NEHRP RECOMMENDED PROVISIONS SEISMIC DESIGN OF STEEL STRUCTURES

- Context in *NEHRP Recommended Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics
- Summary

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 1

Table of contents. Note that most of the substance, except the first two items, is taken from AISC's *Seismic Provision for Steel Buildings*, which is referenced by the *NEHRP Recommended Provisions.*

Some of the examples in this topic draw heavily on the FEMA 451, *NEHRP Recommended Provisions: Design Examples*. Please see this CD for additional details.

Steel Design: Context in Provisions

Design basis: Strength limit state

Using the 2003 *NEHRP Recommended Provisions*:

Load combination	Chap. 4
Seismic load analysis	Chap. 5
Components and attachments	Chap. 6
Design of steel structures	Chap. 8
	AISC Seismic
	and others

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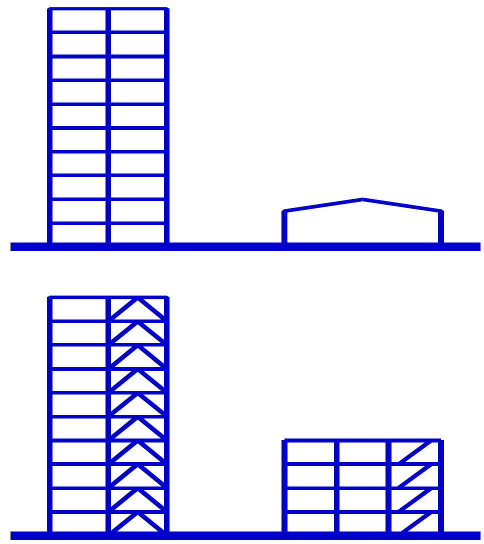
The *NEHRP Recommended Provisions* requirements affect design loads, limit states, and specific details required for members and connections. The bulk of the detailing rules are in the reference documents; the BSSC steel technical committee and the AISC seismic committee have been structured to work very closely together; thus, BSSC's recommendations are incorporated into AISC Seismic very quickly.

Seismic Resisting Systems

Unbraced Frames
- Joints are:
  - Rigid/FR/PR/
  - Moment-resisting
- Seismic classes are:
  - Special/intermediate/
  - Ordinary/not detailed

Braced Frames
- Concentric bracing
- Eccentric bracing



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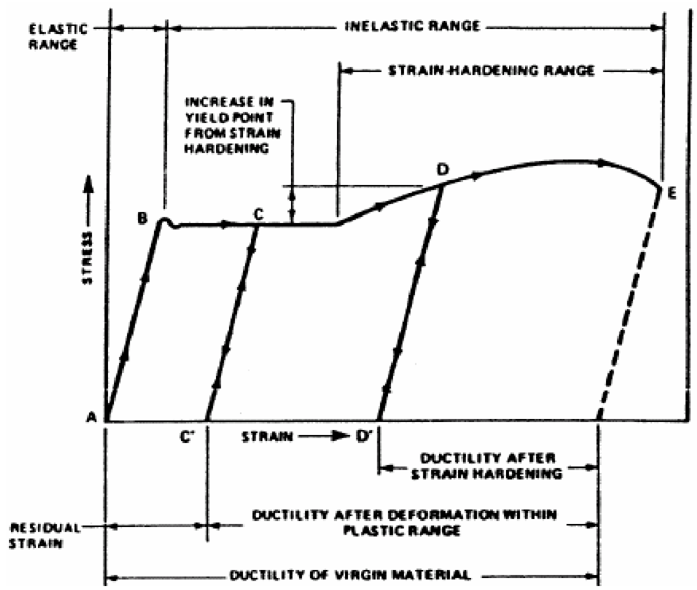
Unbraced ("moment resisting") frames resist lateral forces through flexural actions of members framing into (fully or partially) rigidly connected joints. Concentrically braced frames (no moment resisting connections) resist lateral forces through truss action that causes axial forces in members. Eccentric bracing creates high moments and shears in short links intended to yield first. Overall structural deformation in tall buildings: "shear building" pattern for unbraced frame but flexural pattern in braced frame.

Monotonic Stress-Strain Behavior



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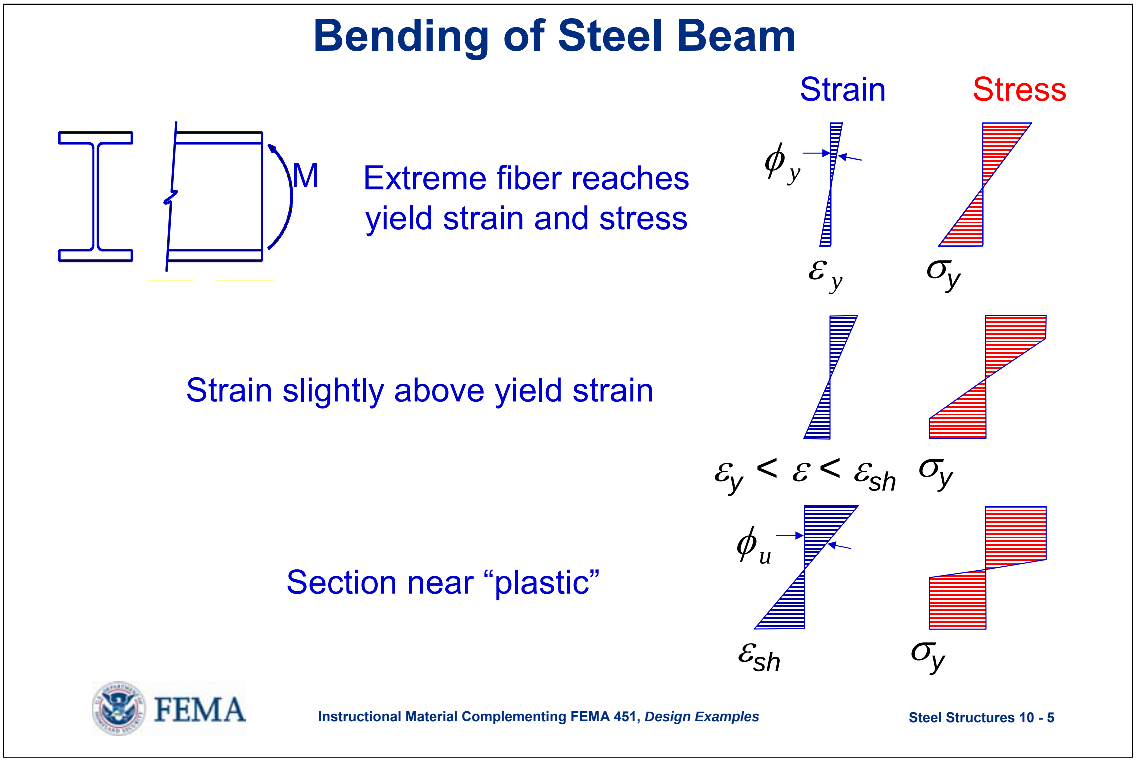
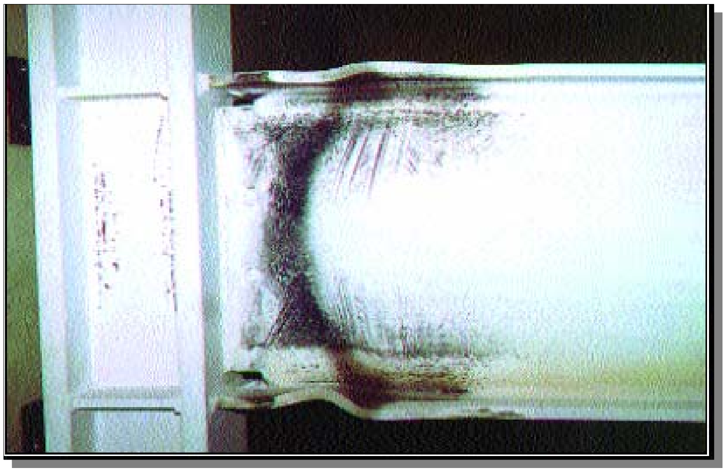
Stress-strain diagram for mild steel. Note the elastic range, yield point, plastic range, and region of strain hardening. Define ductility ratio; note values of 100 or more. Compare high- and low-strength steels.



For the elastic-plastic model, regardless of strain increases at extreme fibers, stress never exceeds yield stress. However, more material yields as strain increases; stress distribution becomes partially plastic. Note that the strain diagram near plastic is prior to the onset of strain hardening.

Plastic Hinge Formation



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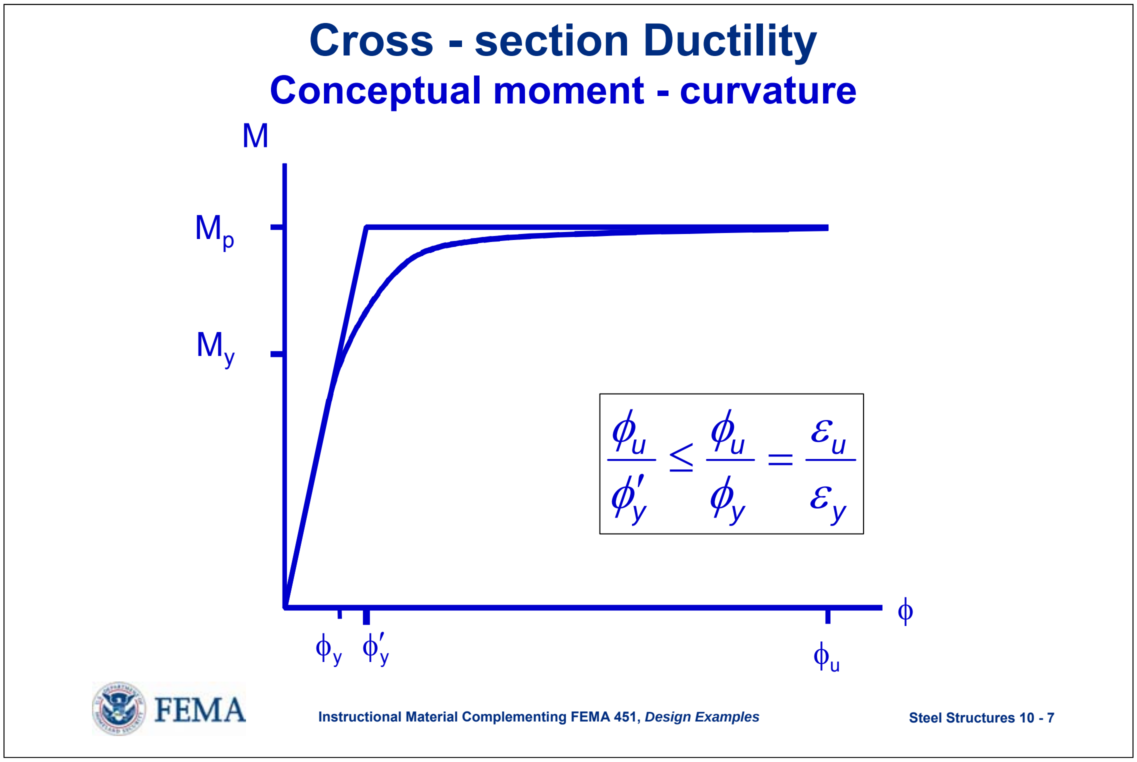
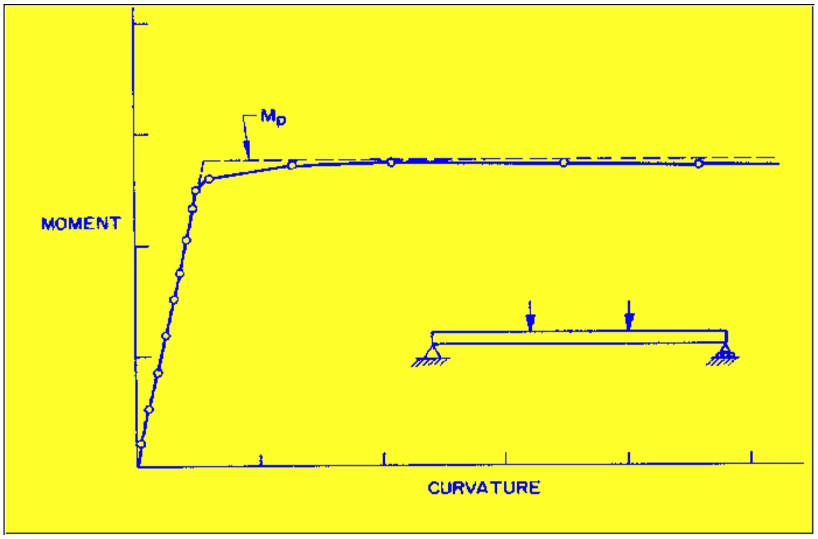
From *Modern Steel Construction* – formation of a plastic hinge in a laboratory test of a RBS (reduced beam section) specimen.



Moment-curvature diagram shows definitions for various curvatures. Yield curvature: at first yield (at M_y). As moment approaches M_p, curvature increases rapidly. At M_p, curvature increases without additional moment. Ultimate curvature: curvature at failure. Behavior often idealized by extending elastic and plastic portions of curve to intersection. Note that the strain hardening is not reflected in the plot.

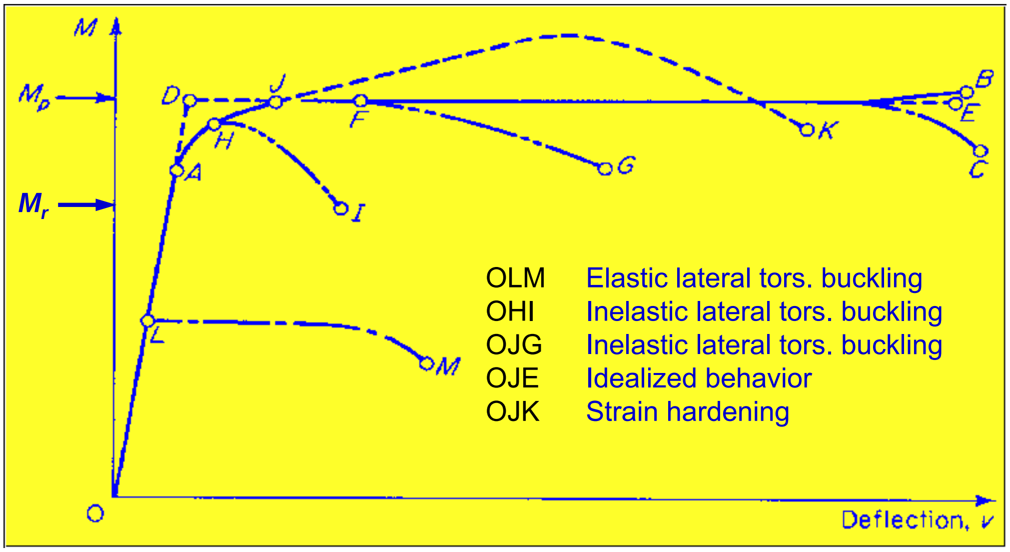
Moment Curvature

Laboratory Test -- Annealed W Beam



Beam is a well braced compact section with a large region of constant moment. Closest possible to idealized elastic-plastic behavior. Residual stresses in non-annealed beam would cause a larger deviation from the idealized behavior. Because the beam response shown is from an annealed section, the residual stresses are low and the moment curvature relationship closely approximates the ideal elastic-plastic behavior.

Behavior Modes For Beams



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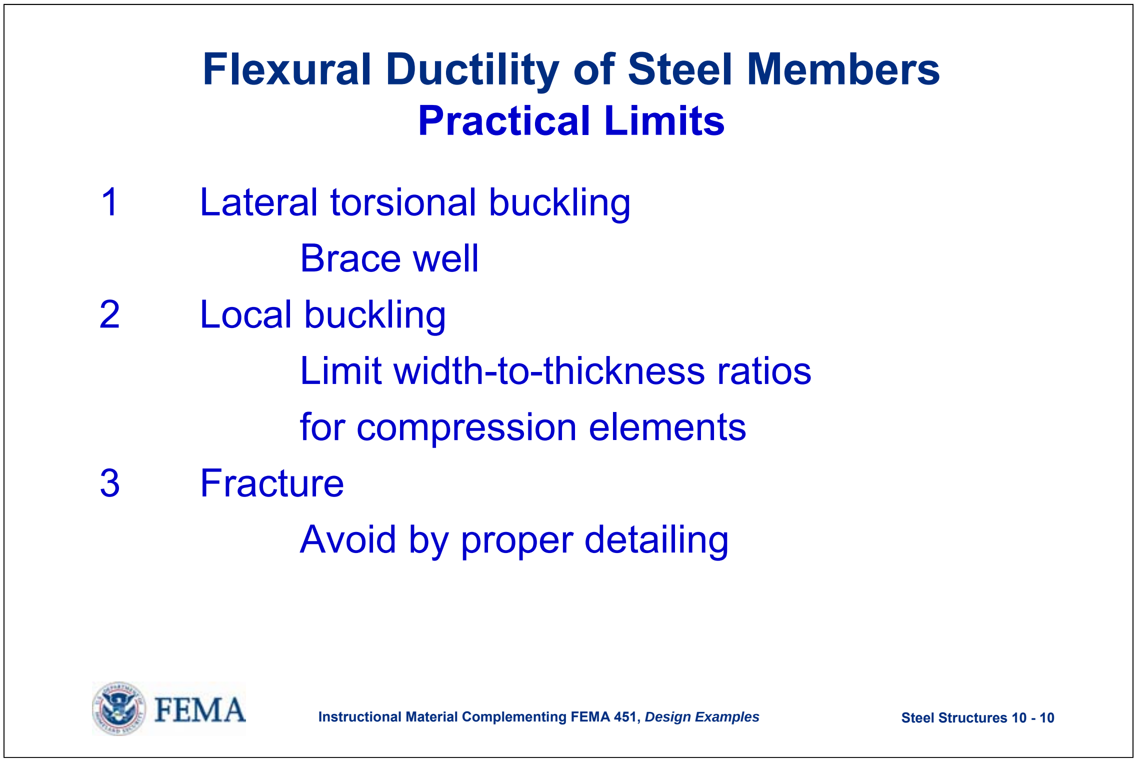
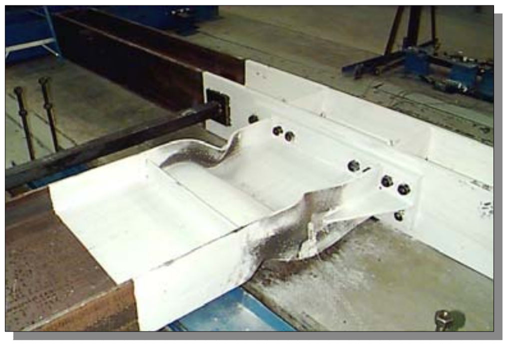
Note the various possible limit states (as on slide).

Strain hardening behavior is typical if hinge occurs in a region of significant shear (moment diagram is not constant). This is the typical situation, whereas the constant moment diagram in previous slide is really a laboratory idealization. M_r is the yield moment that includes the effects of residual stress.



Recall from prior lesson (inelastic behavior) that structural ductility < section ductility < material ductility. Structural ductility of **steel** members further limited, as noted on slide, by local and lateral buckling.

Local and Lateral Buckling



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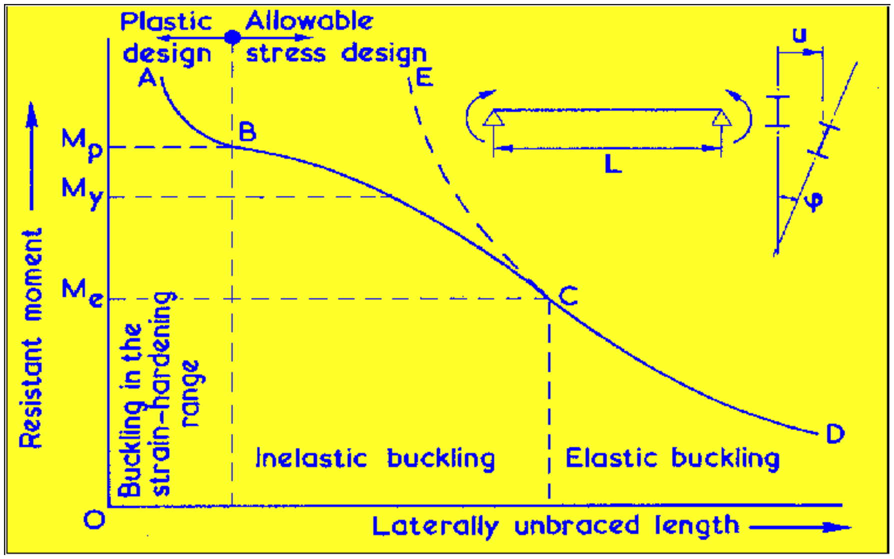
Photograph on the left is a laboratory test of a moment end-plate connection test exhibiting local buckling. Photo courtesy of Professor T. M. Murray, Virginia Tech. Photo on right illustrates lateral torsional buckling of a cold-formed steel hat section test specimen.

Lateral Torsional Buckling



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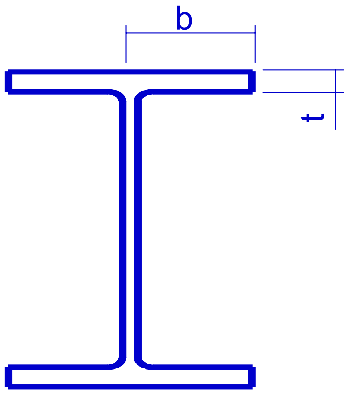
Effect of lateral-torsional buckling on moment capacity. Diagram for uniform bending of simple beam. Capacity above M_p due to strain hardening.

Local Buckling



Classical plate buckling solution:

$$\sigma_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)(b/t)^2} \leq \sigma_y$$

Substituting μ *= 0.3 and rearranging*:

$$\frac{b}{t} \leq 0.95\sqrt{\frac{kE}{F_y}}$$

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Example correlates width to thickness limits for compression flanges with ductility ratios. The mathematical derivation is not truely correct because linear mechanics are extrapolated beyond the yield level by using strain as a substitute for stress. The point is to show that stability depends on width to thickness ratios and that high strains require stubby elements.

Local Buckling
continued

With the plate buckling coefficient taken as 0.7 and an adjustment for residual stresses, the expression for b/t becomes:

$$\frac{b}{t} \leq 0.38\sqrt{\frac{E}{F_y}}$$

This is the slenderness requirement given in the AISC specification for compact flanges of I-shaped sections in bending. The coefficient is further reduced for sections to be used in seismic applications in the AISC Seismic specification

$$\frac{b}{t} \leq 0.3\sqrt{\frac{E}{F_y}}$$

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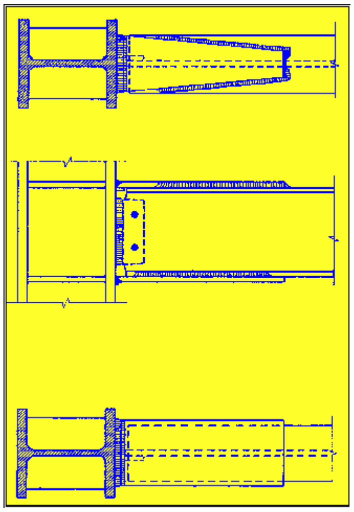
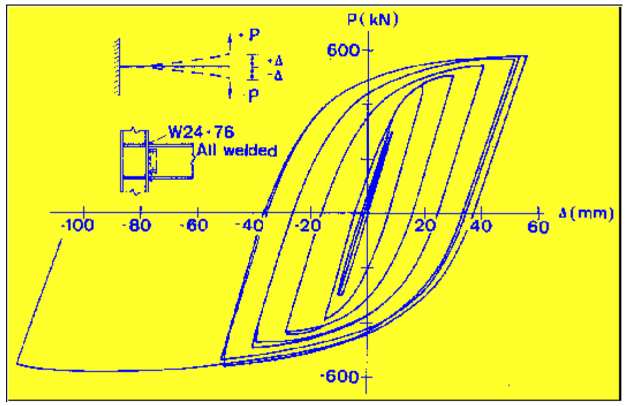
Note that AISC limits are for members expected to undergo significant inelastic deformation in a seismic event. The limits for plastic design are more liberal. These limits correspond very roughly to ductility ratio of about 10. Also note that several sections do not satisfy the limit for grade 50 steel.

Welded Beam to Column Laboratory Test - 1960s





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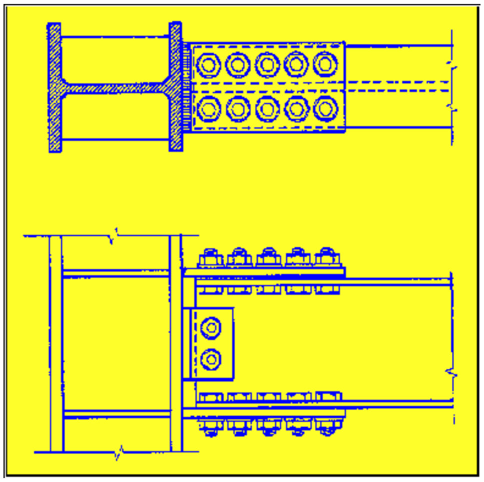
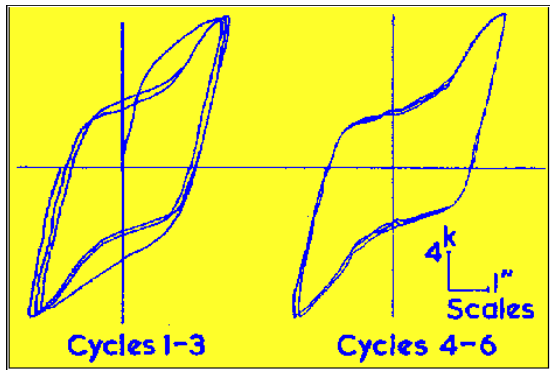
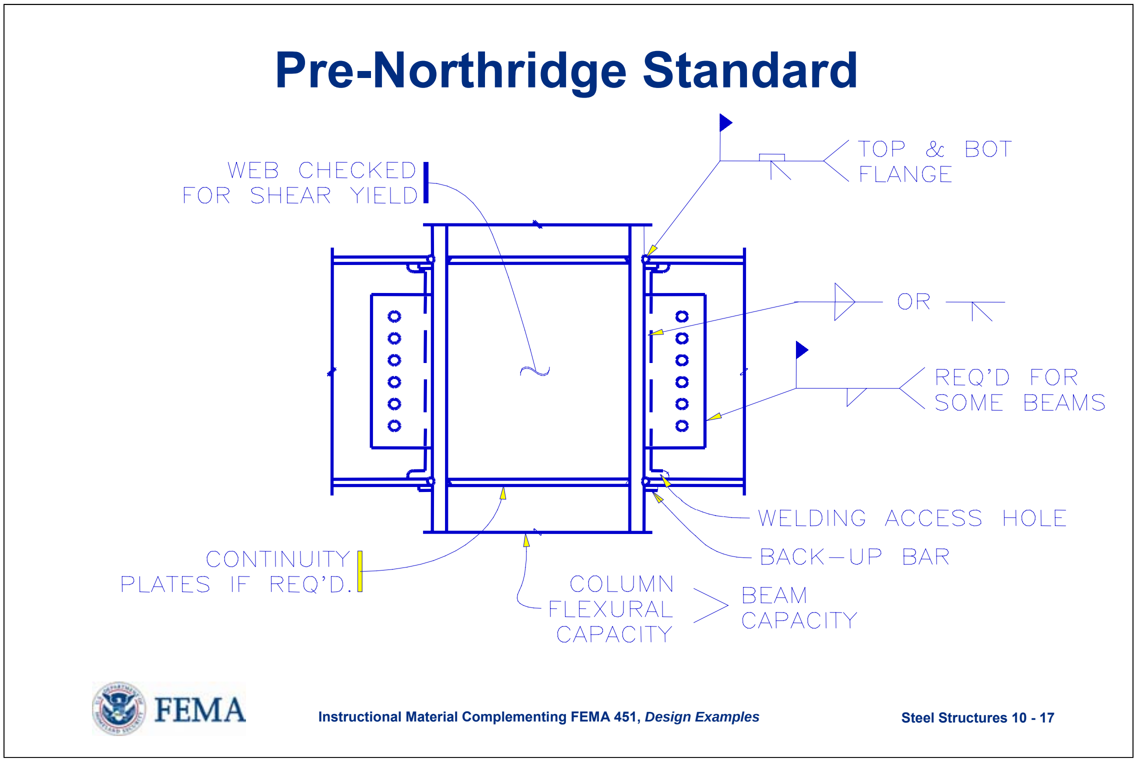
Bolted Beam to Column Laboratory Test - 1960s







The Northridge earthquake revealed serious flaws in the performance of welded joints in steel moment frames. The drawing illustrates the particular detail that had received prescriptive acceptance for seismic-resistant design in the decade prior to Northridge. The only real variable was the weld from the shear tab to the beam web, which was required in beams in which the web represented more than 30% of the plastic flexural capacity.

Following the 1994 Northridge earthquake, numerous failures of steel beam-to-column moment connections were identified. This led to a multiyear, multimillion dollar FEMA-funded research effort known as the SAC joint venture. The failures caused a fundamental rethinking of the design of seismic resistant steel moment connections.

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Text on slide is self-explanatory. The photograph shows a fracture resulting in a divot being removed from the column flange adjacent to the toe of the full penetration weld at the beam flange.

Bottom Flange Weld Fracture Propagating Through Column Flange and Web

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Slide is self-explanatory. It should be noted that many of the buildings that experienced failures similar to that shown in the slide exhibited very little obvious signs of distress.

Beam Bottom Flange Weld Fracture Causing a Column Divot Fracture

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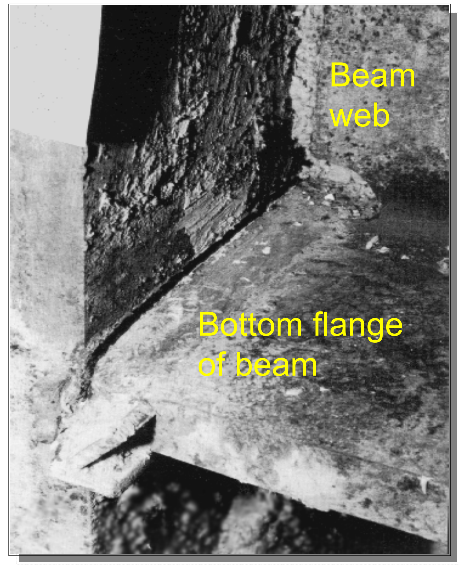
Photograph and title are self-explanatory. Note that the beam web and the weld access hole for the beam appear at the right side of the photo.

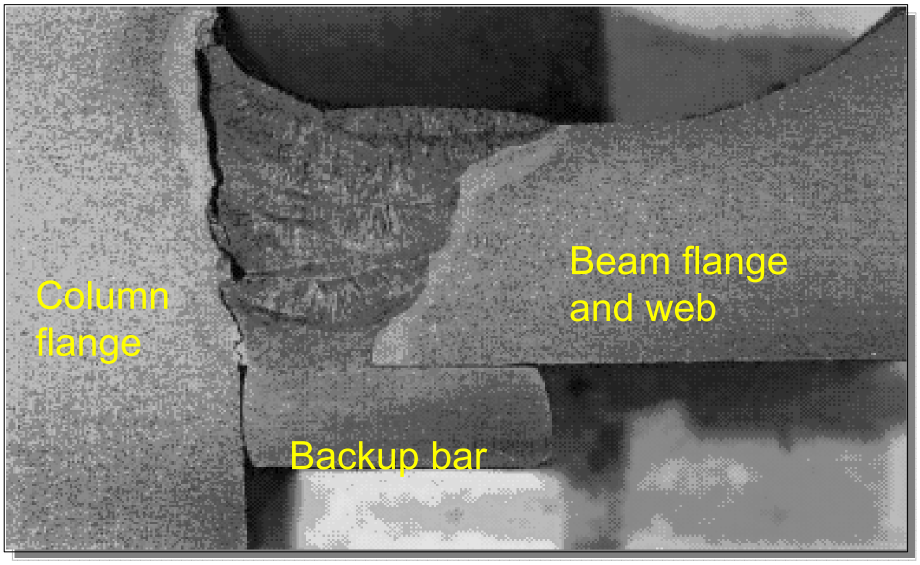
Northridge Failure

- Crack through weld
- Note backup bar and runoff tab



The photo is a closeup view of the weld between the bottom flange of a beam and the flange of a column. Note the crack between the weld and the column flange. Also note the backup bar and the runoff tab (the diagonal bar) at the left edge of the beam flange. The joint had been covered with spray-on fireproofing.

Northridge Failure



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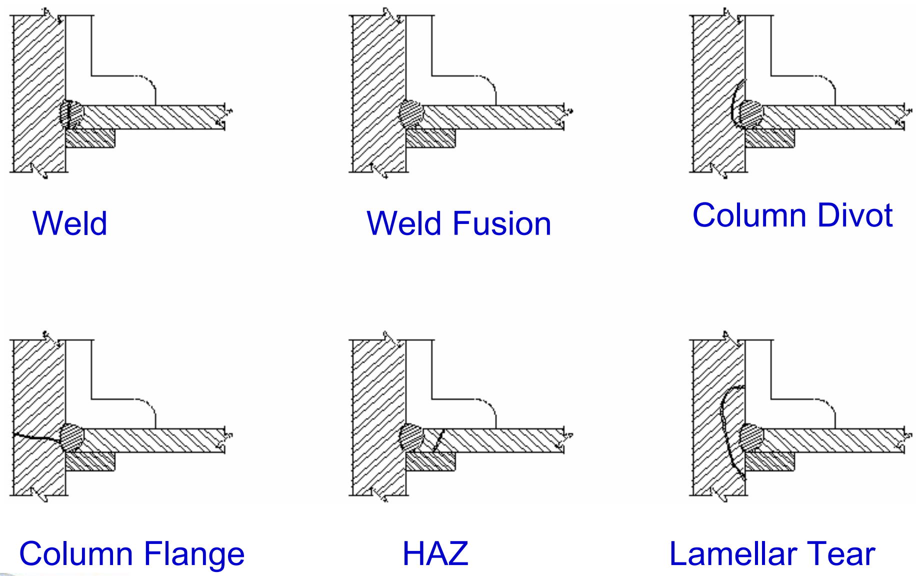
The photo shows a cut and polished specimen with a crack at the fusion line between the weld and the column flange. Note that the backup bar is not attached to the column flange, which creates a notch normal to the stress in the flange. The section is cut at the centerline of the beam; thus, the beam web beyond the weld access hole appears as the upward curving line at the right. Note that at the instant the beam flange is in tension due to lateral sway, the column flange at the weld is also likely to be in tension.

Northridge Failures



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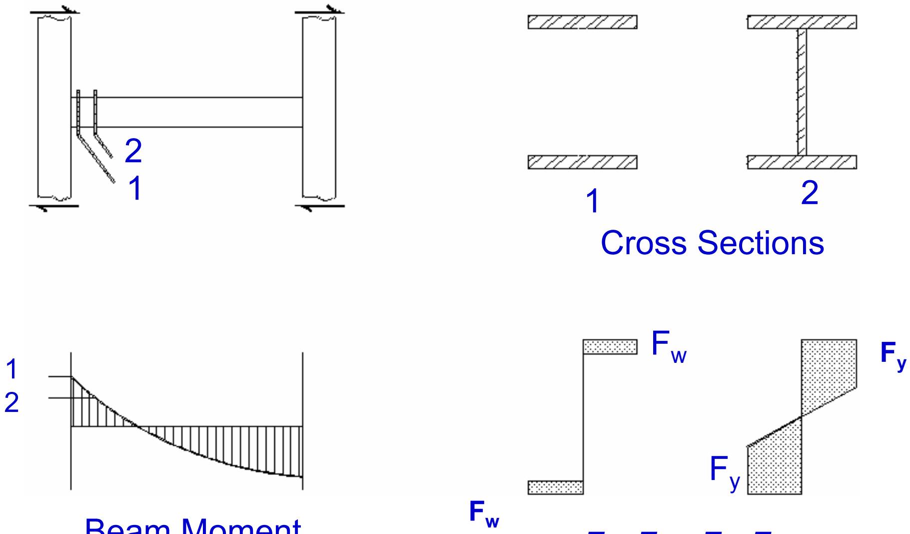
The figures are abstracted from *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, the early SAC report (95-02) on the problems found at Northridge. It is also available as FEMA 267, August 1995. The majority of the cracks began at the root of the weld directly above the notch at the end of the backup bar. Subsequent investigation determined that significant cracks exist in such joints that have not been exposed to seismic ground motion -- due to a combination of restraint of weld shrinkage, a ready made notch, use of materials that are not “notch tough,” difficulties in inspection at the point in question, and other factors. (HAZ is heat affected zone.)

Flexural Mechanics at a Joint



$$F_w \cdot Z_1 > F_y \cdot Z_2$$

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One of the inherent weaknesses of the joint is that the flange welds must be strong enough to develop substantial yielding in the beam away from the flange in order to dissipate significant energy. The plastic section modulus is less at the welds and the gradient in the moment as one moves away from the column flange both combine to require that the ultimate strength of the weld be substantially more than the yield of the beam. Another factor is that most available beams now have yield strengths of about 50 ksi (even though sold under A36 specification) whereas the tests done at the time the joint was developed were made on material much closer to A36 ksi yield; the strength of weld metal has not changed in the same fashion.

Welded Steel Frames

- Northridge showed serious flaws. Problems correlated with:
  - Weld material, detail concept and workmanship
  - Beam yield strength and size
  - Panel zone yield
- Repairs and new design
  - Move yield away from column face (cover plates, haunches, "dog bone")
  - Verify through tests
- SAC Project: FEMA Publications 350 through 354

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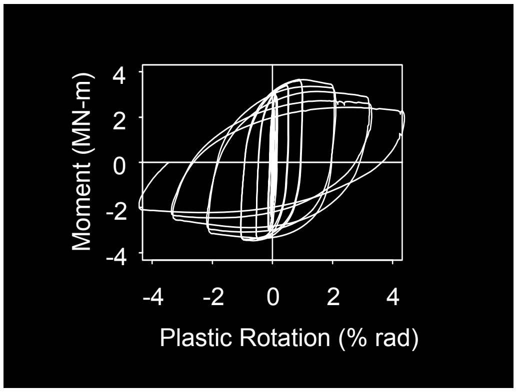
The slide summarizes the major points learned in the first phase of the SAC project (a FEMA-funded joint venture of SEAOC, ATC, and CUREE, which is California Universities for Earthquake Engineering). The second phase of the SAC project, also funded by FEMA, was a very substantial undertaking, and much more was learned. The profession and the steel industry will continue to feel profound effects of the Northridge event for years to come. "Dog bone" is the reduction in width of beam flange at a point away from the column face in order to force first yield at that point.

Reduced Beam Section (RBS) Test Specimen SAC Joint Venture



Graphics courtesy of Professor Chia-Ming Uang, University of California San Diego

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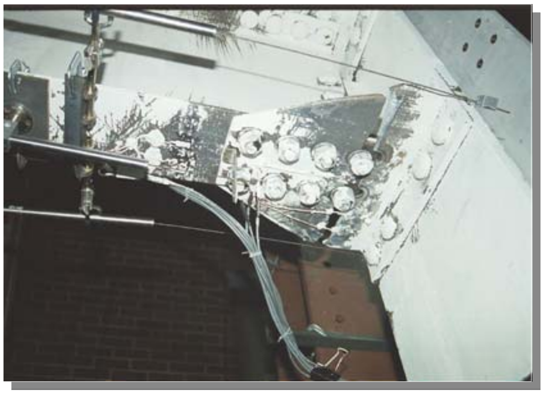
A beam-to-column joint test that utilizes the reduced beam section (RBS) detail is shown in the photograph. The experimental hysteresis plot is also shown. The RBS detail has been recognized as a pre-qualified connection by FEMA-350 and subsequent AISC documents. The behavior of this connection, actually member, detail is ductile and forces the inelasticity to occur in the beam section away for the beam to column welding and bolting. Graphics courtesy of Professor Chia-Ming Uang, University of California San Diego.

T-stub Beam-Column Test
SAC Joint Venture

Photo courtesy of Professor Roberto Leon, Georgia Institute of Technology

This slide shows a photo of a T-stub connection prior to testing. A hydraulic actuator was used to load the system and the column was restrained by load sensing pins. A series of six beam-column tests that incorporated T-stubs very similar to those tested in a component test series were tested to failure. The beam-column tests were intended to provide benchmarks for relating the component test data to actual connections. Courtesy of Professor Roberto Leon, Georgia Institute of Technology and Professor James Swanson, University of Cincinnati.

T-Stub Failure Mechanisms



Net section fracture in stem of T-stub

Plastic hinge formation -- flange and web local buckling

Photos courtesy of Professor Roberto Leon, Georgia Institute of Technology

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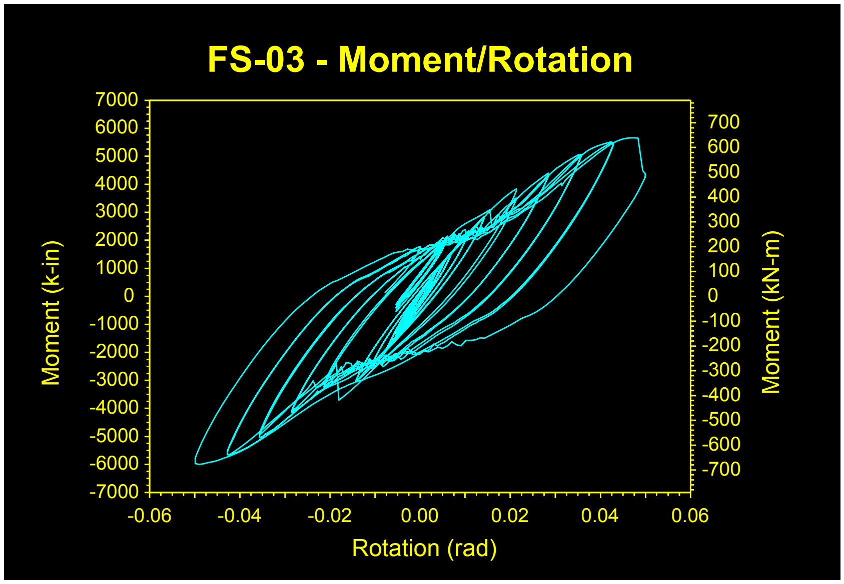
Two of the beam -column tests failed when the stems of the T-stubs fractured as is shown in the left photo on this slide. Because the T-stubs failed before the beams, these two connections would be classified as partial strength. Four of the connections failed with plastic hinges forming in the beam as is shown in the photo on the right in this slide. Also notice the severe flange and web local buckling of the beam. These connections would be considered full strength. Courtesy of Professor Roberto Leon, Georgia Institute of Technology and Professor James Swanson, University of Cincinnati.

T-Stub Connection Moment Rotation Plot



Graphic courtesy of Professor Roberto Leon, Georgia Institute of Technology

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This slide shows the moment rotation curve obtained during the first beam-column test. The connection failed with a net section fracture of the bottom T-stub. The connection exhibited substantial rotation capacity and ductility. Notice, too, that there is relatively little pinching and that the loss of slip resistance is not as pronounced as it was in the component tests. Courtesy of Professor Roberto Leon, Georgia Institute of Technology and Professor James Swanson, University of Cincinnati.

Extended Moment End-Plate Connection Results

Photo courtesy of Professor Thomas Murray, Virginia Tech

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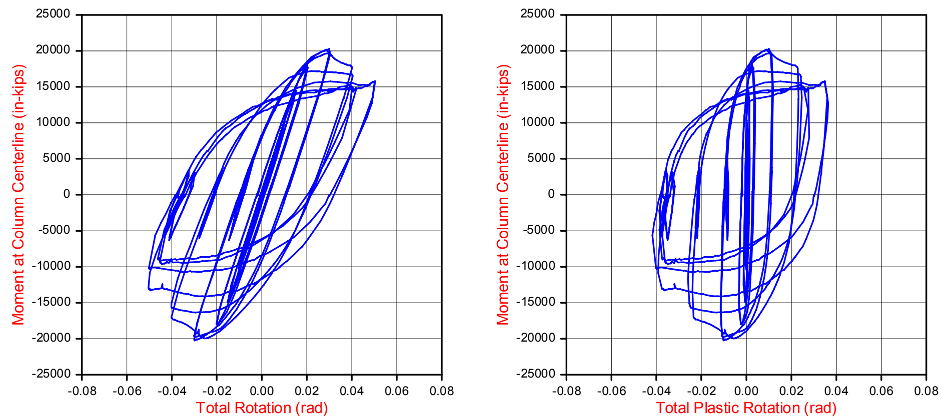
Photo is of a laboratory test specimen of an extended moment end plate connection. The “thick plate” behavior results in the development of a plastic hinge at the end of the stiffened section of the beam as the significant local flange and web buckling illustrate. The photograph is courtesy of Professor Thomas Murray, Virginia Tech.

Extended Moment End-Plate Connection Results



(a) Moment vs Total Rotation

(b) Moment vs Plastic Rotation

Graphics courtesy of Professor Thomas Murray, Virginia Tech

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Hysteresis plots from an extended, stiffened, moment end plate connection. Graphics courtesy of Professor Thomas Murray, Virginia Tech

Ductility of Steel Frame Joints
Limits

Welded Joints
- Brittle fracture of weld
- Lamellar tearing of base metal
- Joint design, testing, and inspection

Bolted Joints
- Fracture at net cross-section
- Excessive slip

Joint Too Weak For Member
- Shear in joint panel

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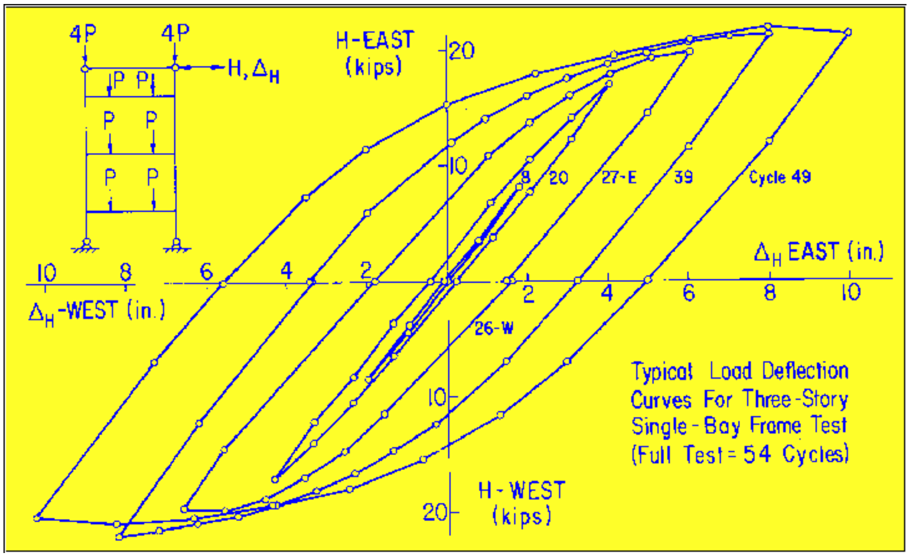
Text on slide is self-explanatory. Other failures not on slide were due to crushing or buckling of column web, distortion of column flange, or panel zone shear yielding or buckling.

Multistory Frame
Laboratory Test



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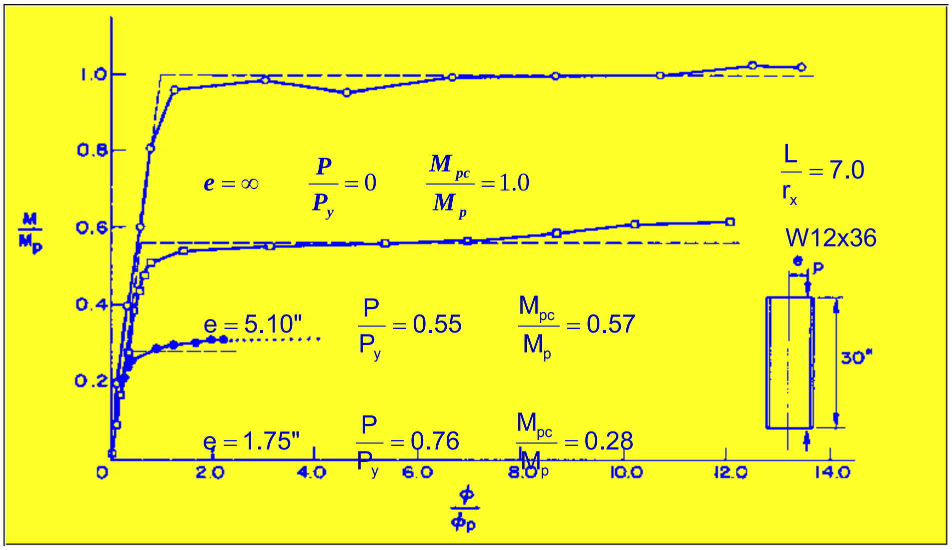
Load deflection plot for three-story, single-bay frame test. Heavier axial loads would limit stability at large lateral deflections.

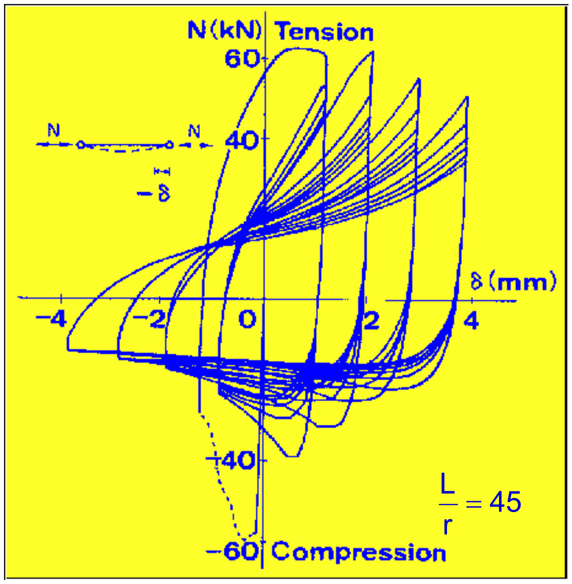
Flexural Ductility
Effect of Axial Load



Moment capacity depends on axial load. Diagram shows moment curvature for three different eccentricities on short column.

Axial Strut

Laboratory test



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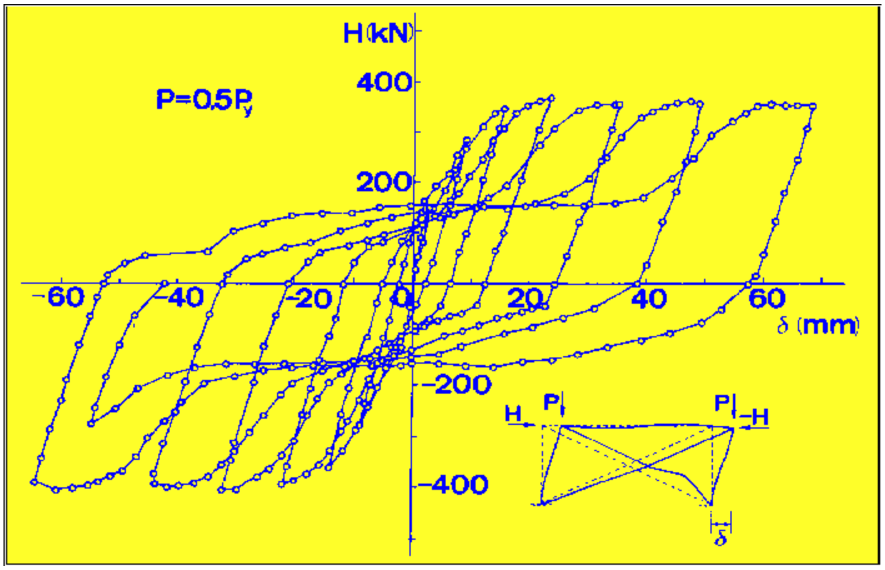
Plot representative of column or brace. Plot of axial load versus axial deflection for low-to-medium slenderness member (*L*/*r* = 45). Tension in upper right quadrant and compression in lower left. Theoretical buckling load reached on only the first cycle. Note the degrading stiffness, decreasing compressive strength, and permanent set caused by tension yielding. Also mention that other testing has shown prevention of local buckling is crucial to maintaining any post-buckling strength.

Cross Braced Frame

Laboratory test



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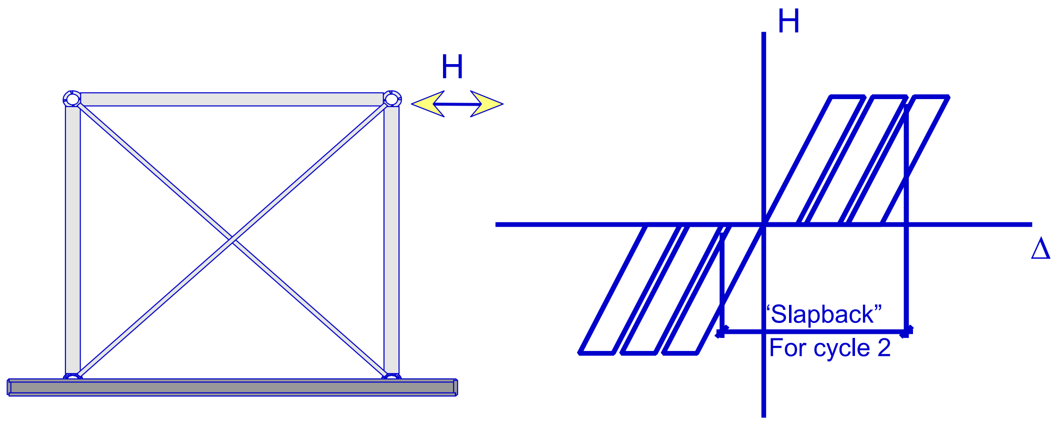
Overall performance of concentrically braced frame governed by axial behavior of braces. Frame has braces in both directions. Fatness of loops depends on slenderness of bracing members; lower slenderness ratios give fatter loops. Note the degrading stiffness.

Tension Rod (Counter) Bracing
Conceptual Behavior



Tension ties stressed beyond yield experience permanent deformation. Each load reversal results in a further increment of deformation; structure moves, unrestrained, through larger range of motion. If braces do not have equal strength, structure will accumulate deflections in one direction, "racheting" sideways as cycles go on.

Eccentrically Braced Frame

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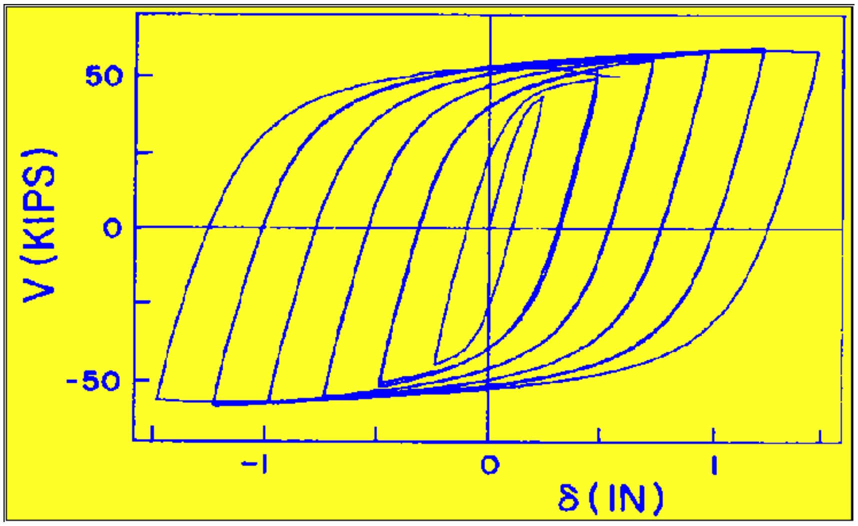
Photograph of eccentrically braced frame (EBF) under construction. Photo from *Modern Steel Construction.*

Eccentrically Braced Frame

Lab test of link



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Yield in eccentrically braced frames occurs in shear in link (short portion of beam between braces). Shear yielding produces very fat and stable hysteresis loops. Braces, beams, and columns designed to remain elastic at load level that causes link yielding.

Steel Behavior

- Ductility
  - Material inherently ductile
  - Ductility of structure < ductility of material
- Damping
  - Welded structures have low damping
  - More damping in bolted structures due to slip at connections
  - Primary energy absorption is yielding of members

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Steels used in construction are typically extremely ductile materials. Ductility of the steel structure or system is less (sometimes many times less) than the ductility of the material because only a portion of material in the system will actually experience yielding. Damping is typically not high in steel structures, especially in welded structures. Bolted structures exhibit a higher degree of damping due to slip at connections. Unlike structures of other materials, the primary energy absorption mode in steel structures is yielding of members, usually in bending but sometimes in shear, axial tension, or compression.

Steel Behavior

- Buckling
  - Most common steel failure under earthquake loads
  - Usually not ductile
  - Local buckling of portion of member
  - Global buckling of member
  - Global buckling of structure
- Fracture
  - Nonductile failure mode under earthquake loads
  - Heavy welded connections susceptible

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Steel Structures 10 - 41

Typical failure in steel structure subjected to earthquake is buckling, either global buckling of member, local buckling of portion of member, or global buckling of entire structure. Another potential weakness in steel structures is susceptibility to fracture; primarily a concern in heavy welded connections, members with notches, and in cold environments. Northridge earthquake has created substantially more concern about fracture.

NEHRP Recommended Provisions
Steel Design

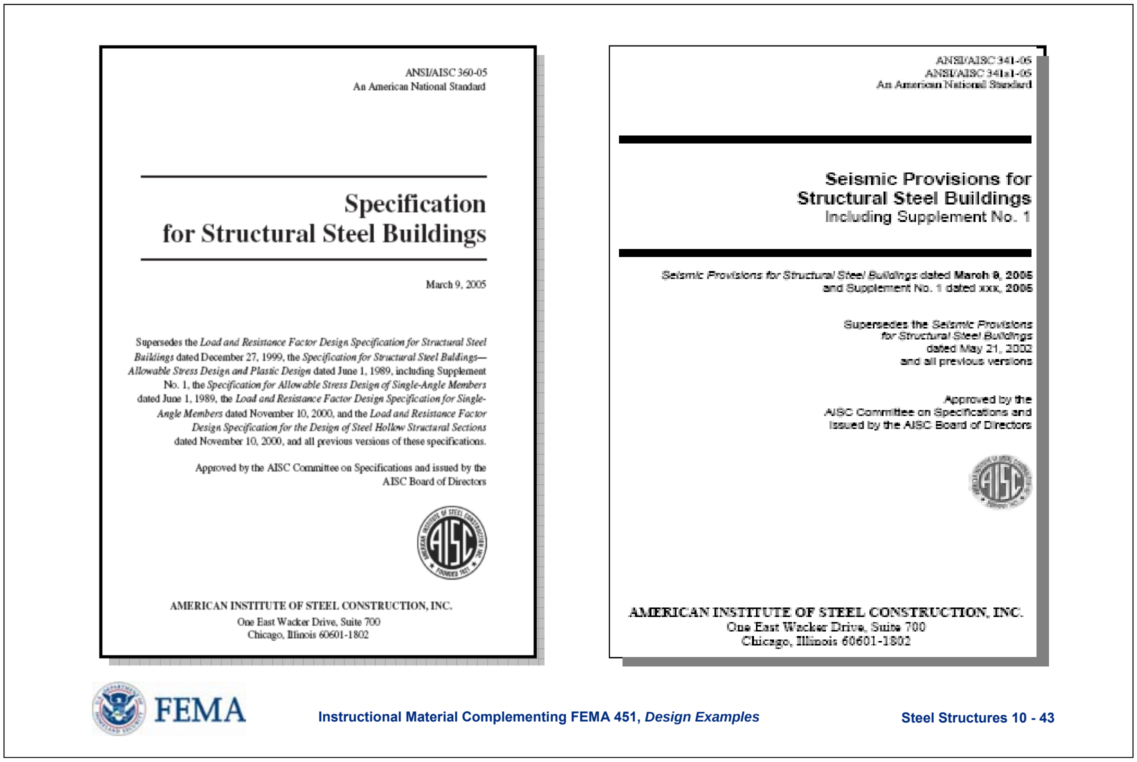
- Context in *NEHRP Recommended Provisions*
- Steel behavior
- Reference standards and design strength

FEMA

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 42

Table of contents: Reference standards and design strength. The *Provisions* references commonly accepted codes for each material and indicates how requirements in those standards must be modified for use with the *NEHRP Recommended Provisions* seismic design procedure. Special additional provisions are also included in each materials chapter.



The referenced standard for structural steel (hot rolled shapes and plates, and sections built up from hot rolled shapes and plates) is the *Specification for Structural Steel Buildings*, March 2005. The specification governing seismic design of steel buildings is *Seismic Provisions for Structural Steel Buildings*, March 2005 and the Supplement, which was not complete at the time of this writing. These documents supersede the earlier specifications from AISC.

Using Reference Standards
Structural Steel

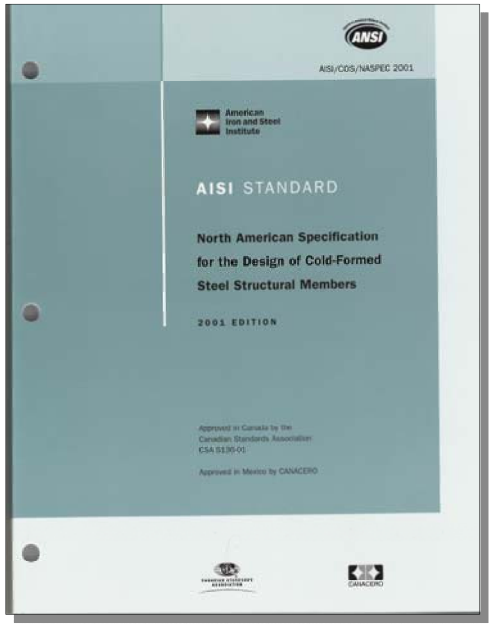
Both the AISC LRFD and ASD methodologies are presented in a unified format in both the *Specification for Structural Steel Buildings* and the *Seismic Provisions for Structural Steel Buildings*.

FEMA

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 44

Slide is self-explanatory.



Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 45

Reference standard for cold formed steel is the AISI *North American Specification for the Design of Cold-formed Steel Structural Members* (shown on slide). It includes both ASD and LRFD. ASCE 8-90 is a similar standard (LRFD only) for cold-formed stainless steel. The *Provisions* adjusts the load factor to be compatible.

Other Steel Members

Steel Joist Institute

Standard Specifications, 2002

Steel Cables

ASCE 19-1996

Steel Deck Institute

Diaphragm Design Manual, 3rd Ed., 2005

Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 46

For design of other specialized steel members, the *Provisions* refers user to standards published by Steel Joist Institute and the ASCE 19 standard for cables. The *Provisions* indicates how SJI allowables should be increased to maximum strengths for use in seismic resistant design and give guidelines for modification of cable allowables. Note the Steel Deck Institute's *Diaphragm Design Manual* is no longer referenced in the *Provisions* although it is a good source (do **not** use the 2.75 "factor of safety"). There are also other sources for steel deck diaphragms.

NEHRP Recommended Provisions
Steel Design

- Context in *NEHRP Recommended Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 47

Table of contents: Moment resisting frames.

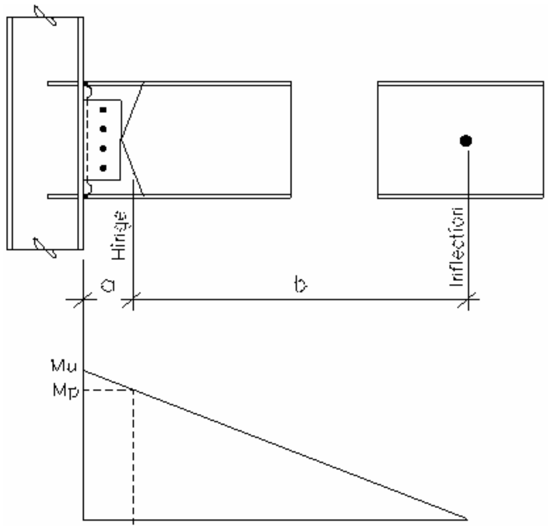
Steel Moment Frame Joints

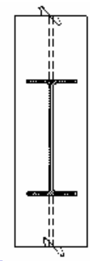
Frame	Test	θ_i	Details
Special	Req'd	0.04	Many
Intermediate	Req'd	0.02	Moderate
Ordinary	Allowed	NA	Few

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 48

θ_i is shorthand for the total (elastic plus inelastic) rotation at the beam to column connection. Elastic drift at yield is on the order of 0.01 radians. The capacity is very sensitive to detail; thus, the requirement for testing representative joints. AISC Seismic includes an appendix with a testing protocol. Mention that there is a disconnect in terminology between a SAC document (FEMA 350) and AISC Seismic, 2005. The “ordinary” moment frame in FEMA 350 is the same as the “intermediate” moment frame in AISC. FEMA 350 does not address the frame called “ordinary” in AISC. In the 1997 *Provisions* (and the 1997 edition of AISC Seismic), there was an “intermediate” frame that was between the “special” and “ordinary” on this slide.

Steel Moment Frame Joints





$$M_u \approx M_p \cdot \frac{a+b}{b}$$

$$F_y^* = R_y \cdot F_y$$

$$F_u \approx F_y^* Z \cdot \frac{a+b}{b} \cdot \frac{1}{A_f d} \approx 1.7 F_y^*$$

FEMA

Instructional Material Complementing FEMA 451, *Design Examples*

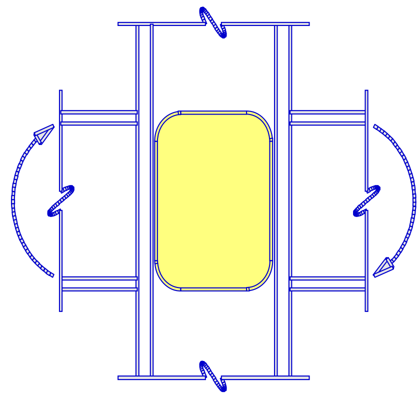
Steel Structures 10 - 49

This illustrates one aspect of the problem with the welded joint. The old connection usually must develop its flexural strength solely through the flange welds at the face. The moment is somewhat higher than M_p in order to develop an appreciable hinge in the beam. The larger moment and the reduced section both increase the demand stress above the actual yield of the beam. The number 1.7 is derived from common proportions for the contribution of the flange to the total plastic section modulus and the ratio of hinge length to inflection point distance plus overstrength commonly found in A36 steel (which is no longer the steel of choice for hot-rolled W shapes.) Additional issues include the geometric stress concentration, weld imperfections, shear lag from beam web to flange tip, residual stresses from welding heat, loss of ductility where triaxial tension exists, etc.

Panel Zones

Special and intermediate moment frame:

- Shear strength demand:
  Basic load combination or $\phi R_y M_p$ of beams
- Shear capacity equation
- Thickness (for buckling)
- Use of doubler plates



FEMA

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 50

Panel shear often controls and this may require larger column or "doubler" plate on web, which is expensive to fabricate. SAC testing has shown that panel zone yield contributes to fracture of beam flange welds, apparently due to the concentration of strains at that location. However, some yielding in the panel zone is considered to be a good way to dissipate energy. The final SAC recommendations require that the yield strength of the shear panel zone be at least strong enough to reach the beginning of yield in the beam flexural hinge areas but not full development of the flexural hinge.

Steel Moment Frames

- Beam shear: $1.1R_yM_p$ + gravity
- Beam local buckling
  - Smaller b/t than LRFD for plastic design
- Continuity plates in joint per tests
- Strong column - weak beam rule
  - Prevent column yield except in panel zone
  - Exceptions: Low axial load, strong stories, top story,and non-SRS columns

FEMA

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 51

Note the tighter restrictions on b/t and h/t than required for plastic design. Beam flange continuity plates are now required if included in the test specimens. It is not an analytical check. The b/t limit rules out a few common beam shapes for grade 50 steel.

Steel Moment Frames

- Lateral support of column flange
  - Top of beam if column elastic
  - Top and bottom of beam otherwise
  - Amplified forces for unrestrained
- Lateral support of beams
  - Both flanges
  - Spacing < $0.086 r_y E/F_y$

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 52

Limits to prevent member buckling. The limit is for special frames and it is different from the general limits in the main LRFD spec, which applies to the compression flange and depends on the square root of Fy. Refer to Section 9.8 of the 2005 AISC Seismic specification for additional details. The Commentary to the 2005 AISC specification includes the following:

"Spacing of lateral braces for beams in SMF systems is specified not to exceed 0.086 ry E / Fy. This limitation, which is unchanged from previous editions, was originally based on an examination of lateral bracing requirements from early work on plastic design and based on limited experimental data on beams subject to cyclic loading. Lateral bracing requirements for SMF beams have since been investigated in greater detail in Nakashima, Kanao and Liu (2002). This study indicates that a beam lateral support spacing of 0.086ryE/Fy is appropriate, and slightly conservative, to achieve an interstory drift angle of 0.04 radian."

Prequalified Connections

See FEMA 350: *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*

- -Welded Unreinforced Flange
- -Welded Free Flange Connection
- -Welded Flange Plate Connection
- -Reduced Beam Section Connections
- -Bolted Unstiffened End Plate Connection
- -Bolted Stiffened End Plate Connection
- -Bolted Flange Plate Connection

See ANSI/AISC 358-05, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*

- -Reduced Beam Section Connections
- -Bolted Stiffened and Unstiffened Extended Moment End Plate Connections

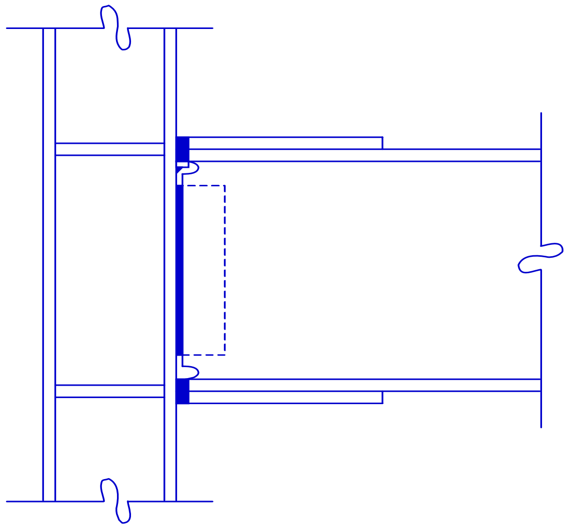
FEMA

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 53

FEMA 350 resulted from the SAC Joint Venture project. Subsequently, an ANSI/AISC committee was established to create ANSI/AISC 358-05. The work by this committee is continuing and additional connections are being addressed. The ANSI/AISC 358 standard is available for download from the AISC website.

Welded Coverplates



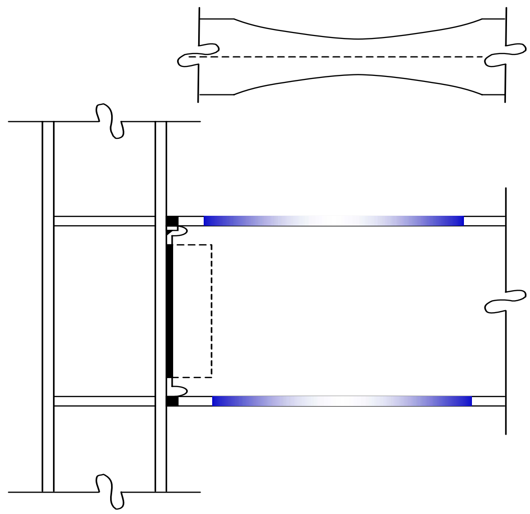
FEMA

Instructional Material Complementing FEMA 451, *Design Examples*

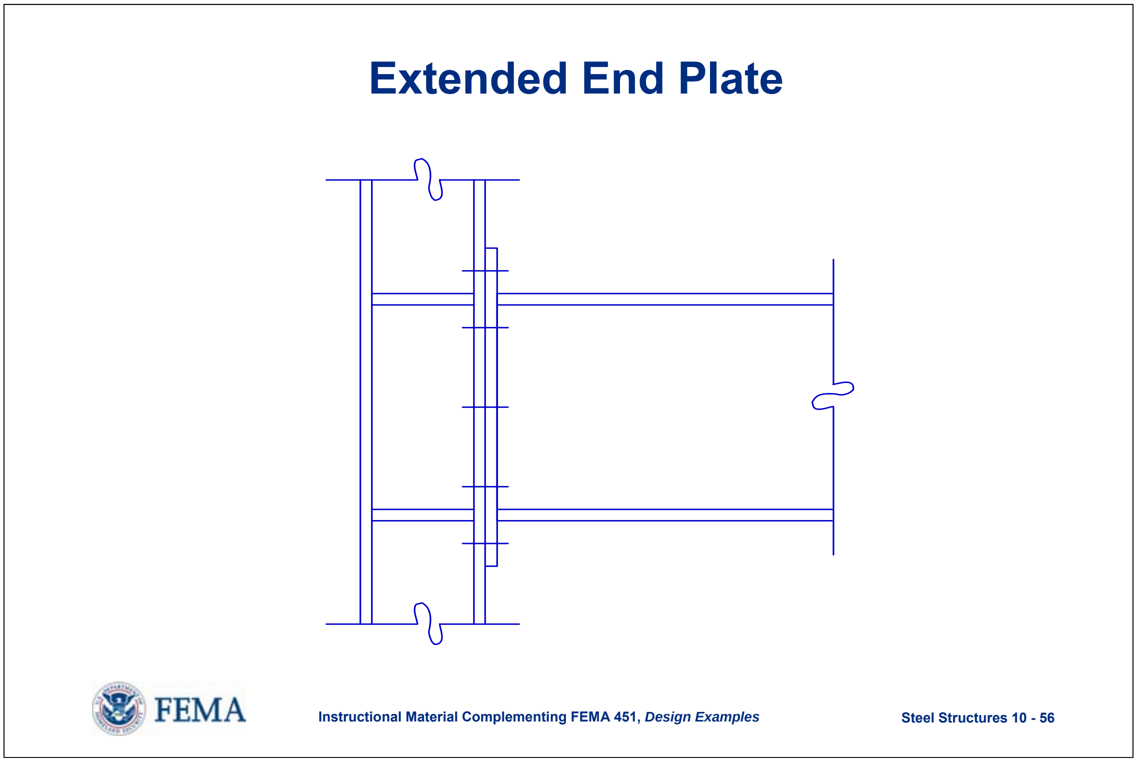
Steel Structures 10 - 54

Slide illustrates a beam-to-column connection using welded cover plates on the beam flanges.

Reduced Beam Section (RBS)

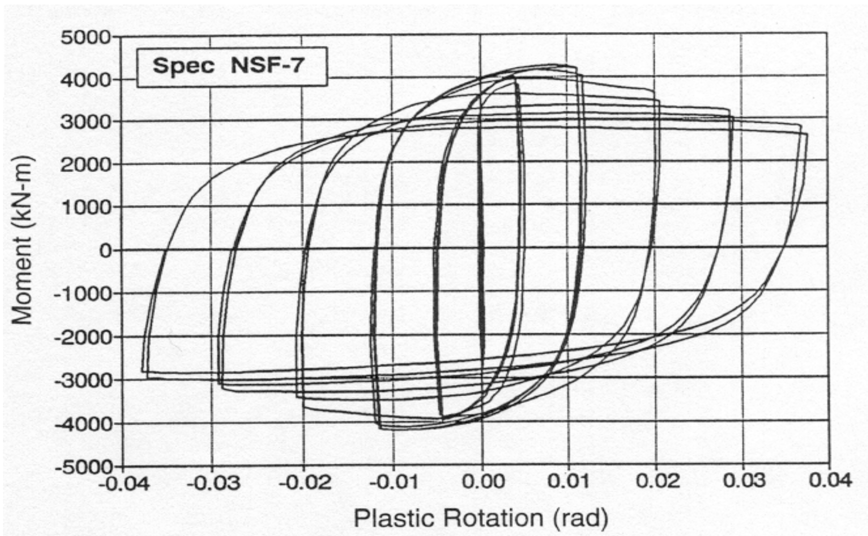


The slide illustrates a beam-to-column connection using the reduced beam section (RBS) connection. Note that the beam flanges and shear plate are welded to the column. The reduction of the beam section insures that the hinging will occur in the beam away from these connection details.



The slide illustrates an extended moment end plate connection. Note that this example does not have stiffeners between the end plate and the beam flange.

Excellent Moment Frame Behavior



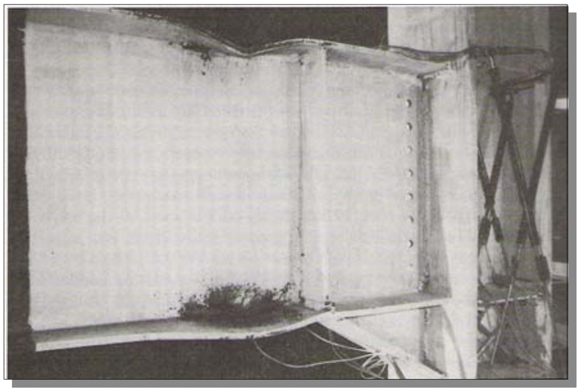
FEMA

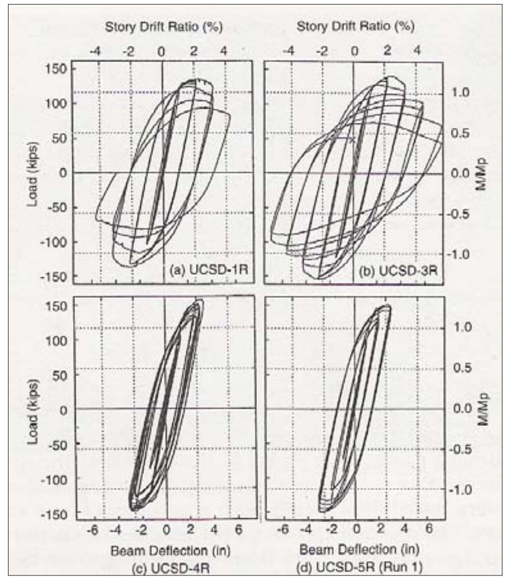
Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 57

This slide illustrates excellent moment frame behavior as noted by the large, full hysteresis loops that exhibit very little strength degradation out to a plastic rotation of approximately 0.04 rad. The test was of a cover plated beam with fully welded web conducted at the University of Texas under the direction of Professor Michael Englehardt

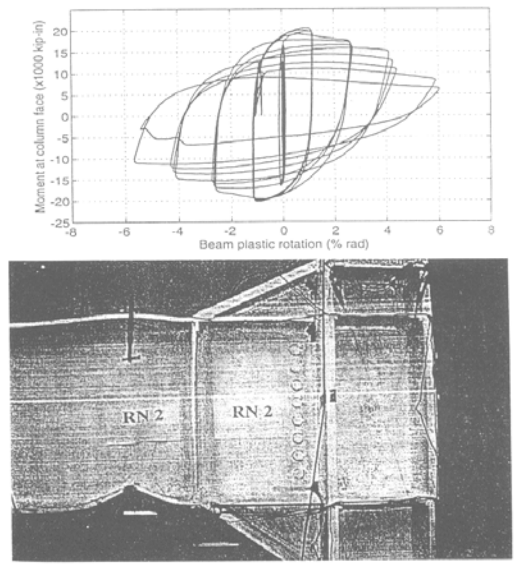
Excellent Moment Frame Behavior





The photograph is of a welded bottom haunch connection that exhibits excellent hysteretic behavior. The test was conducted at the University of California at San Diego under the direction of Professor Chia-Ming Uang.

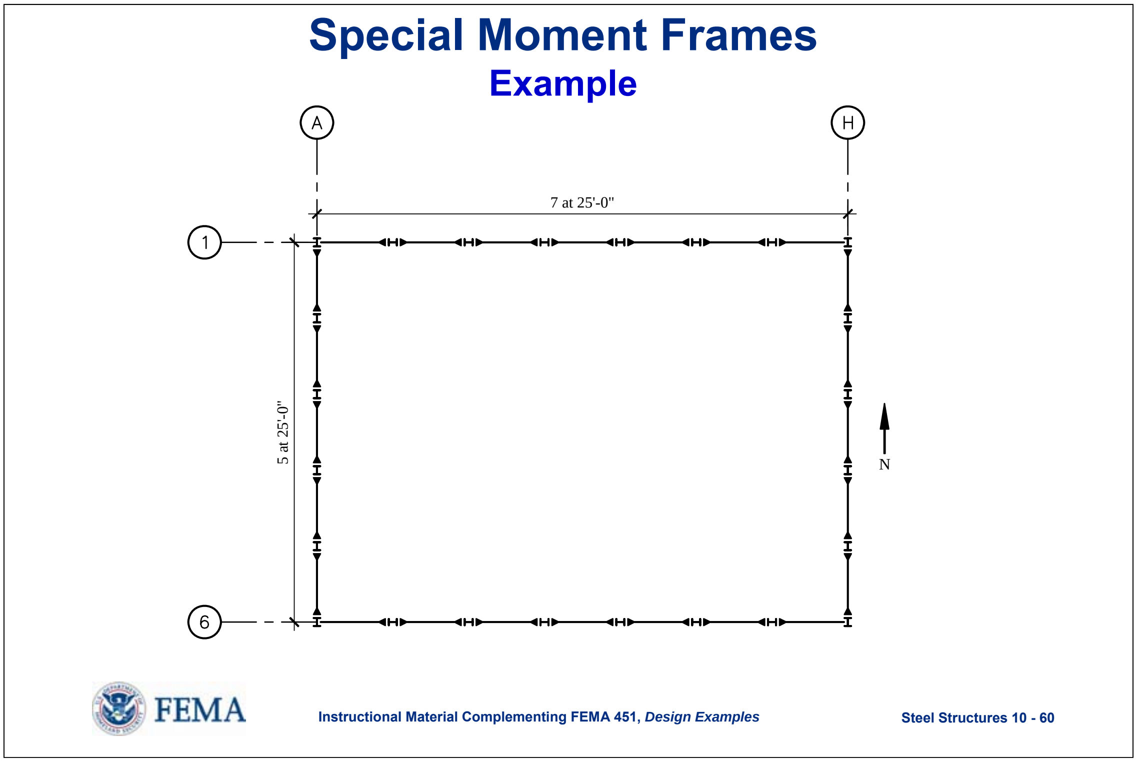
Excellent Moment Frame Behavior



Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 59

The photograph is of a welded top and bottom haunch connection that exhibits excellent hysteretic behavior. The test was conducted at the University of California at Berkeley under the direction of Andrew Whitaker and Vitelmo Bertero.



The information on the following several slides are taken from Example 5.2 (Alternate A) of the NEHRP *Design Examples* volume (FEMA 451). Only the SMF framing is shown in the figure on this slide. Recall that FEMA 451 was written based on the 2000 *NEHRP Recommended Provisions*. However, the example has been modified where necessary to comply with the 2005 AISC Seismic specification. The building configuration, loads, materials and analysis are given in FEMA 451.

Special Moment Frames

The following design steps will be reviewed:

- Select preliminary member sizes
- Check member local stability
- Check deflection and drift
- Check torsional amplification
- Check the column-beam moment ratio rule
- Check shear requirement at panel zone
- Select connection configuration

FEMA   Instructional Material Complementing FEMA 451, *Design Examples*   Steel Structures 10 - 61

The design steps in this slide will be briefly reviewed in the subsequent slides. Additional explanation and detail is available in the FEMA 451. A portion of the East-West frame at level two will be used to illustrate various calculations.

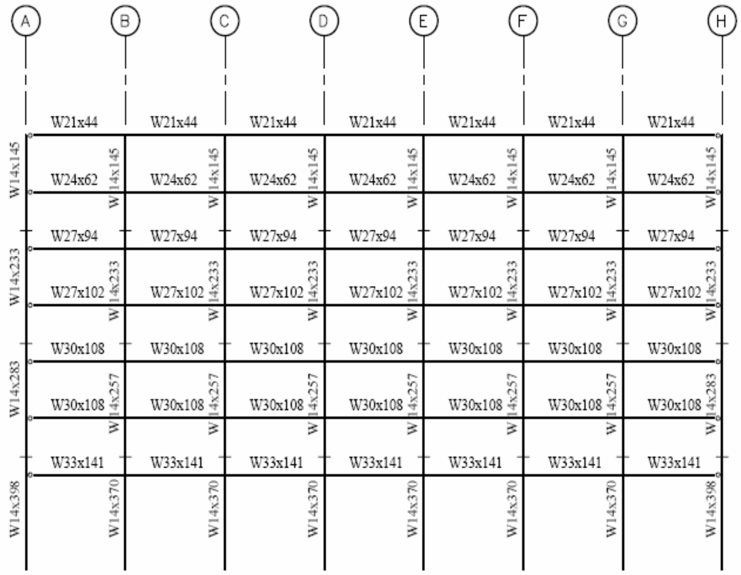
Special Moment Frames

Select preliminary member sizes – The preliminary member sizes are given in the next slide for the frame in the East-West direction. These members were selected based on the use of a 3D stiffness model in the program RAMFRAME. As will be discussed in a subsequent slide, the drift requirements controlled the design of these members.

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 62

No additional comments are necessary for this slide.

SMF Example – Preliminary Member Sizes



The preliminary member sizes were determined from the use of a structural analysis model and were based on satisfying the drift limits for this design. FEMA 451 indicates that the drift requirements govern over the strength requirements, which is often the case for SMF designs.

Note that this example is based on Chapter 5.2 of the FEMA 451 Examples document.

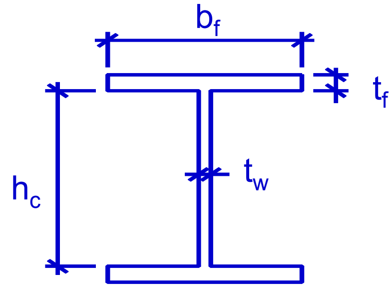
SMF Example – Check Member Local Stability

Check beam flange: $\dfrac{b_f}{2t_f} = 6.01$
(W33x141 A992)

Upper limit: $0.3\sqrt{\dfrac{E}{F_y}} = 7.22$ OK

Check beam web: $\dfrac{h_c}{t_w} = 49.6$

Upper limit: $3.76\sqrt{\dfrac{E}{F_y}} = 90.6$ OK



Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 64

The beam flange local stability limits are prescribed in the AISC Seismic specification, which is more stringent for the W shape in the SMF application. The web local stability limits are prescribed in the AISC *Specification for Structural Steel Buildings*. The column check is similar. Note that the provision for checking the web slenderness under combined bending and compression no longer appears in the AISC *Specification for Structural Steel Buildings*.

SMF Example – Check Deflection and Drift

The frame was checked for an allowable story drift limit of $0.020h_{sx}$. All stories in the building met the limit. Note that the *NEHRP Recommended Provisions* Sec. 4.3.2.3 requires the following check for vertical irregularity:

$$\frac{C_d \Delta_{x \ \ story\ 2}}{C_d \Delta_{x \ \ story\ 3}} = \frac{\left(\dfrac{5.17in.}{268in.}\right)}{\left(\dfrac{3.14in.}{160in.}\right)} = 0.98 < 1.3$$

Therefore, there is no vertical irregularity.

Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 65

No additional comments are required.

SMF Example – Check Torsional Amplification

The torsional amplification factor is given below. If Ax < 1.0 then torsional amplification is not required. From the expression it is apparent that if δ_{max} / δ_{avg} is less than 1.2, then torsional amplification will not be required.

$$A_x = \left(\frac{\delta_{\max}}{1.2\delta_{avg}} \right)^2$$

The 3D analysis results, as shown in FEMA 451, indicate that none of the δ_{max} / δ_{avg} ratios exceed 1.2; therefore, there is no torsional amplification.

The torsional amplification equation given in the slide is described in Sec. 5.2.4.3 of FEMA 451. As noted, the structural analysis results indicate that torsional amplification need not be considered in this design. This is not surprising given the regularity (symmetry) of the floors and frames (placed around the perimeter.) The same regular and symmetrical building with braced frames in the core (and no moment frames) falls into the “extreme torsional irregularity” category.

SMF Example – Member Design NEHRP Guide

Member Design Considerations - Because $P_u/\varphi P_n$ is typically less than 0.4 for the columns, combinations involving Ω_0 factors do not come into play for the special steel moment frames (re: AISC Seismic Sec. 8.3). In sizing columns (and beams) for strength one should satisfy the most severe value from interaction equations. However, the frame in this example is controlled by drift. So, with both strength and drift requirements satisfied, we will check the column-beam moment ratio and the panel zone shear.

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 67

The content of this slide is taken directly from FEMA 451, Example 5.2.

SMF Example – Column-Beam Moment Ratio

Per AISC Seismic Sec. 9.6

$$\frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} > 1.0$$

where $\Sigma \boldsymbol{M^*_{pc}}$ = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. $\boldsymbol{\Sigma M^*_{pc}}$ is determined by summing the projections of the nominal flexural strengths of the columns above and below the joint to the beam centerline with a reduction for the axial force in the column.

$\Sigma \boldsymbol{M^*_{pb}}$ = the sum of the moments in the beams at the intersection of the beam and column centerlines.

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 68

It is permitted to take $\Sigma M^*_{pc} = \Sigma Zc(F_{yc}\text{-}P_{uc/}A_g)$ (LRFD). ΣM^*_{pb} is determined by summing the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to take $\Sigma M^*_{pb} = \Sigma(1.1R_yF_{yb}Z_b+M_{uv})$. Refer to AISC Seismic specification Sec. 9.6 for remainder of definitions.

SMF Example – Column-Beam Moment Ratio

Column – W14x370; beam – W33x141

$$\Sigma M_{pc}^{*} = \Sigma Z_c\left(F_{yc} - \frac{P_{uc}}{A_g}\right) = 2\left[736in^2\left(50ksi - \frac{500kips}{109in^2}\right)\right]$$

$$\Sigma M_{pc}^{*} = 66,850in - kips$$

Adjust this by the ratio of average story height to average clear height between beams.

$$\Sigma M_{pc}^{*} = 66,850in - kips\left(\frac{268in. + 160in.}{251.35in. + 128.44in.}\right) = 75,300in - kips$$

Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 69

The story height for the first story is 268 in. (top of concrete to mid-depth of first floor beam and the clear height is 251.35 in. The values for the second story are 160 in. and 128.44 in. For diagrams and additional details the reader is referred to FEMA 451 and the AISC Seismic specification.

SMF Example – Column-Beam Moment Ratio

For beams:

$$\Sigma M^*_{pb} = \Sigma(1.1R_y M_p + M_v)$$

$$\text{with } M_v = V_p S_h$$

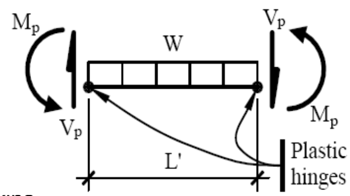
$$S_h = \text{dist. from col. centerline to plastic hinge}$$

$$= d_c/2 + d_b/2 = 25.61 in.$$

$$V_p = \text{shear at plastic hinge location}$$

$$V_p = \left[2M_p + \left(wL^{'2}/2\right)\right] = \frac{2M_p + \frac{wL^{'2}}{2}}{L^{'}}$$

$$= \frac{(2)(25{,}700 in - kips) + \left(\frac{(1.046 klf)}{12}\frac{(248.8 in.)^2}{2}\right)}{248.8 in.} = 221.2 kips$$



Calculations for V_p. Figure from FEMA 451. The calculation for the factored uniform dead load, w, is given and is found to be 1.406 klf.

SMF Example – Column-Beam Moment Ratio

$$M_v = V_p S_h = (221.2 kips)(25.61 in.) = 5{,}665 in - kips$$

and

$$\Sigma M^*_{pb} = \Sigma(1.1 R_y M_p + M_v)$$
$$= 2[(1.1)(1.1)(25{,}700 in - kips) + 5{,}665 in - kips] = 73{,}500 in - kips$$

The ratio of column moment strengths to beam moment strengths is computed as:

$$Ratio = \frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} = \frac{75{,}300 in - kips}{73{,}500 in - kips} = 1.02 > 1.00 \quad \therefore OK$$

Other ratios are also computed to be greater than 1.0

FEMA

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 71

The column-beam strength ratio for all the other stories are determined in a similar manner. They are summarized in Table 5.2-4 of FEMA 451. All cases are found to be acceptable.

SMF Example –Panel Zone Check

The 2005 AISC Seismic specification is used to check the panel zone strength. Note that FEMA 350 contains a different methodology, but only the most recent AISC provisions will be used. From analysis shown in the NEHRP *Design Examples* volume (FEMA 451), the factored strength that the panel zone at Story 2 of the frame in the EW direction must resist is 1,883 kips.

$$R_v = 0.6F_y d_c t_p \left[1+\frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right] = (0.6)(50ksi)(17.92in.)(t_p)\left[1+\frac{(3)(16.475in.)(2.66)^2}{(33.3in.)(17.92in.)(t_p)}\right]$$

$$R_v = 537.6t_p + 315$$

The required total (web plus doubler plate) thickness is determined *by*:

$$\phi R_v = R_u$$

$$(1.0)(537.6t_p + 315) = 1{,}883kips$$

$$t_{p_{required}} = 2.91in.$$

The column web thickness is 1.66*in.*, *therefore the required doubler plate thickness is*:

$$t_{p_{doubler}} = 1.25in. \quad (\textit{therefore use one } 1.25in. \textit{ plate or two } 0.625in. \textit{ plates})$$

FEMA

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 72

The calculations shown use the 2005 AISC Seismic specification methodology. The calculations are self-explanatory and require no further comment.

SMF Example – Connection Configuration

Beam-to-column connections used in the *seismic load resisting system* (SLRS) shall satisfy the following three requirements:

(1) The connection shall be capable of sustaining an *interstory drift angle* of at least 0.04 radians.

(2) The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 radians.

(3) The *required shear strength* of the connection shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1R_yM_p]/L_h \qquad (9\text{-}1)$$

FEMA

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 73

The text on this slide is taken from the 2005 AISC Seismic specification. Connections that accommodate the required interstory drift angle within the connection elements and provide the measured flexural resistance and shear strengths specified above are permitted. In addition to satisfying the requirements noted above, the design shall demonstrate that any additional drift due to connection deformation can be accommodated by the structure. The design shall include analysis for stability effects of the overall frame, including second-order effects.

SMF Example – Connection Configuration

Beam-to-column connections used in the SLRS shall satisfy the requirements of Section 9.2a by one of the following:

(a) Use of SMF connections designed in accordance with ANSI/AISC 358.

(b) Use of a connection prequalified for SMF in accordance with Appendix P.

(c) Provision of qualifying cyclic test results in accordance with Appendix S. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(i) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Appendix S.

(ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix S.

FEMA

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 74

The following is taken from the Commentary to the 2005 AISC Seismic specification. Note that at the time of writing, ANSI/ASCE 358 was not available in approved form. This section provides requirements for demonstrating conformance with the requirements of Section 9.2a. This provision specifically permits the use of prequalified connections meeting the requirements of ANSI/AISC 358, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC, 2005a) to facilitate and standardize connection design. Other prequalification panels may be acceptable but are subject to the approval of the *authority having jurisdiction*. Use of connections qualified by prior tests or project specific tests may also be used although the engineer of record is responsible for substantiating the connection. Published testing, such as that conducted as part of the SAC project and reported in FEMA 350 and 355, or project-specific testing may be used to satisfy this provision.

Special Moment Frames
Summary

Beam to column connection capacity
Select preliminary member sizes
Check member local stability
Check deflection and drift
Check torsional amplification
Check the column-beam moment ratio rule
Check shear requirement at panel zone
Select connection configuration

- Prequalified connections
- Testing

FEMA

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 75

Review SMF requirements. FEMA 350 is the current best resource for prequalified connections. AISC will eventually become the official repository for such information. FEMA 350 is available electronically for download at the AISC ePubs site (www.aisc.org).

NEHRP Recommended Provisions
Steel Design

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Seismic design category requirement
- Moment resisting frames
- Braced frames

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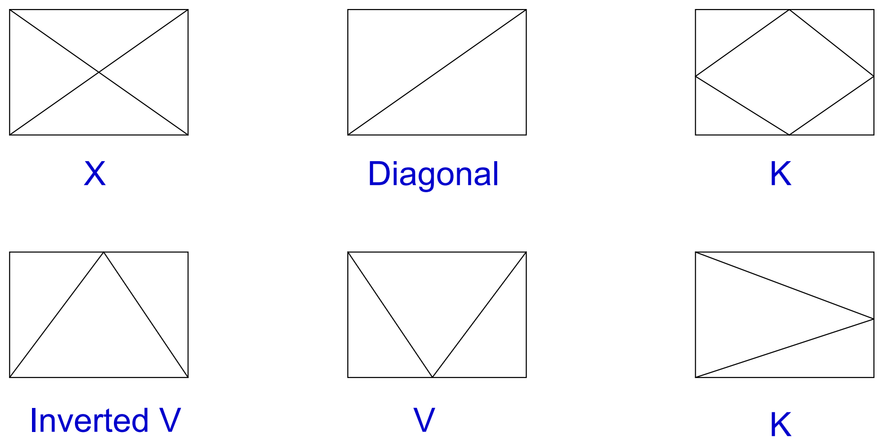
Table of Contents: braced frames.

Concentrically Braced Frames

Basic Configurations



K bracing will introduce bending into columns following buckling of compression brace and is often prohibited. V and inverted V bracing require continuity of beams.

Braced Frame Under Construction

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Steel Structures 10 - 78

A braced frame using inverted V type bracing is shown under construction.

Braced Frame Under Construction

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Braced frame using diagonal bracing under construction.

Concentrically Braced Frames

Special AISC Seismic R = 6

Chapter 13

Ordinary AISC Seismic R = 3.25

Chapter 14

Not Detailed for Seismic R = 3

AISC LRFD

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The special and ordinary systems are very similar. Chapter 13 and Chapter 14 refer to sections in the 2005 AISC Seismic specification.

Concentrically Braced Frames

Dissipate energy after onset of global buckling by avoiding brittle failures:

- Minimize local buckling
- Strong and tough end connections
- Better coupling of built-up members

Most common failures were from: (1) fracture at local buckling where global buckling created a hinge and (2) fracture at connections.

Concentrically Braced Frames

Special and Ordinary

Bracing members:

- Compression capacity = $\phi_c P_n$
- Width / thickness limits

  Generally compact

  Angles, tubes and pipes very compact
- Overall $\frac{KL}{r} < 4\sqrt{\frac{E}{F_y}}$
- Balanced tension and compression

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Previous versions of these the AISC Seismic specification have required that the members of OCBF be designed for the amplified seismic load, effectively reducing the effective *R* factor by half. To make the design of OCBF consistent with other systems, this requirement has been eliminated from the 2005 AISC Seismic specification consistent with a corresponding reduction in the *R* factor for these systems in SEI/ASCE 7-05, Supplement Number 1. The required strength of the members of OCBF will now be determined using the loading combinations stipulated by the applicable building code (and the reduced *R* factors prescribed in SEI/ASCE 7-05, Supplement Number 1) without the application of the amplified seismic load.

Balanced tension and compression means that the sum of horizontal components of tension braces is between 30% and 70% of total horizontal force. This check is a design level, not post-buckling level. It is intended to prevent the “racheting” accumulation of inelastic deformation in one direction and the “impact” possible in a “slapback” phenomenon.

Concentrically Braced Frames

Special concentrically braced frames

Brace connections

Axial tensile strength > smallest of:

- Axial tension strength = $R_yF_yA_g$
- Maximum load effect that can be transmitted to brace by system.

Axial compressive strength $\geq 1.1R_yP_n$ where P_n is the nominal compressive strength of the brace.

Flexural strength > $1.1R_yM_p$ or rotate to permit brace buckling while resisting A_gF_{CR}

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These requirements are all for special concentrically braced frames. Block shear and tensile rupture on net section must both be considered. Refer to the 2005 AISC Seismic specification for ordinary concentrically braced frame details.

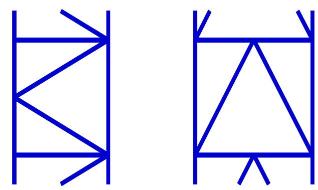
Concentrically Braced Frames

V bracing:

- Design beam for D + L + unbalanced brace forces, using $0.3\phi P_c$ for compression and $R_y F_y A_g$ in tension
- Laterally brace the beam
- Beams between columns shall be continuous.

K bracing:

- Not permitted



Limits on use and design of V and K braces. Refer to Sec. 13.4a in the 2005 AISC Seismic specification for additional description. The following text is taken from the Commentary to the 2005 AISC Seismic specification. Refer to the Commentary for additional information and description of the behavior.

V-braced and inverted-V-braced frames exhibit a special problem that sets them apart from braced frames in which both ends of the braces frame into beam-column connections. The expected behavior of SCBF is that upon continued lateral displacement as the brace in compression buckles, its force drops while that in the brace in tension continues to increase up to the point of yielding. In order for this to occur, an unbalanced vertical force must be resisted by the intersecting beam as well as its connections and supporting members. In order to prevent undesirable deterioration of lateral strength of the frame, the SCBF provisions require that the beam possess adequate strength to resist this potentially significant post-buckling load redistribution (the unbalanced load) in combination with appropriate gravity loads.

Concentrically Braced Frames

Built-up member stitches:

- Spacing < 40% KL/r
- No bolts in middle quarter of span
- Minimum strengths related to P_y

Column in CBF:

- Same local buckling rules as brace members
- Splices resist moments

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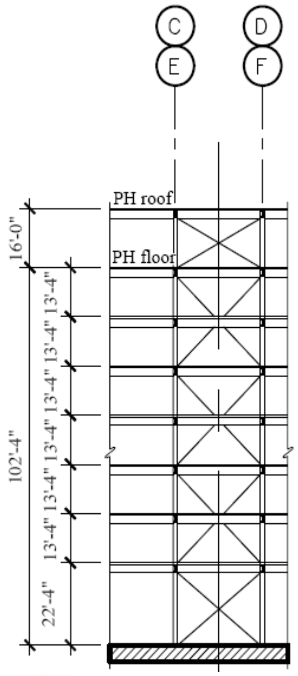
The following text is taken from the Commentary to the 2005 AISC Seismic specification. Closer spacing of stitches and higher stitch strength requirements are specified for built-up bracing members in SCBF than those required for OCBF. These are intended to restrict individual element bending between the stitch points and consequent premature fracture of bracing members. Wider spacing is permitted under an exception when buckling does not cause shear in the stitches. Bolted stitches are not permitted within the middle one-fourth of the clear brace length as the presence of bolt holes in that region may cause premature fractures due to the formation of a plastic hinge in the post-buckling range.



Concentrically Braced Frame Example

E-W direction

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This example is the same building as was considered in the SMF example, which is Example 5.2 in FEMA 451.

Concentrically Braced Frame Example

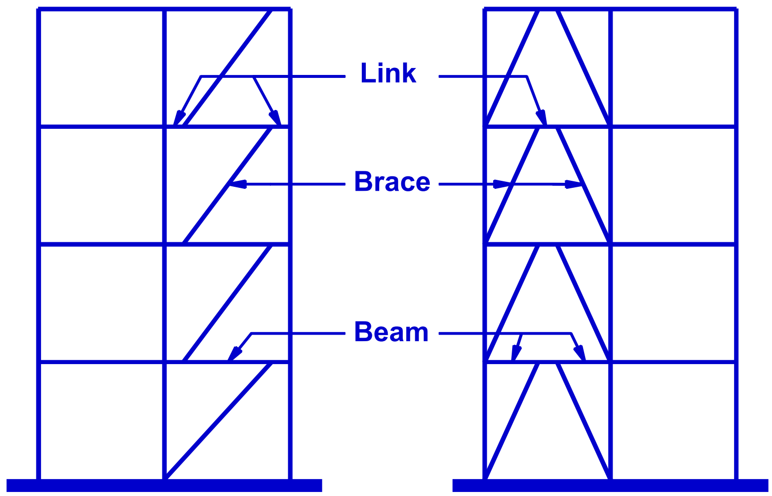
The following general design steps are required:

- Selection of preliminary member sizes
- Check strength
- Check drift
- Check torsional amplification
- Connection design

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 87

The general design steps for a SCBF are indicated on the slide. The load and analysis calculations, along with a summary of each of these steps are given in FEMA 451.

Eccentrically Braced Frames



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Eccentrically braced frames. Define terms.

Eccentrically Braced Frame Under Construction

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Photograph of eccentrically braced frame under construction.

Eccentrically Braced Frame Under Construction

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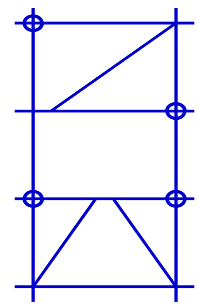
Photograph of eccentrically braced frame under construction.

Eccentrically Braced Frames

Eccentric bracing systems	R	C_d
Building frame system or part of dual system w/ special moment frame		
With moment resisting connections at columns away from links	8	4
Without moment resisting connections at columns away from links	7	4



These connections determine classification

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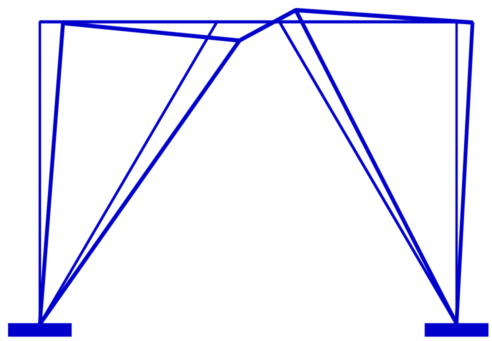
Steel Structures 10 - 91

R and C_d values for eccentric bracing systems vary depending on moment resisting capacity of connections away from link beam; locations identified in diagram. Connections at links must be fully restrained.

Eccentrically Braced Frames
Design Procedure



1. Elastic analysis
2. Check rotation angle; reproportion as required
3. Design check for strength
4. Design connection details

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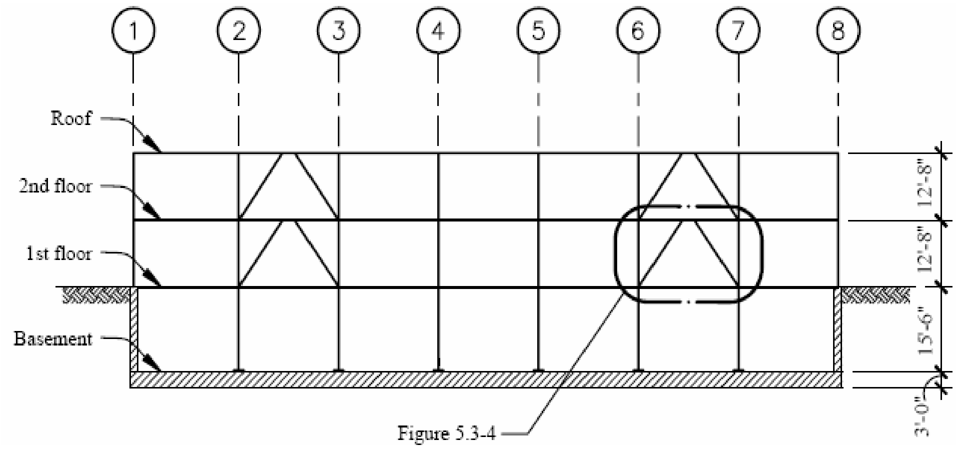
Basic design procedure and special requirements for eccentrically braced frames (EBFs). Elastic analysis of frame determines member forces and elastic deflections; inelastic deflection determined using C_d. EBF deformed this inelastic amount as though if rigid, ideally plastic mechanism; rotation angle of link beam determined. Columns, beams, and braces designed to remain elastic at force level that yields link beam; approximates behavior of rigid, ideally plastic mechanism. Ductility assured and buckling prevented by designing stiffeners and connections at link beam to accommodate link beam rotation in mechanism.

Eccentrically Braced Frames
Example



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Example 5.3 in FEMA 451: two-story hospital in Oakland, California. Building elevation shown. Other design details given in FEMA 451.

Eccentrically Braced Frames
Rotation Angle

1. Compute total $\Delta = C_d \Delta_E$

2. Deform model as rigid-plastic mechanism with hinges at ends of line

3. Compute rotation angle at end of link

4. Check limits (Sec. 15.2g)

$$\alpha \le 0.08 \ radians \ when \ \ L \le \frac{1.6M_p}{V_p}$$

$$\alpha \le 0.02 \ radians \ when \ \ L \ge \frac{2.6M_p}{V_p}$$

$$Interpolate \ \ for \ \alpha \ when \ \ \frac{1.6M_p}{V_p} < L < \frac{2.6M_p}{V_p}$$

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Limits on link beam length related to M_p/V_p ratio. For "balanced" link beam (yields in shear and flexure at same load), e=2.0M_p/V_p. Comparing e divided by M/V at yield to 2.0 indicates type of yield; $(eM_p)/V_p$>2.0 indicates yield in flexure; $(eM_p)/V_p$<2.0 indicates yield in shear. Since shear yield is more ductile and stable, larger rotation angle is allowed. The rotation angle limits 0.08 and 0.02 (radians) are related to e at M_p/V_p ratios of 1.6 and 2.6. Use linear interpolation between these values.

Eccentrically Braced Frames
Rotation Angle Example

From computer analysis:

$$\Delta_e = 0.247\ in$$

Total drift:

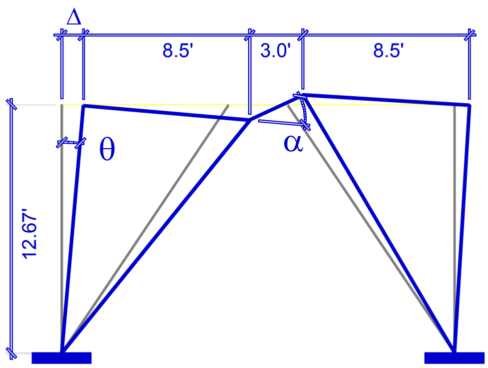
$$\Delta = C_d \Delta_e = 4(0.247) = 0.99\ in.$$

From geometry:

$$\alpha = \left(\frac{L}{e}\right)\theta = \left(\frac{20}{3}\right)\left(\frac{0.99}{12.67(12)}\right) = 0.043 \quad rad$$

$$Because \quad e = 3.0' < \frac{1.6M_p}{F_y} = 3.52'$$

$$\alpha_{\max} = 0.08\ rad > 0.043\ rad \quad OK$$



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This example shows how to calculate link beam rotation angle from drift. Point out the terms:

L is bay width = 8.5 + 3 + 8.5 = 20 ft

e is link length = 3 ft

Eccentrically Braced Frames
Rotation Angle

- Rotation angle limits based on link beam equivalent length
  - Short links yield in shear and are allowed greater rotation
- Rotation angle may be reduced in design by:
  - Increasing member size (reducing Δ_e)
  - Changing geometric configuration (especially changing length of link beam)

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 96

Limits on link beam angle rotation affected by several design choices. May reduce rotation by: increasing member sizes to reduce calculated elastic deformation; changing geometric configuration of system (especially length of link beam). AISC Seismic requires link beams **at columns** to be "short" (yield in shear) unless special reinforcing provided at connection.

Eccentrically Braced Frames
Link Design

- Provide strength V and M per load combinations
- Check lateral bracing per AISC L_{pd}
- Local buckling (width to thickness of web and flange) per AISC Seismic
- Stiffeners (end and intermediate) per AISC Seismic

Summary of link design procedure.

Eccentrically Braced Frames
Brace Design

$$\text{Strength} > 1.25R_y \cdot \begin{pmatrix} \text{axial force from design} \\ \text{shear strength of link} \end{pmatrix}$$

Instructional Material Complementing FEMA 451, *Design Examples*

Steel Structures 10 - 98

Brace strength requirement for EBFs. Member buckling should not occur in EBFs.

Eccentrically Braced Frames

Brace Design Example

Check axial strength of 15.26 ft long TS 8 x 8 x 5/8 F_y = 46 ksi:

$$\frac{KL}{r} = \frac{(1)(15.26)(12)}{2.99} = 61.2$$

$$61.2 < 4.71\sqrt{\frac{E}{F_y}} = 118.3 \qquad \therefore F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29{,}000)}{61.2^2} = 76.4\ ksi$$

$$F_{cr} = \left(0.658^{\frac{46}{76.4}}\right)46 = 35.8 ksi$$

$$\phi_c P_n = \phi_c A_g F_{cr} = 0.9(16.4)(35.8) = 528\ kip$$

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Section 4.3.4.2 in FEMA 451. Calculate brace axial capacity.

Eccentrically Braced Frames

Brace Design Example

$$\phi V_n = 0.9(0.6F_y)d\,t_w = 0.9\left[0.6(50)(16.4)(0.43)\right] = 190\ kip$$

or

$$\phi V_n = 2(0.9)M_p / e = \frac{2(0.9)(50)(105)}{3(12)} = 262.5\ kip$$

$$V_{e(link)} = 85.2\ kip \quad and \quad P_{e(brace)} = 120.2\ kip$$

$$\therefore P_u = 1.25(1.1)\left(\frac{190}{85.2}\right)(120.2) = 369 < 528 \quad OK$$

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 100

Shear yield occurs first. Factor up elastic analysis results and multiply by 1.25 to check brace. Axial brace strength is adequate. Note that brace should also be checked for combined axial load and flexure.

NEHRP Recommended Provisions
Steel Design

- Context in *NEHRP Recommended Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics

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Instructional Material Complementing FEMA 451, *Design Examples*

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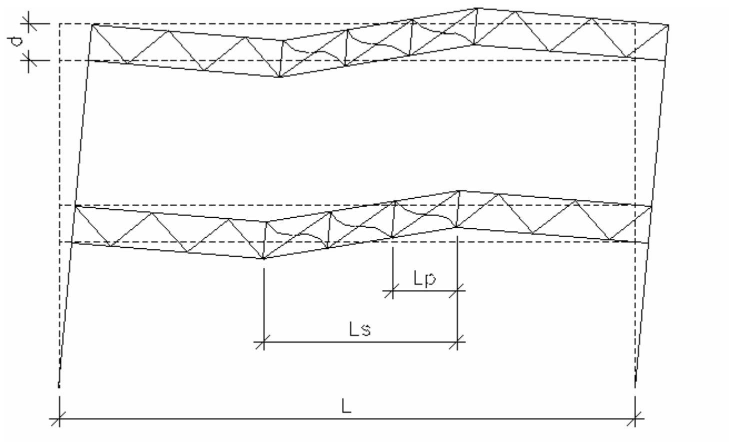
Table of contents: other topics.

Special Truss Moment Frame

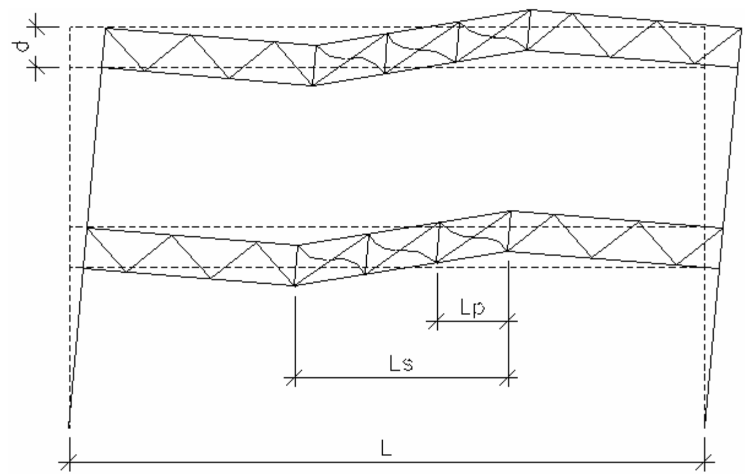


- **Buckling and yielding in special section**
- **Design to be elastic outside special section**
- **Deforms similar to EBF**
- **Special panels to be symmetric X or Vierendeel**

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 102

Special truss moment frames are a new introduction to seismic resistant construction. The concept is that a set of panels near midspan of horizontal trusses, where the gravity load shear is low, are design to yield and buckle in a controlled manner to dissipate energy of strong ground shaking. It should be anticipated that the bucked members may have to be replaced following strong ground shaking.

Special Truss Moment Frame



Geometric Limits:

$L \leq 65'$ $\quad d \leq 6'$

$$0.1 < \frac{L_s}{L} < 0.5$$

$$\frac{2}{3} < \frac{L_p}{d} < \frac{3}{2}$$

Flat bar diagonals, $\frac{b}{t} \leq 2.5$

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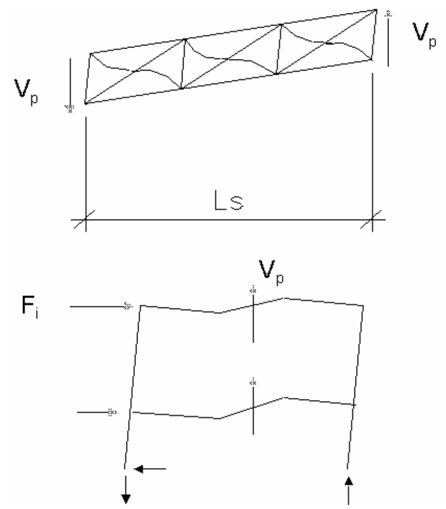
The system has been tested, but only a few real buildings have yet been constructed. The geometric limits are mostly based upon the tested specimens. The use of flat bar diagonals in the special panels prevents local buckling problems.

Special Truss Moment Frame



$$V_p = 2\left(\frac{2\,M_{pc}}{L_s}\right) + \sin\alpha\,\left(P_{nt} + 0.3P_{cd}\right)$$

$$\sum F_i\,h_i = \sum V_p\,L$$

The special section is shown. The plastic shear capacity depends on flexural hinging of the chords at the ends of the section and the buckling and tensile yield capacities of the webs. The plastic shear demand is directly related to the horizontal forces on the structure. The angle between the diagonal and the chord is given as α.

Special Truss Moment Frame

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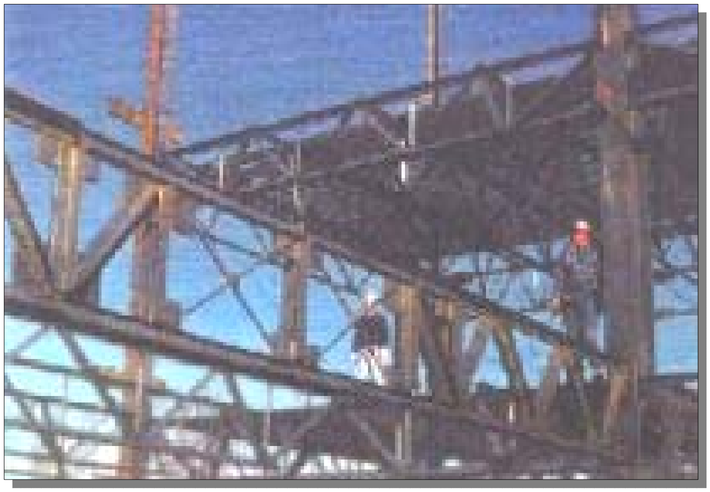
Special truss moment frame experimental test specimen. This specimen was part of a research program at the University of Michigan under the direction of Professor Subash Goel. This research led to the inclusion of the STMF as a recognized structural system in the *NEHRP Recommended Provisions*. Photo courtesy of Nucor Research and Development

Special Truss Moment Frame



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Special truss moment frame under construction.

General Seismic Detailing

Materials:

- Limit to lower strengths and higher ductilities

Bolted Joints:

- Fully tensioned high strength bolts
- Limit on bearing

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Steel Structures 10 - 107

Seismic detailing per AISC Seismic is more than simply the rules for the specific systems. There are also general rules for several subjects that apply to all types of steel seismic resisting systems.

General Seismic Detailing

Welded Joints:

- AWS requirements for welding procedure specs
- Filler metal toughness
  - CVN > 20 ft-lb @ -20°F, or AISC Seismic App. X
- Warning on discontinuities, tack welds, run offs, gouges, etc.

Columns:

- Strength using Ω_o if $P_u / \phi P_n > 0.4$
- Splices: Requirements on partial pen welds and fillet welds

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Steel Structures 10 - 108

The requirements for weld metal toughness are new. The reference to AWS welding procedure specs is a new emphasis. It is likely that structural engineers will have to become more well educated about welding in the future. The rules on columns are intended to prevent premature failure in these vulnerable components.

Steel Diaphragm

Example

$\phi V_n = \phi$ (approved strength)

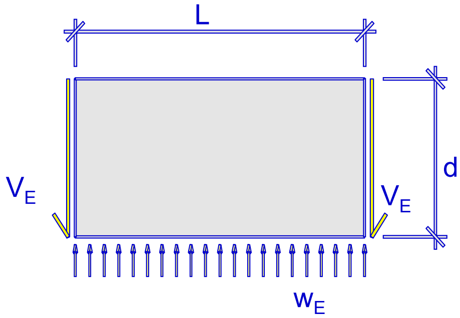
$\phi = 0.6$

For example only:

Use approved strength as 2.0 x working load in SDI *Diaphragm Design Manual*

This example not in FEMA 451; for example only, use capacities from SDI *Diaphragm Design Manual*.

Steel Deck Diaphragm Example



$L = 80'$ $\quad d = 40'$

$w_D = w_L = 0$ $\quad w_E = 500 \text{ plf}$

$$V_E = \frac{w_E L}{2} = 20 \text{ kip}; \quad v_E = \frac{20000}{40} = 500 \text{ plf}$$

$$v_{SDI} = \frac{v_E}{2\phi} = \frac{500}{2(0.6)} = 417 \text{ plf}$$

Deck chosen:
1½ ", 22 gage with welds on 36/5 pattern and 3 sidelap fasteners, spanning 5'-0"

Capacity = 450 > 417 plf

Shear in diaphragm due to dead and live loads is zero. Given the seismic load, w_E=500 plf, calculate total shear at end of diaphragm; compute shear flow at end of diaphragm. Required shear capacity is shear demand divided by 2.0ϕ. Choose deck and fasteners. Fasteners and their spacing greatly affect diaphragm capacity; must carefully inspect deck welds and sidelap fasteners. Inspector needs to make sure that deck has not been burned through, and that connection has been made

Welded Shear Studs

Instructional Material Complementing FEMA 451, *Design Examples*

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Welded shear studs have been a part of the AISC Specifications since 1961. Beginning in the late 1980s, the strength model for shear studs used in cold-formed steel deck begin to come under scrutiny. In particular, the reduction factor, which was a function of deck geometry, in the AISC specifications was thought to be unconservative and need of change. Research teams from several places around the world were working on the problem. This was in part due to the fact that the AISC procedures had been widely adopted. A series of beam tests and several hundred push-out tests were conducted at Virginia Tech over a number of years. This comprehensive research program led to changes in the 2005 AISC specification. These changes will be highlighted in the following slides.

Shear Stud Strength - AISC 2005 Specification

$$Q_n = 0.5 A_{sc} (f_c' E_c)^{1/2} \le R_g R_p A_{sc} F_u$$

R_g = stud geometry adjustment factor
R_p = stud position adjustment factor

Note that the strength reduction factor for bending has been increased from 0.85 to 0.9. This results from the strength model for shear studs being more accurate, although the result for Q_n is lower in the 2005 specification.

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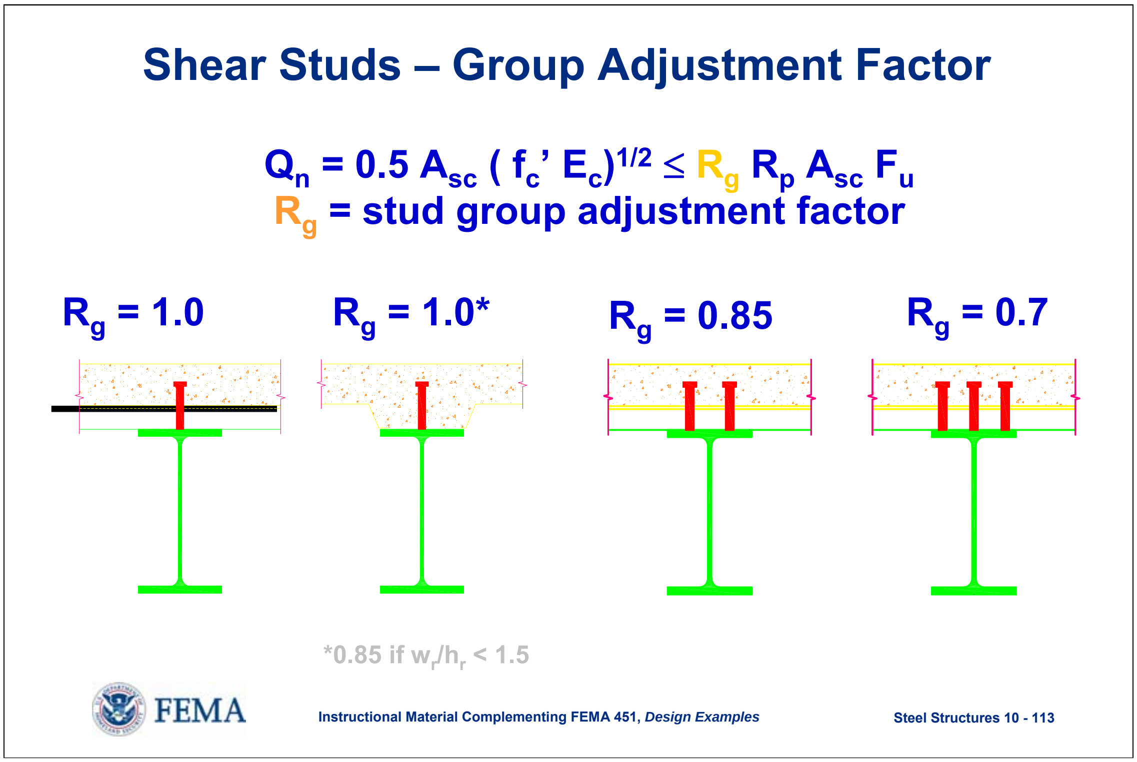
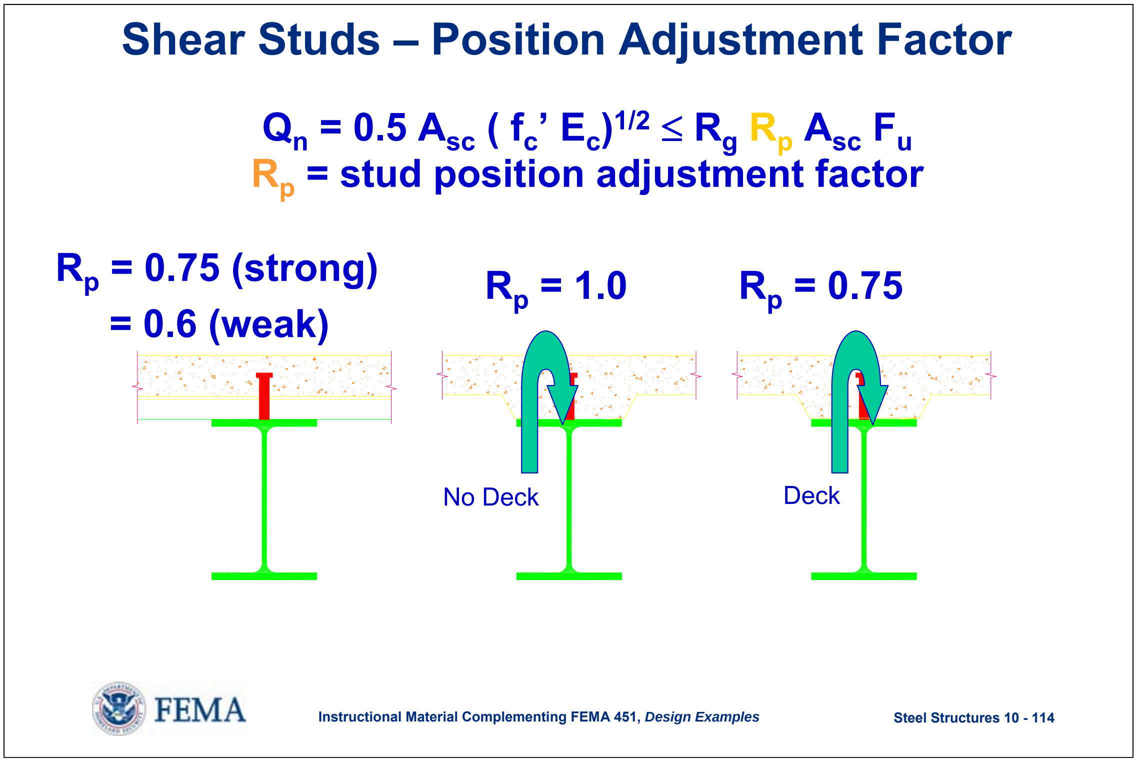
The shear stud strength model is no longer a direct function of the deck geometry, although the limitations of the empirical based formulation is a function of the deck depths that were used in the study. The reader is referred to the 2005 AISC specification and commentary for a more complete discussion. The flexural design procedure is unchanged, other than the calculation of the shear stud strength. The strength reduction factor for bending has been increased from 0.85 to 0.9. This reflects the use of a more accurate shear stud strength model. As in past specifications, there is no strength reduction factor applied separately to the shear studs.

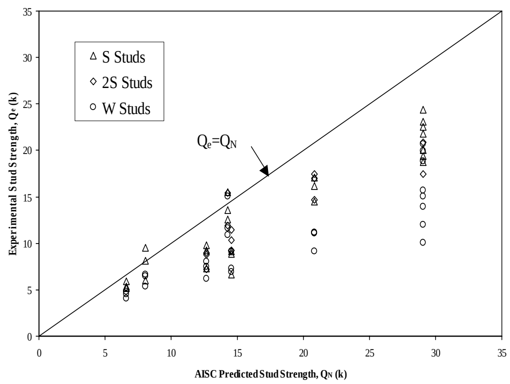


This slide illustrates the application of the group adjustment factor, R_g. The first, third and fourth figures illustrate "beam" applications – that is the deck is perpendicular to the steel section. The second figure illustrates a "girder" for which the deck is parallel to the steel section. A narrow rib in the girder application, as defined by the w_r/h_r ration being less than 1.5, results in a reduction in R_g to a value of 0.85.



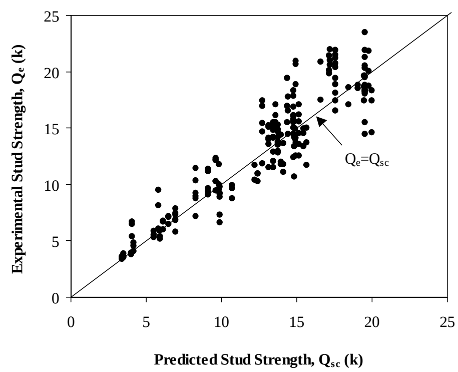
This slide illustrates the applications of the position adjustment factor. Refer to AISC for a further description of the "no deck" and "deck" cases for the "girder" applications in the second and third figure. The recommended value of R_p for the beam case, illustrated in the first figure, is the weak position case of 0.6. Use of the higher value of 0.75 requires close control and inspection during the stud installation process during construction. This control is difficult to obtain.

Shear Studs – Strength Calculation Model Comparison



AISC Seismic prior to 2005

Virginia Tech strength model



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The two graphs illustrate shear stud strength calculation model comparisons with experimental push-off test data. The graph on the left is a comparison between experimental shear stud strengths and the AISC calculation model prior to 2005. Note that virtually all the data indicates that the model is unconservative by as much as 50% in a number of cases. The reader should note that the relationship between shear stud strength and flexural strength is non-linear. A reduction of shear stud strength by 50% may equate to a reduction in flexural strength of approximately 20%. The graph on the right is a comparison between a large data base of push out tests with what is referred to as the Virginia Tech calculation model. The Virginia Tech model formed the basis for the 2005 AISC specification provisions. Note that the comparison with the new model is much better.

Shear Studs – Diaphragm Applications

Shear studs are often used along diaphragm collector members to transfer the shear from the slab into the frame. The shear stud calculation model in the 2005 AISC specification can be used to compute the nominal shear strengths. A strength reduction factor should be used when comparing these values to the factored shear. There is no code-established value for the strength reduction factor. A value of 0.8 is recommended pending further development.

Text in slide is self-explanatory and requires no further comment.

Inspection and Testing
Inspection Requirements

- Welding:
  - Single pass fillet or resistance welds
    - > PERIODIC
  - All other welds
    - > CONTINUOUS
- High strength bolts:
    - > PERIODIC

Instructional Material Complementing FEMA 451, *Design Examples*

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Inspection requirements: Periodic inspection of single pass fillet welds and continuous inspection of all other welds. AWS D1.1 is a good reference for what items should be noted during inspection of welding. Additional inspection methods include ultrasonic inspection, x-ray inspection, and other non-destructive methods. High strength bolts must be periodically inspected.

Inspection and Testing
Shop Certification

- Domestic:
  - AISC
  - Local jurisdictions
- Foreign:
  - No established international criteria

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AISC can certify fabricating shops. Program includes scheduled and surprise inspections to review quality control procedures, personnel, and results. Some local jurisdictions have adopted similar certification procedures. As there is no internationally accepted quality certification, foreign fabricated steel should be given close inspection.

Inspection and Testing
Base Metal Testing

- More than 1-1/2 inches thick
- Subjected to through-thickness weld shrinkage
- Lamellar tearing
- Ultrasonic testing

Base metal testing required: where base metal more than 1-1/2 in. thick subjected to through-thickness weld shrinkage (lamellar tearing is possible); ultrasonic testing required to identify presence of discontinuities in base metal.

NEHRP Recommended Provisions
Steel Design

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics
- Summary

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Summary