PC UFC Briefing
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Steel Requirements

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Steel Requirements

- This presentation provides the specific requirements for designing a structural steel building to resist progressive collapse.
- Appendix D in the UFC demonstrates the application of the structural steel design requirements for a 5-story office building.

Steel Requirements

- Composite Construction
  - ◊ Steel is often used in composite construction, such as
    - concrete deck slabs on steel beams,
    - sheet steel decking with an integral slab, and
    - columns reinforced with structural steel shapes
  - ◊ For steel/concrete composite construction, the application of both the requirements of this chapter and those provided for reinforced concrete in Chapter 4 are required.

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Steel Requirements

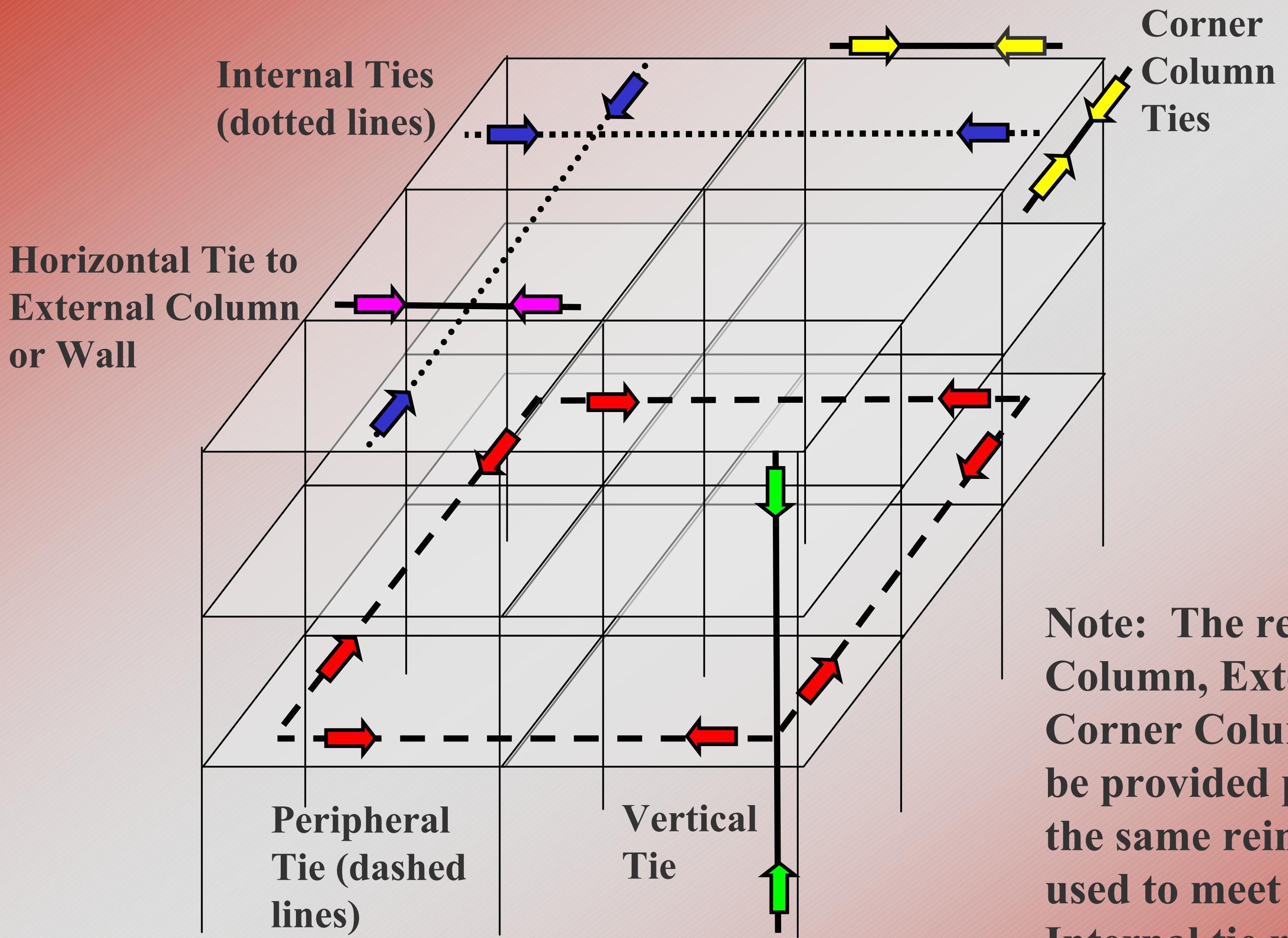
- Material Properties For Structural Steel
  - ◊ Apply the appropriate over-strength factors to the calculation of the design strengths for **both** Tie Forces and the Alternate Path method.

Structural Steel	Ultimate Over-Strength Factor, Ω	Yield Over-Strength Factor, Ω
Hot-Rolled Structural Shapes and Bars		
ASTM A36/A36M	1.05	1.5
ASTM A573/A572M Grade 42	1.05	1.3
ASTM A992/A992M	1.05	1.1
All other grades	1.05	1.1
Hollow Structural Sections		
ASTM A500, A501, A618 and A847	1.05	1.3
Steel Pipe		
ASTM A53/A53M	1.05	1.4
Plates	1.05	1.1
All other products	1.05	1.1

Steel Tie Force Requirements

- Steel Tie Force Requirements
  - ◊ General
    - All buildings must be effectively tied together at each principal floor level.
    - Each column must be effectively held in position by means of horizontal ties in two directions, approximately at right angles, at each principal floor level supported by that column.
    - Horizontal ties must similarly be provided at the roof level, except where the steelwork only supports cladding that weighs not more than 0.7 kN/m^2 (14.6 lb/ft^2) and that carries only imposed roof loads and wind loads.

Steel Tie Force Requirements



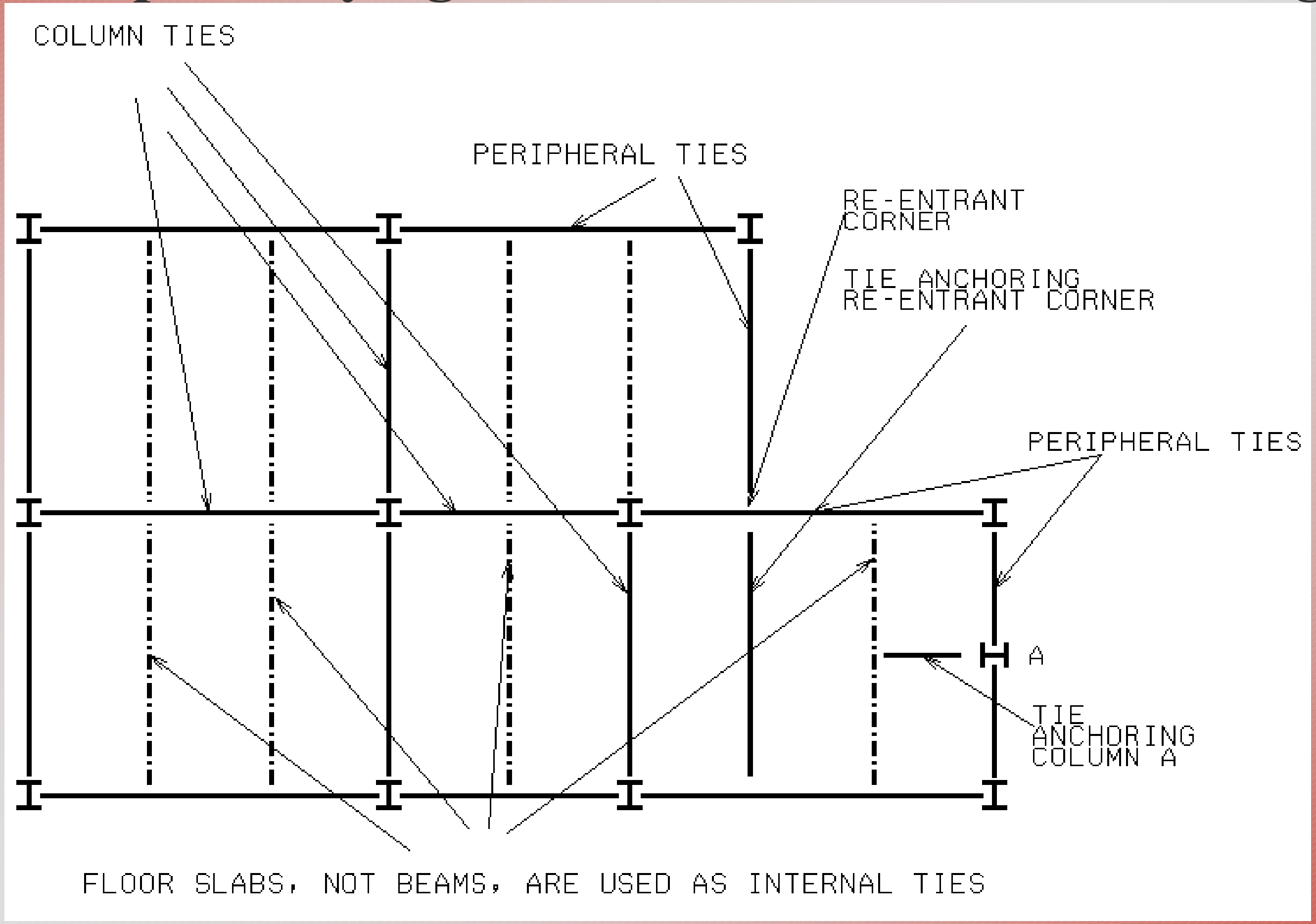
Note: The required External Column, External Wall, and Corner Column tie forces may be provided partly or wholly by the same reinforcement that is used to meet the Peripheral or Internal tie requirement.

Steel Tie Force Requirements

- Steel Tie Force Requirements, cont'd
  - ◊ General, cont'd
    - Continuous lines of ties must be arranged as close as practical to the edges of the floor or roof and to each column line.
    - At re-entrant corners, the tie members nearest to the edge must be anchored into the steel framework.

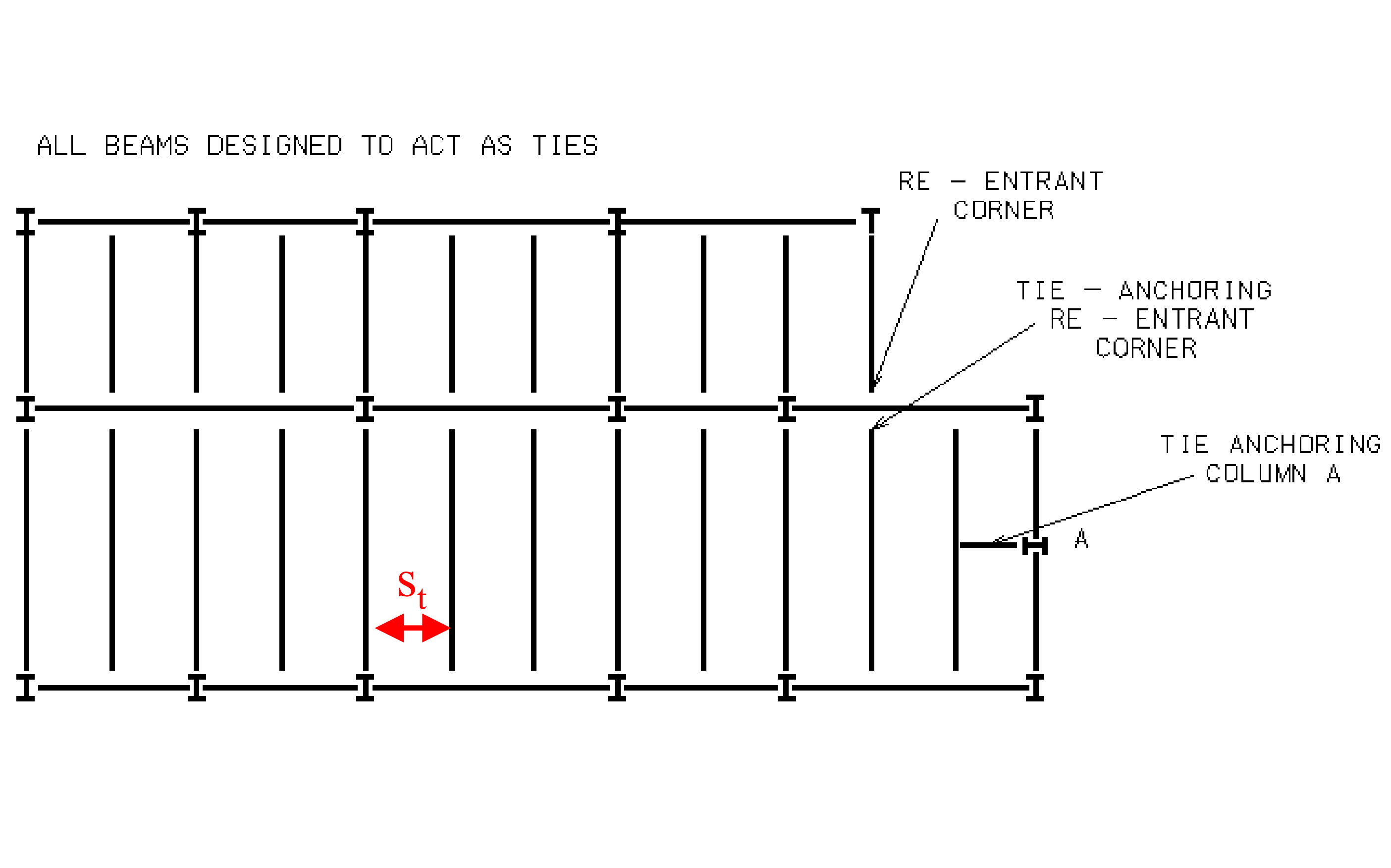
Steel Tie Force Requirements

Example of Tying the Columns of a Steel Building



Steel Tie Force Requirements

Example of General Tying of a Steel Building



Steel Tie Force Requirements

- Strength Reduction Factor Φ for Steel Tie Forces
  - ◊ For the steel members and connections that provide the design tie strengths, use the appropriate tensile strength reduction factors Φ from the 2003 version of the *Manual of Steel Construction, Load and Resistance Factor Design* from the American Institute of Steel Construction.
  - ◊ For example, use a strength reduction factor of 0.75 for block shear at a bolted connection.

Steel Tie Force Requirements

- Horizontal Steel Ties
  - ◊ The horizontal ties may be either
    - steel members, including those also used for other purposes, or,
    - steel reinforcement that is anchored to the steel frame and embedded in concrete, designed per ACI 318-02 and meeting the continuity and anchorage requirements of Section 4-2.4.

Steel Tie Force Requirements

- Internal Ties
  - ◊ *In English units*, the required internal tie strength is

$$0.5\,(1.2D + 1.6L)\,s_t\,L_l \quad \text{but not less than 16.9 kips}$$

where: D = Dead Load (lb/ft^2)
L = Live Load (lb/ft^2)
L_l = Span (ft) of that particular tie
s_t = Mean transverse spacing of the ties adjacent to the ties being checked (ft)

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Steel Tie Force Requirements

- Peripheral Ties
  - ◊ *In English units*, peripheral ties must be capable of resisting:

$$0.25\ (1.2D + 1.6L)\ s_t\ L_l \quad \text{but not less than 8.4 kips}$$

Steel Tie Force Requirements

- Tying of External Columns
  - ◊ The horizontal ties anchoring the column nearest to the edges of a floor or roof must be capable of resisting a factored tensile load, acting perpendicular to the edge, equal to the greater of the load calculated in Section 5-2.4 or 1% of the maximum factored vertical dead and live load in the column that is being tied, considering all load combinations used in the design.

Steel Tie Force Requirements

- Vertical Ties
  - ◊ All columns must be carried through at each beam-to-column connection.
  - ◊ All column splices must be capable of resisting a tensile force equal to the largest factored vertical dead and live load reaction (from all load combinations used in the design) applied to the column **at a single floor** level located between that column splice and the next column splice down.

Steel Tie Force Requirements

- Background
  - ◊ In Burnett 1975, the review of the British Tie Force requirements was limited to reinforced concrete.
  - ◊ Similar details for steel design were not uncovered in development of this UFC.
  - ◊ The tie force requirements for steel are often different from reinforced concrete; this is not surprising given that separate organizations generated the steel and concrete building standards.

Steel Tie Force Requirements

- Background, cont'd
  - ◊ Differences between RC and Steel Tie Forces
    - For steel tie forces, a "basic strength" as a function of building height is not used, as it is for concrete.
    - The loads used to determine the horizontal steel tie forces are based on factored loads, whereas the upper limit on the basic strength and the scaling of the concrete tie forces are based on un-factored loads.

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Steel Tie Force Requirements

- Background, cont'd
  - ◊ Differences between RC and Steel Tie Forces, cont'd
    - For both steel and reinforced concrete, the horizontal ties to external columns are based on factored loads; however, 1% is used for reinforced concrete and 3% is used for steel.
    - The lower limit on the internal steel tie force of 75 kN (16.9 kips) is slightly different from the lower limit of 60 kN (13.9 kips) for reinforced concrete.

Steel Tie Force Requirements

- Columns with Deficient Vertical Tie Forces
  - ◊ If it is not possible to provide the minimal vertical tie force in any of the columns, then the Alternate Path method is applied for each deficient column.
  - ◊ Remove each deficient column from the structure, one at a time in each story in turn, and perform an AP analysis to verify that the structure can bridge over the missing column.

Steel Tie Force Requirements

- Columns with Deficient Vertical Tie Forces, cont'd
  - ◊ The specific details for AP analysis of structural steel construction are provided in the next section.
  - ◊ Note that the only type of load-bearing element that can be removed is a column, so no table defining the extent of the removed elements is needed.

AP Method for Steel

- Alternate Path Method For Steel
  - ◊ The Alternate Path method must be used to verify that the structure can bridge over removed elements.
  - ◊ The general procedure provided in Section 3-2 in the UFC must be followed.

AP Method for Steel

- Acceptability Criteria for Structural Steel
  - ◊ The acceptability criteria are provided in Table 5-2.
  - ◊ The design strengths must be calculated per AISC LRFD 2003.
  - ◊ The subsequent actions for the AP model after violation of the acceptability criteria are detailed in the following slides.

AP Method for Steel

Acceptability Criteria and Subsequent Action for Steel

Structural Behavior	Acceptability Criteria	Subsequent Action for Violation of Criteria
Element Flexure	$\Phi M_n{}^A$	Section 5-3.1.1
Element Combined Axial and Bending	AISC LRFD 2003 Chapter H Interaction Equations	Section 5-3.1.2
Element Shear	$\Phi V_n{}^A$	Section 5-3.1.3
Connections	Connection Design Strength[A]	Section 5-3.1.4
Deformation	Deformation Limits, defined in Table 5-3	Section 5-3.2

[A] Nominal strengths are calculated with the appropriate material properties and over-strength factor Ω; all Φ factors are defined per AISC LRFD 2003.

AP Method for Steel

- Flexural Resistance of Steel
  - ◊ A flexural member can fail by reaching its full plastic moment capacity, or it can fail by lateral-torsional buckling (LTB), flange local buckling (FLB), or web local buckling (WLB).
  - ◊ Calculate nominal moment strength, M_n, in accordance with AISC LRFD 2003.
  - ◊ If a flexural member's capacity is governed by a buckling mode of failure, remove the element when the internal moment reaches the nominal moment strength and distribute the loads associated with the element in accordance with 3-2.4.3.

AP Method for Steel

- Flexural Resistance of Steel, cont'd
  - ◊ If the member strength is not governed by buckling, the strength will be governed by plastification of the cross section and it may be possible for a plastic hinge to form.

AP Method for Steel

- Formation of Plastic Hinge
  - ◊ If hinge formation is included in the AP analysis, the requirements of Section A5.1 of the AISC LRFD 2003 for plastic design must be met.
  - ◊ AISC LRFD 2003 permits plastic analysis only when the structure can remain stable, both locally and globally, up to the point of plastic collapse or stabilization.
  - ◊ Where the analysis indicates the formation of multiple plastic hinges, the analyst must ensure each cross section or connection assumed to form a plastic hinge is capable of not only forming the hinge, but also capable of the deformation demands created by rotation of the hinge as additional hinges are formed in the element or structure.

AP Method for Steel

- Formation of Plastic Hinge
  - ◊ Since the element could be required to undergo large deformations as plastic hinges are being formed, special lateral bracing is required.
  - ◊ The magnitude of the plastic moment, M_p, used for analysis must consider the influence of axial or shear force when appropriate.
  - ◊ Further information on plastic design are provided in *The Plastic Methods of Structural Analysis* by Neal (Neal 1963) and *Plastic Design of Steel Frames* by Beedle (Beedle 1958).

AP Method for Steel

- Modeling of a Plastic Hinge
  - ◊ For Linear Static analyses, if the calculated moment exceeds the nominal moment strength and it is determined the element is capable of forming a plastic hinge, insert an "equivalent" plastic hinge into the model:
    - insert a discrete hinge in the member at an offset from the member end and ,
    - add two constant moments, one at each side of the new hinge, in the appropriate direction for the acting moment.
    - The magnitude of the constant moments is equal to the determined plastic moment capacity of the element.

AP Method for Steel

- Modeling of a Plastic Hinge, cont'd
  - ◊ The designer can determine the location of the plastic hinge through engineering analysis and judgment or with the guidance provided for seismic connections in *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings* from the Federal Emergency Management Agency (FEMA 350) and *Seismic Provisions for Structural Steel Buildings*, from AISC (AISC 341-02).
  - ◊ For Nonlinear Static and Dynamic Analysis, the software must be capable of representing post-peak flexural behavior and consider interaction effects of axial loads and moment.
  - ◊ The designer must ensure that shear failure will not occur prior to developing the full flexural design strength.

AP Method for Steel

- Combined Axial and Bending Resistance of Structural Steel
  - ◊ The response of an element under combined axial force and bending moment can be **force controlled** (i.e. non-ductile) or **deformation controlled** (i.e. ductile).
  - ◊ If the element is sufficiently braced to prevent buckling and the ratio of applied axial force to the axial force at yield (P_u/P_y where $P_y = A_g F_y$) is less than 0.15, the member can be treated as **deformation controlled** with no reduction in plastic moment capacity, i.e. as a flexural member in accordance with Section 5-3.1.1.
  - ◊ For all other cases, treat the element as a beam-column and make the determination of whether the element is deformation or force controlled in accordance with the provisions of FEMA 356 Chapter 5.

AP Method for Steel

- Combined Axial and Bending Resistance of Steel, cont'd
  - ◊ If the controlling action for the element is **force controlled**, evaluate the strength of the element using the interaction equations in Chapter H of AISC LRFD 2003, incorporating the appropriate strength reduction factors Φ and the over-strength factor Ω.
  - ◊ Remove the element from the model when the acceptability criteria is violated and redistribute the loads associated with the element per Section 3-2.4.3.

AP Method for Steel

- Combined Axial and Bending Resistance of Steel, cont'd
  - ◊ If the controlling action for the element is deformation controlled, the element can be modeled for inelastic action using the modeling parameters for nonlinear procedures in Table 5-6 in FEMA 356.
  - ◊ The linear static and dynamic procedures specified in FEMA 356 are not consistent with the analysis approach of this UFC; however, the nonlinear modeling parameters provided in FEMA 356 can be utilized to determine the equivalent plastic hinge properties (see 5-3.1.1.2) for use in the linear static analysis procedure of this UFC.

AP Method for Steel

- Combined Axial and Bending Resistance of Steel, cont'd
  - ◊ In linear analyses, take the force deformation characteristics of the elements as bilinear (elastic – perfectly plastic), ignoring the degrading portion of the relationship specified in FEMA 356.
  - ◊ The modeling of plastic hinges for beam-columns in linear static analyses must include a reduction in the moment capacity due to the effect of the axial force (see FEMA 356 Equation 5-4).

AP Method for Steel

- Combined Axial and Bending Resistance of Steel, cont'd
  - ◊ For nonlinear analysis, the modeling of elements, panel zones, or connections must follow the guidelines in FEMA 356.
  - ◊ Nonlinear analyses must utilize coupled (P-M-M) hinges that yield based on the interaction of axial force and bending moment.
  - ◊ In no cases shall the deformation limits established in FEMA 356 exceed the deformation limits established in Table 5-3 of this UFC.

AP Method for Steel

- Shear Resistance of Structural Steel
  - ◊ The acceptability criteria for shear of structural steel is based on the nominal shear strength of the cross-section, per AISC LRFD 2003, multiplied by the strength reduction factor Φ and the over-strength factor Ω.
  - ◊ If the element violates the shear criteria, remove the element and redistribute the loads associated with the element per Section 3-2.4.3.

AP Method for Steel

- Connections
  - All connections must meet the requirements of AISC LRFD 2003; employ the appropriate strength reduction factor Φ for each limit state and over-strength factor Ω.
  - As detailed in AISC LRFD 2003, multiple limit states for the connections must be considered.
  - If a connection violates one of the limit states criteria, remove it from the model.
  - If both connections at the ends of an element fail, remove the element and redistribute the loads associated with the element per Section 3-2.4.3.

AP Method for Steel

- Deformation Limits for Structural Steel
  - ◊ The Deformation Limits are given in the following table.

Deformation Limits for Structural Steel

	AP for Low LOP		AP for Medium and High LOP	
Component	Ductility (μ)	Rotation, Degrees (θ)	Ductility (μ)	Rotation, Degrees (θ)
Beams--Seismic Section[A]	20	12	10	6
Beams--Compact Section[A]	5	-	3	-
Beams--Non-Compact Section[A]	1.2	-	1	-
Plates	40	12	20	6
Columns and Beam-Columns	3	-	2	-
Steel Frame Connections; Fully Restrained				
Welded Beam Flange or Coverplated (all types)[B]	-	2.0	-	1.5
Reduced Beam Section[B]	-	2.6	-	2
Steel Frame Connections; Partially Restrained				
Limit State governed by rivet shear or flexural yielding of plate, angle or T-section[B]	-	2.0	-	1.5
Limit State governed by high strength bolt shear, tension failure of rivet or bolt, or tension failure of plate, angle or T-section[B]	-	1.3	-	0.9

[A] As defined in AISC 341-02.

[B] See Appendix B.

AP Method for Steel

- Deformation Limits for Structural Steel, cont'd
  - With the exception of the connection limits, the structural steel deformation limits were provided by PDC.
  - The values for the fully restrained and partially restrained connections for MLOP and HLOP were taken from GSA 2003.

AP Method for Steel

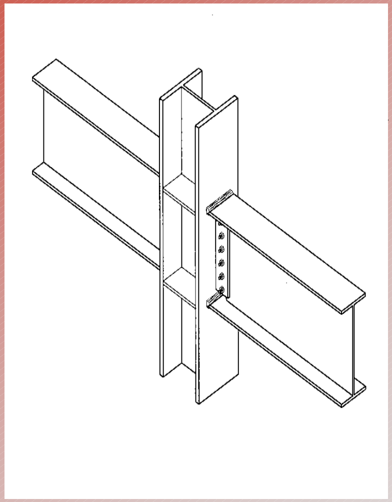
- Deformation Limits for Structural Steel, cont'd
  - These values were developed by GSA based on the FEMA series of documents, with modifications to account for: the results of post-Northridge full-scale cyclic testing; corroborative nonlinear analyses performed by FEMA and others; and monotonic test results from Georgia Tech on riveted connections.
  - The magnitudes were also modified to account for the fact that failure of only one or two connections can trigger a progressive collapse, whereas, in seismic engineering, the category of Life Safety permits the failure of up to 10-15% of the connections.

AP Method for Steel

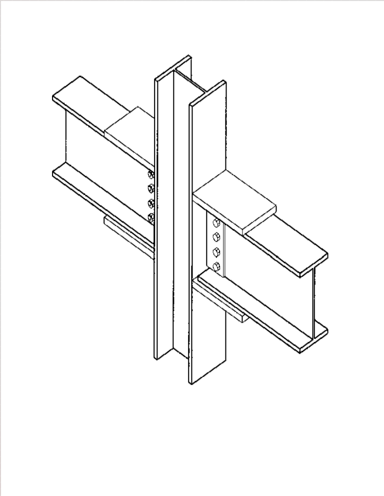
- Deformation Limits for Structural Steel, cont'd
  - The values for connection deformation limits for LLOP were derived from the MLOP/HLOP values by using the ratio of ductilities for Life Safety and Collapse Prevention from the FEMA values, i.e., the increase from MLOP/HLOP to LLOP is the same ratio as FEMA uses to go from Life Safety to Collapse Prevention.
  - Examples of partially restrained and fully restrained steel connections are shown on the next slides.

AP Method for Steel

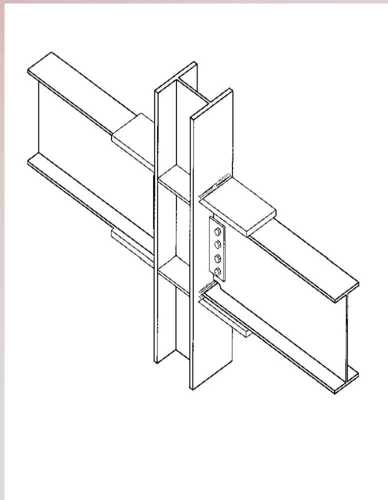
Fully Rigid Moment Connections



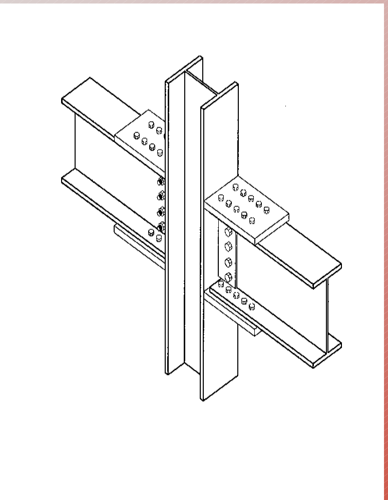
WUF Fully Rigid Connection



Welded Flange Plate



Welded Cover Plated Flanges

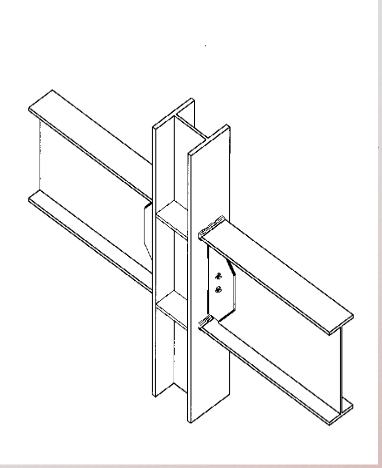


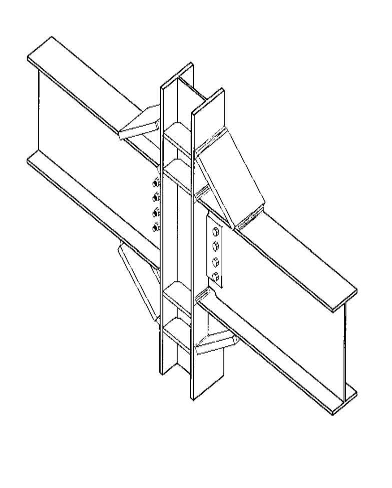
Bolted Flange Plate

AP Method for Steel

Fully Rigid Moment Connections

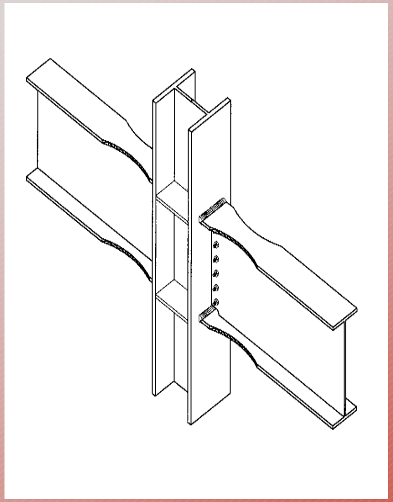
Free Flange





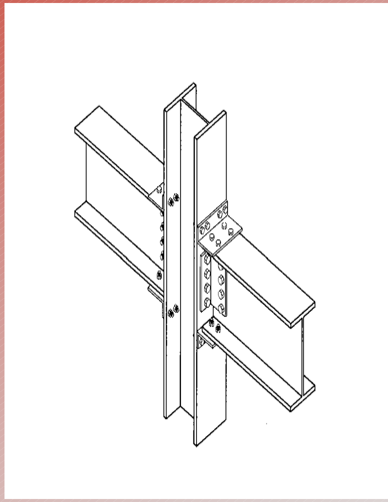
Top and Bottom Haunch

Reduced Beam Section



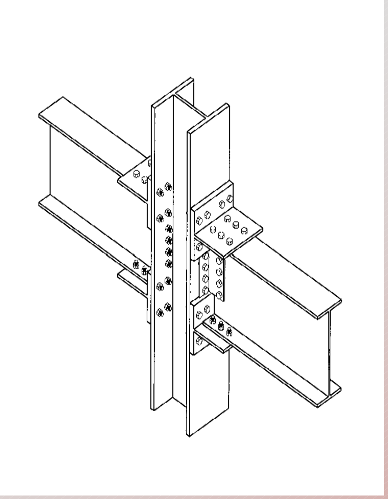
AP Method for Steel

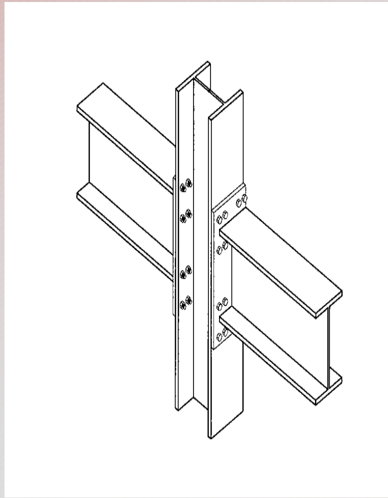
Partially Rigid Moment Connections



Bolted or
Riveted Angle

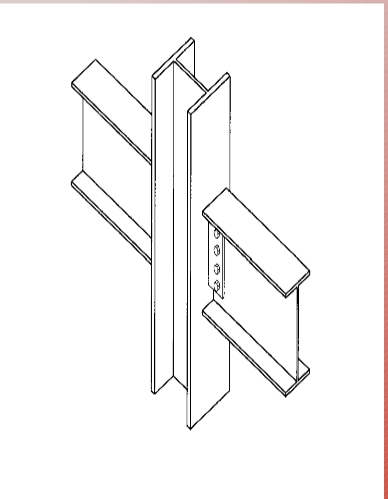
Double
Split Tee





End Plate
(Unstiffened)

Typical Shear
Connection
(without slab)



Steel Requirements

- Additional Ductility Requirements
  - ◊ For MLOP and HLOP structures, all perimeter ground floor columns and load-bearing walls must be designed such that the shear capacity is greater than the flexural capacity, including compression membrane effects where appropriate.
  - ◊ Methods for calculating the compression membrane effects can be found in Park and Gamble 1999 and UFC 3-340-01.

Questions/Comments?