Alternative Designs for Steel Ordinary Moment Frames

Dave K. Adams, S.E., M.ASCE

dadams@bwesd.com

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Dave K. Adams

- Education:
  - B.S. "Structural Engineering" from University of California, San Diego (1990)
- Experience:
  - 2012 - Present: Structural project manager at BWE (San Diego, CA)
  - 1990 – 2012: Structural engineer at Lane Engineers, Inc. (Tulare, CA)
- Affiliations/Registrations:
  - California and Illinois licensed structural engineer
  - Seismic safety assessment program evaluator with the California Office of Emergency Services
  - Subject matter expert (structural) for California Board of Registration for Professional Engineers, Land Surveyors and Geologists
  - Code Advisory Committee member (Structural Design/Lateral Forces) for California Building Standards Commission

Learning Outcomes

- Understand the provisions of ASCE 7-10, AISC 360-10, and AISC 341-10 relative to ordinary steel moment frames
- Recognize and defend alternative ordinary moment frame designs
- Compare some of the characteristics of ordinary, intermediate, and special moment frame systems
- Examine the intent behind code provisions for ordinary moment frame systems
- Learn how to design members of steel ordinary moment frame systems
- Learn how to calculate ordinary moment frame connections

Codes Referenced in This Presentation

- IBC 2012, "International Building Code"
- ASCE 7-10, "Minimum Design Loads for Buildings and Other Structures"
- AISC 360-10, "Specification for Structural Steel Buildings"
- AISC 341-10, "Seismic Provisions for Structural Steel Buildings"
- AWS D1.1, "Structural Welding Code - Steel"
- AWS D1.8, "Structural Welding Code – Seismic Supplement"

Understanding Moment Frames

- Vertical load-carrying space frames with lateral load resistance provided by moment-resisting connections between the beams and columns
- They can displace the most of all identified building systems when subjected to load, which meets good ductility requirements for seismic resistance
  - However, uncontrolled deformation can also lead to measurable P-Δ effects on building elements

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Understanding Moment Frames

- Moment frames are classified as "ordinary", "intermediate", or "special"
  - Associated with their ductility and capability to translate into the inelastic range of movement
- Highly prescriptive detailing and significant inspections are necessary to assure the intended performance
- Low and low-moderate seismic design category regions do not require significant inelastic capabilities
  - Ordinary moment frames make economical sense
  - Fewer prescriptive detailing and inspection requirements
  - A larger selection of shapes and materials can be joined
- Ordinary moment frames can be used in higher seismic design category regions, but with restrictions

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Alternative Designs and Methods

- "An alternative material, design, or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code …" (IBC, 104.11)
- "… and that the material, method, or work offered is, for the purpose intended, at least the equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability, and safety." (ibid)

Alternative Designs and Methods

- What is it that makes something "alternative"?
  - The structural system (element, member) is not specifically identified in the body of the code
  - The method for determining forces or stresses within structural elements is not well recognized
  - The material itself is not well documented or may be unrecognizable

Alternative Designs and Methods

- Performance-Based Design (ASCE 7, 1.3.1.3)
  - Allowed as an alternative to the “strength design” or “allowable stress design” prescriptive procedures
  - Demonstrate with analysis and testing that the chosen system provides equivalent (or better) reliability
    - Assumptions used in analysis to represent structural properties are to be based on testing or accepted standards
    - Documentation used to demonstrate compliance (analysis, testing) shall be submitted to the governing agency and also to independent peer review
    - “At the present time, there is no documentation of the reliability intended …” (C1.3.1.3)
  - The goal is to permit the use of alternate methods and special structures by demonstration of acceptable performance without strict compliance with the prescriptive requirements of the code

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Alternative Designs and Methods

- Load fluctuations, variability of material properties, and uncertainties with analytical models all contribute to the statistical relationship of load combinations (“strength”, “allowable stress”)
- The larger picture associated with alternative designs and methods, however, deals with structural reliability
  - Reliability is a measure of the probability of survival through a particular loading event (i.e. “survival” means maintaining stability, reasonable protection of components, and acceptable serviceability)
- The code defines reliability using principles of judgment, fitting, and optimization
  - If after a period of time the code has performed satisfactorily, basic parameters within the code may be judged to be acceptable and correct

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Alternative Designs and Methods

- Strategies for justifying alternatives:
  - Demonstrate through existing research that the proposed alternative (method, system, component, design) is not completely unheard of
  - Describe the assumptions made in analysis and protocols used in testing, demonstrating that they are reasonable
  - Explain that the conclusions of research, analysis, and testing show compliance with the code intent
    - Structural stability
    - Protection of elements and systems
    - Serviceability
    - Reliability

- So what might be considered an "alternative" ordinary moment resisting frame?
  - One in which the beam welding does not comply with the requirements of AISC 341
  - One in which member shapes joined together are different than commonly expected or defined in ASCE 7
  - One that deflects more than prescribed maximum allowable values per ASCE 7

Moment Frames – ASCE 7

- What are the differences between “building” and “nonbuilding” structural systems?
  - A “building structure” is any structure whose intended use (i.e. occupancy) includes shelter of human occupants. (11.2)
  - A “nonbuilding structure similar to a building” is designed and constructed in a manner similar to a building, will respond to ground motion like a building, and has a basic LFRS conforming to one of the types listed in Table 12.2-1 or 15.4-1. (11.2)

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Moment Frames – ASCE 7

- What are the differences between “building” and “nonbuilding” structural systems?
  - For both wind and seismic loads, the design and construction requirements for nonbuilding structures are intended to be somewhat more simplified and flexible
    - Easier to justify potential “alternates”
    - Can accommodate more imaginative structural member configurations and load imbalances
  - Most nonbuilding structures will still have discernible mode shapes in response to lateral forces, making them particularly suitable to static methods of analysis

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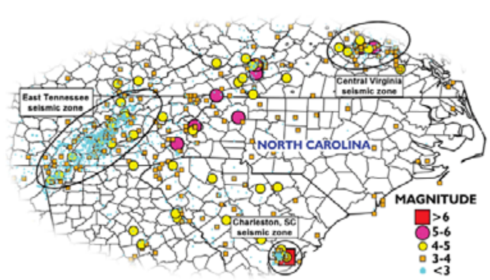
Moment Frames – ASCE 7 (seismic)

- Building systems, Table 12.2-1
  - Steel special moment frames ($R = 8$, $\Omega_0 = 3$, $C_d = 5.5$)
  - Steel intermediate moment frames ($R = 4.5$, $\Omega_0 = 3$, $C_d = 4$)
  - Steel ordinary moment frames ($R = 3.5$, $\Omega_0 = 3$, $C_d = 3$)
- Nonbuilding structures similar to buildings, Table 15.4-1
  - Steel special moment frames
    - AISC 341 ($R = 8$, $\Omega_0 = 3$, $C_d = 5.5$)
  - Steel intermediate moment frames
    - AISC 341 with restrictions ($R = 4.5$, $\Omega_0 = 3$, $C_d = 4$)
    - AISC 341 with permitted height increase ($R = 2.5$, $\Omega_0 = 2$, $C_d = 2.5$)
    - AISC 341 with unlimited height ($R = 1.5$, $\Omega_0 = 1$, $C_d = 1.5$)

Moment Frames – ASCE 7 (seismic)

- Nonbuilding structures similar to buildings, Table 15.4-1
  - Steel ordinary moment frames
    - AISC 341 with restrictions ($R = 3.5$, $\Omega_0 = 3$, $C_d = 3$)
    - AISC 341 with permitted height increase ($R = 2.5$, $\Omega_0 = 2$, $C_d = 2.5$)
    - AISC 360 with unlimited height ($R = 1$, $\Omega_0 = 1$, $C_d = 1$)



Moment Frames – ASCE 7 (seismic)

- Building systems: Single-story, <u>ordinary</u> steel moment frames are allowed in SDC D & E (12.2.5.6.1.a):
  - Maximum height = 65’
  - Roof dead load ≤ 20 psf
  - Dead load of exterior walls more than 35’ above the base tributary to the frame ≤ 20 psf
  - May be of unlimited height for frames which enclose equipment and only incidental repair persons, provided the combined roof dead and equipment load ≤ 20 psf
- Building systems: Multiple-story, <u>ordinary</u> steel moment frames are allowed in SDC D & E (12.2.5.6.1.b):
  - Maximum height = 35’
  - Light-frame construction (wood, metal studs)
  - Roof dead load ≤ 35 psf
  - Floor dead load ≤ 35 psf
  - Dead load of exterior walls ≤ 20 psf

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Moment Frames – ASCE 7 (seismic)

- Building systems: Single-story, <u>ordinary</u> steel moment frames are allowed in SDC F (12.2.5.6.2):
  - Maximum height = 65’
  - Roof dead load ≤ 20 psf
  - Dead load of exterior walls tributary to the frame ≤ 20 psf
- Nonbuilding structural systems:
  - “Similar to buildings” (15.5)
  - No weight restrictions
  - Pipe racks (15.5.2)
  - Steel storage racks (15.5.3)

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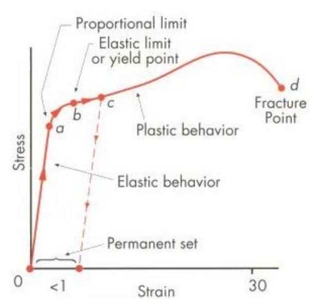
Moment Frames – ASCE 7 (other than seismic)

- Wind pressures:
  - All-heights building systems (Chapter 27)
  - Low-rise building systems (Chapter 28)
  - Nonbuilding structural systems (Chapters 27, 28, 29)
- Other loads:
  - Use load combinations in Chapter 2
  - Ice, snow, lateral earth pressures?

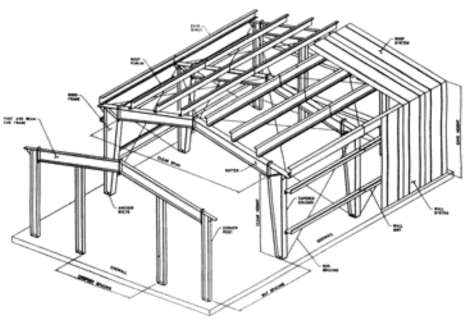
Structural Steel Design

- A complete load path shall be provided, capable of transferring the effect of carried forces from the points of origin to the foundation (IBC 1604.4)
- Connection types:
  - "Simple connections" transmit negligible moment
  - "Fully-Restrained (FR) moment connections" transfer moment with a negligible rotation between the connected members (i.e. members at 90° essentially remain at 90°)
  - "Partially-Restrained (PR) moment connections" transfer moments, but the rotation between connected members is not negligible and contributes to the force-deformation response characteristics of the connection



Structural Steel Design

- AISC 360-10, "Specification for Structural Steel Buildings":
  - A consensus document to provide uniform practice in design and inspection
    - Structural engineers who work in private companies, universities, code agencies, fabrication and production shops
  - Where conditions are not covered, alternative designs may be based on testing and/or analysis, subject to approval (A1)
  - Coordinated documents:
    - ASCE 7-10
    - ACI 318-08
    - AISC 341-10
    - AWS D1.1-10



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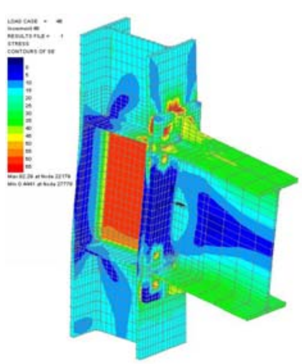
Structural Steel Design



- AISC 341-10, "Specification for Structural Steel Buildings":
  - A consensus document to address the design, fabrication and erection of structural steel and composite structures for high seismic performance
    - Includes provisions for ordinary moment frames
  - Contributions by the Building Seismic Safety Council (BSSC), Federal Emergency Management Agency (FEMA), National Science Foundation (NSF) and the Structural Engineers Association of California (SEAOC)
  - Horizontal diaphragms of steel members are typically design using the provisions of AISC 360, since they are intended to remain mostly elastic in response to lateral forces
    - Deck, rod bracing, beam bending

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Structural Steel Design

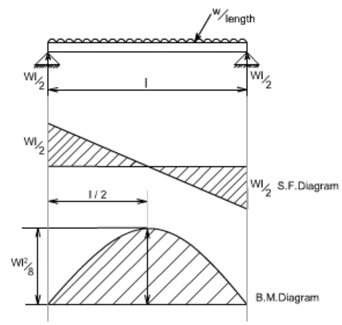
- Seismic design and detailing requirements are dictated by the State's adopted building code:
  - For <u>SDC A - C</u> (IBC 2205.2.1):
    - May use R from ASCE 7, Table 12.2-1 and design/detail per AISC 341
    - May use R = 3 for "systems not specifically detailed for seismic resistance" and follow design/detailing of AISC 360
  - For <u>SDC D - F</u>, use AISC 341 except for certain applications identified in ASCE 7, Table 15.4-1, "Non-Building Structures Similar to Buildings" (2205.2.2)
- AISC 360-10 is generally friendlier than AISC 341 in regards to compactness, lateral bracing, and connection types

Structural Steel Design – AISC 360

- Simplified <u>allowable stresses</u> for flexural members (strong axis bending):
  - Wide flange, channels, and S-shapes:
    - $M_{allow} = 0.66F_yS_x$ (where $L_b \le L_p$)
    - $M_{allow} = 0.42F_yS_x$ (where $L_b = L_r$)
    - For cases where $L_b > L_r$, see AISC manual
    - $L_p = 300r_y/(\sqrt{F_y})$, F_y in ksi, result will match units of "r"
    - r_y = Radius of gyration in the "weak" direction = $\sqrt{(I_y/A)}$
    - S_x = Section modulus in the "strong" direction
    - L_b = Length of member that is unbraced against buckling
    - L_r = Unbraced length for the limit state of inelastic lateral-torsional buckling (see AISC manual)

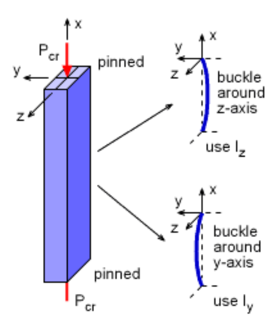
Structural Steel Design – AISC 360

- Simplified nominal strengths (LRFD) for flexural members (strong axis bending):
  - Wide flange, channels, and S-shapes:
    - $M_{LRFD} = 0.99F_yS_x$ (where $L_b \le L_p$)
    - $M_{LRFD} = 0.63F_yS_x$ (where $L_b = L_r$)
- Simplified bending strengths of HSS flexural members (subject to size limits):
  - Rectangular:
    - Allowable Moment = $0.66F_yS_x$
    - LRFD Moment = $0.99F_yS_x$
  - Round:
    - Allowable Moment = $0.78F_yS_x$
    - LRFD Moment = $1.17F_yS_x$



Structural Steel Design – AISC 360

- Simplified allowable stresses for compression members:
  - All types of shapes (except "slender" shapes):
    - $P_{allow} = 0.6F_yA_g(0.658)^P$, when $KL/r \le 800/(\sqrt{F_y})$ using ksi
    - $P_{allow} = 150{,}000A_g/(KL/r)^2$, when $KL/r > 800/(\sqrt{F_y})$
    - P (raised power) = $F_y(KL/r)^2/286{,}000$ (F_y in ksi)
    - KL = Effective length of member between braced points in either axis, which can typically be taken as unity for most conditions, and should be ≤ 200
    - Where shapes are "slender", an additional multiplier is necessary per Section E7
    - Section E4 provides additional requirements for channel shapes



Structural Steel Design – AISC 360

- Simplified nominal strengths (LRFD) for compression members:
  - All types of shapes (except "slender" shapes):
    - $P_{LRFD} = 0.9F_yA_g(0.658)^P$, when $KL/r \le 800/(\sqrt{F_y})$ using ksi
    - $P_{LRFD} = 226{,}000A_g/(KL/r)^2$, when $KL/r > 800/(\sqrt{F_y})$
    - P (raised power) = $F_y(KL/r)^2/286{,}000$ (F_y in ksi)
    - KL = Effective length of member between braced points in either axis, which can typically be taken as unity for most conditions, and should be ≤ 200

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Structural Steel Design – AISC 360

- Simplified allowable shear:
  - Wide flange, channels, and S-shapes = $0.4F_yA_w$
  - Rectangular HSS shapes = $0.36F_yA_w$
  - Round HSS and pipe shapes = $0.18F_yA_g$
- Simplified nominal shear strength (LRFD):
  - Wide flange, channels, and S-shapes= $0.6F_yA_w$
  - Rectangular HSS shapes = $0.54F_yA_w$
  - Round HSS and pipe shapes = $0.27F_yA_g$

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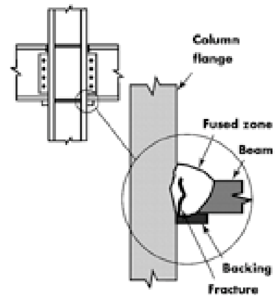
Structural Steel Design – AISC 360

- Simplified allowable stresses for welds:
  - Allowable shear force = $0.3F_{EXX}A_w$
  - Allowable shear force (fillet welds loaded in the transverse direction) = $0.45F_{EXX}A_w$
  - Allowable tension (PJP groove) = $0.32F_{EXX}A_w$
  - Allowable tension (CJP groove) = Strength of base metal
  - F_{EXX} = Tensile strength of weld metal/electrode
  - A_w = Cross-sectional area of weld
    - For fillet welds, multiply leg dimension by 0.7071 to obtain the throat thickness



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Structural Steel Design – AISC 341 (OMF)

- Chapter A
  - Minimum yield stress ≤ 55 ksi
  - R_y & R_t (Table A3.1)
  - AWS D1.8, Seismic Supplement
    - "Demand Critical" weld at beam flanges to column face
- Chapter D
  - Nothing is designated "moderately" or "highly" ductile
    - No specific width-to-thickness requirements (other than AISC 360)
    - No specific lateral bracing requirements (other than AISC 360)
  - No plastic hinges (no protected zones)
  - No special requirements for beams or columns (also see "Commentary" Section E1.2)

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Structural Steel Design – AISC 341 (OMF)

- Chapter D (continued)
  - Connections shall comply with Chapter J of AISC 360 and Section E1.6
  - Column splices shall be designed for load combinations including Em
- Chapter E
  - "Minimal inelastic deformation capacity"
  - FR moment connection options:
    - Required flexural strength = Expected beam flexural strength multiplied by 1.1 (LRFD) or 0.733 (ASD); Required shear strength is based on the load combinations that include E_m as defined by Equation E1-1
      - $E_m = 2[1.1R_yM_p]/(\text{beam clear length})$
    - Maximum moment and corresponding shear that can be transferred to the system
    - Prescriptions for wide flange members noted in Section E1.6b(c)
  - No "strong column/weak beam" requirement

Structural Steel Design – AISC 341 (OMF)

- What is the defined <u>intent</u> of these code provisions?
  - High strength (low R-value) and low ductility
  - Avoid a highly brittle behavior response
  - Connection failure should not be the first significant inelastic event (the controlling limit state)
    - Preferred: Limiting states of flexure or shear of the beam or column; panel zone shear
    - The initial inelastic state can occur in any of the members (not the connections)

Structural Steel Design – AISC 341 (OMF)

- What are maximum loads that can be delivered to the connection by the system (alternative design load for connections)?
  - Column yielding
  - Summation of beam plastic moment capacities
  - Panel zone yielding
  - Foundation uplift
  - A limiting seismic force using R = 1

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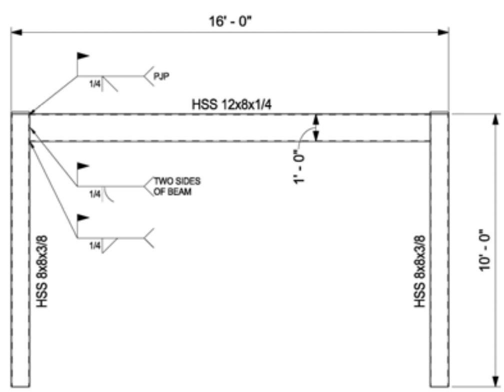
Example: HSS Moment Frame

- Potential qualifications as an "alternate" system:
  - Beam welding includes fillet welds and partial joint penetration welds
  - Lack of continuity plates
- Response: Show that the beam-to-column connection will not be the controlling limit state
  - E_m load combinations (would satisfy the requirement in most cases)



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Example: HSS Moment Frame

- Basic parameters:
  - ASTM A500, Grade B (F_y = 46 ksi)
  - Risk Category II (I_e = 1.0)
  - Ω_0 = 3.0 (ASCE 7, Table 12.2-1)
  - C_d = 3.0
  - Uniformly distributed factored load = 1000 plf (downwards on beam)
  - Factored seismic horizontal force at top = 5000 lbs.
- Drift check:
  - From an elastic analysis, δ_{xe} = 0.928"
  - $\Delta_x = C_d(\delta_{xe})/I_e$ = 3.0(0.928")/1.0 = 2.78"
  - Δ_{all} = 0.025(10')(12) = 3.0" … drift is within limits

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Example: HSS Moment Frame

- Maximum column reactions:
  - M = 37.6 ft-k
  - P = 11.2 k
  - V = 3.8 k
- Maximum beam reactions:
  - M = 37.6 ft-k
  - P = 3.8 k
  - V = 11.2 k

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Example: HSS Moment Frame

- Column check (ASD):
  - HSS 8x8x3/8
  - KL/r = 1.0(10')(12)/3.1 = 38.71 < 800/√(46) = 117.9
  - Allowable axial force = $0.6F_yA_g(0.658)^P$
    - P = $F_y(KL/r)^2/286{,}000$ = 0.241
    - With A_g = 10.4 in^2, allowable axial force = 259.5 k
  - Allowable moment = $0.66F_yS$ = 756 in-k = 63 ft-k
  - Combined stress ratio = 11.2/(2*259.5) + 37.6/(63) = 0.619

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Example: HSS Moment Frame

- Beam check (ASD):
  - HSS 12x8x1/4
  - KL/r = 1.0(16')(12)/3.32 = 57.83 < 800/√(46) = 117.9
  - Allowable axial force = $0.6F_yA_g(0.658)^P$
    - P = $F_y(KL/r)^2/286{,}000$ = 0.538
    - With A_g = 8.96 in^2, allowable axial force = 247.3 k
  - Allowable moment = $0.66F_yS$ = 929 in-k = 77.4 ft-k
  - Combined stress ratio = 3.8/(2*247.3) + 37.6/(77.4) = 0.494

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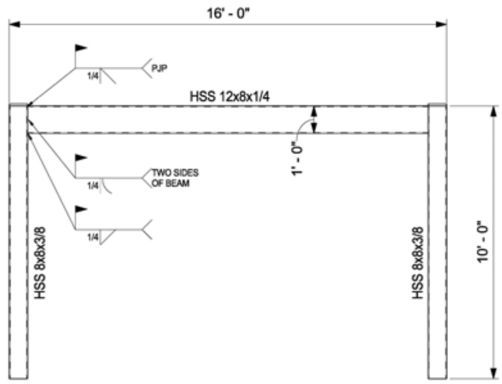
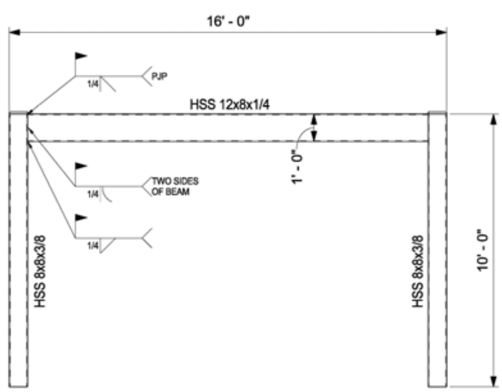
Example: HSS Moment Frame

- Connection check at end of beam (ASD):
  - Ix = 184 in^4
    - Contribution of flanges = $(2/12)(0.25")^3(8") + 2(0.25")(8")(6"-0.125")^2 = 138\ in^4$
    - Contribution of webs = $184 - 138 = 45.9\ in^4$
  - For PJP weld at flanges:
    - $T_{ASD} = 0.32(70\ ksi)(0.25")(8") = 44.8\ k$



Example: HSS Moment Frame

- Connection check at end of beam (ASD):
  - Moment resistance provided by PJP welds at the flanges:
    - $(44.8\ k)(12"- 0.25") = 515.2\ in\text{-}k = 42.9\ ft\text{-}k$
  - Beam capacity at flanges, ASD = $(138/184)77.4\ ft\text{-}k = 58.1\ ft\text{-}k$
- Conclusions:
  - Flanges would typically require CJP welding
  - Webs could potentially require only PJP or fillet welding



Example: Moment Frame with Channel and HSS

- What happens if we change the HSS beam to a channel?
- Say C12x30, ASTM A36 (F_y = 36 ksi)
- Maximum horizontal deflection = 0.969"
  - $\Delta_x = C_d(\delta_{xe})/I_e$ = 3.0(0.969")/1.0 = 2.91"
  - Δ_{all} = 0.025(10')(12) = 3.0" … drift is within limits
- Beam check (ASD):
  - KL/r = 1.0(16')(12)/0.762 = 251 > 200 … provide lateral bracing for beam
  - For an unbraced length of 12'-0", KL/r = 189 > 800/($\sqrt{F_y}$) = 133.3
    - $P_{allow} = 150{,}000A_g/(KL/r)^2$
    - With A_g = 8.81 in^2, allowable axial force = 37 k

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Example: Moment Frame with Channel and HSS

- Beam check (continued):
  - The allowable bending moment will be determined based on the beam's laterally unbraced length
    - Channels do not typically have high buckling strength
    - (*Instructor's recommendation*): In high seismic design category regions where channels are used in moment frames, it may be advisable to restrict the laterally unbraced length to Lp
    - $L_p = 300r_y/(\sqrt{F_y})$ = 300(0.762)/$\sqrt{(36)}$ = 38.1"
    - Adjusted allowable compressive stress = 166.8 k
  - Allowable moment = $0.66F_ySx$ = 641 in-k = 53.5 ft-k
  - Combined stress ratio = 3.8/(2*166.8) + 37.6/(53.5) = 0.714
- Disadvantages of such a system include more lateral bracing of the beam (to keep moment strength higher) and a somewhat weaker and unbalanced beam-to-column connection

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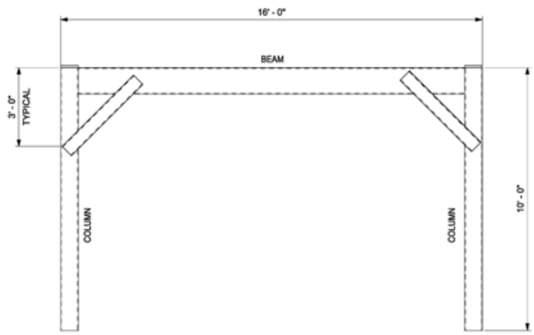
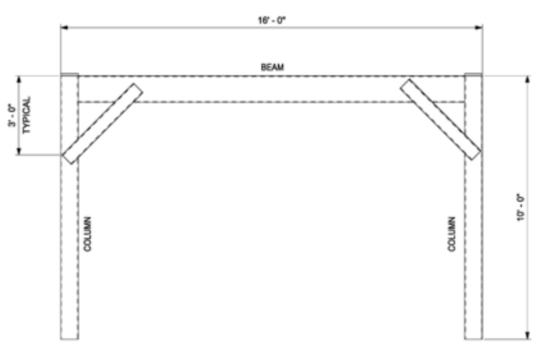
Example: Knee-Braced Moment Frame

- Use ASTM A500, Grade B
- Members:
  - Beam: HSS 12x8x1/4
  - Column: HSS 8x8x3/8
  - Brace: HSS 4x4x1/4
- Construction of the frame should align members at their centroids of action
  - The figure shows a typical elevation view where knee braces are applied to EACH SIDE of the moment frame, which is likely conservative but it balances the forces



Example: Knee-Braced Moment Frame

- Maximum column reactions:
  - M = 32.1 ft-k at the brace location, as the beam connection is “pinned”
  - P = 11.2 k
  - V = 16.1 k
- Maximum beam reactions:
  - M = 19.62 ft-k at the brace location
  - P = 16.1 k
  - V = 10.3 k
- Maximum brace reaction:
  - P = 28.3 k



Example: Knee-Braced Moment Frame

- From an elastic analysis, δ_{xe} = 0.669” … story drift is acceptable
- System advantages:
  - Slightly stiffer than the moment frame without braces (depending on where the braces are located)
  - All connections are “simple” (not moment connections), which makes fillet and partial-joint penetration welds more justifiable
- Knee braces can be placed at the center of the beams and columns or placed along the sides (considering eccentricities)
- Recommended approach from the commentary of AISC 341:
  - Connect the beam to the column, as well as the ends of each brace, based on forces required to develop $1.1R_yM_p$ (LRFD) in the beam
  - Member lateral bracing only needs to comply with AISC 360

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Summary and Closing Remarks

- Steel ordinary moment frames can be economical alternatives to intermediate or special systems
  - Use is limited within high seismic design category regions
  - Design procedures have not been tested to a significant degree, and their reliability is mostly based on judgment and prescriptive measures
- A wide range of possibilities are available, and true “alternatives” are likely to be justified with simple analysis and attention to the intent behind code provisions

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Final Questions?

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THANK YOU!!

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