the top plate), then checked for compression (the bottom plate). • The tension bolt gage (g in Figure 8.51) should be no larger than the beam flange width. To ensure that the steel and concrete act as a unit, there must be a means of sharing the load between the two materials. The 28-day compressive strength of the concrete is fc′ = 4 ksi (Ec = 3492 ksi). Compute the plastic section modulus Z and the plastic moment Mp with respect to the major principal axis. The maximum factored load that can be applied if LRFD is used. For the same reasons, the connection to the support at the bottom may also be treated as a pin connection. EXAMPLE 8.13 Use LRFD and design a four-bolt end-plate connection for a W18 × 35 beam. Diagonal or x bracing is an example of relative bracing. KL > 25, the usual requirements for compression members in AISC E apply. The radius of gyration can be checked by inspection. LRFD SOLUTION Pu = 1.2D + 1.6L = 1.2(9) + 1.6(18) = 39.6 kips The shear strength of the weld per inch of length is 1.392D = 1.392(4) = 5.568 kips冀in. Problems 2″ 2″ 3″ 97 3″ 2″ 3″ 3″ 2″ FIGURE P3.4-2 3.4-3 An MC 9 × 23.9 is connected with 3⁄4-inch-diameter bolts as shown in Figure P3.4-3. If we assume that U = 0.80, An = Ag − Aholes = 4.75 − 2 (43 + 81)(12) = 3.875 in. A procedure for using the tables with Kx L, however, can be developed by examining how the tabular values were obtained. One full turn of a nut corresponds to a fixed length of travel along the bolt threads, which can be correlated to the elongation of the bolt. The live load deflection must not exceed L 冀360. There is a 3-inch-deep formed steel deck supporting 31⁄2 inches of lightweight concrete topping (wc = 115 pcf) for a total depth of 61⁄2 inches. In Figure 6.4a, the member is restrained against sidesway, and the maximum secondary moment is Pd, which is added to the maximum moment within the member. He received the 2011 AISC Special Achievement Award for his contributions to steel design education. SOLUTION Loads to be supported before the concrete cures are Slab: (3.5冀12)(150) = 43.75 psf Weight per linear foot: 43.75(10) = 437.5 lb冀ft Construction load: 20(10) = 200 lb冀ft (The beam weight will be accounted for later.) Loads to be supported after the concrete cures are wD = wslab = 437.5 lb冀ft wL = (100 + 20)(10) = 1200 lb冀ft where the 20 psf partition load is treated as a live load. E., and Malhas, F. The fundamental requirement of structural design is that the required strength not exceed the available strength; that is, Required strength ≤ available strength In allowable strength design (ASD), a member is selected that has cross-sectional properties such as area and moment of inertia that are large enough to prevent the maximum applied axial force, shear, or bending moment from exceeding an allowable, or permissible, value. Composite construction is covered in AISC Specification Chapter I, “Design of Composite Members.” Composite beams can take several forms. 10.7-2 A plate girder must be designed to resist a dead-load bending moment of 1320 ft-kips and a live-load bending moment of 2700 ft-kips. Fy = 50 ksi. At this point, an estimated girder weight can be computed, then Mn req and Af should be recomputed. At these locations, the cross-sectional area will be reduced by an amount equal to the area removed by the holes. 610 kips 3.3-7 a. The factored load is Pu = 1.2D + 1.6L = 1.2(6) + 1.6(18) = 36 kips and 36 = 8.62 in. The member in Example 3.11 is less than 8 inches wide and thus is classified as a bar rather than a plate. E and I were previously defined, and here the moment of inertia I is with respect to the axis of bending (buckling). Thornton, W. The frame is symmetrical, and the gravity loads are symmetrically placed. 1973. Maximum 4.71 KL KL 15 × 12 = = = 105.3 < 200 r ry 1.71 (OK) E 29, 000 = 4.71 = 118 Fy 46 Since 105.3 < 118, use AISC Equation E3-2. For flange local buckling, the relevant slenderness parameters are λ= bf 16 = =8 2t f 2(1.0) λ p = 0.38 E 29, 000 = 0.38 = 10.79 Fy 36 Since l < lp, there is no flange local buckling. Allowable strength per inch of weld = 0.9279D = 0.9279(4) = 3.712 kips冀in. When stage d has been reached, any further increase in the load will cause collapse, since all elements of the cross section have reached the yield plateau of the stress–strain curve and unrestricted plastic flow will occur. The effective length with respect to each axis is 14 feet, and A242 Grade 50 steel is used. For any cross-sectional shape, the section modulus will be a constant. J.: Prentice-Hall, Inc. Ideally, purlins are located at the truss joints so that the truss can be treated as a pin-connected structure loaded only at the joints. For other configurations, some equations may be different, and Design Guide 4 should be consulted. The AISC Specification requires that bolt holes in beam flanges be accounted for when the nominal tensile rupture strength (fracture strength) of the flange is less than the nominal tensile yield strength—that is, when Fu Afn < Fy Afg (5.9) where Afn = net tension flange area Afg = gross tension flange area If Fy 冀Fu > 0.8, the Specification requires that the right hand side of Equation 5.9 be increased by 10%. It is also unreasonable to record the results of every calculator multiplication or division to a predetermined number of significant figures in order to have a consistent degree of precision throughout. Summing forces in the direction of the bolt axis gives T=F+N FIGURE 7.25 418 Chapter 7 Simple Connections The application of force F will increase the bolt tension and cause it to elongate by an amount db. Mechanics of Materials. The probability of failure can then be written as ⎞ ⎛ ⎡ ⎡ ⎛ R⎞ ⎤ ⎤ ⎡ ⎛ R⎞ ⎤ PF = P ⎢ ln ⎜ ⎟ < 0 ⎥ = P ⎨ Us ln(R冀Q)+ ⎢ ln ⎜ ⎟ ⎥ ⎬ < 0 ⎜ ⎟ ⎠ ⎝ ⎠ Q Q ⎠ ⎝ ⎣ ⎦m ⎦ ⎣ ⎦ ⎣ ⎝ ⎣ ⎡ ⎡ ⎛ R⎞ ⎤ ⎡ ⎛ R⎞ ⎤ ⎤ ln ⎜ ⎝ Q ⎟⎠ ⎥ ⎥ ⎥ ⎥ ⎥ ln ⎜ ⎝ Q ⎟⎠ ⎥ ⎥ ⎥ ⎣ ⎦m ⎦m ⎣ = P ⎨U < − ⎬ = Fu ⎨ − ⎬ s ln(R冀Q) ⎥ ⎥ ⎥ s ln(R冀Q) ⎥ ⎥ ⎢ ⎥ ⎣ ⎦ ⎦ where Fu is the cumulative distribution function of U, or the probability that U will not exceed the argument of the function. Sometimes staggered fasteners are required by the geometry of a connection, such as the one shown in Figure 3.14b. SOLUTION Determine the compressive force C in the concrete (horizontal shear force at the interface between the concrete and steel). ∴ Use 9 in. Steel anchors must be present in the negative moment region (between the point of zero moment and the point of maximum negative moment). Designing Steel for Serviceability. 50.5 kips, Combination 3 e. 1): 11–20. If a live load is applied slowly and is not removed and reapplied an excessive number of times, the structure can be analyzed as if the load were static. Compute this as illustrated in Example 9.7. b. Maximum longitudinal spacing is 8t = 8(3.5) = 28 in. ST5): 155–180. Ω (The load is shared by two thicknesses of web, one on each side of the stiffener pair.) From Equation 7.38, the shear rupture strength is Rn = 0.3Fu t × 2 = 0.3(65)(0.360)(2) = 14.04 kips冀in. The design strength based on fracture is ft Pn = 0.75(293.8) = 220 kips The design strength based on yielding is ft Pn = 0.90(244.8) = 220 kips ANSWER Design strength = 220 kips. Bearing stiffeners will be used at the ends and at the load points, but do not proportion them. 260 kips 7.6-3 For both LRFD and ASD: a. This support system forms a cantilever structure that is resistant to horizontal displacements and also provides horizontal support for the other bays. If a compression member is supported differently with respect to each of its principal axes, the effective length will be different for the two directions. The tension member is of A572 Grade 50 steel and the gusset plate is A36. If a second-order analysis is not available, the moment amplification method, which is an acceptable direct analysis approach, can be used. The shear design strength per bolt is frn = 17.9 kips > 11 kips (OK) Compute the tensile force per bolt and then check the tension–shear interaction. (normal-weight concrete) Partition load = 20 psf Weight of ceiling = 5 psf Live load = 100 psf Fy = 50 ksi The maximum live load deflection cannot exceed L冀360. Although the connection shown in Figure 8.37a is simple in concept, its execution requires close tolerances. The layout shown in Figure 9.11 will be recommended. Therefore, fb Mn = 130 ft-kips Check the interaction formula. The required column flange thickness for LRFD is 1.11fM n f b FycYc t fc = where fMn = moment design strength based on bolt tension fb = 0.90 Fyc = yield stress of the column Yc = yield-line mechanism parameter for an unstiffened column flange = b fc 2 3c ⎞ ⎛ ⎛ 1⎞ ⎤ 2 ⎡ ⎛ ⎡ ⎛ 1⎞ ⎤ h1 ⎜ ⎝ s ⎟⎠ + h0 ⎜ ⎝ s ⎟⎠ ⎥ + g ⎢ h1 ⎜ ⎝ s + 4 ⎟⎠ + h0 ⎜ ⎝ s + ⎣ ⎣ c ⎞ c2 ⎤ g + ⎟+ 4 ⎥⎠ 2 ⎦ 2 8.8 End Plate Connections 571 bfc = column flange width 1 b fc g 2 c = p fo + t fb + Pfi s= For ASD, t fc = 1.11(M n 冀Ω) FycYc 冀Ω where Ω = 2.00 Ωb = 1.67 If the existing column flange is not thick enough, another column shape can be selected or stiffeners can be used. Pa = D + L = 140 + 420 = 560 kips Enter the column load tables with KL = 12 feet. The limit state of yielding will control, and Fcr = Fy, resulting in Mn = Rpg Fy Sxc (AISC Equation F5-1) If lp < l ≤ l r , the flange is noncompact. In the latter part of the eighteenth century and in the early nineteenth century, cast iron and wrought iron were used in various types of bridges. lateral-torsional buckling (LTB), either elastically or inelastically; 2. Only when a third hinge forms will a mechanism be created. Beam Bearing Plates and Column Base Plates 5.14-1 A W14 × 61 must support a concentrated service live load of 150 kips applied to the top flange. 120 k 120 k 2k/ft (includes girder wt.) 16′-0″ 16′-0″ 16′-0″ 48′-0″ FIGURE P10.7-4 10.7-5 Use LRFD and design the bearing stiffeners for the girder of Problem 10.7-4. Ω 2.00 2.00 (7.36) 7.11 Fillet Welds 451 Using this constant, the allowable shear strength of the 3⁄16-inch fillet weld in Example 7.12 is fRn = 0.9279 × 3 sixteenths = 2.784 kips冀in. Round HSS (AISC F8): a. 5′ 8 @ 6′-0″ = 48′-0″ FIGURE P6.9-1 6.9-2 Use A992 steel and select a structural tee shape for the top chord of the truss shown in Figure P6.9-2. Design 9.5-1 A fully composite floor system consists of 36-foot-long steel beams spaced at 6′-6″ center-to-center supporting a 41⁄2-inch-thick reinforced concrete floor slab. The dead and live loads are, therefore, wD = 0.350 lb冀ft (excluding the beam weight) wL = 0.560 + 0.140 = 0.700 lb冀ft LRFD SOLUTION The total factored load is wu = 1.2wD + 1.6wL = 1.2(0.350) + 1.6(0.700) = 1.540 kips冀ft The typical floor-beam connection will provide virtually no moment restraint, and the beams can be treated as simply supported. For ASD, the allowable strength is Rn冀Ω, where Ω = 1.67. Steel fabricators who furnish open-web steel joists must certify that a particular joist of a given designation, such as a 10K1 of span length 20 feet, will have a safe load capacity of at least the value given in the table. Part of the flange will therefore be in compression to augment the 602 Chapter 9 Composite Construction compressive force in the slab. The loads are service loads, and the uniform load includes the weight of the member. Having made this assumption, we can take the following steps to complete the design of an unshored floor system. Therefore, Mnx冀Ωb = 1.30(22.0) = 28.60 ft-kips From the uniform load tables, Mpx冀Ωb = 28.6 ft-kips ∴ Use Mnx冀Ωb = 28.6 ft-kips. FIGURE P8.8-2 8.8-3 Design a four-bolt unstiffened end-plate connection for a W18 × 40 beam to a W8 × 40 column. The slip-critical strength is frn = 9.49 kips冀bolt Bearing strength: There are two tables in the Manual available for bearing strength: Table 7-4 for strength based on bolt spacing and Table 7-5 for strength based on bolt edge distance. The ratio is closer to 3.069 = 0.6225 4.93 This corresponds to a required Pn冀Ωt (based on rupture), for purposes of using Table 5-2, of 0.75 (105) = 127 kips 0.6225 Using this as a guide, try L6 × 4 × 5⁄8, with Pn冀Ωt = 126 kips based on yielding of the gross section and Pn冀Ωt = 128 kips based on rupture of the net section. A major difference is that bending in beam bearing plates is in one direction, whereas column base plates are subjected to two-way bending. V. For a member braced against sidesway and transversely loaded, Cm can be taken as 1.0. A more accurate value can be found in the Commentary to AISC Appendix 8: ⎛ aP ⎞ Cm = 1 + Ψ ⎜ r ⎟ ⎝ Pe1 ⎠ (AISC Equation C-A-8-2) From Commentary Table C-A-8.1, Ψ = -0.2 for the support and loading conditions of this beam–column. An analysis consistent with the effective length method was performed; therefore, the flexural rigidity, EI, was unreduced. W., and Struik, J. The member is 26 feet long and pinned at each end. 46.6′ 12′ = 24′ 3′ 3′ 3′ 3′ 42′ 90′ FIGURE P6.9-2 12′ = 24′ 12′ lrafael / Shutterstock Structural steel members are connected either by welding, shown here, or bolting. The stress–strain relationship for the bolt material can then be used to compute the tension in the bolt. A good design requires the evaluation of several framing plans—that is, different arrangements of members and their connections. Check Equation 6.10 with p = 2.09 × 10−3 and bx = 3.04 × 10−3. Limiting values of width-to-thickness ratios are given in AISC B4.1, “Classification of Sections for Local Buckling.” For compression members, shapes are classified as slender or nonslender. = 0.6 for emid-ht < 2 in. The available strengths are computed by the methods of this chapter “Compression Members”, Chapter 5 “Beams”, and Chapter 6 “Beam–Columns”. The ASD tables use the same format, but the loads are unfactored. The strength for Cb = 1.36 is fb Mn = 1.36(335) = 456 ft-kips > fb Mp = 420 ft-kips ∴ fb Mn = fb Mp = 420 ft-kips > Mu = 406.1 ft-kips (OK) Check the beam weight. The publisher reserves the right to remove content from this title at any time if subsequent rights restrictions require it. 3. This is considered in Section 9.7 of this book, “Composite Beams with Formed Steel Deck.” For studs used as anchors in composite beams, the tensile strength Fu is 65 ksi (AWS, 2008). They are arranged in groups, with the lightest shape in each group at the top of that group. A method for handling this case is illustrated in Figure 3.19. If the beam and angles are considered separately from the column, as shown in Figure 8.1b, it is clear that the reaction R acts at an eccentricity e from the centroid of the areas of the fasteners in the beam web. Manual Tables 7-1 through 7-5 will be used in these examples. The stress block depth will generally be very small; consequently, an error in the estimate of a冀2 will have only a slight effect on the estimated value of As. An assumed value of a冀2 = 0.5 inch is suggested. The two most common types of welds are the fillet weld and the groove weld. No shoring is used, and there is a construction load of 20 psf. In the present case, 1.40 E 29, 000 = 1.40 = 51.8 f 21.16 Since 66 > 51.8, AISC Equation E7-18 does apply. W., Griffis, L. LRFD SOLUTION The factored load is 1.2D + 1.6L = 1.2(18) + 1.6(54) = 108.0 kips 7.7 Design Examples 405 FIGURE 7.15 We will select the number of bolts based on shear and verify that the bearing strength is adequate once a final bolt layout has been determined. The effective length with respect to each axis is 16 feet. The specimen diameter was 0.5 inch and the gage length was 8 inches. What conclusions can you draw from this table? For ASD, the analysis results for the controlling load combination are Pnt = 200 kips, Mntx = 90 ft-kips, and Mnty = 20 ft-kips. 2 2 Enter the tables with ΣQn = 530 kips and Y2 = 4.104 inches. Determine the total number of 3⁄4-inch × 31⁄2-inch shear studs required. Select Y 2. Flexural strength b. Equate the design strength to the factored load moment and solve for As: fb M n = f b As Fy y = Mu As = Mu fb Fy y or As = FIGURE 9.12 Mu d a fb Fy ⎛ + t − ⎞ ⎝ 2 2⎠ (9.3) 9.5 Design 615 Equation 9.3 can also be written in terms of weight rather than area. KCS joists are designed to support both concentrated loads and distributed loads (including nonuniform distributions). A continuous connection is not required. Muy ⎞ Pu 8 ⎛ Mux 8 189.8 ⎞ + ⎜ + = 0.4113 + ⎛⎜ ⎟ = 0.885 < 1.0 ⎟ φc Pn 9 ⎝ φ b M nx φb M ny ⎠ 9 ⎝ 356 ⎠ ANSWER ASD SOLUTION (OK) This member satisfies the AISC Specification requirements. Tees Loaded in the Plane of Symmetry (AISC F9): a. Table 5.3 also applies to channels, except that l for the flange is bf冀tf . 0 - 0 5 1 0 - An = 6.80 - 0.5(1.0) - 0.5 ⎟ 1.0 - - . When the compression at the top has been completely overcome, the components will separate, and the couple Pe will be resisted by tensile bolt forces and compression on the remaining surface of contact, as shown in Figure 8.13b. The following step-by-step procedure is recommended. ASD load combination 2 controls. Consequently, we devote the remainder of this chapter to unshored composite construction. K., Cornell, C. The additional strength gained from considering composite action for negative moment is relatively small. Beams that do not satisfy these criteria are considered in Section 5.15. Finally, the various components must be connected by properly designed welds. What is the maximum tensile stress? for larger bolts. These hole locations are given in Table 1-7A in Part 1 of the Manual. SOLUTION The slab weight is wslab = t 4 (150) = (150) = 50 psf 12 12 Assume that each beam supports a 7-ft width (tributary width) of floor. In these cases, the load is not resisted equally by each fastener or each segment of weld, and the determination of the distribution of the load is the complicating factor in the design of this type of connection. Approximately 10,000 ksi d. SOLUTION The factored load to be resisted is Pu = 1.2D + 1.6L = 1.2(8) + 1.6(24) = 48.0 kips Because the bolt size and layout will affect the net area of the tension member, we will begin with selection of the bolts. He holds a B.S.C.E, M.S., and Ph.D. from the University of South Carolina. For width-to-thickness ratio limits for welded shapes, refer to Table B4.1b in Chapter B of the AISC Specification, “Design Requirements.” Shear Strength 5.8-1 Compute the nominal shear strength of an S24 × 121 of A572 Grade 65 steel. The results of all searches are summarized in the following tabulation. From Equation 7.35, the base-metal shear yield strength per unit length is ⎛ 9⎞ φ Rn = 0.6 Fy t = 0.6(36) ⎜ ⎟ = 12.2 kips冀in. ∑A 11 A y Ay 8 6.5 52 Web 3 3.0 Sum 11 Component Flange 9 61 FIGURE 5.7 Fy –y Centroid of half-area –y x –y Fy Figure 5.7 shows that the moment arm of the internal resisting couple is a = 2¯y = 2(5.545) = 11.09 in. The tabulated value of Lp is the value of unbraced length at which the nominal strength based on inelastic lateral-torsional buckling equals the nominal strength based on flange local buckling, that is, the maximum unbraced length for which the nominal strength can be taken as the strength based on flange local buckling. Bending is about the strong axis, and Kx = Ky = 1.0. Lateral support is provided only at the ends. AISC Table B4.1a shows the upper limit, lr, for nonslender members of various cross-sectional shapes. The member length is 15 feet. Assume that Ae = An and compute the following. Column HI, Pu = 480 kips. 3): 107–116. Although the engineer is encouraged to investigate the actual loading conditions and attempt to determine realistic values, the structure must be able to support these specified minimum loads. 122 kips 738 Answers to Selected Problems 10.5-1 a. Does the connection have enough capacity based on shear and bearing? Shear provisions for all flexural members are covered in AISC Chapter G, “Design of Members for Shear.” Other requirements are given in AISC F13, “Proportions of Beams and Girders.” A plate girder cross section can take several forms. They are employed when large loads or span lengths preclude the use of a standard hot-rolled steel shape. 69.3 kips, Combination 3. Use LRFD and the tables in Part 7 of the Manual to determine the following. FIGURE 3.23 3.6 SOLUTION Design of Tension Members 67 The shear areas are Agv = 3 (7.5) = 2.813 in.2 8 and, since there are 2.5 hole diameters, Anv = 3⎡ 7 1 ⎤ 7.5 − 2.5⎛ + ⎞ ⎥ = 1.875 in.2 ⎢ ⎝ 8⎣ 8 8⎠ ⎦ The tension area is Ant = 3⎡ 7 1 ⎤ 1.5 − 0.5⎛ + ⎞ ⎥ = 0.3750 in.2 ⎢ ⎝ 8⎣ 8 8⎠ ⎦ (The factor of 0.5 is used because there is one-half of a hole diameter in the tension section.) Since the block shear will occur in an angle, Ubs = 1.0, and from AISC Equation J4-5, Rn = 0.6Fu Anv + UbsFu Ant = 0.6(58)(1.875) + 1.0(58)(0.3750) = 87.00 kips with an upper limit of 0.6 Fy Agv + Ubs Fu Ant = 0.6(36)(2.813) + 1.0(58)(0.3750) = 82.51 kips The nominal block shear strength is therefore 82.51 kips. Equate the allowable strength to the service load moment and solve for As: M n As Fy y = = Ma Ωb Ωb Ωb M a As = Fy y or As = Ωb M a d a Fy ⎛ + t − ⎞ ⎝ 2 2⎠ (9.5) Using w= As (490) = 3.4 As lb冀ft (for As in square inches) 144 we obtain from Equation 9.5 the estimated weight per foot: w= 3.4 Ωb M a lb冀ft d a⎞ ⎛ Fy +t− ⎝ 2 2⎠ (9.6) where Ma is in in.-kips; Fy is in ksi; and d, t, and a are in inches. In positive moment regions, the concrete will be in compression continuously and is subject to a phenomenon known as creep. Beginning with the smallest shapes, the first one found that will work is a W8 × 58 with an allowable strength of 422 kips. ANSWER Use a PL 1 × 7 × 2′ − 0 3⁄4″ and four 7⁄8-inch diameter Group A fully tightened bolts at each flange. and Johnston, Bruce G. 9.3-2 A fully composite floor system consists of W18 × 40 floor beams of A992 steel supporting a 5-inch-thick reinforced concrete floor slab, with f c′ = 4 ksi. Because shear lag affects both bolted and welded connections, the effective net area concept applies to both types of connections. The bolts are 7⁄8-inch in diameter. For parts 1⁄4 inch or more in thickness, the maximum size is t - 1⁄16 inch, where t is the thickness of the part. They are presented in a slightly different form as follows: Combination 1: Combination 2: Combination 3: Combination 4: Combination 5: Combinations 6 and 7: 1.4D 1.2D + 1.6L + 0.5(Lr or S or R) 1.2D + 1.6(Lr or S or R) + (0.5L or 0.5W) 1.2D +1.0W + 0.5L + 0.5(Lr or S or R) 1.2D ± 1.0E + 0.5L + 0.2S 0.9D ± (1.0W or 1.0E) Here, the load factor on L in combinations 3, 4, and 5 is given as 0.5, which should be increased to 1.0 if L is greater than 100 pounds per square foot or for garages or places of public assembly. Bolt tension: 21.9 kips < 29.8 kips; shear: 9.38 kips < 17.9 kips; bearing: 9.38 kips < 38.2 kips b. A.3 Analysis 725 FIGURE A.5 To investigate the mechanism in span AB, impose a virtual rotation q at A. ANSI/AISC 360-10, Chicago. 85.1 kips b. Cengage Learning products are represented in Canada by Nelson Education Ltd. IV. The maximum permissible live load deflection is L 30 × 12 = = 1.0 in. For shapes not in the column load tables, a trial-and-error approach must be used. Compression member strength depends on both cross-sectional area and member slenderness. (Combination 4 could not control unless B2, the amplification factor for M t , were unrealistically large.) For purposes of selecting a trial shape, assume that B1 = 1.0. The value of B2 can be computed using the design drift index and a value of Pe story from AISC Equation A-8-7: Pe story = RM HL H ⎛ 2.7 ⎞ = RM = 0.85 ⎜ = 918.0 kips ⎝ 1冀400 ⎟⎠ ΔH Δ H 冀L (The unfactored horizontal load, H = 2.7 kips, is used because the drift index is based on the maximum drift caused by service loads.) B2 = 1 1 ≈ = 1.068 1.0(58.4) α Pstory 1− 1− 918.0 Pe story The amplified axial load is Pr = Pu = Pnt + B2Pt = 58.4 + 0 = 58.4 kips and the amplified bending moment is Mu = B1Mnt + B2 Mt = 1.0(210.4) + 0 = 210.4 ft-kips Without knowing the frame member sizes, the alignment chart for the effective length factor cannot be used. The connection will be to a 3⁄8-inch-thick gusset plate, as shown in Figure 7.19. Note that L p = 1.76ry E Fy could still be used for noncompact shapes. Ω 2.00 b. 8 482 Chapter 8 Eccentric Connections FIGURE 8.6 The horizontal and vertical components of the load are Px = 1 2 (50) = 22.36 kips ← and Py = (50) = 44.72 kipss ↓ 5 5 Referring to Figure 8.6a, we can compute the moment of the load about the centroid: M = 44.72(12 + 2.75) – 22.36(14 – 6) = 480.7 in.-kips (clockwise) Figure 8.6b shows the directions of all component bolt forces and the relative magnitudes of the components caused by the couple. An assumed mechanism is subjected to virtual displacements consistent with the possible mechanism motion, and the external work is equated to the internal work. The individual members of the frame can be classified according to the type of behavior represented by this deformed shape. 4 3 1⁄2″ 1 3 @ 3″ = 9″ 11⁄2″ 10″ FIGURE P8.2-1 20k Problems 8.2-2 573 The bolt group shown in Figure P8.2-2 consists of 3⁄4-inch-diameter, Group A slipcritical bolts in single shear. ΔL = 5 wL L4 5 (4.5冀12)(30 × 12)4 = = 1.19 in. These requirements are based on experience and test programs for forged eyebars, but they are conservative when applied to eyebars thermally cut from plates (the present fabrication method). Three tables are presented: strengths of various combinations of shapes and slabs; tables of lower-bound moments of inertia; and a table of steel headed stud anchor strength Qn for various combinations of stud size, concrete strength, and deck geometry. Ω From Equation 7.38, the shear rupture strength is Rn = 0.3Fu t × 2 = 0.3(65)(0.360)(2) = 14.04 kips冀in. 104 Chapter 3 3.7-2 Tension Members A W16 × 36 is supported by two tension rods AB and CD, as shown in Figure P3.7-2. Try an 8-inch transverse weld and two 7-inch longitudinal welds. Kulak and Timler. The load to be resisted is Pa = D + L = 6 + 18 = 24 kips and 24 = 8.62 in. The interval is a function of the truss spacing, the slope of the top chord, the resistance of the purlin 80 Chapter 3 Tension Members FIGURE 3.28 FIGURE 3.29 to this type of bending (most shapes used for purlins are very weak in this respect), and the amount of support furnished by the roofing. Other factors, such as welding and cold-bending to create curvature in a beam, can contribute to the residual stress, but the cooling process is its chief source. In fact, materials account for a relatively small portion of the cost of a typical steel structure as compared with labor and other costs (Ruby and Matuska, 2009). 2.784 ⎛ 3⎞ Minimum length = 4 w = 4 ⎜ ⎟ = 0.75 in. Enter the table with the effective length and move horizontally until you find the desired available strength (or something slightly larger). From the dimensions and properties tables, 362 Chapter 6 Beam–Columns l= bf = 6.31 2t f l p = 0.38 29, 000 E = 0.38 = 9.152 50 Fy l < l p ∴ The flange is compact. (versus 0.455 inch required in Example 8.5) 16 ANSWER Use a 1⁄4-inch fillet weld, E70 electrode. If a stiffener is placed at the end of the girder, what is the required distance to the next stiffener? ⎞ 2⎠ Ω From Equation 7.38, the shear rupture strength is Rn ⎛ 1⎞ = 0.3Fu t = 0.3(58) ⎜ ⎟ = 8.70 kips冀in. V., Ravindra, M. 402 Chapter 7 Simple Connections In Example 7.5, the tensile strength was not required. From Equation 6.9, pPa + bx Max + by May = (3.19 × 10−3)(201) + (5.97 × 10−3)(67) + 0 = 1.04 > 1.0 (N.G.) Try a W12 shape. The end shear consists of 4 kips of dead load and 18 kips of live load. As strength is the property of primary concern to the design engineer, the last two digits are usually represented by XX, and a typical designation would be E70XX or just E70, indicating an electrode with an ultimate tensile strength of 70 ksi. When the first possibility controls, the steel is being fully utilized, and the stress distribution of Figure 9.7a applies. If the shape has no slender unstiffened elements, Qs = 1.0. If the shape has no slender stiffened elements, Qa = 1.0. Many of the shapes commonly used as columns are not slender, and the reduction will not be needed. Also from Example 7.12, the base-metal shear design strength is fRn = 5.4 kips冀in. fillet welds for intermediate stiffeners, spaced as shown in Figure 10.24. Shear areas: 3 Agv = 2 × (2.5 + 2.5 + 1.25) = 4.688 in.2 8 3 Anv = 2 × [6.25 − 2.55(1.0)] = 2.813 in.2 8 Tension area: Ant = 3 (3 − 1.0) = 0.7500 in .2 8 For this type of block shear, Ubs = 1.0. From AISC Equation J4-5, Rn = 0.6Fu Anv +U bs Fu Ant = 0.6(58)(2.813) + 1.0(558)(0.75) = 141.4 kips with an upper limit of 0.6Fy Agv + Ubs Fu Ant = 0.6(36)(4.688) + 1.0(58)(0.75) = 144.8 kips The nominal block shear strength is therefore 141.4 kips, and the allowable strength is Rn 141.4 = = 70.7 kips < 72 kips Ω 2.00 (N.G.) The simplest way to increase the block shear strength for this connection is to increase the shear areas by increasing the bolt spacing or the edge distance; we will 7.7 Design Examples 411 increase the spacing. Because of this, the ASD solution presented here will make use of most of the LRFD calculations, which will not be repeated. A certain number of anchors will be required to make a beam fully composite. If the live load is twice the dead load, what is the maximum total service load, in kips冀ft, that can be supported? The joist girder is designated by specifying its depth, the number of joist spaces, the load at each loaded top-chord panel point of the joist girder, and a letter to indicate whether the load is factored (“F”) or unfactored (“K”). Compute kv and Cv. 3. – ⎢ ⎥ 4(3) ⎥⎦ 4(2.5) ⎥⎦ 4(4.75) ⎥⎦ ⎣ ⎣ ⎣ = 5.065 in.2 The last case controls; use An = 5.065 in.2 Both legs of the angle are connected, so Ae = An = 5.065 in.2 The nominal strength based on fracture is Pn = Fu Ae = 58(5.065) = 293.8 kips The nominal strength based on yielding is Pn = FyAg = 36(6.80) = 244.8 kips a. g = 3.50 in. Whether that occurs will depend on h冀tw, the width-to-thickness ratio of the FIGURE 5.18 218 Chapter 5 Beams web. 2″ 21⁄2″ 7⁄8-in.-diameter 21⁄2″ 21⁄2″ C12 × 20.7 bolts FIGURE P3.3-6 3.3-7 A double-angle tension member, 2L4 × 3 × 1⁄4 LLBB, is connected with welds as shown in Figure P3.3-7. Assume that the slab provides continuous lateral support of the floor beams. Mn = My = Fy Sx = 50(3.23) = 161.5 in.-kips Check lateral-torsional buckling. Although there is no specification requirement, a common practice is to use a minimum diameter of 5⁄8 inch. From the LRFD solution, the reduced nominal moment strength is M n = 4498 in.-kips M n 4498 = = 2693 in.-kips = 224 ft-kips > 140 ft-kips Ω 1.67 ANSWER 8.7 (OK) Use the connection shown in Figure 8.42 (column stiffener requirements are considered in Section 8.7). After some simple modifications, the corresponding ASD equation will be given. The required strength is computed at the factored load level, and the 1.6 factor is used to adjust the ASD service load level to a factored load level. The design of complete penetration groove welds is a trivial exercise in that the weld will have the same strength as the base metal and the connected parts can be treated as completely continuous at the joint. Although the force will be different in each segment of rod, the usual practice is to use one size throughout. In addition, you must still refer to the dimensions and properties tables to find the radius of gyration. According to Section G2.2 of the Specification, the length should be within limits established by the distance between the weld connecting the stiffener to the web and the weld connecting the web to the tension flange. Since this is greater than 0.8, use Yt = 1.1. Yt Fy Afg = 1.1(50)(6.188) = 340.3 kips Since Fu Afn < Yt Fy Afg, the holes must be accounted for. The beams are located at the one-third points. AISC J3.6 requires that prying action be included in the computation of tensile loads applied to fasteners. (The upper limit of controls.) b. If the connected parts are sufficiently rigid, however, this shifting of forces will not occur, and there will be no prying action. ASCE 7 combinations 6 and 7 arise from the expression shown by considering combination 6 to use 1.0W and combination 7 to use 1.0E. Fe = p 2E (KL冀r)2 Fcr = 0.658 = (Fy冀Fe) p 2 (29, 000) (96.77)2 = 30.56 ksi Fy = 0.658(50 冀30.56) (50) = 25.21 ksi (OK) 4.4 Local Stability 121 The nominal strength is Pn = Fcr Ag = 25.21(21.8) = 549.6 kips LRFD SOLUTION The design strength is ASD SOLUTION From Equation 4.7, the allowable stress is fc Pn = 0.90(549.6) = 495 kips Fa = 0.6Fcr = 0.6(25.21) = 15.13 ksi The allowable strength is Fa Ag = 15.13(21.8) = 330 kips ANSWER Design compressive strength = 495 kips. 474 Chapter 7 7.11-7 Simple Connections Design a welded connection for an MC 9 × 23.9 of A572 Grade 50 steel connected to a 3⁄8-inch-thick gusset plate. Use A36 steel and E70XX electrodes and assume that the girder has continuous lateral support. The maximum moment in a beam–column therefore depends on the distribution of bending moment within the member. Pr = Pa = D + L = 100 + 200 = 300 kips Py = Fy Ag = 50(15.8) = 790 kips 4.7 More on Effective Length 153 From AISC Equation C2-2b, 1.6(300) ⎞ ⎛ 1.6(300) ⎞ ⎛ α Pr ⎞ ⎛ α Pr ⎞ τb = 4 ⎜ 1− = 4⎛ 1− = 0.954 ⎟ ⎟ ⎝ 790 ⎠ ⎝ ⎝ Py ⎠ ⎝ Py ⎠ 790 ⎠ ANSWER τb = 0.954. ⎞ 1 ⎛ π 2 ECw Fez = ⎜ + GJ ⎟ ⎠ Ag r 02 ⎝ (K z L)2 H = 1− x 02 + y02 r 20 (AISC Equation E4-9) (AISC Equation E4-10) where z is the longitudinal axis and x0, y0 are the coordinates of the shear center of the cross section with respect to the centroid (in inches). M., Colaco, J. 6⬙ 16⬙ 10⬙ y-axis x-axis FIGURE P4.7-2 4.7-3 A W12 × 65 of A572 Grade 60 steel is used as a compression member. Use an elastic analysis and compute the maximum bolt shear force. Compute Fcr and the strength for the trial shape. 9.10 COMPOSITE COLUMNS Composite columns can take one of two forms: a rolled steel shape encased in concrete with both longitudinal reinforcing bars and transverse reinforcement in the form of ties or spirals (as in a reinforced concrete column) or a hollow shape filled with plain concrete. AISC J10.8 requires that full-depth stiffeners be used in pairs and analyzed as axially loaded columns subject to the following guidelines: • The cross section of the axially loaded member consists of the stiffener plates and a length of the web (see Figure 10.4). • A majority of the problems have been revised. This design is summarized in Figure 8.54. Lightweight concrete with a 28-day compressive strength of 4 ksi is used (unit weight = 115 psf). (In Example 4.1, the stress at buckling is Pcr 冀A = 278.9冀14.6 = 19.10 ksi, which is well below the proportional limit for any grade of structural steel.) This difficulty was initially resolved by Friedrich Engesser, who proposed in 1889 the

use of a variable tangent modulus, Et , in Equation 4.3. For a material with a stress–strain curve like the one shown in Figure 4.5, E is not a constant for stresses greater than the proportional limit Fpl. This placement will give a strength that approximately equals the maximum shear of 156 kips. Part of the beam flange area will be lost because of the bolt holes. The detailing value is the largest.) 1.49 ANSWER 29, 000 E = 1.49 = 35.9 > 25.4 50 Fy (OK) Local instability is not a problem. ∂ 3-in. This load is superimposed on the beam; it does not include the weight of the beam itself. To do so requires the structural analysis of the building frames and the computation of forces and bending moments in the individual members. The 30-kip load is a service live load. Do not consider eccentricity. Two values of K are given: a theoretical value and a recommended design value to be used when the ideal end condition is approximated. This can be accomplished by summing the moments of the resultants about any convenient point. 6.8 6.8 Design of Beam–Columns 339 DESIGN OF BEAM–COLUMNS Because of the many variables in the interaction formulas, the design of beam– columns is essentially a trial-and-error process. The usual designation for both is the abbreviation PL (for plate, even though it could actually be a bar) followed by the thickness in inches, the width 3 1 in inches, and the length in feet and inches; for example, PL⁄8 × 5 × 3′-2⁄2″. (This number also satisfies Equation 4.14.) ANSWER Use 2L 31⁄2 × 3 × 1⁄4 SLBB with three intermediate connectors within the 14-foot length. For example, if there is bending about only one axis, we can write for that axis required moment strength ≤ available moment strength or required moment strength ≤ 1.0 available moment strrength If there is bending about both the x and y axes, the interaction approach requires that the sum of ratios for the two effects be less than 1.0; that is, required y-axis moment strength required x - axis moment strength + ≤ 1.0 available x -axis moment strength available y-axis moment strength (5.21) In effect, this approach allows the designer to allocate to one direction what has not been "used up" in the other direction. The bolts are 7⁄8-inch-diameter Group A, type N, and eight bolts are used in each flange. If this ratio is too large—that is, if the web is too slender—the web can buckle in shear, either inelastically or elastically. For the stiffener, the shear yield strength is fRn = 0.6Fytst × 2 stiffeners = 0.6(36)(3鼻4)(2) = 32.4 kips鼻in. (See the related discussion in Section 4.2, "Column Theory.") For convenience, we first categorize beams as compact, noncompact, or slender, and then determine the moment resistance based on the degree of lateral support. The weight of the roofing materials is 16 psf, and the snow load is 20 psf of horizontal projection of the roof surface. This is for the purpose of resisting uplift. Tables for the Design of Tension Members Part 5 of the Manual contains tables to assist in the design of tension members of various cross-sectional shapes, including Table 5-2 for angles. The stress Fe can be defined as the elastic buckling stress corresponding to the controlling mode of failure, whether flexural, torsional, or flexural-torsional. In the above equations, the z-axis is the longitudinal axis. Use an elastic analysis and assume that the bearing strengths of the connected parts are adequate. Vol. The axial stiffness of the brace can be written as F Prb 鼻cos θ Prb 1 = = • δ Δ cos θ Δ cos 2θ or F Prb cos 2θ = Δ δ That is, Axial stiffness × cos2q = lateral stiffness From elementary mechanics of materials, the axial stiffness is AE鼻L, where L = length of brace = (18 × 12)鼻sin(21.80) = 581.6 in. Compute the available strength for ASD. Salmon, et al. From the column load tables, for KL = 10 ft, Pn = 238 kips Ωc 322 Chapter 6 Beam–Columns Determine which interaction formula to use: 28 Pa = = 0.1176 < 0.2 Pn 鼻Ωc 238 ∴ Use Equation 6.6 (AISC C Equation H1-1b). Pe1x = π 2 EI x* π 2 (0.8 EI x) π 2 (0.8)(29, 000)(248) = = = 1753 kips (K x L)2 (K x L)2 (15 × 12)2 348 Chapter 6 Beam–Columns B1x = Pe1 y = B1 y = Cmx 0.6 = = 0.656 < 1.0 Pu 150 1− 1− Pe1x 1753 ∴ B1x = 1.0 as assumed. Lateral support is provided at A, B, and C. Use an elastic analysis and assume that the bearing strengths of the bracket and the column flange are adequate. Most composite construction is unshored, however, because the additional cost of the shores, especially the labor cost, outweighs the small savings in steel weight that may result. 5.5k/ft 6′ 18′ 6′ FIGURE P5.5-4 5.5-5 The beam shown in Figure P5.5-5 is a two-span beam with a pin (hinge) in the center of the left span, making the beam statically determinate. CL Col. Cooper, P. We consider only compact shapes in this chapter. It is important to recognize, 109 110 Chapter 4 Compression Members however, that these three methods are used to determine the required strengths of the members (axial loads and bending moments). LRFD SOLUTION Pu = 1.2 D + 1.6 L = 1.2(9) + 1.6(27) = 54 kips M = Pe = 54(6.714) = 362.6 in.-kips (clockwise) 54 pcy = = 7.714 kips ↓ pcx = 0 7 From the directions and relative magnitudes shown in Figure 8.9, the lower right bolt is judged to be critical, so 362.6(3.857) My = 17.00 kips ← 2 2 = 82.29 ∑(x + y) 362.6(1.714) Mx = = 7.553 kips ↓ 2 2 = 82.29 ∑(x + y) pmx = pmy 488 Chapter 8 Eccentric Connections ∑ px = 17.00 kips ∑ py = 7.714 + 7.553 = 15.27 kips p = (17.00)2 + (15.27)2 = 22.9 kips The bolt design shear strength is fRn = 0.75(32.47) = 24.4 kips > 22.9 kips (OK) A N S W E R The connection is satisfactory by elastic analysis. In Part 4 of the Manual, "Design of Compression Members," the column load tables contain constants that expedite the evaluation of the need for stiffeners. The two concentrated loads are service live loads. Tables for Analysis and Design Part 4 of the Manual, "Design of Compression Members," contains tables that give the compressive strength of concrete-filled HSS (rectangular, square, and round) and steel pipe. It must resist a service dead load of 20 kips, a service live load of 60 kips, and a service wind load of 20 kips. Use A36 steel and fc′= 3 ksi. Connect GA and GB with a straight line, and read the value of K on the middle scale. CHAPTER 8 Eccentric Connections 8.1 EXAMPLES OF ECCENTRIC CONNECTIONS An eccentric connection is one in which the resultant of the applied loads does not pass through the center of gravity of the fasteners or welds. Detailing for Steel Construction. From AISC J4.4, for compression elements with KL鼻r < 25, the nominal strength is Pn = Fy Ag and for LRFD, f = 0.90 fPn = 0.90(36)Ag = 32.4Ag 8.7 Column Stiffeners and Other Reinforcement 551 Equating this strength to the required strength, we obtain the required area of stiffener: 32.4 Ag = 130.3 Ag = 4.02 in.2 Try two stiffeners, 3⁄4 × 3, one on each side of the web: ⎛ 3⎞ Ast = 2(3)⎜ ⎟ = 4.50 in.2 > 4.02 in.2 required ⎝ 4⎠ (OK) Check for tst ≥ b鼻16: b 3 3 = = 0.188 in. To obtain the critical stress for elastic columns, the Euler stress is reduced as follows to account for the effects of initial crookedness: Fcr = 0.877Fe (4.8) For inelastic columns, the tangent modulus equation, Equation 4.6b, is replaced by the exponential equation Fy ⎛ ⎞ Fcr = ⎜ 0.658 Fe ⎟ Fy ⎝ ⎠ (4.9) With Equation 4.9, a direct solution for inelastic columns can be obtained, avoiding the trial-and-error approach inherent in the use of the tangent modulus equation. For rx 鼻ry = 2.44 (the approximate ratio for all likely possibilities), Kx L 20 = = 8.197 ft > K y L = 8 ft rx 鼻ry 2.44 For KL = 9 ft, a W14 × 74, with a capacity of 854 kips, is the lightest W14-shape. Chapter 8 Eccentric Connections 2″ 8″ 60k 47k 4 sp @ 31⁄2″ 580 WT12 × 31 2″ W12 × 72 FIGURE P8.3-8 8.3-9 For the following designs, use A36 steel for the angles and A992 steel for the beam and column. C-shapes are different only in that the width-to-thickness ratio of the flange is bf鼻tf rather than bf鼻2tf. Bolts have enough shear strength; 180 kips < 195 kips b. Referring to Figure A.4d, we see that M2 = + Mp and M1 = − Mp . Kx L 18 = = 10.53 ft > 9 ft rx 鼻ry 1.71 The interpolated strength is Pn = 444 kips > 400 kips Ωc (OK) Check the W12 shapes. Check the strong axis: Kx L 18 = = 10.34 ft > 9 ft rx 鼻ry 1.74 ∴ K x L controls for this shape. This expression reduces to 1.2D + 0.5L + 0.5S, and by inspection, we can see that it produces a smaller result than combination 3. From Equations 7.9 and 7.10, we can write Tb¢ – Ma–a = Mb–b where b′ has been substituted for b. Consequently, the resultant resisting force provided by the fasteners also acts along this line, and the geometry is consistent with the definition of a simple connection. That can be accomplished if the horizontal shear at the interface is resisted by connecting devices known as anchors (sometimes called shear connectors). Use LRFD and select the lightest W10 shape of A992 steel. 1994. Lateral support of the compression flange is provided only at the ends. EXAMPLE 3.6 Compute the smallest net area for the plate shown in Figure 3.15. This places the bolts at the "workable 536 Chapter 8 Eccentric Connections gage" location in the beam flange (see Part 1 of the Manual). Use b = 90 inches as the effective flange width. As E = 0, this expression reduces to 1.2D + 0.5L + 0.2S, which produces a smaller result than combination 4. By interpolation, f b Mn = 477 ft-kips which compares favorably with the results of Example 9.2 but involves about the same amount of effort. of lightweight concrete fill (wc = 115 pcf) for a total depth of t = 5 in. Compression members with any type of cross-sectional configuration can fail in this way. For the gage distance, use the workable gage distance given in Part 1 of the Manual. controls. From Example 7.12, the allowable weld shear strength is 2.784 kips鼻in., and the allowable base metal shear strength is 3.6 kips鼻in. Satisfactory: Result of interaction equation = 0.831 6.6-7 a. This calls for a maximum clear spacing of d ≤ 0.75 E t f , but no greater than 12 in. Problems 4″ 4″ 4″ 99 4″ 21⁄2″ 21⁄2″ 3⁄8″ FIGURE P3.4-6 Block Shear 3.5-1 The tension member is a PL3⁄8 × 51⁄2 of A242 steel. Shop welding is less expensive and can be more closely controlled. Example 8.10 illustrates the design of a three-plate moment connection, including the requirements for connecting elements, which are covered by AISC J4. If this process proceeds unabated, collapse can occur. This is less than the shear strength of two welds (using two welds for each plate), so use a weld strength of 5.400 kips鼻in. fc Pn = 1170 kips, Pn鼻Ωc = 777 kips b. Except for such specialized steel products as cold-formed steel, which is covered by a different specification (AISI, 2007), the AISC Specification is the standard by which virtually all structural steel buildings in this country are designed and constructed. Substituting into the differential equation, we get − p2 2 L B sin px P px Pe px + B sin =− sin L EI L EI L Solving for the constant gives Pe −e EI = B= = 2 2 P p p EI Pe – 1 1− – P EI L2 PL2 – where p 2 EI = the Euler buckling load L2 px ⎡ e px ⎤ ∴ y = B sin = ⎢ sin L L ⎣ (Pe 鼻P) – 1 ⎦⎥ M = P(y0 + y) Pe = px⎤ e px ⎡ ⎤ ⎡ sin ⎬ = P ⎨e sin + ⎢ ⎥ L ⎣ (Pe 鼻P) − 1 ⎦ L⎦ ⎣ 6.4 The Moment Amplification Method 309 The maximum moment occurs at x = L鼻2: e ⎡ ⎤ M max = P ⎢e + (Pe 鼻P) − 1 ⎥⎦ ⎣ ⎡ (P 鼻Pe) − 1 + 1 ⎤ = Pe ⎢ e ⎥ ⎣ (Pe 鼻P) − 1 ⎦ 1 ⎡ ⎤ = M0 ⎢ ⎥ ⎣ 1 − (P鼻Pe) ⎦ where M0 is the unamplified maximum moment. Slip-critical connection: assume that the slip load controls. Finally, I want to thank my wife, Angela, for her valuable suggestions, patience, and support during the production of the five editions of this book. (2009) and Blodgett (1966) give procedures for determining the depth that incorporate the required moment strength and a specified h鼻tw ratio. If an external load P is applied, a friction force will develop between the connected parts. This means that for laterally supported beams, the Zx table can be used for design without regard to whether the shape is compact or noncompact. Ma = MD = 13 + 77 = 90 ft-kips From the Zx table in Part 3 of the Manual, Mp Mn = = 179 ft-kips > 90 ft-kips Ωb Ωb (OK) After the concrete has cured, the required moment strength is M a = M D + M L = 13 + 77 + 150 = 240 ft-kips M n 523.8 = = 314 ft-kips > 204 ft-kips (OK) Ωb 1.67 ANSWER The beam has sufficient flexural strength. Editorial review has deemed that any suppressed content does not materially affect the overall learning experience. Specification for Structural Joints Using High-Strength Bolts. For other bolts, use c = s – h, where e = edge-distance to center of the hole s = center-to-center spacing of holes h = hole diameter AISC Equation J3-6a is valid for standard, oversized, short-slotted and long-slotted holes with the slot parallel to the load. Compute this as illustrated in Example 9.7. 9.6-3 For the beam of Problem 9.3-1, do the following. Close. 7⁄8″ 10″ 1′-0″ 1″ FIGURE P5.2-3 5.2-4 Verify the value of Zx given in the Manual for a W18 × 46. The centroid of the top half-area can be found by the principle of moments. 2 2 Figure 9.15 shows the corresponding transformed section. 7.11 SOLUTION 455 The base metal in this connection is A36 steel, so E70XX electrodes will be used. In terms of rectangular components, f2 x = FIGURE 8.17 My J and f2 y = Mx J 498 Chapter 8 Eccentric Connections Also, J = ∫ r 2 dA = ∫ (x 2 + y 2) dA = ∫ x 2 dA + ∫ y 2 dA = I y + I x A A A A where Ix and Iy are the rectangular moments of inertia of the shear area. 2″ 3″ 3″ 3″ 2″ 2″ t = 3⁄8″ C8 × 18.75 FIGURE P7.4-1 7.4-2 A 1⁄2-inch-thick tension member is spliced with two 1⁄4-inch-thick splice plates as shown in Figure P7.4-2. (Note that two values of k are given in the dimensions and properties tables: a decimal value, called the design dimension, and a fractional value, called the detailing dimension. These requirements are represented graphically in Figure 4.8. 120 Chapter 4 Compression Members FIGURE 4.8 AISC Equations E3-2 and E3-3 are a condensed version of five equations that cover five ranges of KL鼻r (Galambos, 1988). Problems 53⁄8″ 1⁄2″ 469 diameter Group A bolts 2L4 × 4 × 5⁄8″ A36 3⁄8″ D = 6k L = 15k 7″ FIGURE P7.8-2 Combined Shear and Tension in Fasteners A bracket must support the service loads shown in Figure P7.9-1, which act through the center of gravity of the connection. 16). Plot these values and draw a best-fit line to obtain a stress–strain curve. (OK) 6.6 Members in Braced Frames 319 Note that the value of Cb is nearly the same regardless of whether factored or unfactored moments are used. In addition, the determination of the thickness of a column base plate requires consideration of flexure, so it logically belongs in this chapter rather than in Chapter 4. (The strength corresponding to flexural buckling about the x axis is 42.0 kips, based on an effective length of 14⁄2 = 7 feet.) Note that this shape is a slender-element section, but this is taken into account in the tabular values. EXAMPLE 9.13 A composite compression member consists of a W12 × 136 encased in a 20-in. To ensure that the built-up member acts as a unit, AISC E6.2 requires that the slenderness of an individual component be no greater than three-fourths of the slenderness of the built-up member; that is, Ka 3 KL ≤ ri 4 r (4.14) where a = spacing of the connectors ri = smallest radius of gyration of the component Ka鼻ri = effective slenderness ratio of the component KL鼻r = maximum slenderness ratio of the built-up member If the member buckles about the axis of symmetry—that is, if it is subjected to flexural-torsional buckling about the y-axis—the connectors are subjected to shearing forces. The nominal tensile strength of the threaded rod can therefore be written as Pn = AsFu = 0.75AbFu (3.5) where As = stress area Ab = nominal (unthreaded) area The AISC Specification, in Chapter J, presents the nominal strength in a somewhat different form: Rn = Fn Ab (AISC Equation J3-1) where Rn is the nominal strength and Fn is given in Table J3.2 as Fnt = 0.75Fu. This associates the 0.75 factor with the ultimate tensile stress rather than the area, but the result is the same as that given by Equation 3.5. For LRFD, the resistance factor f is 0.75, so the strength relationship is Pu ≤ ftPn or Pu ≤ 0.75(0.75AbFu) and the required area is Ab = Pu 0.75(0.75Fu) (3.6) For ASD, the safety factor Ω is 2.00, leading to the requirement Pa ≤ Pn 2.00 or Pa ≤ 0.5Pn Using Pn from Equation 3.5, we get Pa ≤ 0.5(0.75AbFu) If we divide both sides by the area Ab, we obtain the allowable stress Ft = 0.5(0.75Fu) = 0.375Fu (3.7) EXAMPLE 3.14 A threaded rod is to be used as a bracing member that must resist a service tensile load of 2 kips dead load and 6 kips live load. 1947. 4. To keep notation simple, however, we use Mp in all examples until the final step, when we substitute fbMp for it. For the other hole, 13 = 2.188 in. ANSWER (OK) Use a W12 × 53. Solve for the required value of a鼻h from kv = 5 + 5 (a鼻h)2 If tension-field action is used, either a trial-and-error approach or the design curves in Part 3 of the Manual can be used to obtain the required a鼻h. From the principle of virtual work, External work = internal work Pu(10q) = Mp(2q) + Mpq (No internal work is done at A because there is no plastic hinge.) Solving for the collapse load gives Pu = 3M p 10 The mechanism for span BC is slightly different: All three hinges are plastic hinges. 2.64 rx 鼻ry Pu = pPu = (3.16 × 10 –3)(54.97) = 0.1737 < 0.2 f c Pn ∴ Equation 6.10 controls as assumed. The installation of steel headed stud anchors is done in the same way as without the deck; the studs are welded to the beam flange directly through the deck. LRFD SOLUTION ANSWER ASD SOLUTION From Table 4-22 from Part 4 of the Manual and with KL鼻r = 54.55, fc Fcr = 36.24 ksi fc Pn = fc Fcr Ag = 36.24(17.0) = 616 kips Design strength = 616 kips. Try a plate 5⁄8 × 61⁄2. Beam web-to-plate weld design:To develop the yield stress in the web near the tension bolts, let 1.392D × 2 = 0.6Fy tw for two welds, one on each side of the web. In Figure 5.3, a simply supported beam with a concentrated load at midspan is shown at successive stages of loading. The gusset plate is the thinner of the two parts in bearing and will control. An unbraced length of 30 ft with Cb = 1.0. SOLUTION To determine the yield stress of a W14 × 68 of A242 steel, we refer to Table 2-3 in Part 2 of the Manual. Therefore, Mn = Mp as assumed. ⎝ 8⎠ and the shear rupture strength of the base metal is ⎛ 3⎞ 0.45Fu t = 0.45(58) ⎜ ⎟ = 9.788 kips鼻in. The holes are for 1⁄2-inch-diameter bolts. 2008. In a design problem, therefore, if the charts are entered with a given unbraced length and a required strength, curves above and to the right of the point correspond to acceptable beams. 5.10-5 The beam shown in Figure P5.10-5 has lateral support only at the ends. There is lateral support at the ends and at the one-third points. The approach is the same for both, but the differences have to be considered throughout the process. The ASCE 7 basic load combinations are also given in Part 2 of the AISC Steel Construction Manual (AISC 2011a), which will be discussed in Section 2.6 of this chapter. Since the partitions are moveable, they will be treated as live load. We must first find the steel anchor requirements for full composite behavior and then reduce the number of anchors. Use the 0.2% offset method to determine the yield strength. = 22.3 kips Ω ANSWER Allowable strength of the weld is 22.3 kips. If the member buckles about the x-axis (flexural buckling), the connectors are not subjected to any calculated load, and the connection problem is simply one of maintaining the relative positions of the two components. Assume a concrete cover of 2.5 inches to the center of the longitudinal renforcement. ⎝8⎠ The base metal shear strength is therefore 8.100 kips鼻in., and the weld strength of 6.960 kips鼻in. That is not the case if applied end moments produce reverse-curvature bending as shown in Figure 6.7. Here the maximum primary moment is at one of the ends, and maximum moment amplification occurs between the ends. W., Galambos, T. This is the same as for compact tees. The purpose of such a large tensile force is to achieve the clamping force illustrated in Figure 7.12. 2001. Check the shear strength of the stiffener (the thinner of the two connected parts). The total deflection must not exceed L 鼻240. This fact leads to the requirement that ⎛ ft ⎞ ⎛ fv ⎞ ⎜ F ⎟⎠ + ⎜⎝ F ⎟⎠ ≤ 1.0 t v 2 2 430 Chapter 7 Simple Connections ft FIGURE 7.32 ft Ft Fv + fv Fv = 1.3 fv The AISC Specification approximates the elliptical curve with three straight line segments as shown in Figure 7.32. McCormac and Stephen F. The strength of the W6 × 25 will control. The reinforcement parallel to the beam within the effective width of the slab must be adequately developed (anchored). What is the maximum normal strain? 44 studs per beam. The connection to the column flange is with eight 7⁄8-inch-diameter, Group A bearing-type bolts. A decision about whether to use intermediate stiffeners must be made early in the process because it will affect the web thickness. If both the front and back sides of the joint are to be welded, all information should be shown on each side of the reference line. In other words, the strength could be reduced because of local buckling. If the end were simply supported (pinned end), the moment would be zero. 9.1 Introduction 603 FIGURE 9.8 The resultant compressive force can also be expressed as C = 0.85 fc′ab from which we obtain a= C 530 = = 1.792 in. Adequate: For shear, 15 kips < 24.4 kips; For tension, 25 kips < 27.8 kips. From AISC Equation A-8-6, B2 = 1 ≥1 α Pstory 1− Pe story where Pe story may be obtained from AISC Equation A-8-7 as Pe story = RM HL ΔH where RM = 1 − 0.15 Pmf Pstory (AISC Equation A-8-8) Pmf = sum of vertical loads in all columns in the story that are part of moment frames L = story height ΔH = drift (sidesway displacement) of the story under consideration H = sum of all horizontal forces causing ΔH (story shear) The columns in a story may be designed for a specified drift index, ΔH /L, which is the ratio of the story drift to story height. Determine the thickness t so that the plate has sufficient bending strength. 1′-8″ 13′ W12 × 96 4 #9 #3 ties @ 13″ 1′-8″ 26′ 13′ Section Support in weak direction Support in strong direction FIGURE P9.10-2 9.10-3 Use the tables in Part 4 of the Manual and select a concrete-filled round HSS for the conditions shown in Figure P9.10-3. If the ratio of live load to dead load is 3, compute the available strength and determine the maximum total service load, in kips鼻ft, that can be supported. What conclusion can you draw from your answer? The end moments are service load moments, and bending is about the strong axis. "Plate Girder Design Using LRFD." Engineering Journal, AISC 24 (no. It has a stress–strain curve of the type shown in Figures 1.3b and 1.4 and has the following tensile properties. The channel is bolted to a 3⁄8-inch gusset plate with 7⁄8-inch-diameter, A307 bolts. If the abscissa values are random events, and enough samples are used, each ordinate can be interpreted as the probability, expressed as a percentage, of that sample value or event occurring. The edition that followed, which for the first time incorporated both ASD and LRFD, was named the thirteenth edition, because it was the thirteenth manual that had been published. If the distributions of Q and R are combined into one function, R–Q, positive values of R–Q correspond to survival. Lateral support is provided at each joint by simply connected bracing in the direction perpendicular to the frame. EXAMPLE 8.3 Use the tables in Part 7 of the Manual to determine the available strength based on bolt shear, for the connection shown in Figure 8.11. 8.2-9 a. 8.4-13 A connection is to be made with the weld shown in Figure P8.4-13. Because of the support conditions normal to the frame, Ky can be taken as 1.0. The applied load is Pa = D + L = 35.5 + 142 = 177.5 kips 4.8 Torsional and Flexural-Torsional Buckling 155 Enter Table 4-21 in Part 4 of the Manual with Pa 177.5 = = 18.28 ksi Ag 9.71 and obtain the stiffness reduction factor tb = 0.9703 by interpolation. Selection of the correct cable for a given loading is usually based on both strength and deformation considerations. The allowable strength per inch is 0.9279D = 0.9279(3) = 2.784 kips鼻in. Tests have shown that placement on the side farthest from the point of maximum moment results in a higher strength. The angles are A36 and the column is A992. ANSWER wa = wD + wL = 9 kips鼻ft ASD SOLUTION Ra = wa L 9(15.83) = = 71.24 kips 2 2 Determine the length of bearing b required to prevent web yielding. Design of Members Subject to Combined Loading. EXAMPLE 5.14 The shape shown in Figure 5.33 is a W18 × 71 with holes in each flange for 1-inch-diameter bolts. 51.7 kips b. Check all spacing and edge-distance requirements. Professor Segui is a member of the American Institute of Steel Construction and is a member of the Committee on Manuals. Of course, at this stage of the design, the flange thickness must also be estimated, but the consequences of a poor estimate are minor. Bearing: The bearing force on the 5⁄8-inch-thick tension member will be twice as large as the bearing force on each of the 1⁄4-inch splice plates. The connection of a bracket to a column flange, as in Figure 7.3d, whether by fasteners or welds, subjects the connection to shear when loaded as shown. The factored external load per bolt, excluding prying force, is T = 120鼻4 = 30 kips. The third case governs only when there are fewer studs than required for full composite behavior, resulting in partial composite behavior. 1971. Special requirements that apply when the ribs are parallel to the beam are presented in AISC I3.2c(3) and I8.2a. Figure 8.36b w FIGURE 8.36 L wL2鼻8 (a) wL2鼻24 wL2鼻8 wL 鼻12 2 (b) wL2鼻16 wL2鼻8 wL2鼻16 (c) 8.6 Moment-Resisting Connections 523 shows the same beam and loading with fixed supports. To determine the critical element, first compute the deformation of each element at maximum stress: Δ m = 0.209(θ + 2) −0.32 w FIGURE 8.19 8.4 Eccentric Welded Connections: Shear Only 503 where Δm = deformation of the element at maximum stress q = angle that the resisting force makes with the axis of the weld segment (see Figure 8.19) w = weld leg size Next, compute Δm鼻r for each element, where r is the radius from the IC to the centroid of the element. Otherwise, a new trial value of r0 must be selected and the process repeated. Compute the nominal flexural strength with two 3⁄4-inch × 31⁄2-inch studs per rib. In 1963, he joined Wilbur Smith Associates, where he was a highway bridge designer. Determine the nominal strength based on the net section as follows: a. 50.6 kips Answers to Selected Problems 7.9-3 a. Dividing both sides of AISC Equation J10-9 by Ω and substituting the plate thickness td for tw , we get Rn 0.60 Fy dc td = Ω Ω 556 Chapter 8 Eccentric Connections where the left side of the equation is the required allowable strength of the doubler plate. As the weld cools, impurities rise to the surface, forming a coating called slag that must be removed before the member is painted or another pass is made with the electrode. EXAMPLE 9.9 Determine the shear strength of a 1⁄2-in. Try a 1⁄4-inch E70 fillet weld: Design strength per inch of weld = 1.392D = 1.392(4) = 5.568 kips鼻in. 69.3 kips c. • Concentrated service live load = 125 kips applied at midspan. ANSWER Design strength = 42.0 kips. Then fc(0.85fc′A1) ≥ Ru 0.65(0.85)(3.5)A1 ≥ 99.73 A1 ≥ 51.57 in.2 The minimum value of dimension B is B= A1 51.57 = = 8.60 in. As in the case of eccentric bolted connections loaded in shear, welded shear connections can be investigated by either elastic or ultimate strength methods. Determine a. The factored loads and moments corresponding to one of the load combinations to be investigated are given for this member in the following table: Type of analysis Nonsway Sway Pu (kips) Mtop (ft-kips) Mbot (ft-kips) 400 — 45 40 24 95 Bending is about the strong axis, and all moments cause double-curvature bending (all end moments are in the same direction, that is, all clockwise or all counterclockwise). The welds are 3⁄16-inch fillet welds made with E70XX electrodes. The provisions of AISC Appendix 6 give equations for the required strength and stiffness (resistance to deformation) of stability bracing. • On the compression side, the stiffeners need only bear on the flange but may be welded. ANSWER The W16 × 31 is satisfactory. Obviously, shored construction is more efficient than unshored construction because the steel section is not called on to support anything other than its own weight. The maximum permissible live load deflection is L 30 × 12 = = 1.5 in. The width-to-thickness ratio is given the generic symbol l. The supporting walls can be reinforced concrete, concrete block, brick, or a combination of these materials. For bolted connections, the effective net area is Ae = AnU (AISC Equation D3-1) For welded connections, we refer to this reduced area as the effective area (rather than the effective net area), and it is given by Ae = AgU where the reduction factor U is given in AISC D3, Table D3.1. The table gives a general equation that will cover most situations as well as alternative numerical values for specific cases. If the maximum bending stress is less than the proportional limit when buckling occurs, the failure is said to be elastic. Qn 17.23 Total no. Sufficient steel anchors are provided to make the beam fully composite. Calibrated wrench tightening. In the direction perpendicular to the frame, the frame is braced at the joints. The allowable tensile strength. 7″ 16k 3″ 12″ 5″ FIGURE P8.4-2 8.4-3 Use an elastic analysis and determine the maximum load per inch of weld. (Since a W12 × 65 is noncompact for Fy = 50 ksi, 356 ft-kips is the design strength based on FLB rather than full yielding of the cross section.) The factored load moments are Mnt = 82.4 ft-kips Mt = 0 From AISC Equation A-8-1, the required moment strength is Mr = Mu = B1Mnt + B2Mt = 1.023(82.4) + 0 = 84.30 ft-kips = Mux From Equation 6.3 (AISC Equation H1-1a), Muy ⎞ Pu 8 ⎛ Mux 8 ⎛ 84.30 ⎞ + ⎜ + = 0.6131 + ⎜ + 0 ⎟ = 0.824 < 1.0 ⎠ fc Pn 9 ⎝ f b Mnx fb Mny ⎟⎠ 9 ⎝ 356 ANSWER ASD SOLUTION (OK) The member is satisfactory. Although section G2.1 of the Specification does not give h鼻tw = 260 as an upper limit, no value of kv is given when h鼻tw ≥ 260. 32 + 0.75 1(32)3 I= + 32(16 − 15.63)2 + 0.75(15.63)2 = 2918 in.4 12 y= For the two welds, I = 2(2918) = 5836 in.4 Mc 2.831R(15.63) ft = = = 0.007582 R kips鼻 in. 50.6 kips 7.8-1 a. Since the inner bolts are not near an edge or adjacent bolts, the outer bolts control. The moment strength of compact shapes is a function of the unbraced length, Lb, defined as the distance between points of lateral support, or bracing. The strength per unit length can be computed for a 1⁄16-inch increment of weld size (and fillet welds are specified to the nearest 1⁄16 inch). Yura, Joseph A. 3.2 Tensile Strength 49 What is the difference in computational effort for the two different approaches? (This method is the same as Case II in Part 7 of the Manual.) EXAMPLE 8.4 A beam-to-column connection is made with a structural tee as shown in Figure 8.14. By inspection, load combination 3 will control. Select a member with a rectangular cross section. In addition to ordinary elastic elongation, an initial stretching is caused by seating or shifting of the individual wires, which results in a permanent stretch. If we assume that the upper limit in AISC Equation J4-5 does not control, the required allowable strength is Rn 0.6 Fu Anv + Ubs Fu Ant = Ω Ω 0.6(58) Anv + 1.0(58)(0.7500) = 72 kips = 2.00 Required Anv = 2.888 in.2 3 Anv = [s + s + 1.25 − 2.5(1.0)](2) = 2.888 in.2 8 Required s = 2.55 in. A special coating on the electrode vaporizes and forms a protective gaseous shield, preventing the molten weld metal from oxidizing before it solidifies. The nodal and relative categories were defined in Chapter 4, "Compression Members." Appendix 6 of the AISC Specification gives the strength and stiffness requirements for beam bracing. A conservative, simplified method will be used here. This approach is referred to as the cantilever method. This material must resist the bearing load applied by the steel plate. For LRFD, the design strength of a fillet weld is fRn, where f = 0.75. 4.7-4 Use A992 steel and select a W12 shape for an axially loaded column to meet the following specifications: The length is 24 feet, both ends are pinned, and there is bracing in the weak direction at a point 10 feet from the top. This connection is similar to the one shown in Figure 8.37a but requires the use of column stiffeners to make the connections to the beam flanges. If the original length is used to compute the strain, it is called engineering strain. Assume that the girder is supporting beams on each side, and assume that the beams weigh 35 lb鼻ft. The limits given in the International Building Code (ICC, 2009) are typical. For example, Table 3-2, "W Shapes, Selection by Zx," (hereafter referred to as the "Zx table"), lists shapes commonly used as beams, arranged in order of available flexural strength—both fbMpx and Mpx 鼻Ωb. (The effective throat thickness for a weld made with the submerged arc welding process is larger. FIGURE 9.23 EXAMPLE 9.11 Compute the available strength of the composite beam in Examples 9.1 and 9.2. Use the tables in Part 3 of the Manual. 0.85 fc′b 0.85(4)(87) The force C will be located at the centroid of the compressive area at a depth of a鼻2 from the top of the slab. From Equation 5.20, t≥ ANSWER 5.15 2 Pa 2(243) = 2.5 = 0.913 in.. 2″ 3″ 21⁄2″ 2″ 21⁄2″ 21⁄2″ 2″ MC 9 × 23.9 FIGURE P3.4-3 3.4-4 A992 steel is used for the tension member shown in Figure P3.4-4. An additional requirement is that the shear on the base metal cannot exceed the shear strength of the base metal. At the other hole, the shearing strength is smaller. Such a case is illustrated in Figure 5.45, where a single concentrated load acts normal to the longitudinal axis of the beam but is inclined with respect to each of the principal axes of the cross section. 665 666 Chapter 10 Plate Girders FIGURE 10.1 Before the widespread use of welding, connecting the components of the cross section was a major consideration in the design of plate girders. The steel yield stress is 50 ksi, and fc′ = 8 ksi. H., and Chesson, E. LRFD Manual of Steel Construction, Metric Conversion of the 2nd edition. Use a distance of 36 inches from the center of the end bearing stiffener to the center of the first intermediate stiffener. 2 2 From Table 3-19 in the Manual, by interpolation, φb M n = 237 ft-kips > 214 ft-kips (OK) Adjust Mu for the beam weight: wu = 1.900 + 1.2(0.026) = 1.931 kips鼻ft 1 1 Mu = wu L2 = (1.931)(30)2 = 217 ft-kips < 237 ft-kips 8 8 (OK) Check shear: From the Zx table, fvVn = 106 kips. In many cases, a structural member must be investigated for various positions of a live load so that a potential failure condition is not overlooked. Assume that the strengths of the connected parts do not govern. 3 2 3 2 Use a maximum width of b≤ 8.07 − 0.360 = 3.86 in. ASTM A242 steel is used. If the member is subjected to dead load and live load only, what is the total service load capacity if the live-to-dead load ratio is 3? Specify whether your selection is Standard, Extra-Strong, or Double-Extra Strong. Do not consider the spacing of the studs. SOLUTION AISC E4(b) must be used, because this shape is nonslender and is neither a doubleangle shape nor a tee shape. If the hypothetical structure under consideration is subjected to a uniformly distributed roof load, the portion carried by one frame will be a uniformly distributed line load measured in force per unit length, as shown in Figure 1.2b. Although partial composite action can exist with either solid slabs or slabs with formed steel deck, it will be covered in Section 9.7, "Composite Beams with Formed Steel Deck." EXAMPLE 9.2 Compute the available strength of the composite beam of Example 9.1. Assume that sufficient stud anchors are provided for full composite behavior. A36 steel is used for both the angle and the gusset plate. Do not balance the welds. As previously shown, when a steel is stressed beyond its elastic limit and then unloaded, the path followed to zero stress will not be the original path from zero stress; it will be along a line having the slope of the linear portion of the path followed during loading—that is, a slope equal to E, the modulus of elasticity. For truss analysis, member lengths are measured from working point to working point. The required strength is actually the required design strength, so Required fb Mp = 128.2 ft-kips Before making a selection, remember that the shape must be compact. fRn = 4.176 (4 + 4 + 4) = 50.1 kips 2. If the shape is noncompact because of the flange, the nominal strength will be the smaller of the strengths corresponding to flange local buckling and lateraltorsional buckling. 70° 6″ 1′-0″ t = 3⁄8″ FIGURE P8.5-4 588 Chapter 8 8.5-5 Eccentric Connections A WT7 × 41 bracket is connected to a W14 × 159 column with 5⁄16-inch E70 fillet welds as shown in Figure P8.5-5. • For ASD, the allowable strength is Mn鼻Ωb , where Ωb = 1.67. P 15′ 15′ FIGURE P5.5-1 5.5-2 The beam in Figure P5.5-2 has continuous lateral support. Design of Steel and Composite Beams with Web Openings is a useful guide to this topic (Darwin, 1990). Our solution manuals are written by Chegg experts so you can be assured of the highest. 2 2 Line abcd controls. Determine the required weld size. To compute the plate girder strength reduction factor Rpg, the value of aw will be needed: aw = hc t w 62(5鼻16) = = 0.7176 < 10 b fc t fc 18(1.5) From AISC Equation F5-6, Rpg = 1 − = 1− ⎛ hc aw E⎞ ⎜ ⎟ − 5.7 ⎟ ≤ 1.0 1200 + 300 aw ⎝ t w Fy ⎠ ⎛ 0.7176 29, 000 ⎞ ⎜ 198.4 − 5.7 ⎟ = 0.9814 1200 + 300(0.7176) ⎝ 36 ⎠ From AISC Equation F5-7, the nominal flexural strength for the compression flange is Mn = RpgFcr Sxc = 0.9814(36)(1866) = 65,930 in.-kips = 5494 ft-kips and the design strength is f b Mn = 0.90(5494) = 4945 ft-kips > 4749 ft-kips (OK) Although this capacity is somewhat more than needed, the excess will compensate for the weight of stiffeners and other incidentals that we have not accounted for. ANSWER Use a W12 × 58. Timoshenko, Stephen P. At any location, the moment is M = P (y0 + y) FIGURE 6.5 308 Chapter 6 Beam–Columns Substituting this equation into the differential equation, we obtain d2y dx =− 2 px ⎞ P ⎛ e sin + y⎟ ⎠ EI ⎜⎝ L Rearranging gives d2y dx 2 px P Pe sin y=− EI EI L + which is an ordinary, nonhomogeneous differential equation. For this reason, unbraced frames are frequently referred to as moment frames. I 36 P 15 fv = a = = 1.250 kips鼻in. A member is selected by using the criterion that the structure will fail at a load substantially higher than the working load. Modify bx to account for Cb. bx 6.54 × 10 −3 = = 3.92 × 10 −3 Cb 1.67 For Lb = 0, bx = 5.90 × 10−3. The nominal bearing strength for the connection is therefore Rn = 122.3 kips LRFD SOLUTION The design strength is fRn = 0.75(122.3) = 91.7 kips. M1 + M 2 − M p + M p = =0 2 2 M1 + M 2 M mid > ∴ M1⬘ = 2 M mid − M 2 = 2(585) − 1170 = 0 2 From AISC Equation A-1-5, the maximum permitted unbraced length is M1⬘ ⎞ ⎤ ⎛ E ⎞ ⎡ L pd = ⎢ 0.12 + 0.076 ⎛ ry ⎝ M 2 ⎠ ⎥⎦ ⎝ Fy ⎠ ⎣ = [0.12 + 0.076(0)] ⎛ ⎝ 29, 000 ⎞ (2.10) = 146.2 in. 4.6-9 For the conditions shown in Figure P4.6-7, use LRFD and select the lightest W21 shape of A992 steel. Before the concrete cures, there is only dead load (there is no construction load in this example). 514 Chapter 8 Eccentric Connections FIGURE 8.27 For the beam setback of 3⁄4 inch, the beam is supported by 3.25 inches of the 4-inch outstanding leg of the angle. The force R is the beam reaction. 0.860 k/ft 1.45 k/ft P = 19 k M = 79 ft-k 18′ P = 33 k M = 130 ft-k 45′ Roof live load Dead load 0.510 k/ft 3.6 k P = -11 k M = -46 ft-k P = 1.4 k M = 32 ft-k Wind (roof uplift) Wind (lateral) FIGURE P6.8-9 Design of Bracing 6.8-10 Use A36 steel and design single-angle diagonal bracing for the frame of Problem 6.8-9. For a doubly-symmetric, compact I- or C-shaped section bent about its major axis, AISC F2.1 gives the nominal strength as Mn = Mp (AISC Equation F2-1) where Mp = Fy Z x EXAMPLE 5.3 The beam shown in Figure 5.11 is a W16 × 31 of A992 steel. 5⎡ 7 ⎤ Ant = ⎢1.5 − 0.5 ⎛⎜ ⎞⎟ ⎥ × 2 = 1.328 in.2 ⎝ 8⎣ 8 ⎠⎦ Rn = 0.6Fu Anv + Ubs Fu Ant = 0.6(58)(9.279) + 1.0 (58)(1.328) = 399.9 kips Upper limit = 0.6Fy Agv + Ubs Fu Ant = 0.6(36)(13.13) + 1.0(58)(1.328) = 360.6 kips fRn = 0.75(360.6) = 271 kips > 121.2 kips (OK) Check block shear in the beam flange. The seated beam connection consists primarily of a short length of angle that serves as a "shelf" to support the beam. What additional force is experienced as a consequence of the fasteners not being on the centroidal axis of the member? Although this designation provides all of the dimensions, it does not provide the weight per foot. × 12-in. In summary, to investigate the need for column reinforcement, three limit states should be checked: 1. If Table C-A-7.1 is used for this frame, the lowerstory columns are best approximated by condition (f), and a value of K = 2 might be used. 10.3 AISC REQUIREMENTS FOR PROPORTIONS OF PLATE GIRDERS Whether a girder web is noncompact or slender depends on h鼻tw, the width-tothickness ratio of the web, where h is the depth of the web from inside face of flange to inside face of flange and tw is the web thickness. Neal, B. The superimposed loads consist of a construction load of 20 psf, a partition load of 20 psf, a miscellaneous dead load of 10 psf, and a live load of 150 psf. As an alternative, an approximate second-order analysis, as given in Appendix 8, can be used. The web is the thinner of the connected parts and controls. It is based on research by Butler et al. Assume full lateral support during construction and an additional construction load of 20 psf. For maximum stress, Equation 5.3 takes the form: fmax = Mc M M = = Ix I x 鼻c S x (5.4) 192 Chapter 5 Beams where c is the perpendicular distance from the neutral axis to the extreme fiber, and Sx is the elastic section modulus of the cross section. Compare the available strength of the steel shape to the required moment strength acting before the concrete cures. Threads reduce the cross-sectional area, and upsetting the end produces a larger gross area to start with. The free-body diagrams in Figure 7.12a show that the total compressive force acting on the connected part is numerically equal to the tension in the bolt. SOLUTION The following dimensions and properties from Part 1 of the Manual will be needed: design wall thickness = 0.116 in., As = 2.51 in.2, D鼻t = 60.3 in.4, and Is = 14.9 in.4. The following values will also be required: 2 π d inside π (7.000 − 2 × 0.116)2 = = 35.98 in.2 4 4 Ec = w1..5 fc⬘ = (145)1.5 5 = 3904 ksi Ac = Ic = 4 π d inside π (7.000 − 2 × 0.116)4 = = 103.0 in.4 64 64 652 Chapter 9 Composite Construction Determine the classification of the section: D = 60.3 t 0.15E 0.15(29, 000) λp = = = 103.6 Fy 42 λ= Since l < lp, the section is compact, and from AISC Equation I2-9b, Es ⎞ Pno = Pp = Fy As + C2 f c⬘ ⎛⎜ Ac + Asr ⎟ ⎝ Ec ⎠ = 42(2.511) + 0.95(5)(35.98 + 0) = 276.3 kips Next, compute (EI)eff and Pe . The presence of the axial load produces secondary moments, and unless the axial load is relatively small, these additional moments must be accounted for. Noncompact shapes: The strength will be the lower value obtained for flange local buckling, web local buckling, and LTB. The lightest one with a possibility of working is a W14 × 61. The maximum live-load deflection cannot exceed L鼻240. The connection illustrated also uses column stiffeners, which are not always required (see Section 8.7). 3.944 3.944 10.74 P2 = = 2.72 in. 400 k 182 ft-k 12′ 140 ft-k 400 k FIGURE P6.8-2 6.8-3 A member is subjected to the loads shown in Figure P6.8-3. Hence, load combination A4-1 controls, and the factored load moment is Mu = 1.4(MD) = 1.4(13 + 77) = 126 ft-kips From the Zx table in Part 3 of the Manual, fb Mn = fb Mp = 270 ft-kips > 126 ft-kips (OK) After the concrete has cured, the factored load moment that must be resisted by the composite beam is Mu = 1.2MD + 1.6ML = 1.2(13 + 77) + 1.6(150) = 348 ft-kips 606 Chapter 9 Composite Construction The design moment is fb Mn = 0.9(523.8) = 471 ft-kips > 348 ft-kips ANSWER ASD SOLUTION (OK) The beam has sufficient flexural strength. 1.47 in. Solving for Ra , we get Ra = 326.5 kips Check the shear capacity of the base metal (the angle controls). Connected through the flange with three or more fasteners in the direction of loading, with a width less than 2⁄3 of the depth: U = 0.85. Typical units would be kips per foot. studs (Asa = 0.4418 in.2), one at each section. 469 kips 10.5-3 a. = np, n = 0, 1, 2, 3, . They were introduced long after A325 bolts had been in general use, primarily for use with high-strength steels (Bethlehem Steel, 1969). CHAPTER 2 Concepts in Structural Steel Design 2.1 DESIGN PHILOSOPHIES As discussed earlier, the design of a structural member entails the selection of a cross section that will safely and economically resist the applied loads. The bolts are in single shear. Because the frame has a plane of symmetry parallel to the page, we are able to treat the frame as two-dimensional and represent the frame members by their centerlines. Use A36 steel and design the sag rods and the tie rod at the ridge for the following service loads. Only those provisions not already discussed are listed. Next, refine the value of Y2. Using AISC Equation J8-2, Pp ≥ Pa Ωc 1 ⎡ 18(18) ⎤ 0.85(3) A1 ≥ 243 ⎢ A1 ⎥⎦ 2.31 ⎣ A1 ≥ 150 inn.2 The upper limit is 1 1 (1.7 fc′ A1) = [(1.7)(3)(150)] = 331 kips > 243 kips (OK) 2.31 Ωc The plate must be as large as the column: bf d = 10.0(10.0) = 100 in.2 < 150 in.2 (OK) Try B = N = 13 in., with A1 provided = 13(13) = 169 in.2 >150 in.2 N − 0.95d 13 − 0.95(10.0) = = 1.75 in. Fcr (inelastic) π 2 Et 鼻(KL鼻r)2 Et = 2 = = τb Fcr (elastic) E π E鼻(KL鼻rr)2 From Galambos (1998), Fcr (inelastic) and Fcr (elastic) can be expressed as ⎛ λ2 ⎞ Fcr (inelastic) = ⎢ 1 − ⎢ Fy ⎝ 4⎠ Fcr (elastic) = Fy λ2 where λ= KL 1 r π Fy E Since τb = Fcr (inelastic) Fcr (elastic) then Fy Fcr (inelastic) = τ b Fcr (elastic) = τ b ⎛⎜ 2 ⎞ ⎝ λ ⎠ λ2 = τ b Fy Fcr (inelastic) From Equation 4.13, τ b Fy ⎞ ⎛ ⎛ λ2 ⎞ Fcr (inelastic) = ⎢ 1 − ⎢ Fy = ⎢ 1 − Fy ⎝ ⎝ 4 Fcr (inelastic) ⎠⎦ 4⎠ (4.13) 152 Chapter 4 Compression Members Using the notation Fcr = Fcr (inelastic) and solving for tb, we obtain Fcr ⎞ ⎛ Fcr ⎞ ⎛ τ b = 4 ⎢ ⎥⎢1 − ⎝ Fy ⎠ ⎝ Fy ⎠⎦ This can be written in terms of forces as Pn ⎞ ⎛ Pn ⎞ ⎛ τ b = 4 ⎢ ⎥⎢1 − ⎢ Py ⎠ ⎝ Py ⎠ where Pn = nominal compressive strength = Fcr Ag Py = compressive yield strength = Fy Ag Substituting the required strength, aPr , for the available strength, Pn, we have ⎛ α Pr ⎞ ⎛ α Pr ⎞ τb = 4 ⎢ 1− ⎝ Py ⎠⎝ Py ⎠ (AISC Equation C2-2b) where a = 1.0 for LRFD and 1.6 for ASD. π 2 EI x* π 2 (0.8 EI x) π 2 (0.8)(29, 000)(248) = = = 1753 kips (K x L)2 (K x L)2 (15 × 12)2 Cmx 0.6 B1x = = = 0.660 < 1.0 ∴ B1x = 1.0 as assumed. (We also addressed web crippling in Section 5.14.) When the load is applied at a distance of at least d鼻2 from the end of the column, ⎡ b ⎞ ⎛ t w ⎞ ⎛ d ⎠⎦ ⎝ t f ⎠ Rn = 0.80 t w2 ⎢1 + 3 ⎛⎜ ⎥⎢ 1.5 ⎤ EFyw t f ⎥ tw ⎦⎥ (AISC Equation J10-4) where d = total column depth. = 384 EI LB 384(29, 000)(759.4) The total deflection is Δ1 + Δ 3 + Δ 4 = 1.098 + 0.9931 + 0.0828 = 2.17 in. Compute the nominal strength based on shear and bearing. Block shear must also be examined in beam-to-column connections in which the top flange of the beam is coped. From the dimensions and properties tables in Part 1 of the Manual, rmin = 0.863 in. Two relatively unskilled workers can install high-strength bolts, whereas four skilled workers were required for riveting. Again, the connection prevents translation, but no restraint against rotation is furnished. The maximum available factored load reaction, Ru , for LRFD. Figure 6.4 illustrates these two components of deflection. The lateral load produces an M t moment. Next, check the shear: Vu = wu L 7.301(30) = = 110 kips 2 2 From the Zx table, fvVn = 340 kips > 110 kips (OK) Finally, check the deflection. Two plots are shown here; the first one shows the entire test range, and the second shows a portion near the proportional limit. For the edge bolts, use c = e − h鼻2. Equation 8.1 can then be written as fv = Md A∑ d2 and the shear force in each fastener caused by the couple is Md Md pm = Afv = A = 2 A∑ d ∑ d2 The two components of shear force thus determined can be added vectorially to obtain the resultant force, p, as shown in Figure 8.3b, where the lower right-hand fastener is used as an example. • Minimum height of stud above the top of the deck = 11⁄2 inches. For a member with a rectangular cross section, the calculations are relatively straightforward. What is the required nominal strength of the column for a safety factor Ω of 1.67? Design the flange-to-web welds , stiffener-to-web welds , and any other connections (flange segments, web splices, etc.). Not all high-strength bolts need to be tightened to the fully tensioned condition. In such cases, plate girders can be used. From AISC Equation F5-3, ⎡ ⎛ Lb − L p ⎞ ⎤ Fcr = Cb ⎢ Fy − 0.3Fy ⎜ ⎟ ⎥ ≤ Fy ⎝ Lr − L p ⎠ ⎦⎥ ⎣ For the computation of Cb, refer to Figure 10.12, which shows the loading, shear, and bending moment diagrams based on factored loads. 3″ 4″ 30k 15k 2″ 3″ 3″ 2″ FIGURE P8.2-5 8.2-6 The load on the bracket plate consists of a service dead load of 22 kips and a service live load of 44 kips. (Properties of the double-angle shape are given in Part 1 of the Manual.) In this example, we consider one angle and double the result. 5.14 BEAM BEARING PLATES AND COLUMN BASE PLATES The design procedure for column base plates is similar to that for beam bearing plates, and for that reason we consider them together. If this concept is used, a sufficient number of steel anchors must be provided to achieve a degree of continuity between the steel shape and the reinforcing. The relationship between stress and strain is linear up to the proportional limit; the material is said to follow Hooke's law. ANSWER Use 14 bolts (one extra to provide symmetry). Assume that the shear in the base metal is acceptable. Hence, Mu = 1 1 wu L2 = (1.540)(30)2 = 173 ft-kips 8 8 Since the beams have continuous lateral support, the Zx table can be used. In either case, any cross section passing through holes will pass through fewer holes than if the fasteners are not staggered. (OK) 8.7 ANSWER Column Stiffeners and Other Reinforcement 555 Use two plates 5⁄16 × 3 × 0′ – 41⁄2″. See Figure 10.5. Mp = plastic moment = Fy Zx My = yield moment = Fy Sx 10.3 FIGURE 10.5 hc鼻2 AISC Requirements for Proportions of Plate Girders 671 Plastic neutral axis hp鼻2 Centroid Elastic neutral axis To prevent vertical buckling of the compression flange into the web, AISC F13.2 imposes an upper limit on the web slenderness. The design and analysis of each frame in the system begins with the idealization of the frame as a two-dimensional structure, as shown in Figure 1.2b. 360 360 ΔL = 5 wL L4 5 (0.700鼻112)(30 × 12)4 = = 1.17 in. For a W14 × 68, it is given as 2.80 in.) ho = d − tf = 14.0 − 0.720 = 13.28 in. The Specification user note for square and rectangular sections permits a value of f = Fy to be used in lieu of determining f by iteration. L = 20 ft L W10 × 33 A992 steel FIGURE P4.3-1 4.3-2 Compute the nominal axial compressive strength of the member shown in Figure P4.3-2. For these cases, Rn = 0.6Fu Anv + 0.5Fu Ant (3.4) The AISC Specification limits the 0.6Fu Anv term to 0.6Fy Agv, where 0.6Fy = shear yield stress Agv = gross area along the shear surface or surfaces and gives one equation to cover all cases as follows: Rn = 0.6Fu Anv + Ubs Fu Ant ≤ 0.6Fy Agv + Ubs Fu Ant (AISC Equation J4-5) where Ubs = 1.0 when the tension stress is uniform (angles, gusset plates, and most coped beams) and Ubs = 0.5 when the tension stress is nonuniform. 1.21 in., 1.73 in. Lb = 20 ft, Cb = 1.14 LRFD SOLUTION a. Some have noncompact flanges, but none have slender flanges. The minimum edge distance required by AISC Table J3.4 is 1 inch, and this requirement is satisfied in both the longitudinal and transverse directions. From a practical standpoint, if a compression member to be analyzed can be found in the column load tables, then these tables should be used. Compact shapes: For compact shapes, the strength will be based on the limit state of yielding. Web area: Flange area: 62(5鼻16) = 19.38 in.2 2(1.5 × 18) = 54.00 in.2 73.38 in.2 Total: Weight = 73.38 144 (490) = 250 lb鼻ft 700 Chapter 10 Plate Girders The adjusted bending moment is Mu = 4614 + (1.2 × 0.250)(60)2 = 4749 ft-kips 8 Figure 10.17 shows the trial section, and Figure 10.18 shows the shear and bending moment diagrams for the factored loads, which include the girder weight of 250 lb鼻ft. Any point on this graph, such as the intersection of the two dashed lines, represents FIGURE 5.25 FIGURE 5.26 232 Chapter 5 Beams an available moment strength and an unbraced length. Modify bx to account for Cb. bx 5.07 × 10 −3 = = 3.04 × 10 −3 Cb 1.67 For Lb = 0, bx = 4.32 × 10−3. Use the 50% increase for the transverse weld, but reduce the basic strength by 15% for the longitudinal welds. L., and Ravindra, M. The tables for design do, however, provide all other information in a compact form, and the search may go more quickly. New York: John Wiley and Sons. Although Lb = 0 is not on this particular chart, the smallest value of Lb shown is less than Lp for all shapes on that page. 7.9 ANSWER ASD SOLUTION Combined Shear and Tension in Fasteners 439 For symmetry, use fourteen 3⁄4-inch-diameter, Group A bolts. 1): 47–55. In the upper story, members AC and BD will bend under the influence of P1. The Euler stress is Fe = Pe p 2E = A (KL鼻r)2 (AISC Equation E3-4) With a slight modification, this expression will be used for the critical stress in the elastic range. D = 30 k L = 70 k 10⬘ FIGURE P4.8-4 Built-Up Members 4.9-1 Verify the value of ry given in Part 1 of the Manual for the double-angle shape 2L4 × 31⁄2 × 1⁄4 SLBB. From Equation 6.9, (4.45 × 10−3)(201) + (5.26 × 10−3)(67) = 1.25 > 1.0 (N.G.) There are no other possibilities in the deeper W-shape groups. ANSWER 8.5 Use the weld shown in Figure 8.23. Any shape bent about its weak axis cannot buckle in the other direction, so lateral-torsional buckling is not a limit state. The plastic moment can be computed for a given yield stress and cross-sectional shape, as indicated in Figure A.2. If the stress distribution in the fully plastic condition is replaced by two equal and opposite statically equivalent concentrated forces, a couple is formed. The weld is a 5⁄16-inch fillet weld, E70 electrode. EXAMPLE 7.4 The connection shown in Figure 7.13a uses 3⁄4-inch-diameter Group A bolts with the threads in the shear plane. A building code has the force of law and is administered by a governmental entity such as a city, a county, or, for some large metropolitan areas, a consolidated government. The concentrated loads are service live loads. 4.71 29, 000 E = 4.71 = 124.7 0.8989(46) QFy KL = 105.3 < 124.7 r ∴ Use AISC Equation E7-2 QFy ⎞ ⎛ 0.8989 (46) ⎞ ⎛ Fcr = Q ⎢ 0.658 Fe ⎥ Fy = 0.8989 ⎢ 0.658 25.81 ⎥ 46 ⎥ ⎝ ⎠ ⎝ ⎠ = 21.15 ksi ≠ 21.26 ksi Try f = 21.15 ksi: be = 1.92(0.116) 29, 000 21.15 ⎡ 0.38 29, 000 ⎤ ⎥ 1 − (66.0) 21.15 ⎥ = 6.489 in. For one bolt, Rn µ Du h f Tb ns 0.30(1.13)(1.0)(28)(1.0) = = = 6.328 kips Ω Ω 1.50 Setting the total shear load to the reduced slip-critical strength for nb bolts, 4.741 ⎞ 50 = nb ⎛⎜ 1 − ⎢ (6.328) ⎝ nb ⎠ = 6.328nb − 30.00 nb = 12.66 From Part a, the connection is adequate in shear, bearing, and tension with eight bolts, so it will be adequate if more bolts are used. Assume that the bolt threads are in the plane of shear and bearing strength is controlled by the upper limit of 2.4dtFu. a. The design strength based on yielding of the gross area is f t Pn = 0.90(86.76) = 78.08 kips The design strength based on fracture of the net area is ft Pn = 0.75(87.81) = 65.86 kips ANSWER Because 65.86 kips < 78.08 kips, fracture of the net section controls, and the design strength for the two angles is 2 × 65.86 = 132 kips. • Steel is A572 Grade 50. SOLUTION Loads to be supported before the concrete cures: 4.5 (150) = 56.25 psf (conservatively) 12 = 56.25(5.5) = 309.4 lb鼻ft Slab wt. Compute the nominal strength of the column. It supports a reinforced concrete floor slab that provides continuous lateral support of the compression flange. Because this shape is symmetrical about the x-axis, this axis divides the cross section into equal areas and is therefore the plastic neutral axis. Note that the upper limit is independent of c and is the same for all bolts. Adequate: Mu = 433 ft-kips < f bMn = 446 ft-kips, Vu = 21.6 kips < f vVn = 175 kips b. From the LRFD solution, Δ = 1.11 in. 14.77 The slenderness ratio is KL Kh 0.75(63) = = = 12.05 < 25 r r 3.922 ∴ Pn = Fy Ag = 36(14.77) = 531.7 kips and fPn = 0.90(531.7) = 479 kips > 60 kips (OK) For the bearing stiffeners at the supports, the nominal bearing strength is (refer to Figure 10.14) Rn = 1.8Fy Apb = 1.8(36)[4(6.5)(0.75)] = 1264 kips and fRn = 0.75(1264) = 948 kips > 234 kips (OK) 10.6 Bearing Stiffeners 691 FIGURE 10.14 Check the stiffener–web assembly as a compression member. However, determining which element has the maximum deformation and computing the force in each element at failure is more difficult, because unlike the bolted case, the weld strength is a function of the direction of the load on the element. From Table 6-1, rx 鼻ry = 2.11 and K x L 2.0(15) = = 14.2 ft < K y L = 15 ft 2.11 rx 鼻ry ∴ KL = 15 ftt as assumed. Design the stud anchors. This type of buckling can be either compression flange buckling, called flange local buckling (FLB), or buckling of the compression part of the web, called web local buckling (WLB). SOLUTIONS MANUAL: Analysis and Design of Analog Integrated Circuits 5th Ed (vol.1) SOLUTIONS MANUAL: Structural Steel Design 4th Ed by McCormac Do u have solutions manual for Applied Structural steel design - 4th edition Leonard Spiegel and George Limbrunner #. fc Pn To be sure that we have found the lightest W12, try the next lighter one, a W12 × 53, with p = 2.21 × 10−3 and bx = 3.52 × 10−3. It can be summarized as follows. A complication arises if the beam is subject to sustained loading, such as the weight of partitions, after the concrete cures. ASTM A500 is usually specified for hollow structural sections (HSS). Others suggest a range of 1⁄6 to 1⁄20 (Galambos, et al., 1980). Use A36 steel for both the tension member and the gusset plate. Show the results on a sketch similar to Figure 9.11. An inadequate connection, which can be the "weak link" in a structure, has been the cause of numerous failures. Check the strong axis: Kx L 18 = = 10.53 ft > 9 ft rx 鼻ry 1.71 ∴ K x L controls for this shape. The design thickness, which is 0.93 times the nominal thickness, should be used. Maximum total deflection after composite behavior has been attained. 8.4 ECCENTRIC WELDED CONNECTIONS: SHEAR ONLY Eccentric welded connections are analyzed in much the same way as bolted connections, except that unit lengths of weld replace individual fasteners in the computations. ECCENTRIC BOLTED CONNECTIONS: SHEAR PLUS TENSION In a connection such as the one for the tee stub bracket of Figure 8.12, an eccentric load creates a couple that will increase the tension in the upper row of fasteners and decrease it in the lower row. Design of Flexural Members. From AISC J2.2b, 3 ⎞ Minimum length = 4 w = 4 ⎛⎜ ⎞ 16 ⎠⎟ = 0.75 in., but not less than 1.5 in. K. 29,000 E Use intermittent welds. Torsional bracing prevents twist directly; it can be either nodal or continuous, and it can take the form of either cross frames or diaphragms. Use E70 electrodes and determine the required fillet weld size. • Compute KL鼻r and Fe as usual. To prevent damage during construction, rods should not be too slender. That is, Pstory Pu 212 + 56 268 ≈ = = 3767 3767 Pe story Pe1 Then from AISC Equation A-8-6, B2 = 1 1 = = 1.077 α Pstory 1.0(268) 1− 1− 3767 Pestory 334 Chapter 6 Beam–Columns The amplified axial load is Pr = Pu = Pnt + B2Pt = 212 + 1.077(56) = 272.3 kips The total amplified moment is Mr = Mu = B1Mnt + B2Mt = 1.0(47.6) + 1.077(132) = 189.8 ft-kips Although the moments Mnt and Mt are different, they are distributed similarly, and Cb will be roughly the same; at any rate, they are large enough that fbMp = 356 ft-kips will be the design strength regardless of which moment is considered. Using a ratio of live load-to-dead load of 3.0, determine the maximum service load, P, that can be applied. D = 20k L = 54k 2″ 3″ 10″ 1″-diameter Group A bolts A992 steel Slip is permitted 14″ W10 × 100 WT5 × 24.5 2″ FIGURE P8.3-1 8.3-2 A beam is connected to a column with 7⁄8-inch-diameter, Group A bearing-type bolts, as shown in Figure P8.3-2. If the planks are unconnected, they will slip along the surface of contact when loaded and will function as two separate beams. It can occur only with doubly symmetrical cross sections with very slender cross-sectional elements (Figure 4.18b). ©iStockphoto.com/sack Beams incorporating steel headed stud anchors are called composite beams. The wrenches are calibrated for the size and strength of the bolt, and when the required tension is reached, the end twists off. Regardless of the method used, the two nominal strengths must be computed (if a stress approach is used with ASD, an equivalent computation must be made). From Table 6-1, Try a W10 ∂ 60, with p = 1.89 × 10−3 and bx = 3.51 × 10−3. If there is a tension field, the width-to-thickness ratio limit of AISC Equation G3-3 and the moment of inertia requirement of AISC Equation G3-4 must be satisfied. All connections are to be made with 7⁄8-inch-diameter, Group B slip-critical bolts. Z= (A2) a = 8.85(8.432) = 74.62 in. The location of the shear center can be determined from elementary mechanics of materials by equating the internal resisting torsional moment, derived from the shear flow on the cross section, to the external torque. 10.2 GENERAL CONSIDERATIONS Structural steel design is largely a matter of providing for stability, either locally or in an overall sense. The connection is to a 3⁄8-in.-thick gusset plate. Ignore the beam weight initially, then check for its effect after a selection is made. The basis of both types is a plate that is shop welded to the end of a beam and field bolted to a column or another beam. No: 133 kips > 130 kips b. The tie rod between ridge purlins must resist the load from all of the sag rods on either side. Use a plate 5⁄8 × 61⁄2. To be sure that we have found the lightest W12, try the next lighter one, a W12 × 53, with p = 3.32 × 10−3 and bx = 5.29 × 10−3. The tensile load at the top can distort the column flange (exaggerated in Figure 8.43c), creating additional load on the weld connecting the beam flange to the column flange. The labor costs associated with doubler plates and stiffeners may outweigh the additional cost of material for a larger column. To attach the flange plates to the web plate, angle shapes were used to transfer load between the two elements. D = 340 kips L = 670 kips 10⬘ 35⬘ 15⬘ 10⬘ Strong axis Weak axis FIGURE P4.7-5 4.7-6 Select a square HSS for use as a 15-foot-long compression member that must resist a service dead load of 35 kips and a service live load of 80 kips. This can be understood by considering the horizontal components of the tension fields shown in Figure 10.8. (The vertical components are resisted by the stiffeners.) The tension field in panel CD is balanced on the left side in part by the tension field in panel BC. Stiffener-to-column flange welds: Since the beam tension flange force is an issue (for both column flange bending and web yielding), the stiffener must be welded to the column flange. If web stiffeners were needed, pairs of angles were used for that purpose also. The procedure used in Example 4.15, which is based on AISC Specification E4(a), should always be used for double angles and tees. Ω 2.00 DESIGN OF TENSION MEMBERS The design of a tension member involves finding a member with adequate gross and net areas. We cover this topic in Chapter 6. LRFD Φ=f and Fnt′ = 1.3Fnt − Fnt frv ≤ Fnt φ Fnv (AISC Equation J3-3a) ΩFnt frv ≤ Fnt Fnv (AISC Equation J3-3b) where f = 0.75. For a wind direction that produces uplift, 0.9D – 1.0W = 0.9(85) – 1.0(56) = 20.5 kips The positive result means that the net load is compressive, and we need not consider this load combination further. Griffis, Lawrence G. The web thickness tw can then be found by using the following limitations as a guide. As previously mentioned, the two properties usually needed in structural steel design are Fu and Fy, regardless of the shape of the stress–strain curve and regardless of how Fy was obtained. 510 kips 8.5-1 1.57 kips鼻in. (The upper limit of Mpx鼻Ωb controls.) b. The steel is A36. All steel is A36. For LRFD, the design strength is fc Pn , where fc = 0.75. From AISC J3.3, ⎛ 7⎞ Minimum spacing = 2.667 ⎜ ⎟ = 2.33 in. 14th ed. To which shapes does this value apply? To determine which one controls, compute the strength corresponding to each axis and use the smaller value. Ruddy, John L. Built-up welded shapes, however, can have noncompact or slender webs as well as noncompact or slender flanges. (Wind or earthquake load may tend to overturn a structure, but the dead load will have a stabilizing effect.) As previously mentioned, the load factor for a particular load effect is not the same in all load combinations. Eight bolts connect the tee to the column. A frame must resist not only the tendency to sway under the action of lateral loads but also the tendency to buckle, or become unstable, under the action of vertical loads. The length of the welds in this case may not be less than the distance between them—that is, L ≥ W. Adequate: Mu = 19,500 ft-kips < f bMn = 20,900 ftkips b. For reasons to be discussed later, doubly symmetric shapes such as the standard rolled W, M, and S shapes are the most efficient. 4 spaces Then, from Equation 4.14, 48 Ka Ka = = = 74.77 < 0.75(154.8) = 116.1 (OK) 0.642 ri rz Compute the modified slenderness ratio, (KL鼻r)m: 48 a = = 74.77 > 40 ri 0.642 K i a 0.5(48) = = 37.38 0.642 ri 2 ∴ Use AISC Equation E6-2b 2 ⎛ KL ⎞ = ⎛ KL ⎞ + ⎛ K i a ⎞ = ⎝ r ⎠m ⎝ r ⎠ o ⎜⎝ ri ⎟⎠ (154.8)2 + (37.38)2) = 159.2 This value should be used in place of KL鼻ry for the computation of Fcry : Fe = Since π 2E π 2 (29, 000) = = 11.29 ksi (KL鼻r)2 (159.2)2 KL E > 4.71 = 134, r Fy Fcry = 0.877 Fe = 0.877(11.29) = 9.901 ksi From AISC Equation E4-3, Fcrz = GJ 11, 200(2 × 0.322) = = 152.6 kssi Ag r o2 7.50(2.51)2 Fcry + Fcrz = 9.901 + 152.6 = 162.5 ksi 4.9 Fcr = ⎛ ⎞ = Built-Up Members 169 Fcry + Fcrz ⎞ ⎡ 4 Fcry Fcrz H ⎤ 1− 1− ⎢ 2H ⎠ ⎣ (Fcry + Fcrz)2 ⎦ 162.5 ⎡ 4(9.832)(152.6)(0.646) ⎤ 1− 1− ⎢ ⎥ = 9.599 ksi 2(0.646) ⎣ (162.5)2 ⎦ The nominal strength is Pn = Fcr Ag = 9.599(7.50) = 71.99 kips Therefore the flexural-torsional buckling strength controls. The loads are 50% live load and 50% dead load. PD = 20k PL = 10k wD = 7k/ft wL = 3k/ft 10′ 20′ 30′ FIGURE P5.10-3 Problems 289 5.10-4 Same as Problem 5.10-3, except that lateral support is provided only at the ends and at the concentrated load. Equating the required section modulus to the approximate value, we have M n req t w h 2 = + Af h 6 Rpg Fcr Af = M n req t h − w 6 hRpg Fcr If we assume that Rpg = 1.0 and Fcr = Fy, the required area of one flange is Af = M n req Aw − 6 hFy (10.5) 10.7 Design 697 where Mn req = Required nominal flexural strength = Mu鼻fb for LRFD = ΩbMa for ASD Aw = web area Once the required flange area has been determined, select the width and thickness. In this book, we consider deformation to be a design consideration. Use partial composite action and do not interpolate in the composite beam tables. 656 Chapter 9 Composite Construction 9.1-5 Compute the nominal flexural strength of the composite beam in Problem 9.1-2. CHAPTER 4 Compression Members 4.1 INTRODUCTION Compression members are structural elements that are subjected only to axial compressive forces; that is, the loads are applied along a longitudinal axis through the centroid of the member cross section, and the stress can be taken as f = P/A, where f is considered to be uniform over the entire cross section. Jean Buttrom Production Service: RPK Editorial Services, Inc. First, the reduction in the cross section is usually small. If an 8-inch-diameter standard weight steel pipe full of water is used, what size A36 steel rod is required? Alternatively, the loads can be factored at the outset: wu = 1.2 wD + 1.6 wL = 1.2(0.481) + 1.6(0.550) = 1.457 kips鼻ft 1 1 Mu = wu L2 = (1.457)(30)2 = 164 ft-kips 8 8 The design strength is fb Mn = 0.90(225.0) = 203 ft-kips > 164 ft-kips ANSWER ASD SOLUTION (OK) The design moment is greater than the factored-load moment, so the W16 × 31 is satisfactory. or s = 4.58 in. To compute the bending strength reduction factor Rpg, the value of aw will be needed. Maximum longitudinal spacing is 8t = 8(4.5) = 36 in. The bolts are 3⁄4-inch Group A bearing-type with the threads in the plane of shear. In the end panels, a is less than 12 feet because of the double stiffeners at the supports.) Because a鼻h is greater than 1.5, AISC Equation F13-4 applies: 0.40 E 0.40(29, 000) h = = 322 > tw Fy 36 (OK) 684 Chapter 10 TABLE 10.1 Plate Girders Component A I¯ d I¯ + Ad 2 Web Flange Flange — 16 16 7814 — — — 32 32 7,814 16,380 16,380 40,574 a. The analysis gives the required strength of members. In terms of effective length, the Euler load can be written as Pe = p 2 EI (KL)2 = p 2 EI x (K x L)2 = p 2 (29, 000)(272) (1.00 × 17 × 12)2 = 1871 kips 310 Chapter 6 Beam–Columns From the LRFD solution to Example 6.1, Pu = 200.4 kips, and 1 1 = = 1.12 1 − (Pu 鼻Pe) 1 − (200.4鼻1871) which represents a 12% increase in bending moment. Although partial composite design can be used without formed steel deck, it is covered here because it is almost a necessity with formed steel deck. w W33 × 141 25′ FIGURE P5.5-2 5.5-3 A simply supported beam (Figure P5.5-3) is subjected to a uniform service dead load of 1.0 kips鼻ft (including the weight of the beam), a uniform service live load of 2.5 kips鼻ft, and a concentrated service dead load of 45 kips. Amrine, J.J. and Swanson, J.A. 2004, "Effects of Variable Pretension on the Behavior of Bolted Connections with Prying." Engineering Journal, AISC 41 (no. Maximum L 20(12) = = 122.4 r 1.96 Pcr = ANSWER p 2 EA (L鼻r)2 = p 2 (29, 000)(14.6) (122.4)2 = 278.9 kips Because the applied load of 145 kips is less than Pcr , the column remains stable and has an overall factor of safety against buckling of 278.9鼻145 = 1.92. The horizontal members AB and BC are subjected primarily to bending, or flexure, and are called beams. SOLUTION K x L 24(12) = = 54.55 5.28 rx K y L 8(12) = = 38.25 2.51 ry Kx L鼻rx , the larger value, controls. (Different manufacturers will produce this shape with different values of k. This determination is the basis for the two major categories of connections. Figure 6.16 further illustrates the superposition concept. Ae = AgU = 7.50(0.80) = 6.000 in.2 Pn = Fu Ae = 58(6.000) = 348.0 kips For A36 steel, the appropriate electrode is E70XX, and 3 in. When connected by bolts (or any other fasteners, such as nails), the two planks will behave as a unit, and the resistance to slip will be provided by shear in the bolts. These constants are evaluated by applying the following boundary conditions: At x = 0, y = 0: 0 = A cos(0) + B sin(0) At x = L, y = 0: 0 = B sin(cL) A=0 This last condition requires that sin(cL) be zero if B is not to be zero (the trivial solution, corresponding to P = 0). The span length is 22 feet and the joist spacing is 4 feet. The relevant shape here is the WT5 × 30, and the distance from the outside face of the flange to the centroid is 0.884 inch, as shown in Figure 5.8. FIGURE 5.8 a = d − 2(0.884) = 10.2 − 2(0.884) = 8.432 in. Adequate: Result of interaction equation = 0.952 6.6-9 a. This location will make the strength analysis somewhat more difficult than if the PNA were in the slab, but the basic principles are the same. To check web crippling strength, use AISC Equation J10-4: 1.5⎡ b ⎛ t w ⎞ ⎤ EFyw t f fRn = f 0.80 t w2 ⎢1 + 3 ⎛⎜ ⎞⎟ ⎜ ⎟ ⎥ ⎝ d ⎠ ⎝ tf ⎠ tw ⎥⎦ ⎣ 1.5 ⎡ 0.525 ⎞ ⎛ 0.360 ⎞ ⎤ 29, 000(50)(0.560) = 0.75(0.80)(0.360)2 ⎢1 + 3 ⎛⎜ ⎟ ⎜ ⎟ ⎥ 8.25 ⎠ ⎝ 0.560 ⎠ ⎦ 0.360 ⎣ = 128.3 kips < 134.7 kips (N.G.) ∴ Stiffeners are required to prevent web crippling. Is the tee-to-column connection adequate? AISC Equation I2-2 then becomes ⎛ Fy ⎞ ⎡ ⎤ ⎢ F ⎥ Pn = As 0.658⎝ e ⎠ ⎢ Fy ⎥ ⎣ ⎦ (9.7) and AISC Equation I2-3 becomes Pn = As(0.877)Fe (9.8) Equations 9.7 and 9.8 give the same strength as the equations in AISC Chapter E for noncomposite compression members. A 30′ B 4 @ 5′ = 20′ FIGURE P5.11-5 5.11-6 Use LRFD and design a typical girder for the floor system of Problem 5.11-5. 32 + 2(2.5) and e = 3 − 0.1689 = 2.831 in. ASTM A36 is usually specified for angles, plates, S, M, and channel shapes; A572 Grade 50 for HP shapes; and A992 for W shapes. The bearing strength is Rn = 1.8Fy Apb = 1.8(36)(0.75)(8 − 0.5) × 2 = 729.0 kips fRn = 0.75(729.0) = 547 kips > 223.4 kips Check the stiffener as a column. For ASD, the allowable strength is Pn 鼻Ωc = 3274鼻2.00 = 1637 kips. The connections of these elements will now be designed. They can also simultaneously serve as intermediate stiffeners. Ω where D is the weld size in sixteenths of an inch. Determine kv and Cv. kv = 5 + 5 (a 鼻 h) 2 = 5+ 5 (2.286)2 = 5.957 Establish the range of h鼻tw: 1.10 kv E 5.957(29,000) = 1.10 = 76.20 Fy 36 1.37 kv E 5.957(29,000) = 1.37 = 94.90 Fy 36 688 Chapter 10 Plate Girders Since h鼻tw = 168 > 1.37 k v E鼻Fy , Cv is found from AISC Equation G2-5: Cv = 1.51kv E 1.51(5.957)(29,000) = = 0.2567 2 (h鼻t w) Fy (168)2 (36) AISC Equation G3-2, which accounts for tension-field action, will be used to determine the nominal shear strength (except for the end panels): ⎛ ⎞ 1 − Cv Vn = 0.6 Fy

Aw ⎢ Cv + ⎢⎝ 1.15 1 + (a 鈭 h)2 ⎥⎦ ⎡ 1 − 0.2567 = 0.6(36)(23.63) ⎢ 0.2567 + ⎥ = 263.2 kips ⎢⎣ 1.15 1 + (2.286)2 ⎥⎦ The design shear strength is fvVn = 0.90(263.2) = 237 kips From Figure 10.12, the maximum factored load shear in the middle half of the girder is 102 kips, so the shear strength is adequate where tension-field action is permitted. Since S is larger than Lr, and R = 0, this combination reduces to D + S = 109 + 20 = 129 kips D + 0.75L + 0.75(Lr or S or R). The axial dead load supported by column AB is 155 kips, and the axial live load is 460 kips. >1.0 in. 1922. If a steel column is connected to girders having large cross sections, the girders will effectively prevent rotation of the column. The service dead loads consist of a 5-inch-thick reinforced-concrete floor slab (normal weight concrete), a partition load of 20 psf, and 10 psf to account for a suspended ceiling and mechanical equipment. Once all rectangular components have been found, they can be added vectorially to obtain the resultant shearing stress at the point of interest, or fv = (∑ f x) 2 + (∑ f y) 2 As with bolted connections, the critical location for this resultant stress can usually be determined from an inspection of the relative magnitudes and directions of the direct and torsional shearing stress components. A doubler plate with sufficient thickness to make up the deficiency can be welded to the web or a pair of diagonal stiffeners can be welded to the web. The strength of a fastener depends on whether it is subjected to shear or tension, or both. It is therefore essential that students have a copy of the 14th edition of the Steel Construction Manual. One Ӎ × 3 stud will be used every third rib. West, M.A. and Fisher, J.M. 2003. From the charts, the first solid curve above the 810 0 ft-kip mark is for a W24 × 84, the same as selected in Example 5.10. Figure 5.31 shows a typical floor framing plan for a multistory building. SOLUTION The wood-truss system is the same type as the one shown in Manual Table 8-8 (angle = 0°.), and the loading situation is similar. The length is 20 feet, and the ends are pinned. Southern Building Code Congress International. In all girders without web stiffeners, AISC F13.2 requires that 鈭tw be no greater than 260 and that the ratio of the web area to the compression flange area be no greater than 10. Continuous lateral support is provided. The torque required to attain a specified tension in a bolt of a given size and grade is determined by tightening this bolt in a tension-indicating device. 2 Line abcd: An = Ag − t w (d for hole at b) − t w (d ′ for hole at c) = 3.82 − 0.437 ANSWER (43) − 0.437 ⎡⎢ (43 − (42()3) ⎤⎥⎦ = 3.31 in. This shape can be fabricated from plates as shown in the figure, or built up from four angles placed back to back. Determine whether the bolts are adequate. Whenever possible, the designer should provide extra support for the weak direction of a column. Instead of giving the nominal strength, Mn, however, both the allowable moment strength Mn鈭Ωb and the design moment strength fb Mn are given. (The grade of steel need not be known: The critical buckling load is a function of the modulus of elasticity, not the yield stress or ultimate tensile strength.) 114 Chapter 4 Compression Members For a W10 × 50, Minimum r = ry = 1.96 in. Structural Stability Research Council. = 1 1 + (45/65) Weight of deck and slab = 51 psf + construction load = 20 psf + partition load = 20 psf + miscellaneous dead load = 10 psf + live load = 80 psf • Fy = 50 ksi, fc′ = 4 ksi Assume continuous lateral support. The use of a maximum permissible drift index is a serviceability requirement that is similar to a limit on beam deflection. With biaxial bending and no axial load, the formula for small axial load is the appropriate one, and FIGURE 5.47 y x 268 Chapter 5 Beams it reduces to the form of Equation 5.21. ANSWER Use a W14 × 132. When using the column load tables for unsymmetrical shapes, there is no need to account for slender compression elements, because that has already been done. A., and Galambos, T. Select the girder cross section and the required spacing of intermediate stiffeners. One reason for the popularity of steel headed stud anchors is their ease 593 594 Chapter 9 Composite Construction FIGURE 9.1 of installation. Solution manual Steel Structures : Behavior and LRFD (1st Ed., Ramulu Charles Salmon) Solution manual Reinforced Concrete Design (7th Ed., ChuKia Wang, products, you can visit this great site that provides many structural steel design 5th edition mccormac solution manual. The corresponding rotations at the plastic hinges will be as shown in Figure 4.5b, and the vertical displacement of the load will be 10q. The calculated load can then be multiplied by 0.707 times the weld size to obtain the actual load. LRFD: The dead weight of the truss members is: wD = 0.75(0.707 wFnw) = 0.75(0.707) ⎛⎝ (0.6 × 70) = 1.392 kips鈭in. 10 Chapter 1 Introduction Steel exhibiting the behavior shown in Figure 1.3b is called ductile because of its ability to undergo large deformations before fracturing. The loading consists of 180 lb鈭ft dead load (not including the weight of the beam), 95 lb鈭ft roof live load, 275 lb鈭ft snow load, and 180 lb鈭ft wind load acting upward. 15♠ HSS 8 × 8 × 1/4 ASTM A500, Grade B steel (Fy = 46 ksi) FIGURE P4.3-4 4.3-5 A W18 × 119 is used as a compression member with one end fixed and the other end fixed against rotation but free to translate. SOLUTION Slenderness ratio: Maximum 4.71 KL KL 1.0(20 × 12) = = = 96.77 < 200 r ry 2.48 E 29, 000 = 4.71 = 113 Fy 50 Since 96.77 < 113, use AISC Equation E3-2. 5.15 Biaxial Bending 269 EXAMPLE 5.18 A W21 × 68 is used as a simply supported beam with a span length of 12 feet. When wind effects are included and consideration is given to different wind directions, the force in some web members may alternate between tension and compression. At the instant of failure, equilibrium must be maintained, and the following three equations of equilibrium will be applied to the fastener group (refer to Figure 8.7): n ∑ Fx = ∑ (Rn)x − Px = 0 n = 1 (8.3) 8.2 Eccentric Bolted Connections: Shear Only 485 m M IC = Pr0 + e) − ∑ (rn × Rn) = 0 n = 1 (8.4) and n ∑ Fy = ∑ (Ry) N − Py = 0 n =1 (8.5) where the subscript n identifies an individual fastener and m is the total number of fasteners. the distance from the beam centerline to the edge of the slab. For example, the ultimate tensile strength of a Group A bolt is 120 ksi, so the nominal shear strength with the threads not in the shear plane is Fnv = 120(0.625)(0.90) = 67.5 ksi If the threads are in the plane, Fnv = 0.8(67.5) = 54 ksi The nominal shear strength of ASTM A307 bolts is based on the assumption that the threads will always be in the plane of shear. There are many other possibilities, some of which we illustrate throughout this book. We will assume class A surfaces, and for a 5⁄8-inch-diameter Group A bolt, the minimum tension is Tb = 19 kips FIGURE 7.19 = 3⁄8 in. For any size and type of bolt, therefore, the number of turns of the nut required to produce a given tensile force can be computed. They are periodically revised and updated by the issuance of supplements or completely new editions. Classification of Shapes 5.4-1 Determine whether a W14 × 90 is compact, noncompact, or slender for Fy = 60 ksi. 1 1 γm = 47.3t = 47.3 ⎛⎝ ⎞⎠ = 23.7 kips鈭bolt ⎝ 4 ⎠ To obtain the strength of the connection based on shear and bearing, we consider the total strength at each bolt location (AISC J3.6 and J3.10 User Notes). The number of studs required between the end of the beam and midspan is N1 = V ′ 384.0 = = 22.3 Qn 17.23 Use 23 for half the beam, or 46 total. r Fy Fcr = 0.877 Fe = 0.877(21.42) = 18.79 ksi 0.6 Fcr Ag = 0.6(18.79)(38.3) = 432 kips < 400 kips (OK) This shape is not slender (there is no footnote in the dimensions and properties table to indicate that it is), so local buckling does not have to be investigated. If there is a reduction in strength because of the holes, what is the percent reduction? The computation of flexural strength is illustrated in Example 10.1, Part a. All plates are to be shop welded to the column with E70XX electrodes and field bolted to the beam with 7⁄8-inch-diameter, Group A bolts. Although the charts for compact and noncompact shapes are similar in appearance, Mp and Lp are used for compact shapes, whereas M′p and L′p are used for noncompact shapes. If the compression flange is restrained against rotation, the nominal strength is 3 Cr t w3 t f ⎡ ⎛ h鈭t w ⎞ ⎤ Rn = ⎢1 + 0.4 ⎜ ⎟ ⎥ ⎛ Lb 鈭b f ⎞ ⎦ 3 ⎣ ⎝ Lb 鈭b f ⎠ h2 ⎢⎣ (AISC Equation J10-6) [This equation need not be checked if (h鈭tw)鈭(Lb鈭bf) > 2.3.] If the flange is not restrained against rotation, Rn = Cr t w3 t f h2 ⎡ ⎛ h鈭tw ⎞ 3 ⎤ ⎢ 0.4 ⎜ ⎟ ⎥ ⎣ ⎝ Lb 鈭b f ⎠ ⎦ (AISC Equation J10-7) (This equation need not be checked if (h鈭tw)鈭(Lb鈭bf) > 1.7.) where Cr = 960,000 ksi when Mu < My (for LRFD) or 1.5 Ma < My (for ASD) = 480,000 ksi when Mu ≥ My (for LRFD) or 1.5 Ma ≥ My (for ASD) (All moments are at the location of the load.) Lb = largest unbraced length of either flange (at the load point) For LRFD, f = 0.85. A 4-inch-thick reinforced-concrete floor slab of normal-weight concrete is supported by floor beams spaced at 7 feet. From AISC Equation F9-5, ⎛ d ⎞ B = 2.3 ⎜ ⎟ ⎝ Lb ⎠ Iy ⎝ (6.25 ⎞ = −2.3 ⎜ ⎟ 10 × 12 ⎠ J 12.2 = −0.6888 0.369 Problems 363 (The minus sign is used when the stem is in compression anywhere along the unbraced length.) From AISC Equation F9-4, M n = Mcr = p EI y GJ ⎡⎣ B + 1 + B 2 ⎤⎦ ⎡ ⎤ Lb p 29, 000(12.2)(11, 200)(0.369) ⎣ − 0.6888 + 1 + (− 0.6888)2 ⎦ 10 × 12 = 526.0 in.-kips = The limit state of web local buckling controls. Murphy, G. Compression in the flange of the structural tee will be reduced, resulting in a distortion df in the same sense and amount as db. The allowable shear rupture strength per unit length is Rn 0.6 Fu t 1.50 Ω and for a one-inch length (7.38) the base metal allowable shear rupture strength per unit length is Rn 0.6 Fu t = = 0.3Fu t Ω 2.00 for a one-inch length (7.38) To summarize, for E70 electrodes, the design strength of a fillet weld can be taken as ⎛ 1.392 D ⎞ ⎝ fRn = min ⎛⎝ 0.6 Fu t ⎞ ⎠ ⎟ ⎠ kips鈭in. This is a function of Pu鈭Ag and can be found in Table 4-21 in Part 4 of the Manual. Applying the parallel axis theorem and tabulating the computations in Table 9.2, we obtain the moment of inertia of the transformed section as Itr = 1530 in.4 TABLE 9.1 Component Concrete W16 × 36 A y Ay 54.40 10.6 2.50 12.95 136.0 137.3 65.00 TABLE 9.2 Component Concrete W16 × 36 273.3 A y − I d − I + Ad 2 54.40 10.60 2.50 12.95 113.3 448.0 1.705 8.745 271.4 1259.0 1530.4 9.1 Introduction 599 The distance from the neutral axis to the top of the steel is yt = y − t = 4.205 − 5.000 = − 0.795 in. Variable flange thicknesses can also be considered. Deflections are always investigated at service loads, and in some cases, the available strength is based on the limit state of first yield. The steel has a yield stress of 50 ksi. The calculations for y-, which is the distance from the top of the steel shape to the tensile force, are summarized in Table 9.6. The depth of the compressive stress block in the concrete is a= C 258.5 = = 0.8448 in. Thus, for compact, laterally-supported beams, the allowable stress can be conservatively taken as 0.66Fy. (This value of allowable stress has been used in ASD allowable stress design specifications since 1963.) We can formulate an allowable stress approach that requires no approximation if we use the elastic section modulus instead of the plastic section modulus. Check Equation 6.9: p = 2.22 × 10−3, bx = 3.92 × 10−3, by = 8.38 × 10−3 pPu + bx Mux + by Muy = (2.22 × 10−3)(150) + (3.92 × 10−3)(75) + (8.38 × 10−3)(30) = 0.878 < 1.0 (OK) Try the next smaller shape. To determine the required length of the longitudinal welds, we investigate the two options specified in AISC J2.4c. Depending on the size of the structure, more than one bay may require bracing. ANSWER Allowable strength of the weld is 35.6 kips. Compute the allowable flexural strength Mn鈭Ωb. For full composite action, the compressive force, C, in the concrete is the smaller of As Fy = 7.68(50) = 384.0 kips or 0.85fc′Ac = 0.85(4)[90(4.75 − 1.5)] = 994.5 kips where only the concrete above the top of the deck has been accounted for in the second equation, as illustrated in Figure 9.18. As the load P is gradually applied, the compression at the top will be relieved and the compression at the bottom will increase, as shown in Figure 8.13a. Equate the required shear strength to the shear strength given by AISC Equation G2-1 and solve for the required value of Cv. b. Research Council on Structural Connections. Use LRFD and Fy = 50 ksi for all components. deck width 3 in. For the bolt hole edge distance, use AISC Table J3.4. For 7⁄8-inch diameter bolts, Minimum Le = 11⁄8 in. Try Lc × 4 × 1⁄2. The design moment for a partially restrained connection corresponds to the intersection of the beam line with the moment-rotation curve for the partially restrained connection. Ruby, J. These include 396 Chapter 7 Simple Connections bearing-type connections (see Section 7.6 of this book), most tension connections (Section 7.8), and most combined shear and tension connections (Section 7.9). Select a round HSS. Studs are the most commonly used type of anchors, and more than one can be used at each location if the flange is wide enough to accommodate them (which depends on the allowable spacing, which we consider in Section 9.4). This table gives values of fcFcr (for LRFD) and Fcr鈭Ωc (for ASD) as a function of KL鈭r for various values of Fy. This table stops at the recommended upper limit of KL鈭r = 200. The bending factor is ⎛ M) ⎛ 90 ⎞ Cm = 0.6 − 0.4 ⎜ 1 ⎟ = 0.6 − 0.4 = 0.2565 ⎝ M2 ⎠ ⎝ 104.8 ⎠ For the axis of bending, Pe1 = = 2 EI* π 2 EI x* = (K1L) 2 (K x L) 2 The modified stiffness is EIx* = 0.8tb EI = 0.8(1.0)EIx = 0.8EIx Therefore, Pe1 = p 2 EI x* π 2 (0.8 EI x) π 2 (0.8)(29, 000)(533) = 3767 kips = = (1.0 × 15 × 12)2 (K x L) 2 (K x L)2 The amplification factor for nonsway moments is B1 = 0.2565 Cm Cm = = 0.292 < 1.0 ∴ Use B1 = 1.0. 1 − (a Pr 鈭Pe1) 1 − (1.00 Pu 鈭Pe1) 1 − (454 鈭3 3767) Mr = Mu = B1 Mnt + B2 Mlt = 1.0(104.8) + 0 = 104.8 ft-kips The required axial compressive strength is Pr = Pu = Pnt + B2Plt = 454 + 0 = 454 kips From the beam design charts, with Lb = 15 ft, Cb = 1.0, Mn = 340 ft-kips Mp = 356 ft-kips (for Cb = 1.0) Figure 6.19 shows the bending moment diagram for the gravity-load moments. This is possible if the member connections do not transmit moment; in other words, if the column is not part of a rigid frame. 0.90 Fy BN 0.90(36)(13)(13) Use a PL 1 × 13 × 13. Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications. Ω 2.00 The required strength is Ra = D + L = 15 + 45 = 60 kips < 61.2 kips ANSWER (OK) Bearing strength, spacing, and edge-distance requirements are satisfied. The applied load is Pa = D + L = 98 + 145 = 243 kips. Specification for Structural Steel Buildings: Allowable Stress Design and Plastic Design. 3): 108–110. In other words, the flexural buckling strength could be reduced to account for local buckling (AISC 12.2b). 12): 2646–2664. These members prevent translation of the column in all directions, but the details of which are not shown, permit small rotations to take place. Example 6.9 illustrates the design of a beam–column of a specific depth. 240 240 ANSWER ASD SOLUTION (OK) Use to exceed a value of 0.75Fu. Load restraint in a W12 × 26 and 20 studs, 3⁄4 × 3, one every third rib. Most shapes will also satisfy the flange requirement and will therefore be classified as compact. Often the overall nominal dimensions will also be known because of architectural or other requirements. For the unit weight of reinforced concrete, we use the weight of plain concrete plus 5 pcf. At midheight, the column is braced, but only in one direction. The distinction between different types of connections can be made by examining the degree of relative rotation that can take place between the connected members. If A572 Grade 50 steel is used, what is the nominal flexural strength? The requirement for pinned ends, however, is a serious limitation, and provisions must be made for other support conditions. Determine whether this beam satisfies the provisions of the AISC Specification. 63.3 kips a. Although the charts are based on Cb = 1.0, they can easily be used for design when Cb is not 1.0; simply divide the required moment strength by Cb before entering the charts. Vu = ASD SOLUTION wu L 2.080(45) = = 46.8 kips < 185 kips (OK K) 2 2 Determine the safety factor Ωv. Try a W12 ᵭ 58, with p = 2.01 × 10−3 and bx = 4.16 × 10−3. 638 Chapter 9 Composite Construction The compressive force in the concrete is the smaller of As Fy = 10.6(50) = 530 kips or 0.85fc′Ac = 0.85(4)(5 × 87) = 1487 kips Use C = 530 kips. For ordinary buildings, the additional creep deflection caused by sustained dead load is very small, and we will not cover it in this book. 5.5 BENDING STRENGTH OF COMPACT SHAPES A beam can fail by reaching Mp and becoming fully plastic, or it can fail by 1. Section FIGURE P3.2-6 3.2-7 A C8 × 11.5 is connected to a gusset plate with 7⁄8-inch-diameter bolts as shown in Figure P3.2-7. The flange weld strength is bf b = (bf b − tw) = 6.00 + (6.00 − 0.300) = 11.70 in. Square structural tubes (HSS) are efficient shapes for compression members because the radius of gyration about each axis is the same for both axes. The loads are 50% dead load. = min. If the member has a bolted connection, the selection of a suitable cross section requires an accounting for the area lost because of holes. There is also available Manual For The Seismic Design Of Steel Concrete Buildings To · Seismic Design Design Of Machine Design Shigley Solution Manual · Professional. When a plastic hinge forms at a support where members of unequal strength meet, it will form when the bending moment equals the plastic moment capacity of the weaker member. 5.5-15 Determine whether a W30 × 99 of A992 steel is adequate for the beam shown in Figure P5.5-15. 9.4-3 36 8.4-5 a. M a = Pe = 15(2.375) = 35.63 ft-kips M a c 35.63(3) = = 2.969 kips鈭in. 8.6-2 A W16 × 45 beam is connected to a W10 × 45 column as shown in Figure P8.6-2. The column is pinned at both ends. Assume that Ae = 0.75An. a. Sag rods are located midway between the trusses. The equivalent system thus obtained will consist of the given force acting through the shear center plus a twisting moment, as shown. SOLUTION Bolt shear strength: From Manual Table 7-1, for 3⁄4-inch Group A bolts, type N (threads included in shear plane), and S (single shear), frn = 17.9 kips鈭bolt (These tables use a lowercase r to denote the strength of an individual bolt.) Slip-critical strength: From Manual Table 7-3 (for Group A bolts and μ = 0.30), use STD (standard hole) and S (single shear). Birkemore, Peter C., and Gilmor, Michael I. For the beam, Fy = 50 ksi, and Fy = 36 ksi for the plates. 286 Chapter 5 Beams PD = 16k PL = 32k wD = 1k/ft wL = 3k/ft A C B 10′ 20′ 30′ FIGURE P5.5-15 5.5-16 The beam shown in Figure P5.5-16 is laterally braced at A, B, C, and D. © Carsten Flieger/Alamy Large beams fabricated with steel plates are called plate girders. Bearing Stiffeners 10.6-1 An interior bearing stiffener consists of two plates, 1⁄2 inch × 6 inches, one on each side of a 5⁄16-inch × 56-inch plate girder web. For continuous lateral support, Lb can be taken as zero. When the bending moment is large enough to cause the entire cross section to yield, any further increase in moment cannot be countered, and the plastic hinge has formed. As long as the connected parts (the two flanges) remain in contact, the bolt deformation, db, and the flange deformation, df , will be equal. 1307 Solutions. Its derivation is given here to illustrate the importance of the end conditions. The required axial load strength is Pa = PD + PL = 25 + 75 = 100 kips The moments at each end are Mntx = MD + ML = 12.5 + 37.5 = 50.0 ft-kips Mnty = MD + ML = 5 + 15 = 20.0 ft-kips If we assume that B1 = 1.0, the required bending moment strengths are Max = B1Mntx = 1.0(50) = 50 ft-kips May = B1Mnty = 1.0(20) = 20 ft-kips Since the unbraced length for compression and the unbraced length for bending are the same: KL = Lb = 15 ft From Table 6-1, try a W10 × 49, with p = 3.35 × 10−3, bx = 6.54 × 10−3, by = 12.6 × 10−6. If the beam is not laterally braced at the load point (in such a way as to prevent relative lateral displacement between the loaded compression flange and the tension flange), the Specification requires that web sidesway buckling be investigated (AISC J10.4). The major flaw in the analysis is the implied assumption that the fastener load–deformation relationship is linear and that the yield stress is not exceeded. 1977. The plastic moment capacity, which is the moment required to form the plastic hinge, can easily be computed from a consideration of the corresponding stress distribution. For the gusset plate, tension failure would be along the transverse section df, and shear failure would occur on two longitudinal surfaces, de and fg. If doing so resulted in the equation for inelastic LTB being used when Lb was not really large enough, the strength based on FLB would control anyway. Note that in Example 5.13, the design was controlled by serviceability rather than strength. ⎛ 1 ⎞ The shear rupture strength Ω ⎛ ⎞ fRn = 0.45Fu t × 2 = 0.45(58) ⎜ ⎟ (2) = 16.31 kips鈭in. Trusses spaced at 18 feet Weight of roofing = 8 psf Snow load = 20 psf of horizontal projection W10 × 33 purlins located only at the joints Total estimated truss weight = 5000 lb 3.8-4 Use LRFD and design the tension members of the roof truss shown in Figure P3.8-4. Does this girder have enough shear strength? Try a W12 ᵭ 58, with p = 3.01 × 10−3 and bx = 4.71 × 10−3. How many Group A bolts are required? In most examples, both LRFD and ASD solutions are given. For sin(cL) = 0, cL = 0, p, 2p, 3p, . To compute stresses, we locate the neutral axis of this composite shape and compute the corresponding moment of inertia. Where possible, equations and expressions are expressed in non-dimensional form by leaving quantities such as yield stress and modulus of elasticity in symbolic form, thereby avoiding givens units. Compact shapes: The strength will be smaller values for the limit states of yielding and LTB. For more 276.3 ⎡ ⎤ Pn = Pno ⎢ 0.658 Pe ⎥ = ⎢ 0.658 294.4 ⎥ 186 = 186.5 kips ⎢⎣ ⎥⎦ ⎢⎣ ⎥⎦ ANSWER For LRFD, the design strength is fc Pn = 0.75(186.5) = 140 kips. All are compact. For a thickness expressed in inches, the slab weight is therefore (鈭t12)(150) psf. For the stiffener shear yield strength, ⎛ 5 ⎞ fRn = 0.6 Fy tst × 2 = 0.6(36) ⎜ ⎟ (2) = 13.5 kips鈭in. For uniform sagrod spacings, the formulas for continuous beams in Part 3 of the Manual can be used. Weld Symbols Welds are specified on design drawings by standard symbols, which provide a convenient method for describing the required weld. In such cases, top chord members will be subjected to significant bending as well as axial compression and must be designed as beam–columns (Chapter 6). Try 5⁄8-inch bolts: From Manual Table 7-1, with the threads not in the shear plane, the shear strength is frn = 12.4 kips鈭bolt No slip is permitted, so this connection is slip-critical. Because W and E are zero, this expression reduces to combination 4. Aminmansour, Abbas. These definitions of U will be presented here in a different format from that in the Specification. Ffu = 0.6Fy(0.60) = 0.6(50)(0.425) = 76.5 kips < 120.3 kips Therefore, use the actual flange force of 120.3 kips. 9th ed. These conditions could occur, for example, in a slack bracing rod subjected to wind loads. 1.5-7 b. The tensile force acts at the centroid of the 632 Chapter 9 TABLE 9.6 Composite Construction A Component W16 × 26 Flange segment 7.68 −0.2282(5.50) −1,255 Sum y = ∑ Ay 15.7 鈭 = 7.85 0.2282鈭2 = 0.1141 60.29 −14 6425 ∑ y ∑A = 60.15 = 15.362 in. The notation LLBB means "long-legs back-to-back," and SLBB indicates "short-legs back-to-back." When a double-shear section is used, various approaches are possible: (1) consider a single shape and double everything, or (2) consider two shapes from the outset. Buckling Strength of Metal Structures. The problem of torsion in rolled shapes is a complex one, and we resort to approximate, although conservative, methods for dealing with it. Consider the lap joint shown in Figure 7.4a. Before the truss members, Slab wt. • There is a uniformly distributed service load consisting of a live load of 0.7 kips鈭ft and a superimposed dead load of 0.3 kips鈭ft. Problems 399 Solutions. Steel that has been previously used is referred to here as an attempt to equalize. Concrete Structures, 4th ed. A PR connection for effect. In effect, however, the design can be summed up as follows: The architect decides how the building should look; the engineer must make sure that it doesn't fall down. As with the bolted connection in Section 8.3, the eccentric load P can be replaced by a concentric load P and a couple M = Pe. The shearing stress is fv = FIGURE 8.25 P A 8.5 Eccentric Welded Connections: Shear Plus Tension 513 where A is the total throat area of the weld. (OK) 384 EIx 384 29, 000(2370) Use a W24 × 84. Also shown in Figure 1.7 are hollow shapes, which can be produced either by bending plate material into the desired shape and welding the seam or by hot-working to produce a seamless shape. Uniform Building Code. Combination 1: Combination 2: ANSWER Combination 2 controls, and the factored load is 214.4 kips. Use the procedure of AISC Section E4(a). Miscellaneous Data and Mathematical Information This part includes properties of standard steel shapes in SI units, conversion factors and other information on SI units, weights and other properties of building materials, mathematical formulas, and properties of geometric shapes. ⎛ 16 ⎞ The base metal shear strength is therefore 6.750 kips鈭in. To transfer moment, plates are bolted to the beam flanges. From AISC Table B4.1b, 鈭= bf 2tf f λ p = 0.38 E Fy λr = 0.95 kc E FL kc = 4 but (0.35 ≤ kc ≤ 0.76) h鈭t w FL = 0.7Fy for girders with slender webss. Building code limitations on the depth-to-span ratio or the deflection could also influence the selection. Adequate: Result of interaction equation = 0.937 b. Problems Notes Unless otherwise indicated, the following conditions apply to all problems. As we discussed in Chapter 5 in conjunction with bending stresses, the sum of the load-to-resistance ratios must be limited to unity. From Figure 9.19, equilibrium of forces requires that C + Cs − T = 0 258.5 + Fy bf te − Fy (As − bf te) = 0 258.5 + 50(5.50)te − 50(7.68 − 5.50te) = 0 te = 0.2282 in. 2 50 Chapter 3 Tension Members The nominal strength based on the net area is Pn = Fu Ae = 58(1.514) = 87.81 kips a. Use a placement of welds that will eliminate eccentricity. We illustrate this procedure, known as balancing the welds, in Example 8.7. *Since Chapter J does not consider fatigue loading, Section J1.7 simply says that the eccentricity can be ignored. The critical buckling load, Pcr , is defined as the load that is just large enough to maintain the deflected shape when the temporary transverse load is removed. 731 Aminmansour, Abbas. From Manual Table 7-1, the shear strength is φrn = 24.4 kips鈭bolt and required bolt strength 108 = = 4.43 bolts Number of bolts required = strength per bolt 24.4 Eight bolts will furnish enough capacity, try six bolts so that a symmetrical layout with two gage lines of three bolts each can be used, as shown in Figure 7.16. Use A36 steel for the bracket, A992 steel for the W6 × 25, and perform the following analyses: (a) elastic analysis, (b) ultimate strength analysis. Get free premium access to download Seismic Design Manual. In general, the result will be no more accurate than the least accurate number used in the computation—and sometimes less because of round-off error. Neglect the weight of the beam and determine whether the beam satisfies the AISC Specification if A992 steel is used. For this type of connection, the length of the welds must be at least equal to the transverse distance between them, or 4 inches in this case. Because of the variability of Er, computation of Pcr in the inelastic range by the use of Equation 4.5 is difficult. "Inelastic K-factor for Column Design." Engineering Journal, AISC 10 (no. Otherwise, the interaction equation given by AISC Equations E3-2 and E3-3 must be reduced. The yield stress, Fy, is 60 ksi. No sag rods are used, so lateral support is at the ends only. FIGURE 5.21 (Unfortunately, there are two parts: unstiffened elements and stiffened elements. Printed in the United States of America 1 2 3 4 5 6 7 19 13 12 11 About the Author William T. 4 (OK) For computation of the bearing strength, use a hole diameter of 1 3 13 = + = in. Use Fy = 50 ksi. The value of Ω is actually 12⁄ 3 = 5鈭3 but has been rounded to 1.67 in the AISC specification. 2d ed. Two types of elements must be considered: unstiffened elements, which are 122 Chapter 4 Compression Members We consider three types of compression elements: unstiffened elements, which are supported along one edge. Quality welding can be more easily assured under the controlled conditions of a fabricating shop. As illustrated in Figure 5.47, the load can be resolved into rectangular components in the x- and y-directions, each producing bending about a different axis. Detailed coverage of this topic, as well as deflections caused by shrinkage of the concrete during curing, can be found in Viest et al. ST6): 1397–1426. Once yielding begins, the distribution of stress FIGURE 5.3 5.2 Bending Stress and the Plastic Moment 193 on the cross section will no longer be linear, and yielding will progress from the extreme fiber toward the neutral axis. (The web is compact for all shapes in the Manual for Fy = 65 ksi.) This shape can also be identified as compact because there is no footnote in the dimensions and properties table to indicate otherwise. Other commonly used processes for shop welding include gas shielded metal arc, flux cored arc, and electroslag welding. Try a W14 × 30: fb Vn = 177 ft-kips > 173 ft-kips (OK) Check the beam weight. In simply supported beam, only one plastic hinge is required. Take the ultimate shear yield stress, we can write the equation for the stress in the web at failure as fv = Vn = 0.6 Fy Aw where Aw = area of the web. Compute B1: p 2 EI x p 2 (29, 000)(16.0) = = 318.0 kips (K x L)2 (10 × 12)2 Cm 1.0 B1 = = = 1.395 a Pr 1.00(90) 1 − 1− Pe1 318.0 Pe1 = The amplified moment is Mu = B1Mnt + B2Mlt = 1.395(3.0) + 0 = 4.185 ft-kips The nominal moment strength of structural tees with compact or noncompact cross sections was introduced in Section 5.16 of this book and is covered in AISC F9. For the axis of bending, Pe1 = π2 EI (K1L)2 = p 2 EI x (K x L) 2 = p 2 (29, 000)(127) (110 × 12)2 = 2524 kips ⎛ aP ⎞ ⎛ 1.00 Pu ⎞ ⎛ 43.8 ⎞ Cm = 1 − 4 ⎜ r ⎟ = 1 − 0.2 ⎜ ⎟ = 1 − 0.2 ⎜ ⎟ = 0.9965 ⎝ Pe1 ⎠ ⎝ Pe1 ⎠ ⎝ 2524 ⎠ 320 Chapter 6 Beam–Columns The amplification factor is B1 = Cm Cm 0.9965 = = = 1.015 1 − (a Pr 鈭Pe1) 1 − (1.00 Pu 鈭Pe1) 1 − (44.8 鈭2524) The amplified moment is Mu = B1Mnt + B2Mlt = 1.015(112.7) + 0 = 114.4 ft-kips Because the beam weight is very small in relation to the concentrated live load, Cb may be taken from Figure 5.15c as 1.32. ANSWER A WT10.5 × 66 is satisfactory. Solution manual A Structures Primer (Harry F. 1980. The loading is the same for all frames. Therefore, Mn = 151 ft-kips Ωb The total axial compressive load is Pa = PD + PL = 35 + 99 = 134 kips The total transverse load is Qa = QD + QL = 5 + 12 = 17 kips The maximum bending moment is at midheight Ma = 17(17) = 72.25 ft-kips 4 Determine which interaction equation controls: 134 Pa = = 0.4963 > 0.2 Pn 鈭Ωc 270 ∴ Use Equation 6.5 (AIS SC Equation H1-1a). The limit states considered are yielding of the tension flange and buckling of the compression flange. The result then can be multiplied by the actual bolt strength to find the strength of the connection. For the inner holes and a spacing of 3 inches, the strength per bolt is 102t = 102 ⎛⎝ 1 ⎞ ⎛ ⎠ = 25.5 kips鈭bolt ⎝ 4 ⎠ For the holes nearest the edge of the tension member, use Manual Table 7-5 and an edge distance of 11⁄4 inches. Add one intermediate stiffener 56 inches from each end of the girder. 30k 3k 8′ 3k 8′ Section 16′ FIGURE P3.2-7 3.2-8 PL 1⁄2 × 7 FIGURE P3.5-12 3.5-12 The 30-kip concentrated load shown in Figure P3.5-12 is a service live load. Connections will fail when these are exceeded. Each flange force can be taken as H= M1 + M 2 dm where dm is the moment arm of the couple (for beams of equal depth on both sides of the column). EXAMPLE 7.10 A WT10.5 × 31 is used as a bracket to transmit a 60-kip service load to a W14 × 90 column, as previously shown in Figure 7.30. Load and resistance factor design (LRFD) is similar to plastic design in that strength, or the failure condition, is considered. If shoring is not used, however, the rolled steel shape must support not only its own weight, but also the weight of the slab and its formwork during the curing period. Because a unit width of weld is used, the computations for centroid and moment of inertia are the same as for a line. Figure 8.36a shows a uniformly loaded beam with simple supports, and Figure 8.36b shows the corresponding bending moment diagram. 2009. 1969. Try a W10 ᵭ 45, with p = 3.00 × 10−3, bx = 5.07 × 10−3, by = 11.7 × 10−3. The design of connections subjected to prying is essentially a trial-and-error process. This procedure is based on the load deformation characteristics of the connected parts. The shear center is the point on the cross section through which a transverse load on a beam must pass if the member is to bend without twisting. From Table 6-1 of the Manual, for Lb = 20 ft, bx = 6.29 × 10−3 bx 6.29 × 10−3 = = 3.77 × 10−3 Cb 1.67 For Cb = 1.0, bx = 4.16 × 10−3 ANSWER Since 4.16 × 10−3 > 3.77 × 10−3, use bx = 4.16 × 10−3. This distance, labeled c in Figure 10.9, should be between four and six times the thickness. SOLUTION The area of transformed concrete to be used is Ac = C 335 = = 6.7 in.2 Fy 50 622 Chapter 9 Composite Construction The thickness of this portion of the slab will be equal to a, the depth of the concrete stress block. Relative bracing can also be provided as follows: If a lateral cut through the frame not only cuts the column to be braced but also the brace itself, the bracing is relative. 115k L3 × 3 × 3⁄8 4″ FIGURE P6.4-7 4.8 Use an elastic analysis and compute the extra load in the weld (in kips per inch of length) caused by the eccentricity. The maximum total service load that can be applied if ASD is used. (Note that a smaller weld spacing is required for the intermediate stiffeners.) ANSWER Use 3⁄16-inch × 11⁄2-inch fillet welds for all bearing stiffeners, spaced as shown in Figure 10.25. 5.5-11 The beam shown in Figure P5.5-11 has lateral support at a, b, c, and d. AISC G3.1 lists all of the conditions under which a tension field cannot be used: a. This design procedure is roughly as follows. Since 6.49 × 10−3 > 4.57 × 10−3, use bx = 6.49 × 10−3. Modify bx to account for Cb. bx 4.35 × 10 −3 = = 2.60 × 10 −3 Cb 1.67 For Lb = 0, bx = 3.92 × 10−3. 1.93 bolts; use two. Bending about the minor axis: Mn = Mp = Fy Z ≤ 1.6My (AISC Equation F11-1) where My = yield moment = Fy S. Sidesway web buckling occurs when the tension flange buckles laterally. Pe1 = π 2 EI x π 2 (29, 000)(425) = = 3754 kips (K x L) 2 (1.0 × 15 × 12)2 M1 ⎞ ⎛ = 0.6 − 0.4 ⎛ = 0.6 Cm = 0.6 − 0.4 ⎜ ⎝ M2 ⎠ ⎝ M2 ⎠ From AISC Equation A-8-3, Cm Cm 1 = = B1 = ⎛ 1 − (a Pr Pe1) 1 − α ⎛(Pnt + Pt) Pe1 ⎞ 0.6 = 0.610 < 1.0 = 1 − 1.0[(58.4 + 0)] 3754 B1 = ∴ Use B1 = 1.0. Since B1 is the value originally assumed and B2 will not be reviewed, the previously computed value of Mu = 205.7 ft-kips is unchanged. If the shape is compact, check for lateral-torsional buckling as follows. FIGURE 9.19 C′ Cs −y T tf bf The moment strength (the internal resisting couple) can then be computed as the sum of the moments of the forces about any convenient location; we will take moments about the tensile force. Therefore, we will use the results of the LRFD solution: Required a = 53.9 in. Rn = 0.852(.784)(4 − 4) + 1.5(2.784)(4) = 35.6 kips Ω The larger value controls. Required weld length on the W-shape from one side of the web is Rn = 0A Fy ≤ × 2 sides of stiffener = 4(50)(0.360)(2) = 14.4 kips鈭in. For a beam, being serviceability always means that the deformations, primarily the vertical sag, or deflection, must be limited. The welds for connecting the components of a plate girder are designed in much the same way as for other welded connections. f (0.60 Fy dc) 0.90(0.60)(36)(8.25) where 134.7 − 80.19 is the extra strength, in kips, to be furnished by the doubler plate. FIGURE 6.14 15′ W12 × 65 A242 steel 6.6 Members in Braced Frames 323 SOLUTION First, determine the yield stress. For A242 steel is available in three different versions. For this beam, fRn = f(mDuhfTb ns) = 1.0(0.30)(1.13)(1)(20)(1) = 9.492 kips Setting the total factored-load shear to the reduced slip-critical strength for nb bolts, 4.741 ⎞ 75 = nb ⎛⎝ ⎠ − 1 (9.492) nb) = 9.492nb − 45.0 nb = 12.6 Since eight bolts are adequate for shear, bearing, and tension (with a reduced tensile strength), these limits also will not have to be checked. Yes: For end panel, Vu = 61.2 kips The total deflection must not exceed L 鈭180. AISC Design Guide 13 (Carter, 1999) contains detailed guidance on this topic. For plate thicknesses greater than 2 inches, use AISC Table J2.4. Problems 461 so the required length of the longitudinal weld is 41.69 = 6.61 in. We illustrate this technique in Example 5.12. If Lb is greater than Lr , the beam is subject to elastic LTB. Is a W18 × 35 of A992 steel adequate? The plate is ⁄8 × 8 in. The dimensions of the steel parts 3.1-1 Determine the properties of a built-up section. M = 10: 2.3-6 (18 − 10 b) ≥ 71.24 21.50 b ≥ 2.00 in. The load can then be entered in the upper row of fasteners. For load and resistance factor design (LRFD), Equation 2.6 can be written as Mu ≤ fbMn (5.1) where Mu = required moment strength = maximum moment caused by the controlling load combination from ASCE 7 fb = resistance factor for bending (flexure) = 0.90 189 190 Chapter 5 Beams FIGURE 5.1 Mn = nominal moment strength The right-hand side of Equation 5.1 is the design strength, sometimes called the design moment. The load to be carried by the longitudinal welds is 86.23 − 23.23 = 41.69 kips The required length of the longitudinal weld is 41.69 kips The AISC Specification covers this in Appendix 8, "Approximate Second-Order Analysis." The approach is the same as the one used in the ACI Building Code for reinforced concrete (ACI, 2008). The connection shown in Figure 7.3f produces both shear and tension in the upper row of fasteners. The nominal moment strength is the load required to cause yielding of the entire cross section. From Table 2-4 in Part 2 of the Manual, we see that A242 steel is available in three different versions. From the beam weight of 20 ft, reduce the hole diameter to ⁄8 in. Loads applied before the concrete cures: Slab: wslab = 0.0 ⎛ 1500 = 56.25 psf ⎞ ⎝ 12 ⎠ (Although normal-weight concrete weighs 145 pcf, we use 150 pcf.) For a beam spacing of 9 feet, the dead load is 56.25(9) = 506.3 lb鈭ft + Beam weight = 35.0 lb鈭ft 541.3 lb鈭ft The construction load is 20(9) = 180 lb鈭ft, which is treated as live load. Table C-A-7.1 in the Commentary to the Specification reveals that case (f) corresponds most closely to the end conditions for the sidesway case of this example and that Kx = 2.0. For the braced condition, Kx = 1.0 will be used. For example, consider a compression member pinned at one end and fixed against rotation and translation at the other, as shown in Figure 4.7. The Euler equation for this case, derived in the same manner as Equation 4.3, is 2.05π 2 EI L2 Pcr = or Pcr = FIGURE 4.7 π 2 EA 2.05π 2 EA (鈭r)2 (0.70 L鈭r)2 Pcr L Pcr 4.3 AISC Requirements 117 This compression member has the same load capacity as a column that is pinned at both ends and is only 70% as long as the given column. In most cases, the provisions for bearing in bolted connections and base metal shear in welded connections must be considered. Load from trusses is applied to the top chord at the joints only. The dead load of wood framing is 20 psf. For the other pertinent data are summarized as follows: Metal deck: Built-up roof: Snow: 2 psf 5 psf 18 psf of horizontal roof projection a. Check the bending strength of the trial section. The maximum permissible factored load. Both of the last two mechanisms evaluated in this design A.4 Design 727 FIGURE A.6 problem require the same controlling value of Mp and will therefore occur simultaneously. Modify bx to account for Cb. bx 3.45 × 10 −3 = = 2.07 × 10 −3 Cb 1.67 For Cb = 1.0, bx = 3.04 × 10−3. In practice, B1 will almost always be equal to 1.0. For beam–columns subject to sidesway, however, the amplification factor B2 is usually greater than 1.0 and must be determined as shown in Section 6.7. The computation of B2 will be reviewed here. In other words, Combination 6: 0.9D ± 1.0W Combination 7: 0.9D ± 1.0E Combinations 6 and 7 account for the possibility of dead load and wind or earthquake load counteracting each other; for example, the net load effect could be the difference between 0.9D and 1.0W or between 0.9D and 1.0E. Deflection is not a design consideration. Although the connection has a particular cross-sectional element, 1 for I shapes is either the ratio bf鈭t or h鈭tw, both of which are defined presently. For example, in equilibrium of forces requires the approach to the design of members subjected to a simple shear connection that is restrained. From Equation 8.5, ∑ Fy = ∑Ry − P = 2.78951 − 2.78951 = 0.000 The applied load has no horizontal component, so Equation 8.3 is automatically satisfied. Cross sections of some of the more commonly used hot-rolled shapes are shown in Figure 1.6. The dimensions and designations of the standard available shapes are defined in the ASTM standards (ASTM, 2010b). If we equate the weld strength to the flange force, 1392D × 11.70 × 1.5 = 120.3, D = 4.92 sixteenths From AISC Table J2.4, the minimum weld size is 3⁄16 in. Use Table 4-22 from Part 4 of the Manual. "Rules for Riveted Hole Deduction in Tension Members." Engineering News Record (Nov. The effective width of the slab is 90 inches. There is a 20-psf construction load, a partition load of 20 psf, a ceiling weight of 5 psf, a mechanical load of 5 psf, and a live load of 120 psf. (hc 鈭2 defines the part of the web that is in compression for elastic bending. The structural steel is A992, and the concrete strength is fc′ = 4 ksi. Connected through the web with four or more fasteners in the direction of loading: U = 0.70. The required weld size can be found by equating fr to the weld capacity per inch of length: fr = 1.392D 4.724 = 1.392D, D = 3.394 where D is the weld size in sixteenths of an inch for E70 electrodes. If stiffeners are used, a different equation for Vc from the design Guide 4 must be used. The column load tables in Part 4 of the Manual give the available strength for selected W-, HP-, single-angle, WT-, HSS, pipe, double-angle, and composite shapes. Use Cb = 1.0. bf = 7.64″ FIGURE 5.33 tf = 0.810″ W18 × 71 SOLUTION To determine the nominal flexural strength Mn , all applicable limit states must be checked. Approximately 0.0121 in. 65.6 kips b. Creep is a deformation that takes place when a material is subjected to a sustained stress. Deflections 621 place under sustained compressive stress. Most often, this selection will entail choosing a standard hot-rolled shape that is widely available rather than requiring the fabrication of a shape with unique dimensions and properties. What is the slope of this line? Double Angles Loaded in the Plane of Symmetry (AISC F9 and F10): a). In this particular case, however, the eccentricities usually are small and can be neglected. Ollgaard, J. Bearing stiffeners: Bearing stiffeners are provided at each concentrated load, so there is no need to check for web yielding, web crippling, or sidesway web buckling. If the unbraced length is no greater than Lp , to be defined presently, the beam is considered to have full lateral support, and Mn = Mp. If Lb is greater than Lp but less than or equal to the parameter Lr , the strength is based on inelastic LTB. In many small structures, column axial forces can be easily computed from the reactions of the beams that they support or computed directly from floor or roof loads. From the dimensions and properties tables in Part 1 of the Manual, rmin = 0.864 in. There are many inherent inaccuracies and uncertainties in structural design, including variations in material properties and loads; load magnitudes sometimes border on educated guesses. The volume of slab contained in one square foot of floor is 1 ft2 × the slab thickness t. This makes inspection of the installation particularly easy. (This reduction is not intended for certain types of eccentric connections; that will be covered in Chapter 8.) The resistance is made by multiplying the slip-critical strength by a factor ksc as follows: For LRFD, ksc = 1 − Tu Du Tb nb (AISC Equation J3-5a) 1.5Ta Du Tb nb (AISC Equation J3-5b) For ASD, ksc = 1 − where Tu = total factored tensile load on the connection Ta = total service tensile load on the connection Du = ratio of mean bolt pretension to specified minimum pretension; default value is 1.13 Tb = prescribed initial bolt tension from AISC Table J3.1 nb = number of bolts in the connection Both the shear and tension strength in bolted connections can be summarized as follows: Bearing-type connections: 1. The noncompact shapes are identified in the dimensions and properties table with a footnote (footnote f). Relative moments of inertia of the members have been assumed for preliminary design purposes. ⎛ Pno ⎞ ⎡ ⎤ Pn = Pno ⎢ 0.658 Pe ⎥ = 3979(0.658)0.46656 = 3274 kips ⎢⎣ ⎥⎦ ANSWER For LRFD, the design strength is fcPn = 0.75(3274) = 2456 kips. STABILITY If a beam can be counted on to remain stable up to the fully plastic condition, the nominal moment strength can be taken as the plastic moment capacity; that is, Mn = Mp Otherwise, Mn will be less than Mp. As with a compression member, instability can be in an overall sense or it can be local. Load and resistance factor design (AASHTO, 2010). Other frequently used cross-sectional shapes are shown in Figure 1.7. Bars can have circular, square, or rectangular cross sections. As we describe later, the exact form of the AISC equation will depend on the method of analysis used to compute the LRFD amplification factor for the beam–column of Example 6.1. SOLUTION Since the Euler load Pe is used as part of an amplification factor for a moment, it must be computed for the axis of bending, which in this case is the x-axis. If the effective length is long enough, local buckling will generally not be an issue; the strength will be governed by lateral-torsional buckling. A circular or slotted hole is cut in one of the parts to be connected and is filled with the weld metal. FIGURE 2.1 2.5 Probabilistic Basis of Load and Resistance Factors 31 The variance, v, is a measure of the overall variation of the data from the mean and is defined as v = n ∑ (xi − x) 2 i =1 The standard deviation s is the square root of the variance, or s = 1 n n ∑ (xi − x) 2 i =1 Like the variance, the standard deviation is a measure of the overall variation, but it has the same units and the same order of magnitude as the data. Ω 2.00 2.00 7.11 Fillet Welds 449 The base-metal shear strength is therefore 3.6 kips鈭in., and the weld shear strength from Equation 7.35 is 3.488 in. ST9): 1337–53. × 2.00 in. Otherwise, it is inelastic. Figure 8.32 shows a moment-rotation curve for a typical rigid connection. This occurs when KL鈭r ≤ 4.71 E Fy To summarize, When KL E ≤ 4.71 , r Fy Fcr = 0.658 When KL E > 4.71 , r Fy Fy Fe F y (4.10) (4.11) The other AISC Specification provides for separating inelastic and elastic behavior based on either the value of KL鈭r (as in equations 4.10 and 4.11) or the value of the ratio Fy鈭Fe. The limiting value of Fy鈭Fe can be derived as follows. For this element, the ultimate (fracture) deformation is Δ u = 1.087(θ + 6) −0.65 w ≤ 0.17 w and the radius is rcrit. Solid Rectangular Bars (AISC F11): The applicable limit states are yielding and LTB for major axis bending; local buckling is not a limit state for minor axis bending. The allowable stress can be written as Fb = 0.6 M n 0.6 Fy Z × = Sx Sx If an average value of Zx 鈭Sx = 1.1 is used (this is conservative), Fb = 0.6Fy(1.1) = 0.66Fy If this value is used in Example 5.3, Fb = 0.66(50) = 33.0 ksi which is conservative by about 4%. Note: 104 kips > 86.2 kips 3.2−7 a. Use a 12 ft to ⎛ Since Lb > Lpd (15 ft > 12.2 ft), bracing is not adequate, and additional brace points must be added. To compute this required moment strength must be used. For example, line abcde in Figure 3.15 must be used. The full coordinates, whereas this will be subjected to 80 t of the one-bolt hole. An average edge distance in any direction is 11⁄8 inches. ST5): 989–1005. The web members will either be in tension or compression, depending on the loading function and orientation and on the location of the member. If the member is part of a wide flange section, member must be determined. A3.4. M1 = smaller end moment of the unbraced length (always taken as positive) Mmid = moment at the middle of the unbraced length (positive when it causes compression in the same flange as M2, negative otherwise) M2 = larger end moment of the unbraced length (always taken as positive) Mmid = moment at the middle of the unbraced length (positive when it causes compression in the same flange as M2, negative otherwise) The equivalent moment M1♠ is defined as follows: when M mid ≤ M1 + M 2 , M1♠ = 2 M mid − M 2 < M 2 (AISC Equation A-1-6b) when M mid > M1 + M 2 , M1♠ = 2 M mid − M 2 < M 2 (AISC Equation A-1-6c) A3 Analysis 721 The one exception to these definitions is that whenever the magnitude of M1 is greater than M2, the unbraced length exceeds M2, M1♠ = 1.0 M2 (AISC Appendix 1, in Section A3.1, "Analysis Requirements," requires that a second-order plastic analysis be performed. Values of Sx for standard rolled shapes are tabulated in the dimensions and properties tables in the Manual. (b) Cross-sectional dimensions are. A relatively small depth of penetration may be sufficient, and tabulated stresses for required shear, not to exceed. ANSWER ASD SOLUTION Use a W14 × 61 ANSWER (b) Hence the total bearing strength = 2(ϕ ≤ 1 fRn = 0.75(0.6)(0.75)(1)(2) = 1.14 × 61 kips A = 0.75 × 0.6 × 0.8 × 24 = 8.64 in.2 (b) 29, 000(170) (4) = 12.90 kips 12.0 Ω 18.3 ≤ 12 (1.0 × 15 × 12)2 B1 + Cm Cmy 0.6 − 1 = (1.60 × 3.0 = 4.10 ft-kips ∴ Use B1 = 1.0 Mr = May = B1 y Mnty + B2 y Mlty = 1.0(20) + 0 = 20 ft-kips Because the flange of this shape is noncompact, the weak axis bending strength is limited by FLB (see Section 5.15 of this book and Chapter F of the AISC Specification). The shop will be done using line AB (Figure 7.39b) and subject an area of IL to shear (Figure 7.39c). For shop welding, an automatic or semiautomatic process is usually used. The basic strength equation is Vn = 0.6Fy AwCv Vn = 0.6Fy AwCv (AISC Equation G2-1) where Aw = area of the web = dtw d = overall depth of the beam Cv = ratio of critical web stress to web shear yield stress The value of Cv depends on whether the limit state is web yielding, web inelastic buckling, or web elastic buckling. Assume that the bearing strength is controlled by the upper limit of 2.4dtFu. a. The provisions for column ends are from the Guide to Stability Design Criteria (Galambos, 1998). This gap is filled at the top flange during erection with shims, which are thin strips of steel used for adjusting the fit at joints. Shear strength c. ∴ Use a = 2 3 4 in. The member must be reinforced with continuous longitudinal lateral support, and in such a case, consisting of lateral ties or spirals. The AISC Specification gives provisions for bearing strength, as well as the requirements for high-strength bolts, and based on the provisions of the Research Council on Structural Connections (RCSC, 2009). The beam and column are A992 steel, and the end plate is A36 steel. Ωb Qb (Since a WI2 × 65 is recommended for Fy = 50 ksi, 237 ft-kips is the strength based on FLB rather than full yielding of the cross section.) The unamplified moments are Mnt = 55 ft-kips Mlt = 0 From AISC Equation A-8-1, the required moment strength is Mr = Ma = B1Mnt + B2Mlt = 1.027(55) + 0 = 56.49 ft-kips = Max From Equation 6.5 (AISC Equation H1-1a), ⎛ M ay ⎞ 280 8 56.49 Pa 8 ⎛ M ax + ⎟ + = + + 0 = 0.826 < 1.0 Pn 鈭Ωc 9 ⎝ M nx 鈭Ωb M ny 鈭Ωb ⎠ 9 456 9 237 ANSWER The member is satisfactory. Although the lightest W shape was found in Example 6.7, in many cases a specific nominal depth, such as 12 inches, is required for architectural or other reasons. 16 2 Use a 12-inch fillet weld, E70 electrode. The nominal flexural strength is therefore Mn = Mp = 3723 in-kips = 2229 ft-kips 1.67 From Figure 10.15, the maximum moment is Ma = 2112 ft-kips < 2229 ft-kips ANSWER (OK) The flexural strength is adequate. The required strength 8 is Mp for a plane of Mp. ANSWER Pn = 2 ab M p = (1040) = 139 kips 15 15 726 Appendix Plastic Analysis and Design DESIGN The design process is simpler than analysis, because each possible failure mechanism can be analyzed simply by multiplying the collapse load to determine the required moment capacity, Mp. The collapse load is known in advance, having been obtained by multiplying the service loads by the load factor. "LRFD Criteria for Plate Girders." Journal of the Structural Division, ASCE 104 (no. Check shear and bearing against the usual strengths. Flexural Strength For most cases, the nominal flexural strength will be reached when the entire steel section yields and the concrete crushes in compression (for positive bending moment). When the support in the single-plate beam-to-column connection shown in Figure 7.2, the plate is shop-welded to the column flange and field-bolted to the beam web. Higdon, A., Ohlsen, E. SOLUTION As in Example 3.4, only part of the cross section is connected and a reduced effective area must be used. Adequate: 86.8 kips < 92.0 kips b. First, assuming the same strength for both the longitudinal and transverse welds, 86.23 = 23.23 in. rmin = ry = ly A = 56.80 = 1.661 in. Two types of beam bearing plates are considered: one that transmits the beam reaction to a support such as a concrete wall and one that transmits a load to the top flange of a beam. ⎛ 4 ⎞ The base-metal shear strength is therefore 5.4 kips鈭in., and the weld shear strength is 65 inches. There is a design dead load of 15 kips. The channels are bolted to a 3⁄8-inch gusset plate with 7⁄8-inch diameter bolts. The design strength of a built-up compression member is a function of the slenderness ratio KL鈭r. It is common practice to use the PL (Plate) designation for both bars and plates. No: Mu = 1480 ft-kips > fbMn = 1420 ft-kips b. Segui is a Professor Emeritus of Civil Engineering at The University of Memphis, where he has been a member of the faculty since 1968. Any fewer than this number will permit some slippage to occur between the steel and concrete; such a beam is said to be partially composite. Consider the beam–column shown in Figure 6.15. The required axial strength is affected by the displaced geometry of the structure during loading. When more than one failure pattern is conceivable, all possibilities should be investigated, and the one corresponding to the smallest load capacity should be used. 'SOCIAL INNOVATION - IT'S OUR FUTURE' Big Data (IT) version : Hitachi. The shear force to be resisted by the web reinforcement is 89.78 − 53.35 = 36.43 kips. Select a trial shape from Table 6-1 of the Manual. 11⁄2″ 11⁄2″ 3″ 3″ 3″ 11⁄2″ 4 FIGURE P8.2-6 Problems 8.2-7 575 What diameter Group A bearing-type bolt is required? Also from the Zx table, Lp = 6.6 ft and Lr = 19.6 ft. The factored strength is the theoretical strength multiplied by a resistance factor. The 0.7 factor applied to the seismic load effect E is used because ASCE 7 uses a strength approach (i.e., LRFD) for computing seismic loads, and the factor is an attempt to equalize the effect for ASD. 18k 1 2 10′ 3 5 5′ 30′ FIGURE P5.10-6 4 10′ 290 Chapter 5 5.10-7 Beams Beam shown in Figure P5.10-7 has lateral support at the ends only. For columns in rigid frames, there are intermediate bending moments as well as axial forces, and a frame analysis is necessary. hc = h for rolled shapes (for built-up sections with equal flanges). The category is based on the worst width-to-thickness ratio of the cross section. Minimum edge distance = 1.5 in. Compute the strength of the fastener group. The shearing stress caused by the couple is found with the torsion formula f2 = Md J where d = polar moment of inertia of the shear area to the point where the stress is being computed J = polar moment of inertia of the shear area. (Rn)x, measured from the center of the plate, are given in AISC Table J3.4 as a function of bolt size. The shearing stress caused by the couple is found with the torsion formula f2 = Md J where d = polar moment of inertia of the shear area to the point where the stress is being computed. Figure 8.17 shows this stress at the upper right-hand corner of the given weld. 306 ◄ 306 ▶ 21 Ω 31 E 13♠ 8 鈭 21 F 11♠ 30♠ 21 31 I F FIGURE P4.7-11 31 Problems 4.7-12 181 An unbraced frame is shown in Figure P4.7-12. LRFD SOLUTION The factored axial load is Pu = 1.2PD + 1.6PL = 1.2(25) + 1.6(75) = 150 kips (There is no amplification of axial loads in members braced against sidesway.) The factored moments are Mntx = 1.2(12.5) + 1.6(37.5) = 75.0 ft-kips Mnty = 1.2(5) + 1.6(15) = 30.0 ft-kips The amplification factor B1 can be estimated to be 1.0 for purposes of making a trial selection. The amplification factor for the sidesway moments, B2, is given by 1 B2 = ≥1 (AISC Equation A-8-6) a Pstory 1 − Pe story where a = 1.00 for LRFD = 1.60 for ASD Pstory = sum of required load capacities for all columns in the story under consideration (factored for LRFD, unfactored for ASD) Pe story = total elastic buckling strength of the story under consideration This story buckling strength may be obtained by a sidesway buckling analysis or as Pe story = RM HL ΔH (AISC Equation A-8-7) 328 Chapter 6 Beam–Columns FIGURE 6.15 where RM = 1 − 0.15 (Pmf 鈭Pstory) Pmf = sum of vertical loads in all columns in the story that are part of moment frames L = story height ΔH = interstory drift = drift (sidesway displacement) of the story under consideration H = story shear = sum of all horizontal forces causing ΔH Note that if there are no moment frames in the story, then Pmf = 0 and RM = 1.00. If all of the columns in the story are members of moment frames, then Pmf = Pstory and RM = 0.85. Other certain loadings combinations it may be necessary to consider the loads as of their moment. Usually, they consist of structural members and connections, for the crosssectional properties. The relationship between resistance factors and safety factors is given by 1.5 Ω= (2.8) φ For reasons that will be discussed later, this relationship will produce similar designs for LRFD and ASD, under certain loading conditions. Figure 9.3 shows a segment of a composite beam with stress and strain diagrams superimposed. 4 Before proceeding, check the conditions of AISC G3.1 to be sure that the tension-field action can be used for this girder and this stiffener spacing. Select a W shape of A992 steel. wu = 1.2wD + 1.6wL = 1.2(0.084) + 1.6(4.5) = 7.301 kips鈭ft Required moment strength = Mu = 1 1 wu L2 = (7.301)(30)2 = 821.4 ft-kips 8 8 The required section modulus is Zx = Mu 821.4(12) = = 219 in.3 < 224 in.3 8 Fy 0.90(50) In lieu of basing the search on the required section modulus, the design for Mp could be used, because it is directly proportional to Zx. Built-up shapes, including plate girders, are covered in Chapter 10 of this textbook. Some members, however, such as when loads are applied to the same web, however, additional shear investigation is needed. 5.5-8 A W16 × 77 is used as a beam with an unbraced length of 15 feet. Washington D.C. American Railway Engineering and Maintenance-ofWay Association. Next, determine the moments required for the computation of Cb . Figure 1.1 shows a truss with vertical concentrated forces applied at the joints along the top chord. The compression element is 2 π 2 Pu 0.90 BNFy where n′ = 1 db f 4 (5.17) These three approaches were combined by Thornton (1990b), and a summary of the resulting unified procedure follows. In the transverse direction, use bolt edge distances of 11⁄2 inches and a spacing of 31⁄2 inches. Compute the strength of this connection. Consider the plate girder of Example 10.1, part b. When rounding numbers where the first digit to be dropped is a 5 with no digits following, two options are possible. Girders are defined as beams that support other beams, although sometimes the term is applied to large beams in general. For Group A bolts with the threads not in the plane of shear (type X), the nominal shear strength is Fnv = 68 ksi, and Rn = FnvAb = 68(0.4418) = 30.04 kips At the hole nearest the edge, the bearing strength of 28.55 kips is less than the shear strength of 30.04 kips. From Figure 8.40, the force at the interface between the beam flange and the plate is H= M 210(12) = = 121.2 kips 20.8 d where d is the depth of the beam. The steel shapes must be

compact with adequate lateral support. The length is 9 feet and all structural steel is A36. 7 3(3) x= = 1.286 in. 7.4 SHEAR STRENGTH While bearing strength is independent of the type of fastener, shear strength is not. Local web yielding (AISC Equation J10-2 or J10-3). Design of Steel and Composite Beams with Web Openings. It is to be attached to a 3⁄8-inch gusset plate, as shown in Figure 7.44. Check the shear strength. Conversely, recording too few figures can lead to meaningless results. Cochrane, V. The values of lp and lr are given in AISC Table I1.1a. 384 EI s 384(29,000)(510) The total immediate deflection is Δ1 + Δ2 = 0.6670 + 0.2218 = 0.889 in. We use only standard holes in this book (holes 1⁄16-inch larger than the bolt diameter). What is the maximum permissible value of P? Moment strength based on bolt strength: Pt = Ft Ab = 90(0.6013) = 54.12 kips鼻bolt Mn = 2Pt (h0 + h1) = 2(54.12) (19.49 + 15.06) = 3740 in.-kips fMn = 0.75(3740) = 2805 in-kips Determine end-plate width. Noncompact shapes are also treated differently in the Zx table in the following way. ANSWER ASD SOLUTION Use a W18 × 130. All steel is A992. For deflection of the composite section, use the lower-bound moment of inertia. LRFD SOLUTION Calculate the factored load: Pu = 1.2D + 1.6L = 1.2(165) + 1.6(535) = 1054 kips ∴ Required design strength fc Pn = 1054 kips. The beam has continuous lateral support, and A572 Grade 50 steel is used. The method used is based on the model shown in Figure 7.28. New York: Ronald Press Co. Thornton, W. In this book, the results of structural analyses will be given in all examples and problems. Determine the effective length factor Kx for columns AB and BC. Maximum moment amplification occurs at the center, where the deflection is largest. If A992 steel is used, does this beam satisfy the provisions of the AISC Specification? A strain of 0.002 is usually selected, and this method of determining the yield strength is called the 0.2% offset method. The Appendix section is followed by the Commentary, which gives background and elaboration on many of the provisions of 24 Chapter 2 Concepts in Structural Steel Design the Specification. Consider the beam connection shown in Figure 8.30. The bolts are 7⁄8-inch, Group A bearing-type. The tables also give values of fb Mp and Mp 鼻 Ωb for the steel shape, which may be needed for checking unshored beams during the curing of the concrete; and Y1, the distance from the top of the steel to the plastic neutral axis. 169 kips 3.4-5 a. 668 Chapter 10 Plate Girders FIGURE 10.2 If an unstiffened web is incapable of resisting the applied shear, appropriately spaced stiffeners are used to develop tension-field action. The ends of the column are approximately fixed, and K is relatively small. 3 φrn = 40.8t = 40.8 ⎛⎜ ⎞⎟ = 15.3 kips鼻bolt ⎝ 8⎠ Using the minimum of shear and bearing strengths for each bolt location, the total connection strength is φ Rn = 2(edge bolt strength) + 4(inner bolt strenngth) = 2(15.3) + 4(24.4) = 128 kips > 108 kipss (OK) The tentative connection design is shown in Figure 7.17 and will now be checked for block shear in the gusset plate (the geometry of the failure block in the channel is identical, but the gusset plate is thinner). The largest required thickness is for the limit state of shear rupture. One of the most commonly used structural steels is a mild steel designated as ASTM A36, or A36 for short. Substituting and solving for ΣAb, we get 150 ⎛ 75 ⎞ = 87.75 − 1.667 ⎜ ⎝ ΣAb ⎟⎠ ΣAb 150 = 87.75ΣAb − 1.667(75) ΣAb = 3.134 in.2 The area of one bolt is Ab = π (3鼻4)2 = 0.4418 in. Generate a table of stress and strain values. 299 300 Chapter 6 Beam–Columns FIGURE 6.1 Another example of beam–columns can sometimes be found in roof trusses. 198 Chapter 5 Beams FIGURE 5.10 Curve 1 is the load-deflection curve of a beam that becomes unstable (in any way) and loses its load-carrying capacity before first yield (see Figure 5.3b) is attained. 78 b. In a connection of the type just considered, however, there is additional compression on the lower part of the connection that increases the friction, thereby compensating for the reduction in the upper part of the connection. Assume continuous lateral support. ANSWER Mn = 5825 in.-kips = 485 ft-kips. Load Combination 2: Pa = 305 kips, Mnt = 70 ft-kips, and Mt = 0 (because of symmetry, there are no sidesway moments). Note that the tension member is a doubleangle shape, and both of the angles are welded as shown in Figure P7.11-2. (N.G.) 138 Chapter 4 Compression Members Try a W18 × 130: Ag = 38.3 in.2 KL 26 × 12 = = 115.6 < 200 rmin 2.70 Fe = Since (OK) π 2E π 2 (29, 000) = = 21.42 ksi (KL鼻r)2 (115.62) KL E > 4.71 = 113, AISC Equation E3-3 applies. A36 steel is used for the bracket, and A992 steel is used for the column. A case in point is the plate girder shown in Figure 7.1. Before welding became widely used, this type of built-up shape was fabricated by riveting. Most of the conditions will usually be approximately satisfied; if they are not, the deviation will be on the conservative side. 5.4 CLASSIFICATION OF SHAPES AISC classifies cross-sectional shapes as compact, noncompact, or slender, depending on the values of the width-to-thickness ratios. fb Mn = 0.90(155.2) = 139.7 in.-kips = 11.64 ft-kips Determine which interaction equation to use: 90 Pu = = 0.6040 > 0.2 fc Pn 149 ∴ Use AISC Equation H1-1a. As discussed in Chapter 4, “Compression Members,” whether either type of local buckling occurs will depend on the width-to-thickness ratios of the compression elements of the cross section. 412 Chapter 7 Simple Connections The bolt layout in Example 7.7 is symmetrical with respect to the longitudinal centroidal axis of the member. Thus, T is the external tension force applied to one bolt, q is the prying force corresponding to one bolt, and Bc is the total bolt force. For h鼻tw = 198 and fvVn鼻Aw = 11 ksi, a ≈ 1.60 h The required stiffener spacing is a = 1.60h = 1.60(62) = 99.2 in. Eyebars were widely used in the past as single tension members in bridge trusses or were linked in chainlike fashion in suspension bridges. ANSWER ASD SOLUTION Use an HSS14 × 10 × 1⁄4. No slip of the connection is permitted. The other two vertical members, AE and CF, must resist not only axial compression from the vertical loads but also a significant amount of bending. D = 95k L = 9k PL 1⁄2 × 8 FIGURE P3.2-5 92 Chapter 3 3.2-6 Tension Members A double-angle tension member, 2L 3 × 2 × 1⁄4 LLBB, of A36 steel is subjected to a dead load of 12 kips and a live load of 36 kips. The International Building Code incorporates much of ASCE 7 in its load provisions. EXAMPLE 4.2 A W14 × 74 of A992 steel has a length of 20 feet and pinned ends. 604 Chapter 9 9.2 Composite Construction SHORED VERSUS UNSHORED CONSTRUCTION Until the concrete has cured and attained its design strength (at least 75% of its 28-day compressive strength, fc′), there can be no composite behavior, and the weight of the slab must be supported by some other means. The standard load tables also include a K-series economy table, which facilitates the selection of the lightest joist for a given load. Tensile strength, min. For convenience, in the following derivation, the member will be oriented with its longitudinal axis along the x-axis of the coordinate system given in Figure 4.3. The roller support is to be interpreted as restraining the member from translating either up or down. Annual Book of ASTM Standards. As pointed out in Example 4.12, for a pinned support, G should be taken as 10.0; for a fixed support, G should be taken as 1.0. The latter support condition corresponds to an infinitely stiff girder and a flexible column, corresponding to a theoretical value of G = 0. (1.5 in. As far as strength is concerned, flexure is almost always more critical than shear, so the usual practice is to design for flexure and then check shear. Design for a drift index of 1鼻400 based on service wind load. Ω 2.00 c. ⎝ 8⎠ and the allowable base metal shear rupture strength is ⎛ 3⎞ 0.3Fu t = 0.3(58) ⎜ ⎟ = 6.525 kips鼻in. Dimensions and Properties. In Example 7.3, the bearing strength was assumed to be adequate. Figure 7.24 shows a hanger connection consisting of a structural tee-shape bolted to the bottom flange of a W shape and subjected to a tensile load. The maximum permissible live load deflection is L鼻360. B 40.59 −1= − 1 = 0.353 T 30 From Equation 7.15, α= [(B鼻T − 1)](a ′鼻b ′) 0.3553(3.469鼻1.988) = = 2.139 δ {1 − [(B鼻T) − 1](a ′鼻b ′)} 0.75[1 − 0.353(3.469鼻1.988)] Because ⎪ a ⎪ > 1.0, use a = 1.0. From Equation 7.18, 4Tb ′ 4(30)(1.988) = φb pFu (1 + δα) 0.90(4)(65)[1 + 0.75(1.0)] = 0.763 in. Bending about the minor axis: The provisions are the same as for bending about the major axis. All of the W-shapes of a given nominal size can be grouped into families that have the same depth from inside-of-flange to inside-of-flange but with different flange thicknesses. 3.7 THREADED RODS AND CABLES When slenderness is not a consideration, rods with circular cross sections and cables are often used as tension members. For the coordinate system shown, the moment–curvature relationship can be written as d2y dx 2 =− M EI The bending moment, M, is caused by the eccentricity of the axial load, P, with respect to the axis of the member. Because of symmetry, the elastic neutral axis (the x-axis) is located at middepth of the cross section (the location of the centroid). That will not always be possible, so the effect of the holes must be accounted for. (Although it is not shown in Figure 1.1, this same idealization is made with trusses, and the members are usually represented by their centerlines.) Note that the supports are represented as hinges (pins), not as fixed supports. ANSWER Use a tension brace with a cross-sectional area of at least 0.0349 in.2 354 Chapter 6 Beam–Columns Notes on an Allowable Strength Solution to Example 6.10 In Example 6.10, the given loads were already factored. The welded version, however, is elegant in its simplicity. Check bolt shear and bearing. V., Kulak, G. Each load consists of 25% dead load and 75% live load. 16 1 1 7 = in. When a plate girder is used, however, the designer must account for factors that in many cases would not be a problem with a hot-rolled shape. The beams are spaced at 10 feet, and the span length is 40 feet. The uniform load is a superimposed dead load, and the concentrated load is a live load. 658 Chapter 9 Composite Construction a. Direct analysis method 2. 2 ⎝ 16 ⎠ ⎝ 2 8 ⎠ The effective net area is Ae = 0.75(2.019) = 1.514 in. Bending is about the strong axis, and Kx = Ky = 1.0. The frame analysis was performed consistent with the effective length method, so the flexural rigidity, EI, was unreduced. Federal Emergency Management Agency. T., and Wyllie, Jr., L. 9.9 CONTINUOUS BEAMS In a simply supported beam, the point of zero moment is at the support. Slip-critical bolt strength has also been modified. Consider the composite section shown in Figure 9.4a (determination of the effective flange width b when the beam is part of a floor system is discussed presently). A convenient procedure for determining K for inelastic columns allows the alignment charts to be used (Yura, 1971; Disque, 1973; Geschwindner, 2010). Adequate: Before concrete has cured, Ma = 182 ft-kips < 322 ft-kips; after concrete has cured, Ma = 596 ft-kips < 643 ft-kips; Va = 59.6 kips < 171 kips c. Although the torsion formula is applicable only to right circular cylinders, its use here is conservative, yielding stresses that are somewhat larger than the actual stresses. LRFD SOLUTION The composite section must resist a factored load and moment of wu = 1.2 wD + 1.6 wL = 1.2(437.5) + 1.6(1200) = 2445 lb鼻ft Mu = 1 1 wu L2 = (2.445)(30)2 = 275 ft-kips 8 8 Try a nominal depth of d = 16 inches. The deformation can be determined by measuring the gap between the nut or bolt head and the undeformed part of the washer surface. This combination also gives the same result as combination 4. 25k 2k/ft A B 20′ 20′ 40′ FIGURE P5.10-5 5.10-6 The beam in Figure P5.10-6 is laterally supported at the ends and at the 1⁄3 points (points 1, 2, 3, and 4). 5.916 5.916 16.18 P2 = = 2.73 in. “Structural Steel Economy: Revisiting the Assumptions.” Modern Steel Construction, AISC 49 (no. Each bolt centerline shown represents a row of bolts in the direction of the width of the plates. If A36 steel is used, is the member adequate for a service dead load of 31 kips and a service live load of 31 kips? The AISC Specification does not cover open-web steel joists; a separate organization, the Steel Joist Institute (SJI), exists for this purpose. Try a flange thickness of t f = 1.5 inches and a web depth of h = 65 − 2(1.5) = 62 in. In this case, however, the load is not in the plane of the fasteners, so the couple will tend to put the upper part of the connection in tension and compress the lower part. Maximum deflection before the concrete has cured. Thus interior panels are anchored by adjacent panels. FIGURE 8.28 1⁄2″ 3⁄4″ 8.5 SOLUTION Eccentric Welded Connections: Shear Plus Tension 517 The beam reaction will be assumed to act through the center of gravity of the connection to the framing angles. The AISC Specification is only a small part of the Manual. 408 Chapter 7 Simple Connections Compute the actual block shear strength. Failure in this context means either collapse or extremely large deformations. If so, the provisions of AISC F5 can be used. π 2 (0.8 EI y) π 2 (0.8)(29, 000)(53.4) = = 377 kips (15 × 12)2 (K y L)2 Cmy 0.6 = 0.996 < 1.0 = 150 Pu 1− 1− 377 Pe1 y ∴ B1 y = 1.0 as assumed. 8.8-4 Design a four-bolt unstiffened end-plate connection for a W12 × 30 beam to a W10 × 60 column. Assume that the bracing will stabilize three frames. For welds on both sides of the plate, the required strength per weld is 4.588鼻2 = 2.294 kips鼻in. 6.2 INTERACTION FORMULAS The relationship between required and available strengths may be expressed as required strength ≤ 1.0 available strength (6.1) For compression members, the strengths are axial forces. Note that if the effective net area must be computed, the tables do not save much effort. (rts can also be found in the dimensions and properties tables. In a shear connection, there is no connection of the flanges, and the web connection is designed to be flexible enough to allow some relative rotation of the members at the joint. Assume that the member tensile and block shear strengths will be adequate. 6.425 area below the PNA. The slight difference is caused by the rounding of the safety factor Ω from 5⁄3 to 1.67 (this rounding was noted in Chapter 2). Compression Flange Strength The compression flange nominal strength is given by Mn = Rpg Fcr Sxc (AISC Equation F5-7) where Rpg = bending strength reduction factor Fcr = critical compressive flange stress, based on either yielding or local buckling Sxc = elastic section modulus referred to the compression side 10.4 Flexural Strength 673 The bending strength reduction factor is given by Rpg = 1 − aw 1200 + 300 aw ⎛ hc E⎞ ⎜ ⎟ − 5.7 ⎥ ≤ 1.0 Fy ⎥⎠ ⎝ tw (AISC Equation F5-6) where aw = hc t w ≤ 10 b fc t fc (AISC Equation F4-12) bfc = width of the compression flange tfc = thickness of the compression flange (The upper limit of 10 in Equation F4-12 is not actually part of the AISC Equation, but AISC F5.2 stipulates that limit.) The critical compression flange stress Fcr depends on whether the flange is compact, noncompact, or slender. The resistance factors account for uncertainties in material properties, design theory, and fabrication and construction practices. Bolt line FIGURE P3.6-2 3.6-3 Select an ST shape to be used as a 20-ft-long tension member to resist the following service loads: dead load = 38 kips, live load = 115 kips, and snow load = 75 kips. From the LRFD solution, the web–slenderness limitation of AISC F13.2 is satisfied. Try 3⁄4-inch Group A bolts.(Since 3⁄4-inch-diameter bolts were selected for the shear connection, try the same size here.) If bolt shear controls, the number of bolts required is 121.2 = 6.77 ∴ Use 8 bolts for symmetry (4 pairss) 17.9 Use edge distances of 11⁄4 inches, spacings of 3 inches, and determine the minimum plate thickness required for bearing. The longer panel will have the smaller strength for both the tension field and the no-tension-field cases defined by Vc1 and Vc2. For example, the connection of the single-angle tension member shown in Figure 3.21 is susceptible to this phenomenon, called block shear. Assume that the cross-sectional elements are connected such that the built-up shape is fully effective. For one angle, the nominal strength based on the gross area is Pn = Fy Ag = 36(2.41) = 86.76 kips There are two holes in each angle, so the net area of one angle is ⎛ 5 ⎞ ⎛ 1 1 ⎞ An = 2.41 − ⎜ ⎜ ⎟ + ⎟ × 2 = 2.019 in. 8.6 Moment-Resisting Connections 531 ⎛ 0.535 ⎞ Agv = 13.13 ⎜ = 11.24 in.2 ⎝ 5鼻8 ⎟⎠ ⎛ 0.535 ⎞ = 7.958 in.2 Anv = 9.297 ⎜ ⎝ 5鼻8 ⎟⎠ ⎛ 0.535 ⎞ = 1.137 in.2 Ant = 1.328 ⎜ ⎝ 5鼻8 ⎟⎠ Rn = 0.6Fu Anv + Ubs Fu Ant = 0.6(65)(7.958) + 1.0(65)(1.137) = 384.3 kips 0.6Fy Agv + UbsFu Ant = 0.6(50)(11.24) + 1.0(65)(1.137) = 411.1 kips > 384.3 kips Since 384.3 kips > 360.6 kips, block shear in the plate controls. Determine the available shear strength of the connection. 4.7 More on Effective Length 143 Enter the tables with KL = 10.34 feet. Steel with Fy = 50 ksi is used for both components. If the axial load in a slender tension member is removed and small transverse loads are applied, undesirable vibrations or deflections might occur. To transform the concrete area, Ac, we must divide by n. Use ASD and determine the maximum service concentrated load that can be supported. For fillet welds other than those along edges (as in Figure 7.41), there is no maximum size specified. Bearing stiffeners: Bearing stiffeners are provided at each concentrated load, so there is no need to check the provisions of AISC Chapter J for web yielding, web crippling, or sidesway web buckling. The braced frame is shown in Figure 6.23. Figure 8.52 shows these forces. Kx = Ky = 1.0. FIGURE 6.10 PD = 70k PL = 210k MD = 11 ft-k ML = 36 ft-k 14′ W12 × 65 MD = 14 ft-k ML = 41 ft-k LRFD SOLUTION The factored loads, computed from load combination 2, are shown in Figure 6.11. First, compute the flexural buckling strength for the x-axis (the axis of no symmetry): K x L 25.5 × 12 = = 87.43 3.50 rx Fe = 4.71 Since π 2E π 2 (29, 000) = = 37.44 ksi (87.43)2 (KL鼻r)2 E 29, 000 = 4.71 = 113 Fy 50 KL E < 4.71 , AISC Equation E3-2 applies: r Fy Fcr = 0.658(Fy 鼻Fe) Fy = 0.658(50/37.44) (50) = 28.59 ksi The nominal strength is Pn = Fcr Ag = 28.59(23.9) = 683.3 kips Compute the flexural-torsional buckling strength about the y-axis (the axis of symmetry): Compute Fcry using AISC E3: K y L 20 × 12 = = 78.69 3.05 ry From AISC Equation E3-4, Fe = π 2E π 2E π 2 (29, 000) = 46.22 ksi 2 = 2 = (KL鼻r) (K y L鼻ry) (78.69)2 160 Chapter 4 Compression Members Since K y L鼻ry < 4.71 E = 113, Fy Fcry = 0.658(Fy鼻Fe) Fy = 0.658(50/46.22)(50) = 31.79 ksi Because the shear center of a tee is located at the intersection of the centerlines of the flange and the stem, x0 = 0 tf 1.22 = 2.70 − = 2.090 in. If a temporary transverse load is applied so as to deflect the member into the shape indicated by the dashed line, the member will return to its original position when this temporary load is removed if the axial load is less than the critical buckling load. The total depth of the girder will be 73 inches, and continuous lateral support will be provided. Mnt will be a factored load moment for LRFD and a service load moment for ASD. Many persons contributed to the production of this revision. Ae = AnU = 3.836(0.80) = 3.07 in.2 < 3.54 in.2 2 (N.G.) (Note that this shape has slightly more gross area than that produced by the previous trial shape, but because of the greater leg thickness, slightly more area is deducted for the holes.) Passing over the next few heavier shapes, Try L8 × 4 × 1⁄2 (Ag = 5.80 in.2 and rmin = 0.863 in.) 3 1 1 An = Ag − Aholes = 5.80 − 2 ⎛ + ⎞ ⎛ ⎞ = 4.925 in.2 ⎝ 4 8⎠ ⎝ 2⎠ Ae = AnU = 4.925(0.80) = 3.94 in.2 > 3.54 in.2 *The notation N.G. means “No Good.” (OK) 3.6 ANSWER ASD SOLUTION Design of Tension Members 73 This shape satisfies all requirements, so use an L8 × 4 × 1⁄2. From Manual Table 7-1, frn = 24.9 kips鼻bolt 7.7 Design Examples 403 Assume that shear controls and then check bearing. With C = 384.0 kips, the depth of the compressive stress distribution in the concrete is a= C 384.0 = = 1.255 in. upper limit) If one stud is used at each section, the approximate spacing will be s= 30(12) = 4.4 in. Your table should have the following form: Bolt diameter (in.) 1⁄2 . Senior print Buyer: Doug Wilke For your course and learning solutions, visit www.cengage.com/engineering. 6.6-3 A W14 × 99 of A992 steel is used as a 14-foot-long beam–column with Kx = 0.9 and Ky = 1.0. (An analysis was performed consistent with the effective length method; therefore, the flexural rigidity, EI, was unreduced.) The member is braced against Problems 365 sidesway, and there are transverse loads between the ends. These tables include values of KL up to those corresponding to KL鼻r = 200. Determine the effective length factor Kx for column AB. (By inspection, the remaining combinations will not govern.) For the component parallel to the roof (Figure 3.32b), 12 = 4.679 kips 46.6 4.679 T = = 0.1434 in.2 Required Ab = ft (0.75Fu) 0.75(0.75)(58) T = (18.17) ANSWER Use a 5⁄8-inch-diameter threaded rod (Ab = 0.3068 in.2). Tension member strength is dependent on the cross-sectional area of the member. Not adequate: 62 kips > 61.3 kips 3.5-1 210 kips 3.5-3 134 kips 3.5-5 a. 46.6 kips Satisfactory: Result of interaction equation = 0.826 Chapter 7: Simple Connections 7.3-1 a. Solutions Manual: Last update Sept 2, 2015. Does this member have enough strength? Ωb Ωb Ωb Compute the axial compressive strength. 450 Chapter 7 Simple Connections To determine the strength of the connection, we will investigate the two options given in AISC J2.4c. Pu 272.3 = = 0.4113 > 0.2 φc Pn 662 ∴ Use Equation 6.3 (AISC Equation H1-1a). = in. The results of the final trial value of r 0 = 1.57104 inches are given in Table 8.1. The coordinate system and bolt numbering scheme are shown in 8.2 TABLE 8.1 Origin at Bolt 1 489 Eccentric Bolted Connections: Shear Only Origin at IC Bolt x y x y r R rR Ry 1 2 3 4 5 6 7 0 3 0 3 0 3 0 0 0 3 3 6 6 9 0.285 3.285 0.285 3.285 0.285 3.285 0.285 −3.857 −3.857 −0.857 −0.857 2.143 2.143 5.143 3.868 5.067 0.903 3.395 2.162 3.922 5.151 0.255 0.334 0.060 0.224 0.143 0.259 0.340 0.956 0.980 0.644 0.940 0.860 0.958 0.982 3.699 4.968 0.582 3.192 1.859 3.758 5.056 0.071 0.636 0.203 0.910 0.113 0.802 0.054 Sum 23.1120 2.78951 FIGURE 8.10 Figure 8.10. 236 k 168 ft-k 15′ FIGURE P6.8-1 Problems 6.8-2 371 The beam–column in Figure P6.8-2 is part of a braced frame and is subjected to the axial load and end moments shown (the end shears are not shown). 2 8 From AISC Equation J4-5, Rn = 0.6Fu Anv + UbsFu Ant = 0.6(58)(5.25) + 1.0(58)(3.0) = 356.7 kips with an upper limit of 0.6Fy Agv + Ubs Fu Ant = 0.6(36)(5.25) + 1.0(58)(3.0) = 287.4 kips (controls) The design strength is fRn = 0.75(287.4) = 216 kips > 129.6 kips ANSWER Use the weld shown in Figure 7.48. 84 Chapter 3 Tension Members FIGURE 3.33 The fact that chord members are continuous and joints are bolted or welded would seem to invalidate the usual assumption that the truss is pin-connected. From Example 8.10, the flange force can be conservatively taken as Pbf = H = 80.77 kips Check local flange bending with AISC Equation J10-1: Rn 6.25Fyf t 2f = Ω Ω 6.25(50)(0.780)2 = = 114 kips > 80.77 kips 1.67 (OK) Check local web yielding with AISC Equation J10-2: Rn Fyw t w (5k + b) = Ω Ω 5 50(0.485) ⎡5(1.38) + ⎤ ⎥ ⎣ 8 ⎦ = 122 kips > 80.77 kips = 1.50 (OK) Check web crippling with AISC Equation J10-4: 2 ⎡ Rn 0.80 t w ⎢⎣1 + 3 = Ω (d)(tt) w f b Ω 1.5 ⎤ EFyw t f tw ⎦⎥ ( )( ) 5鼻8 485 1.5 ⎤ 29,000(50) 0.780 0.80(0.485)2 ⎡⎢1 + 3 14.2 00..780 0.485 ⎦⎥ ⎣ = 2.00 = 153 kips > 80.77 kips (OK) ANSWER Column stiffeners are not required. 1.04. The studs are welded to the beams from above, through the deck. They may or may not be acting on the structure at any given time, and the location may not be fixed. First, check the upper limit on F′nt: 50 50 = = 14.15 ksi ∑ Ab 8(0.4418) Fnt′ = 117 − 3.333 frv = 117 − 3.333(14.15) = 69.8 ksi < 90 ksi frv = Check shear. The superimposed loads are as follows: construction load = 20 psf and live load = 160 psf. The nominal stress is therefore Fnw = 0.60 FEXX AISC Section J2.4a presents an alternative fillet weld strength that accounts for the direction of the load. Although this 10% increase is theoretically possible, tests have shown that the overall strength of a connection with fasteners in tension is not affected by the installation tension (Amrine and Swanson, 2004). With this alternative, use full-depth horizontal stiffeners as shown in Figure 8.49. For the gross area, Pn = Fy Ag = 36(5.51) = 198.4 kips The allowable strength is Pn 198.4 = = 119 kips Ωt 1.67 Tension on the effective net area: ⎛ 7 1⎞ An = 5.51 − 2 ⎜ + ⎟ (0.487) = 4.536 in.2 ⎝ 8 8⎠ The exact length of the connection is not yet known, so Equation 3.1 for U cannot be used. Ec 3492 598 Chapter 9 Composite Construction FIGURE 9.5 Since the modulus of elasticity of concrete can only be approximated, the usual practice of rounding n to the nearest whole number is sufficiently accurate. 384 EI 384 29, 000(510) The maximum permissible live load deflection is L 30(12) = = 1.0 in. Both of these members are assumed to be tension members and not resist any compression. ∑ A 20.60 With the location of the horizontal centroidal axis known, the moment of inertia with respect to this axis can be found by using the parallel-axis theorem: I = I¯ + Ad 2 where I¯ = moment of inertia about the centroidal axis of a component area A = area of the component I = moment of inertia about an axis parallel to the centroidal axis of the component area d = perpendicular distance between the two axes The contributions from each component area are computed and summed to obtain the moment of inertia of the composite area. Use four 7⁄8-inch-diameter, Group A bolts, as shown in Figure 7.23. From equilibrium of forces, C=T Ac Fy = At Fy Ac = At Thus the plastic neutral axis divides the cross section into two equal areas. One stiffener is placed at midspan. EXAMPLE 9.15 A 16-foot-long compression member must support a total service load of 500 kips, consisting of equal parts of dead load and live load. The shear yield strength of the base metal is ⎛ 3⎞ 0.6 Fy t = 0.6(36) ⎜ ⎟ = 8.1 kips鼻in. ⎝ ⎠ 12 12 2 2 ⎣ ⎦ and the area and radius of gyration are 3 A = 4.5 ⎛ ⎞ + 4(0.75)(7.5) = 24.19 in.2 ⎝ 8⎠ and r= I = A 454.3 = 4.334 in. They are used when the cost and weight of a beam would be prohibitive. This is because the ratio of live load to dead load is approximately 3, and the two approaches will give the same results for this ratio. Check the shear strength of the steel shape. EXAMPLE 7.9 An 8-inch-long WT10.5 × 66 is attached to the bottom flange of a beam as shown in Figure 7.29. Determine the controlling load combination for ASD and the corresponding required service load strength. In Chapter 5, “Beams,” we covered hot-rolled shapes, and for all the standard sections in the Manual, the webs are compact. 7.8 High-Strength Bolts in Tension 417 FIGURE 7.24 Before loading, all forces are internal, and a free-body diagram of the assembly is as shown in Figure 7.25a. The bar cross-sectional area to be used in computations is a nominal area equal to the area of a plain bar that has the same weight per foot of length as the deformed bar. As an alternative to this approach, two structural analyses can be performed (Gaylord et al., 1992). Compute the nominal shear strength of the end panel if the first intermediate stiffener is placed 70 inches from the support. Figure 10.1 shows some of the possibilities. It must support a service dead load of 100 kips and a service live load of 100 kips. 63 h = = 168 t w 3鼻8 5.70 29, 000 E = 5.70 = 161.8 36 Fy Since h鼻t w > 5.70 E鼻Fy , the web is slender and the provisions of AISC F5 apply. From AISC Table J3-1, the minimum bolt tension is Tb = 28 kips. This loading subjects the top chord to bending as well as axial compression, so the top chord is designed as a beam–column (see Chapter 6). Impact loading occurs in relatively few buildings, notably industrial buildings, and fatigue loading is rare, with thousands of load cycles over the life of the structure required before fatigue becomes a problem. When the dead load is applied to the cambered beam, the curvature is removed, and the beam becomes level. 3.75(5鼻16) 3 5 ⎤ ⎡ 0.75(8) I= + 2⎥ + 8 ( ) (4 +) ⎦ = 271.3 in. For the loading of Fig. Design 4.6-1 a. AISC I6.3c gives equations for computing the available bond strength. Use of the transformed moment of inertia to compute deflections in composite beams tends to underestimate the actual deflection (Viest, et al., 1997). 172 Chapter 4 Compression Members 16⬘ HP 12 × 53 A572 Grade 50 FIGURE P4.3-3 4.3-4 Determine the available strength of the compression member shown in Figure P4.3-4, in each of the following ways: a. 75 (58) ft u The radius of gyration should be at least L 15(12) = = 0.6 in. No: 784 kips > 781 kips b. In addition, for a given load capacity, although a small weld must be made longer, a larger and shorter weld will require more volume of weld metal. Welders must be properly certified, and for critical work, special inspection techniques such as radiography or ultrasonic testing must be used. A relationship can then be found between the load and the plastic moment capacity, Mp. This technique will be illustrated in Example A.3. EXAMPLE A.3 The continuous beam shown in Figure A.5 has a compact cross section with a design strength fb Mp of 1040 ft-kips. Structural Steel Design Solution Manual Mccormac solutions manual for Structural Steel Design 4th solutions manual for Structural Steel Design 4th Ed. design solutions manual jon kleinberg algorithm design kleinberg solutions manual design solution manual 5th fifth edition steel design solution manual. Although the required spacing can be determined by trial and error, it can be solved for directly, which we do here. As indicated in Figure A.5, they are similar, with each segment undergoing a rigid-body motion. With a diagonal stiffener, use full-depth horizontal stiffeners, as shown in Figure 8.49. • Compute Qa = Ae鼻Ag, where Ae is the effective area, and Ag is the gross or unreduced area. 9.1-2 A W14 × 22 floor beam supports a 5-inch-thick reinforced concrete slab with an effective width b of 81 inches. For ASD, the allowable strength is Rn 58.59 = = 29.3 kips. The bearing load is therefore P = fp dt. Normal-weight concrete weighs approximately 145 pounds per cubic foot. From Chapter 5, the shear flow, based on elastic behavior, is given by f = FIGURE 10.4 VQ Ix 670 Chapter 10 Plate Girders where Q is the moment, about the neutral axis, of the area between the horizontal shear plane and the outside face of the section. 0.85 fc′b 0.85(4)(90) d a 1.683 y = + t − = 8.85 + 4.5 − = 12.51 in. For each of these systems, determine the required ASD load capacity. A., Galambos, T. The maximum permissible live-load deflection is 1⁄360 of the span length. Thus the tabulated strengths are based on the values of KL being equal to Ky L. “Riveted and Bolted Joints: Net Section Design.” Journal of the Structural Division, ASCE 89 (no. The top chord of an open-web steel joist must also be 6.9 Trusses with Top-Chord Loads between Joints 359 designed as a beam–column because an open-web steel joist must support uniformly distributed gravity loads on its top chord. For multiplication and division, each number used in an intermediate calculation should be expressed to four significant figures, and the result should be recorded to four significant figures. Miami. EXAMPLE 6.4 The horizontal beam–column shown in Figure 6.13 is subject to the service live loads shown. 225 kips 3.2-5 a. Inspection of the magnitudes and directions of the horizontal and vertical components of the forces confirms the earlier conclusion that the fastener selected is indeed the critical one. MEMBERS IN UNBRACED FRAMES In a beam–column whose ends are free to translate, the maximum primary moment resulting from the sidesway is almost always at one end. With ΔH caused by H, the ratio H鼻ΔH can be based on either factored or unfactored loads. From Section 5.5 of this book, with the flexural stress based on the plastic section modulus, Fb = 0.6Fy = 0.6(50) = 30.0 ksi and the required section modulus (before the beam weight is included) is Zx = M a 506.3 × 12 = = 203 in.3 Fb 30 Next, check the shear. Since it is difficult to know in advance where the stud will actually be placed, it is conservative to use a value of Rp = 0.6. In this book, we will use Rp = 0.6 when formed steel deck is used. Equilibrium requires that T0 equals N0. Zahn, C. ∂ 2 in. In the negative moment zones, the concrete slab will be in tension and therefore ineffective. As pointed out earlier, all tabulated values correspond to a slenderness ratio of 200 or less. • For shapes with h鼻t w > 3.76 E鼻Fy , Mn is obtained from the elastic stress distribution corresponding to first yielding of the steel. When a beam bends, the compression region (above the neutral axis) is analogous to a column, and in a manner similar to a column, it will buckle if the member is slender enough. Adequate: For shear, 6.3 kips < 16.2 kips; For tension, 18.9 kips < 24.7 kips 8.3-7 a. Ideally, holes should be placed in the web only at sections of low shear, and holes should be made in the flanges at points of low bending moment. Although it is sometimes possible to determine in advance whether bolt threads will be in the plane of shear, it may depend on such things as which side of the connection the bolt is installed from. Load effects, Q, and resistances, R, are random variables and depend on many factors. Compute the elastic section modulus S and the yield moment My with respect to the major principal axis. A B C D E 4 @ 30′ = 120′ FIGURE P10.7-9 a. Figure 9.7a shows the distribution corresponding to full tensile yielding of the steel and partial compression of the concrete, with the plastic neutral axis (PNA) in the slab. Taking moments about the x-axis (the neutral axis of the entire cross section) and tabulating the computations in Table 5.2, we get y = TABLE 5.2 ∑ Ay 61 = = 5.545 in. The second term is the sum of all hole diameters, and the third term is the sum of s2鼻4g for all inclined lines in the failure pattern. 1953. The length available for welding the stiffener to the web is L = (length − clip) × 2 sides × 2 stiffeners 5 = ⎛ 4.5 − ⎞ (2)(2) = 15.5 in. Use Fy = 50 ksi and determine whether a W12 × 50 is adequate. 1978. Wire rope and strand are made from steels of much higher strength than structural steels and are not covered by the AISC Specification. The column is of A992 steel, and the bracket is A36. The gusset plate, which is the thinner element, will control. Which AISC load combination controls? Since the bending moment appears to be dominant in this member, use Equation 6.10. No part of this work covered by the copyright herein may be reproduced, transmitted, stored, or used in any form or by any means graphic, electronic, or mechanical, including but not limited to photocopying, recording, scanning, digitizing, taping, web distribution, information networks, or information storage and retrieval systems, except as permitted under Section 107 or 108 of the 1976 United States Copyright Act, without the prior written permission of the publisher. Shallower beams can be used, and deflections will be smaller than with conventional noncomposite construction. The minimum spacing in any direction is 2 2⁄3 d = 2.33 in. Steel Design Guide Series, No. 4, 2nd edition. Design Considerations for Bolts. When the parts have separated, T=F or T0 + ΔT = F (7.7) At the point of impending separation, the bolt elongation and plate decompression are the same, and ΔT = Ab Eb AE δb = b b δ f Lb Lb (7.8) where df is the elongation corresponding to the release of the initial compression force N0. dead + live load defl. Check shear. A tension test was conducted on a specimen with a circular cross section of diameter 0.5 inch and a gage length of 8 inches. 3.6-6 Use LRFD and select an American Standard Channel shape for the following tensile loads: dead load = 54 kips, live load = 80 kips, and wind load = 75 kips. 140 Chapter 4 Compression Members EXAMPLE 4.9 A W12 × 58, 24 feet long, is pinned at both ends and braced in the weak direction at the third points, as shown in Figure 4.11. Since 3.92 × 10–3 > 2.60 × 10–3, use bx = 3.92 × 10–3. Similarly, an ST10 × 33 is cut from an S20 × 66, and an MT5 × 4 is cut from an M10 × 8. The mode of failure considered so far is referred to as flexural buckling, as the member is subjected to flexure, or bending, when it becomes unstable. Adequate: For shear, 9.25 kips < 24.4 kips; For tension, 27.7 kips < 37.3 kips. Cover plates will be used in spans BC and CD to give the relative moment strengths indicated. The service live load is 550 lb鼻ft. Such loading would occur at an interior column with beams connected to both sides. ANSWER ASD SOLUTION Use 8 inches total, 4 inches on each side. Adequate: Ma = 286 ft-kips < Mn鼻Ωb = 297 ft-kips, Va = 14.3 kips < Vn鼻Ω v = 116 kips 6.6-11 Adequate: Result of interaction equation = 1.00 5.5-7 201 ft-kips 6.6-13 a. 4.640 4.640 16.54 P2 = = 3.56 in. Galambos, T. The nominal shear strength, Fnv, is based on the ultimate tensile stress of the bolt with several modification factors. The bearing strength for the gusset plate is Rn = 2(22.02) + 2(39.15) = 122.3 kips The gusset plate controls. (If the flange thickness is adequate, the bolt strength will also be adequate.) If the actual flange thickness is different from the required value, the actual values of a and Bc will be different from those previously calculated. If we conservatively assume that the full area of the support is used, the required plate area A1 can be found as follows: fcPp ≥ Ru From AISC Equation J8-1, Pp = 0.85fc′A1. 4.9-8 In order to reinforce a column in an existing structure, two channels are welded to the column as shown in Figure P4.9-8. Unshored: After Concrete Cures After composite behavior is achieved, all loads subsequently applied will be supported by the composite beam. This organization was formerly known as the American Railway Engineering Association (AREA). Determine the allowable axial compressive strength of column AB. FIGURE 8.55 (8.16) 572 Chapter 8 Eccentric Connections If the beam is near the top of the column (that is, if the distance from the top face of the beam flange to the top of the column is less than the column depth), the result from Equation 8.16 should by reduced by half. 314 Chapter 6 Beam–Columns Evaluation of Cm The factor Cm applies only to the braced condition. 9.8-8 A composite floor system consists of steel beams supporting a formed steel deck and concrete slab. Using Cb = 1.0, a. 9.4 Steel Headed Stud Anchors 611 FIGURE 9.10 Miscellaneous Requirements for Steel Headed Stud Anchors The following requirements are from AISC I8.1, I8.2, and I8.2d: • Maximum diameter = 2.5 × flange thickness of steel shape (unless placed directly over the web) • Minimum length = 4 × stud diameter • Minimum longitudinal spacing (center-to-center) = 6 × stud diameter • Maximum longitudinal spacing (center-to-center) = 8 × slab thickness ≤ 36″ • Minimum transverse spacing (center-to-center) = 4 × stud diameter • Minimum concrete cover in the direction perpendicular to the shear force, V ′, = 1 inch • Minimum distance from the center of a stud to a free edge in the direction of the shear force = 8 inches for normal-weight concrete and 10 inches for lightweight concrete. 31.2 kips 5.5-3 a. For example, for LRFD, Pu ≤ 1.0 fc Pn and for ASD, Pa ≤ 1.0 Pn 鼻Ωc These expressions can be written in the general form Pr ≤ 1.0 Pc where Pr = required axial strength Pc = available axial strength If more than one type of resistance is involved, Equation 6.1 can be used to form the basis of an interaction formula. Great! We have the file you. Referring to Figure 9.6 and Table 9.3, we can compute the new location of the neutral axis as follows: y= ∑ Ay 5.44 y 2 + 137.3 = ∑A 10.88 y + 10.6 y (10.88 y + 10.6) = 5.44 y 2 + 137.3 5.44 y 2 + 10.6 y − 137.3 = 0 y = 4.143 in. Other limit states resulting from the application of concentrated loads to the top flange are web yielding, web crippling (buckling), and sidesway web buckling. For simplicity, all forces will be assumed to be symmetrical with respect to the axis of the bolt, and any eccentricity will be neglected. ho = 13.3 in. Shear Strength of Anchors Used with Steel Deck The shear strength of one stud depends on the values of Rp and Rg in Qn = 0.5 Asa fc⬘ E c ≤ Rg R p Asa Fu (AISC Equation I8-1) If the studs are welded directly to the top flange of the steel beam (no deck), Rp = 1.0 and Rg = 0.75. If the ultimate stress Fu is substituted for the yield stress Fy in the expression for flexural strength, much better agreement with test results is obtained (Thornton, 1992 and Swanson, 2002). GT STRUDL steel design offers a variety of international steel design codes for Base Verification Manual, and by approximately 4,700 additional general V&V test While other solutions may only allow you to search for training and help. Chicago, AISC. That is, the displacement is zero at each end. For example, consider a number calculated on a 12-digit calculator and recorded to four significant figures. Not adequate: Mu = 1090 ft-kips > f bMn = 952 ft-kips b. Estimate the value of the proportional limit. 1 t Try a 1 × 31⁄2 cross section. American Institute of Steel Construction, 1997a. pPa + bx Max + by May = (3.01 × 10−3)(201) + (4.71 × 10−3)(67) + 0 = 0.921 < 1.0 (OK) Verify that Equation 6.8 is the correct one: Pa = pPa = (3.01 × 10 −3)(201) = 0.605 > 0.2 Pn 鼻Ωc ∴ Equation n 6.9 controls, as assumed. These yielded regions are indicated by the dark areas in Figure 5.3c and d. The strength of a fillet weld can now be summarized for both load and resistance factor design and for allowable strength design. We cover these connection methods in detail in Chapter 7, “Simple Connections.” When the connectors are snug-tight bolts, the modified slenderness ratio is 2 ⎛ KL ⎞ = ⎛ KL ⎞ + ⎛ a ⎞ ⎝ r ⎠m ⎝ r ⎠0 ⎝ ri ⎠ 2 (AISC Equation E6-1) where ⎛ KL ⎞ = original unmodified slenderness ratio ⎝ r ⎠0 When the connectors are fully-tensioned bolts or welds, the modified slenderness ratio depends on the value of a鼻ri: When a鼻ri ≤ 40, the slenderness ratio is not modified; that is, ⎛ KL ⎞ = ⎛ KL ⎞ ⎝ r ⎠ m ⎝ r ⎠0 (AISC Equation E6-2a) When a鼻ri > 40, 2 ⎛ KL ⎞ = ⎛ KL ⎞ + ⎛ K i a ⎞ ⎝ r ⎠m ⎝ r ⎠0 ⎝ ri ⎠ 2 where K i = 0.5 for angles back-to-back = 0.75 for channnels back-to-back = 0.86 for all other cases (AISC Equation E6-2b) 4.9 Built-Up Members 167 The column load tables for double angles are based on the use of welds or fully tightened bolts. For a bolted truss, the bolt lines are the working lines, and in welded trusses the centroidal axes of the welds are the working lines. Determine the location and size of intermediate stiffeners. It is not our purpose to describe in this book all available design aids in detail, but some are worthy of note, particularly the curves of moment strength versus unbraced length given in Part 3 of the Manual. Remember that ΔH is the drift caused by H, so if the drift index is based on service loads, then the lateral loads H must also be service loads. Design of Welded Structures (Blodgett, 1966) contains many useful suggestions for the design of economical welded plate girders. During plastic collapse, the mechanism motion will be as shown in Figure 5.4. Structural analysis based on a consideration of collapse mechanisms is called plastic analysis. Select the lightest cross-sectional shape that has these properties. The following procedure is recommended. The distinction between the two is that rods are solid and cables are made from individual strands wound together in ropelike fashion. Determine the overall depth. The shear is transmitted mostly through the web FIGURE 8.30 ≈ FIGURE 8.31 ≈ 520 Chapter 8 Eccentric Connections connection, as in the shear connection, and the moment is transferred through the flanges. (Clip the inside corners to avoid the fillets at the column flange-to-web intersection. So that we can use the values in Manual Table 7-5, try 11⁄4 in. Connections designed to be moment-free have some moment restraint. To avoid a conflict between the stiffener angles and the flange angles, filler plates were added to the web so that the stiffeners could clear the flange angles, as shown in Figure 10.1c. Using the spacing and edge distances selected, the minimum length is L = 11⁄4 in. Thus each bolt in the connection is capable of resisting a load of P = F, even if the bolt shank does not bear on the connected part. Deep, thin webs account for many of the special problems associated with plate girders, including local instability. problems for which knowledge of the answer leads directly to the solution 2. ANSWER ASD SOLUTION Design strength of the weld is 53.5 kips. If a shape is slender, its strength limit state is local buckling, and the corresponding reduced strength must be computed. 144 Chapter 4 Compression Members EXAMPLE 4.11 The column shown in Figure 4.13 is subjected to a service dead load of 140 kips and a service live load of 420 kips. (The beam web has a thickness of tw = 0.380 in. Bars should be specified to the nearest 1⁄4 inch in width and to the nearest 1⁄8 inch in thickness (the precise classification system is given in Part 1 of the Manual under the heading “Plate Products”). If the floor consists of a metal deck and concrete fill, the combined weight can usually be obtained from the deck manufacturer’s literature. Substituting into Equation 7.17, we get φb pt 2f Fy Tb ′ = 4 (1 + δα) tf = 4Tb ′ φb pFy (1 + δα) where T is the factored load per bolt. 4 4 m= Conservatively, let l = 1.0, resulting in = max(m, n, ln′) = max(1.75, 2.5, 2.5) = 2.5 in. 26 Chapter 2 Concepts in Structural Steel Design is being taken as the dominant effect in combination 2, and one of the three effects, Lr , S, or R, will be dominant in combination 3. If h kE ≤ 1.10 v tw Fy the strength is based on shear yielding, and Vn = 0.6Fy Aw (AISC Equation G3-1) where Aw = cross-sectional area of the web. Eccentric Welded Connections: Shear Plus Tension 8.5-1 Determine the maximum load in the weld in kips per inch of length. We will use the procedure as outlined previously, with one slight modification. SOLUTION Try lateral bracing at midspan. For transversely loaded members, Cm can be taken as 1.0. A more refined procedure for transversely loaded members is provided in the Commentary to Appendix 8 of the Specification. Check Equation 6.9 with p = 3.00 × 10−3, bx = 4.32 × 10−3, by = 11.7 × 10−3. “Economical Use of Cambered Steel Beams.” 1990 National Steel Construction Conference Proceedings. The corresponding bending moments are MA = 234(15) – 60(3) – 6(15)2鼻2 = 2655 ft-kips MB = 234(18) – 60(6) – 6(18)2鼻2 = 2880 ft-kips MC = 234(21) – 60(9) – 6(21)2鼻2 = 3051 ft-kips From AISC Equation F1-1, 12.5 M max 2.5 M max + 3 M A + 4 M B + 3 MC 12.5(3168) = = 1.083 2.5(3168) + 3(2655) + 4(2880) + 3(3051) Cb = FIGURE 10.12 10.6 Bearing Stiffeners 687 ⎛ Lb − L p ⎞ ⎤ ⎡ Fcr = Cb ⎢ Fy − 0.3Fy ⎜ ⎟ ≤ Fy ⎝ Lr − L p ⎠ ⎥ ⎦ ⎣ ⎛ 12 − 10.77 ⎞ ⎤ ⎡ = 38.43 ksi = 1.083 ⎢36 − (0.3 × 36) ⎜ ⎝ 36.75 − 10.77 ⎟⎠ ⎥⎦ ⎣ Since 38.43 ksi > Fy = 36 ksi, use Fcr = 36 ksi (same as for the other limit states). 259 kips 4.3-3 196 kips 4.3-5 a. The concentrated loads are live loads. Therefore, fb Mnx = 1.30(33.0) = 42.90 ft-kips From the uniform load table for C shapes, fb Mpx = 42.9 ft-kips ∴ Use fb Mnx = 42.9 ft-kips. Applied service-load shear = 50 kips Applied service-load tension = 100 kips a. At an arbitrary point O, there is a bending moment caused by the uniform load and an additional moment Py, caused by the axial load acting at an eccentricity from the longitudinal axis of the member. Compute the nominal compressive strength for A500 Grade B steel (Fy = 46 ksi). Do not exclude slender shapes from consideration. Therefore, the net area FIGURE 3.17 3.4 Staggered Fasteners 61 of 5.413 in.2 should be multiplied by 10⁄9 to obtain a net area that can be compared with those lines that resist the full load. The maximum size that can be made with a single pass of the electrode is approximately 5⁄16 inch, and multiple passes will add to the cost. The largest required thickness is therefore required by the limit state of shear rupture. The steel strength will not be fully developed in a partially composite beam, so a larger shape will be required than with full composite behavior. (Shapes built up from plate elements are also covered in Chapter 10 of this book.) Problems 281 Problems Bending Stress and the Plastic Moment 5.2-1 A flexural member is fabricated from two flange plates 1⁄2 × 16 and a web plate 1⁄4 × 20. Kx L 18 = = 10.29 ft > 9 ft rx 鼻ry 1.75 The interpolated allowable strength is Pn = 460 kips > 400 kips Ωc (OK) Next, investigate the W10 shapes. 5.14-4 Design a column base plate for a W10 × 33 column supporting a service dead load of 35 kips and a service live load of 35 kips. 8.4-11 Solve Problem 8.4-4 by the ultimate strength method (use the tables in Part 8 of the Manual). These members are subjected primarily to bending. 614 Chapter 9 Composite Construction 4. The web must satisfy the slenderness limitation of AISC F13.2. The limiting value of h鼻tw will depend on the aspect ratio a鼻h. All connections are with longitudinal welds. The beam curve shown in Figure 5.25 is for a compact shape, so the value of Mn for sufficiently small values of Lb is Mp. As discussed in Section 5.6, if the shape is noncompact, the maximum value of Mn will be based on flange local buckling. Since all elements of the weld receive an equal portion of the direct shear, the direct shear stress is f1 = P L where L is the total length of the weld and numerically equals the shear area, because a unit throat size has been assumed. To encourage use of the Manual, we did not reproduce its tables and graphs in this book. This shape is compact, as assumed (noncompact shapes are marked as such in the table); therefore Mn = Mp, as assumed. 4.7 More on Effective Length 141 The available strengths given in the column load tables are based on the effective length with respect to the y-axis. Ag = 6.5t = 3.741 in.2 or t = 0.576 in. The one exception is continuous beams with no axial compression, which may be analyzed by traditional first-order plastic analysis. Evaluate the selected interaction equation with the values of p, bx , and by for the trial shape. It is not known whether the bolt threads are in the plane of shear, so conservatively assume that they are. Although an end return is usually used for a weld of this type, conservatively for simplicity it will be neglected in the following calculations. (OK) ANSWER Use two plates 5⁄16 × 3 × 0′ – 41⁄2″. At a location of high shear in a girder web, usually near the support and at or near the neutral axis, the principal planes will be inclined with respect to the longitudinal axis of the member, and the principal stresses will be diagonal tension and diagonal compression. They are used in various types of structures and include truss members, bracing for buildings and bridges, cables in suspended roof systems, and cables in suspension and cable-stayed bridges. Verify the assumption on U to be sure that the strength of the member is government by yielding. American Institute of Steel Construction. From Part a, the shear, bearing, and tensile strengths are satisfactory. In most cases the functional design, including the establishment of the number of stories and the floor plan, will have been done by an architect, and the structural engineer must work within the constraints imposed by this design. 2 2 2 2 and the design strength is 0.90(384.0)(11.97) 12 = 345 ft-kips > 305 ftt -kips f b Mn = (OK) Check the shear: Vu = wu L 2.711(30) = = 40.7 kips 2 2 From the Zx table, fvVn = 106 kips > 40.7 kips ANSWER (OK) Use a W16 × 26. LRFD Specification for Single-Angle Members. For ASD, the allowable shear strength is Rn鼻Ω, where Ω = 1.67. (This determination could also be made by observing that there is no footnote in the dimensions and properties tables to indicate that the shape is not compact.) a. AISC G2.2 requires the clear distance between intermittent fillet welds be no more than 16tw or 10 inches. With a formed steel deck, these constants can take on different values. Compute a: 27.06 B −1= − 1 = 0.353 20 T 0.353(3.469鼻1.988) [(B鼻T − 1)](a ′鼻b ′) α= = = 2.139 δ {1 − [(B鼻T) − 1](a ′鼻b ′)} 0.75[1 − 0.353(3.469鼻1.988)] Since ⎪ a ⎪ > 1.0, use a = 1.0. From Equation 7.19, Ωb 4Tb ′ 1.67(4)(30)(1.988) = pFu (1 + δα) 4(65)[1 + 0.75(1.0)] = 0.936 in. In this case, the affected member must be designed to function as both a tension member and a compression member. Le. b. The compression-flange strength is therefore based on yielding, and Fcr = Fy = 36 ksi. If the purlins are simply supported at the trusses (or rigid frame rafters), the maximum bending moment about each axis is wL2鼻8, where w is the appropriate component of load. Verify the value of ry given in the properties table in the Manual. Steel Design by William T. Determine the design strength for LRFD. When > 3 or (Each of these cases corresponds to kv = 5.) h h ⎣ (h鼻t w) ⎦⎥ 2 Aw > 2.5 c. The guiding principle at that time was to read and record numbers as accurately as possible, which meant three or four significant figures. Estimate the 0.2% offset yield strength. The approximate length of each diagonal stiffener is Lst = dc 8.25 = = 19.7 in. If the composite section is inadequate, select another steel shape for trial. This is the tributary area for this member, and it consists of half the floor between beam EF and the adjacent beam on each side. What is the nominal shear strength if no intermediate stiffeners are used? Steel Structures Design: ASD/LRFD — Download - Duration: 0:09. 4): 143–154. Properties of rivets can be found in the ASTM Specification (ASTM, 2010c). 1 FIGURE 3.5 Section SOLUTION 2L5 × 3 × 5⁄16 LLBB Figure 3.5 illustrates the notation for unequal-leg double-angle shapes. The design of the bearing plate consists of three steps. Steel Design Guide Series No. 13, Chicago: AISC. The background and development of the plate thickness equation is presented here in LRFD terms. Use Kx = Ky = 1.0. Assume 3⁄8-inch gusset plates, and use A36 steel. The effective length KL is 16 feet, and there are three fully tightened intermediate connectors. Hence Number of bolts = total load 48.0 = 7.5 bolts = load per bolt 6.44 Eight bolts will be required. If the line of action of the resultant force to be resisted passes through the center of gravity of the connection, each part of the connection is assumed to resist an equal share of the load, and the connection is called a simple connection. We first require that the strain in the concrete at any point be equal to the strain in any replacement steel at that point: ε c = ε s or fc fs = Ec Es and fs = Es fc = nfc Ec (9.1) 596 Chapter 9 Composite Construction e FIGURE 9.3 (a) Strain (e) f (b) Actual Stress (f = Ee) f (c) Stress in Transformed Section where Ec = modulus of elasticity of concrete Es n= = modullar ratio Ec AISC I2.lb gives the modulus of elasticity of concrete as* Ec = wc1.5 fc′ ksi where wc = unit weight of concrete in lb鼻ft3. Note, however, that the theoretical and recommended design values are the same for conditions (d) and (f) in Commentary Table C-A-7.1. The reason is that any deviation from a perfectly frictionless hinge or pin introduces rotational restraint and tends to reduce K. Assume that there is partial lateral support equivalent to bracing at the ends and at midspan. 1 Actual edge distance = 1 in. Proportioning the intermediate stiffeners by the AISC rules does not require the computation of any forces, but a force must be transmitted from the stiffener to the web, and the connection should be designed for this force. The beam and column steel is A992, and the shelf angle is A36. For those shapes, a鼻h does not apply, kv = 5, and there is no tension field. Satisfactory: s = 3 in. To be able to use them, an artifice known as the transformed section is employed to “convert” the concrete into an amount of steel that has the same effect as the concrete. The web will need to resist only the diagonal tension, hence the term tension-field action. Plate tension member connections of the type shown in Figure 7.42, similar to those in the preceding examples, are in the special case category for shear lag in welded connections, covered in Chapter 3. Select ΣQn. This is the Manual’s notation for the compressive force C, which is the smallest of As Fy , 0.85fc′Ac , and the total steel anchor strength (which we have been calling ΣQn). The required strength and stiffness for stability can be added directly to the requirements for bracing to resist lateral loading. The applied loads are wa = 3 kips鼻ft and Pa = 9 kips The left-end reaction is wa L + Pa 3(24) + 9 = = 40.5 kips 2 2 and the bending moment at a distance x from the left end is ⎛ x⎞ M = 40.5 x − 3x ⎜ ⎟ = 40.5 x − 1.5 x 2 (for x ≤ 12 ft) ⎝ 2⎠ Compute the moments required for the computation of Cb: For x = 3 ft, MA = 40.5(3) − 1.5(3)2 = 108.0 ft-kips For x = 6 ft, MB = 40.5(6) − 1.5(6)2 = 189.0 ft-kips For x = 9 ft, MC = 40.5(9) − 1.5(9)2 = 243.0 ft-kips For x = 12 ft, Mmax = 40.5(12) − 1.5(12)2 = 270.0 ft-kip Cb = = 12.5 M max 2.5 M max + 3M A + 4 M B + 3MC 12.5(270) = 1.36 2.5(270) + 3(108) + 4(189) + 3(243) (OK) 238 Chapter 5 Beams Enter the charts with an unbraced length Lb = 12 ft and a bending moment of M a 270 = = 199 ft-kips Cb 1.36 Try W21 × 48. The bending moment is uniform over the unbraced length, so Cb = 1.0. Try a W10 shape. If the actual bolt force, which includes the prying force q, is desired, a will need to be recomputed as follows. Because the use of this section is so widespread, tables of properties of various combinations of angles are included in the AISC Steel Construction Manual. Hendrick, A., and Murray, T. Gravity load can produce sideway if the frame is unsymmetrical or if the gravity loads are unsymmetrically placed. The standard notation for specifying an electrode is the letter E followed by two or three digits indicating the tensile strength in kips per square inch and two digits specifying the type of coating. The connection is through the flange with three 3⁄4-inch diameter bolts in each line. 3⁄8″ PL L4 × 4 × 1⁄4 A36 steel FIGURE P7.11-8 7.11-9 Use ASD to select a double-angle tension member and design a welded connection that will resist a dead load of 12 kips and a live load of 36 kips. Assume that the bearing strength is adequate and use an elastic analysis to determine the following: a. CHAPTER 9 Composite Construction 9.1 INTRODUCTION Composite construction employs structural members that are composed of two materials: structural steel and reinforced concrete. The length is 14 feet. Problems 463 11⁄2″ 5″ 11⁄2″ 11⁄2″ 23⁄4″ 23⁄4″ 11⁄2″ 2 @ 11⁄2″ 1 1⁄4″ PL 1⁄2″⁄4″ PL PL FIGURE P7.4-2 7.4-3 Determine the number of 3⁄4-inch-diameter, Group A bolts required, based on shear and bearing, along line a–b in Figure P7.4-3. For example, in combination 2 the load factor for the live load L is 1.6, whereas in combination 3, it is 0.5. The reason is that the live load * This load does not include ponding, a phenomenon that we discuss in Chapter 5. A large number of columns are in this category. Guide to Stability Design Criteria for Metal Structures, 3rd. Because C is smaller than As Fy , part of the steel section must be in compression, and the plastic neutral axis is in the steel section. Select a trial layout. Sometimes a standard shape is augmented by additional cross-sectional elements, as when a cover plate is welded to one or both flanges of a W-shape. 272 Chapter 5 Beams Design of Roof Purlins Roof purlins that are part of a sloping roof system can be subjected to biaxial bending of the type just described. Compute pPr. If this is greater than or equal to 0.2, use interaction Equation 6.9. If pPr is less than 0.2, use Equation 6.10. • Fcr was computed. He was a recipient of the University of Memphis Distinguished Teaching Award in 2000. Compute the aspect ratio a鼻h and the web slenderness ratio h鼻tw. For the two plates (four welds), use 5.400 × 2 = 10.80 kips鼻in. Select a W16 shape. 1989. Flexural strength: This is the same as the LRFD solution up to the computation of Cb. For this, we need the service load moments. Plain carbon steels: mostly iron and carbon, with less than 1% carbon. 5.15-5(b), the beam is unsatisfactory: Result of interaction equation is 1.06. Because this element is located at the neutral FIGURE 5.17 5.8 Shear Strength 217 axis, it is not subjected to flexural stress. Required Ag = Pu 600 = = 33.3 in.2 φc Fcr 0.90(20) Try a W18 × 119: Ag = 35.1 in.2 > 33.3 in.2 (OK) KL 26 × 12 = 116.0 < 200 = 2.69 rmin Fe = p 2E (KL鼻r)2 = p 2 (29, 000) (116.0)2 (OK) = 21.27 ksi 136 Chapter 4 Compression Members Since KL E > 4.71 = 113, AISC Equation E3-3 applies. To deal with this combined loading, we temporarily look ahead to Chapter H of the Specification, “Design of Members for Combined Forces and Torsion” (also the subject of Chapter 6 of this textbook, “Beam–Columns”). The corresponding bolt force, including the effects of prying, is Bc. In connections of this type, bending caused by the prying action will usually control the design of the connected part. Add the reactions from this fixed-end beam to the actual joint loads to obtain total joint loads. Required Ab = ANSWER T 3.203 = = 0.1473 in.2 Ft 21.75 Use a 5⁄8-inch-diameter threaded rod (Ab = 0.3068 in.2) for the sag rods. From the ASD relationship, Equation 2.7, Rn Ω R 158.5 ≤ n 1.67 Rn ≥ 265 kips Ra ≤ ANSWER The required nominal strength is 265 kips. This is especially true of statically indeterminate structures. ⎝ 16 ⎠ controls). wu = 1.2wD + 1.6wL = 1.2(0) + 1.6(4.5) = 7.2 kips鼻ft 1 1 wu L2 = (7.2)(30)2 = 810.0 ft-kips 8 8 = required fb Mn Required moment strength Mu = Assume that the shape will be compact. SOLUTION First, compute the nominal flexural strength for x-axis bending. A maximum value of stress is reached, after which the specimen begins to “neck down” as the stress decreases with increasing strain, and fracture occurs. (OK) A 2(32.75) ANSWER ASD SOLUTION The maximum factored load reaction = 490 kips. Practical design of welded connections requires a consideration of such details as maximum and minimum weld sizes and lengths. Ω The shear rupture strength of the stiffener is Rn = 0.3Fu t × 2 = 0.3(58)(3鼻4)(2) = 26.1 kips鼻in. The deck is shown in Figure P9.8-8, and the total depth from bottom of deck to top of slab is 61⁄2 inches. (OK) Re quired tf = 7.8 High-Strength Bolts in Tension 427 Both the number of bolts selected and the flange thickness are adequate, and no further computations are required. 5.15a) For a doubly-symmetric I shape, c = 1.0. AISC Equation F2-4 gives Fcr = = Cb p 2 E (Lb 鼻rts)2 1 + 0.078 1.14p 2 (29, 000) ⎛ 45 × 12 ⎞ ⎜ ⎝ 4.11 ⎠ 2 Jc S x ho ⎛ Lb ⎞ ⎜ ⎝ r ⎠ ts 2 2 1 + 0.078 4.06(1.0) ⎛ 45 × 12 ⎞ = 37.20 ksi 143(13.3) ⎜ ⎝ 4.11 ⎠ From AISC Equation F2-3, Mn = Fcr Sx = 37.20(143) = 5320 in.-kips < Mp = 7850 in.-kips This is smaller than the nominal strength based on flange local buckling, so lateraltorsional buckling controls. If fr is derived from factored loads, it may be compared with the design strength of a unit length of weld. For the current case, the shape with a section modulus closest to 203 in.3 is a W18 × 97, but the lightest shape with sufficient section modulus is a W24 × 84, with Zx = 224 in.3 Try a W24 × 84. Use the alternative value of U from AISC Table D3.1. 21⁄4″ 3″ 3″ 3″ 7⁄8-in.-diameter 21⁄4″ bolts FIGURE P3.3-5 W16 × 45 Problems 3.3-6 95 The tension member shown in Figure P3.3-6 is a C12 × 20.7 of A572 Grade 50 steel. Connection Requirements for Built-Up Members Composed of Plates or Both Plates and Shapes When a built-up member consists of two or more rolled shapes separated by a substantial distance, plates must be used to connect the shapes. 38 Chapter 2 Concepts in Structural Steel Design Problems Note All given loads are service loads. Use the tables in Part 4 of the Manual to select a concrete-filled rectangular HSS. Research reported by Munse and Chesson (1963) 3.3 Effective Area 51 FIGURE 3.6 suggests that shear lag be accounted for by using a reduced, or effective, net area. Use LRFD and the minimum size fillet weld to design a connection. Lateral bracing, which prevents lateral translation, should be applied as close to the compression flange as possible. U.S. customary units are used throughout, with no introduction of SI units. For the 45-kip service load, Pu = 1.2(0.25 × 45) + 1.6(0.75 × 45) = 67.5 kips For the 57-kip service load, Pu = 1.2(0.25 × 57) + 1.6(0.75 × 57) = 85.5 kips Three mechanisms must be investigated, one in each span. When the ribs are parallel to the beam, the concrete may be included in determining section properties, and it must be included in computing Ac [AISC I3.2c(3)]. gusset plate connection). In each of the three analysis methods, member out-ofplumbness is accounted for by including fictitious lateral loads, called notional loads, in the load combinations. Determine the available flexural strength. The connected parts are of A36 steel. If possible, the structure or connection geometry should be modified to remove the eccentricity. The nominal bearing strength of a single bolt therefore can be expressed as Rn = 1.2 c tFu ≤ 2.4 dtFu (AISC Equation J3-6a) where c = clear distance, in the direction parallel to the applied load, from the edge of the bolt hole to the edge of the adjacent hole or to the edge of the material t = thickness of the connected part Fu = ultimate tensile stress of the connected part (not the bolt) 384 Chapter 7 Simple Connections FIGURE 7.8 c c 2 1 s e For load and resistance factor design, the resistance factor is f = 0.75, and the design strength is fRn = 0.75Rn For allowable strength design, the safety factor is Ω = 2.00, and the allowable strength is Rn R = n Ω 2.00 Figure 7.8 further illustrates the distance c. Column BC: For joint B, as before, G = 0.95 4.7 More on Effective Length 149 For joint C, a pin connection, the situation is analogous to that of a very stiff column attached to infinitely flexible girders—that is, girders of zero stiffness. Direct tension indicators. Plastic design is based on a consideration of failure conditions rather than working load conditions. In Figure 5.49a, the top flange is assumed to provide the total resistance to the horizontal component of the load. Hence, unless a “fixed” end is perfectly fixed, the more conservative design values are to be used. A procedure developed by Aminmansour (2000) simplifies this process, especially the evaluation of trial shapes. Select the overall depth. From AISC Equation F13-1, Mn = Fu A fn 166.3 (94.5) = 4498 in.-kips Sx = 3.494 A fg fb M n = 0.90(4498) = 4048 in.-kips = 337 ft-kips > 210 ft-kkips (OK) ANSWER ASD SOLUTION Use the connection shown in Figure 8.42 (column stiffener requirements will be considered in Section 8.7).* (Portions of this solution will be taken from the LRFD solution.) Ma = MD + ML = 35 + 105 = 140 ft-kips Va = VD + VL = 6.5 + 19.5 = 26 kips *Figure 8.42 also shows the symbol for a bevel groove weld, used here for the beam flange plate-to-column connection. In this book, we do this and use Section F5 for girders with either noncompact or slender webs. Problems 471 120 k 1 1 W18 × 60 2L5 × 3 × 1⁄2 long legs back-to-back⁄8″ PL 7 2L5 × 3 × 3⁄8 long legs back-to-back Section FIGURE P7.9-4 A bracket cut from a W12 × 120 is connected to a W12 × 120 column flange with 12 Group A bearing-type bolts as shown in Figure P7.9-5. Adequate: Before concrete has cured, Mu = 233 ftkips < 484 ft-kips; after concrete has cured, Mu = 896 ft-kips < 966 ft-kips; Vu = 89.6 kips < 256 kips b. For homogeneous cross sections, the principal axes coincide with the centroidal axes. Get instant access to our step-by-step Steel Design solutions manual. A36 steel is used, and holes are for 7⁄8-inch-diameter bolts. Ma = MD + ML = 54.11 + 61.88 = 116.0 ft-kips Alternatively, the loads can be added before the moment is computed: wa = wD + wL = 0.481 + 0.550 = 1.031 kips鼻ft Ma = 1 1 wa L2 = (1.031)(30)2 = 116.0 ft-kips 8 8 The allowable moment is Mn M = n = 0.6 M n = 0.6(225.0) = 135 ft-kips >116 ft-kips Ωb 1.67 (OK) 202 Chapter 5 Beams Allowable stress solution: The applied stress is fb = M a 116.0(12) = = 29.5 ksi Sx 47.2 The allowable stress is Fb = 0.6 M n 0.6(225.0)(12) = = 34.3 ksi Sx 47.2 Since fb < Fb, the beam has enough strength. From Equation 6.8, EI* = 0.8tb EI = 0.8(1.0)EI = 0.8EI (In this book, we assume that the requirements for using tb = 1.0 have been met.) Pelx = π 2 EI * π 2 (0.8 EI x) π 2 (0.8)(29, 000)(533) = = = 3767 kips (1.0 × 15 × 12)2 (K1 L) 2 (K x L)2 Cmx Cmx 0.6 = 1 − (a Pr 鼻Pe1x) 1 − (1.00 Pu 鼻Pe1x) 1 − (300鼻3767) = 0.652 < 1.0 ∴ Use B1x = 1.0. B1x = The required moment strength is Mr = Mux = B1x Mntx + B2x Mtx = 1.0(135) + 0 = 135.0 ft-kips From the Beam Design Charts with Cb = 1.0 and Lb = 15 feet, the moment strength is fbMnx = 340 ft-kips and fb Mpx = 356 ft-kips From Figure 5.15g, Cb = 1.67 and Cb × (fb Mnx for Cb = 1.0) = 1.67(340) = 567.8 ft-kips This result is larger than fb Mpx; therefore, use fb Mnx = fb Mpx = 356 ft-kips. Consider eccentricity and check the connection designed in part b. The analysis for the required strength should account for the displaced geometry, member out-of-plumbness (deviation from vertical), and inelasticity. Most columns in rigid frames are actually beam–columns, and the effects of bending should not be ignored. Therefore, Pr will be the total load to be stabilized and will be the total column load acting above the brace. If the moment amplification method is used, then we can consider this to be the direct analysis method, and the effective length factor Kx can be taken as 1.0. Use Ky = 1.0. Determine whether this member is in compliance with the AISC Specification. Length available for weld 62 − 2(0.5) From 20.25鼻s = 3.662, s = 5.53 inches. For an interior load, the nominal strength for web crippling is ( ) 1.5 ⎡ b ⎛ t w ⎞ ⎤ EFy t f Rn = 0.80t w2 ⎢1 + 3 ⎜ d ⎟ ⎜ t f ⎟ ⎥ ⎦ tw ⎢ ⎣ (AISC Equation J10-4) For a load at or near the support (no greater than half the beam depth from the end), the nominal strength is ( ) 1.5 ⎡ b ⎛ t w ⎞ ⎤ EFy t f Rn = 0.40t w2 ⎢1 + 3 ⎜ d ⎟ ⎜ t f ⎟ ⎥ ⎦ tw ⎢ ⎣ foor b ≤ 0.2 (AISC Equation J10-5a) d or Rn = 0.40t w2 ( ) 1.5 ⎡ 4 b ⎛ t w ⎞ ⎤ EFy t f − 0.2 ⎥ ⎥ ⎢1 + ⎜ t f ⎟ ⎦ d tw ⎢ ⎣ for b > 0.2 d (AISC Equation J10-5b) The resistance factor for this limit state is f = 0.75. 4 sp. Part 4 includes a discussion of the Specification requirements for compression members and numerous design aids. For ASD, the allowable strength is M n 532.2 = = 319 ft-kips. Figure 10.2 illustrates the concept of tension-field action. The shear areas for both cases are 5 Agv = (3 + 3 + 3 + 1.25) × 2 = 12.81 in.2 8 5⎡ 7 ⎤ Anv = ⎢3 + 3 + 3 + 1.25 − 3.5 ⎛⎜ ⎞⎟ ⎥ × 2 = 8.984 in.2 ⎝ 8⎣ 8 ⎠⎦ 530 Chapter 8 FIGURE 8.41 Eccentric Connections 11⁄2″ 31⁄2″ 11⁄2″ 1⁄2″ 11⁄2″ 3″ 3″ 3″ (a) 11⁄4″ 11⁄2″ 31⁄2″ 11⁄2″ 1⁄2″ 11⁄2″ 3″ 3″ 3″ 11⁄4″ (b) If a transverse tension area between the bolts is considered (Figure 8.41a), the width is 3.5 in. These examples illustrate the application of the Specification and the Manual. This placement will give a design strength that approximately equals the maximum factored load shear of 234 kips. Use A992 steel and select a shape. The depth of the compressive stress block is a= C 530 = = 1.792 in. Check the deflection. Substituting and solving for ΣAb, we get 100 50 ⎞ = 58.5 − 1.667 ⎛⎜ ⎝ ∑ Ab ⎟⎠ ∑ Ab 100 = 58.5 ∑ Ab − 83.35 ∑ Ab = 3.134 in.2 The area of one bolt is Ab = π (3鼻4)2 = 0.4418 in.2 4 And the number of bolts required is nb = ∑ Ab 3.134 = = 7.09 Ab 0.4418 Try eight bolts. In most structures, however, slip is perfectly acceptable, and a bearing-type connection is adequate. Design the welds to develop the available strength of the member. The distribution of moments in the member is therefore a combination of the primary moment, the P-d moment, and the P-Δ moment. 80.9 kips 3.3-1 a. 3.8 Tension Members in Roof Trusses 79 FIGURE 3.27 3.8 TENSION MEMBERS IN ROOF TRUSSES Many of the tension members that structural engineers design are components of trusses. Eccentrically loaded connections, covered in Chapter 8, are those in which the line of action of the load does not act through the center of gravity of the connection. For LTB, M n = Mcr = p EI yGJ Lb ⎡ B + 1 + B2 ⎤ ⎣⎢ ⎦⎥ (AISC Equation F9-4) where ⎛ d⎞ B = ±2.3 ⎜ ⎟ ⎝ Lb ⎠ Iy J (AISC Equation F9-5) The positive sign is used for B when the stem is in tension, and the negative sign is used when the stem is in compression anywhere along the unbraced length. If a variable moment of inertia is desired, decisions must be made regarding the method of varying the flange size—that is, whether to use cover plates or different thicknesses of flange plate at different points along the length of the girder. Compute the nominal strength. Design a composite beam and specify the size and number of stud anchors required. Structural Welding Code – Steel (AWS D1.1:2010). For joint A, Ginelastic = ta × Gelastic = 0.9703(1.52) = 1.47 For joint B, Ginelastic = 0.9703(1.36) = 1.32 ANSWER From the alignment chart, Kx = 1.43. The moment amplification factor is therefore 1 1 − (P鼻Pe) Because the member deflection corresponds to a buckled shape, the axial load corresponds to a failure load—that is, a load corresponding to an LRFD formulation. The bolts are 7⁄8-inch-diameter, Group A and all steel is A36. They are clipped 1 inch at the inside edge, both top and bottom, to clear the flange-to-web welds. Both the tension member and the gusset plate are of A36 steel. Ae = An = Ag

− Ahole ⎛ 7 1⎞ = (1 × 3.5) − ⎪ + ⎪(1) = 2.5 in.2 > 2.409 in.2 ⎝ 8 8⎠ Check the slenderness ratio: Imin = 3.5(1)3 = 0.2917 in.4 12 A = 1(3.5) = 3.5 in.2 From I = Ar2, we obtain I min 0.2917 = = 0.2887 in.2 A 3.5 L 5.75(12) = 239 < 300 Maximum = 0.2887 r rmin = ANSWER ASD SOLUTION (OK) Use a PL 1 × 31⁄2. Continue the search and investigate a W12 × 53 (fc Pn = 611 kips for KL = 9 ft): Kx L 18 = = 8 .53 ft < 9 ft rx 龠ry 2.11 ∴ K y L controls for thhis shape, and fc Pn = 611 kips. Compute the nominal strength of the composite section. = 500 lbft wL = live load + partition load = 1200 + 100 = 1300 1b/ft wa = wD + wL = 0.500 + 1.300 = 1.800 kips/ft Ma = 1 1 wa L2 = (1.800)(30)2 = 202.5 ft-kips 8 8 Assume that d = 16 in., a龠2 = 0.5 in., and estimate the beam weight from Equation 9.6: w= 3.4Ωb M a (Fy d2 +t− a 2) = 3.4(1.67)(202.5 × 12) 50 (126 + 4.75 − 0.5) = 22.53 1b/ft Try a W16 *d* 26. It is to be attached to a 3⁄8-inch gusset plate with 7⁄8-inch-diameter, Group A bolts. Detroit. Design specifications represent what is considered to be good engineering practice based on the latest research. Both will be illustrated. If the required strength is greater than the available strength corresponding to any of these limit states, column web reinforcement must be provided. These recommendations are summarized in Table 1.2. Other steels can be used for these shapes, but the ones listed in Table 1.2 are the most common (Anderson and Carter, 2009). The bottom number is the live load per foot that will produce a deflection of 1龠360 of the span length. The compression flange is restrained against rotation at these same points. 9.1 Introduction 595 FIGURE 9.2 Almost all highway bridges that use steel beams are of composite construction, and composite beams are frequently the most economical alternative in buildings. Design a simply supported beam for the conditions shown in Figure P8.3-9. Try two plates 5⁄16 × 3 × 0′ – 4 1⁄2′′. where D is the weld size in sixteenths of an inch. The design strength per inch is 1.392D = 1.392(4) = 4.176 kips龠in.

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