A note about sources:
The underlying concepts and requirements for disproportionate collapse were drawn from United Kingdom standard, "Building Regulations 2004 (Structure)," Approved Document A, specifically section A3 (UK). The specific technical criteria for masonry, steel and concrete construction were drawn from UFC 4-023-03 (January 25, 2005), Unified Facilities Criteria, "Design of Buildings to Resist Progressive Collapse" (UFC) and from the American Society of Civil Engineers (ASCE) 7, Commentary, 2005 edition (ASCE). Specific technical criteria for concrete construction also were drawn from the American Concrete Institute (ACI) 318-02, "Building Code Requirements for Structural Concrete" and from the commentary for this standard (ACI).

(new) **Section 1604.9 Disproportionate Collapse**. Design for structural integrity to protect against disproportionate collapse shall be in accordance with this section.

Section 1604.9.1 General. The building, structure or portion thereof shall be constructed so the building will not suffer collapse as the result of an accident or incident to an extent disproportionate to the cause. Buildings shall be designed for sufficient robustness to sustain a limited extent of damage or failure, depending on the class of the building, without collapse.

Section 1604.9.2 Definitions:

Disproportionate collapse. Disproportionate collapse shall be deemed to have occurred when the local failure of a primary structural component(s) leads to the collapse of the adjoining structural members, which then leads to additional collapse.

(UK 5.3)Load bearing construction Load bearing construction shall include masonry cross-wall construction and walls of lightweight steel section studs.

(UK 5.3) Key element. A structural element capable of sustaining an accidental design loading of 700 psf applied in the horizontal and vertical directions (in one direction at a time) to the member and any attached components (ie. cladding, etc.).

(UK 5.1a) Section 1604.9.3 Building Class. Buildings shall be classified in accordance with Table 1604.9.3. Buildings with occupancy groups within more than one classification shall be designed as the higher class.

(UK Table 11) **(new) Table 1604.9.3**

Class	Building Type and Occupancy
1	Group R-3 or R-5 not exceeding 4 stories Agricultural buildings Unoccupied buildings that are separated from other buildings by a distance of 1.5 times the buildings height.
2	Group R-3 not exceeding 5 stories Group R-1 not exceeding 4 stories

	Group R-2 not exceeding 4 stories Group B not exceeding 4 stories Group F not exceeding 3 stories Group M not exceeding 3 stories of less than 21,500 square feet floor area in each story. Group E not exceeding one story All buildings of Group A not exceeding 2 stories which contain floor areas not exceeding 21,500 square feet at each story.
3	Group R-1 and R-2 buildings greater than 4 stories but not exceeding 15 stories Group E buildings greater than 1 story but not exceeding 15 stories. Group M buildings greater than 3 stories but not exceeding 15 stories. Group I-2 buildings not exceeding 3 stories. Group B buildings greater than 4 stories but not exceeding 15 stories. Group A buildings which contain floors of more than 21,500 square feet but less than 54,000 square feet per floor. Group S-2 buildings not exceeding 6 stories.
4	All buildings that exceed the limits on area or number of stories for class 1-3. Grandstands accommodating more than 5000 spectators. Building containing hazardous substances and/or processes.

(UK 5.1b) **Section 1604.9.3.1 Class 1 buildings.** Class 1 buildings are not required to comply with this section.

(UK 5.1c) **Section 1604.9.3.2 Class 2 buildings.** Class 2 buildings shall be provided with horizontal ties, as per section 1604.9.3.2.1 or with anchorage as per section 1609.3.2.2:

1604.9.3.2.1 Horizontal ties. Horizontal ties shall be provided in accordance with sections 1604.9.4.1, 1604.9.4.2, and 1604.9.4.3, as applicable.

(UFC 4-2.7.1, 5-2.3, 6-2.7) **1604.9.3.2.2 Anchorage.** Anchorage of suspended floors to walls shall be provided as per sections 1604.9.4.1, 1604.9.4.2, and 1604.9.4.3, as applicable, for load-bearing construction.

(UK 5.1d) **Section 1604.9.3.3 Class 3 buildings.** Class 3 buildings shall be provided with horizontal ties, as per section 1604.9.3.3.1, anchorage as per section 1609.3.3.2, and vertical ties as per section 1604.9.3.3.3 or shall be designed utilizing alternate load path analysis as per section 1609.3.3.4.

1604.9.3.3.1 Horizontal ties. Horizontal ties shall be provided in accordance with sections 1604.9.4.1, 1604.9.4.2, and 1604.9.4.3, as applicable.

(UFC 4-2.7.1, 5-2.3, 6-2.7) **1604.9.3.3.2 Anchorage.** Anchorage of suspended floors to walls shall be provided as per sections 1604.9.4.1, 1604.9.4.2, and 1604.9.4.3, as applicable, for load-bearing construction.

(**UFC 4-2.8, 5-2.7, 6-2.8**) **1604.9.3.3.3 Vertical ties**. Vertical ties shall be provided in accordance with sections 1604.9.4.1, 1604.9.4.2, and 1604.9.4.3, as applicable.

(**UFC 4-3, 5-3, 6-3**) **1604.9.3.3.4 Alternate Load Path Analysis**. An alternate load path analysis shall be performed in accordance with sections 1604.9.4.1.8, 1604.9.4.2.4, 1604.9.4.3.1, as applicable.

(**UK 5.2d**) **1604.9.3.3.4.1 Scope**. For the purpose of applying the alternate load path analysis, collapse shall be deemed disproportionate when the removal of any supporting column or beam supporting one or more columns, or any nominal length of load-bearing wall (one at a time in each story of the building) causes the building to become unstable or the floor area at risk of collapse exceeds 15% of the area of that story or 750 square feet whichever is smallest, or extends furthers than the immediate adjacent story.

(**UK 5.3**) **1604.9.3.3.4.2 Key elements**. Where the removal of columns and lengths of walls would result in an extent of damage in excess of the limit established in 1604.9.3.3.3.1, then such elements shall be designed as "key elements" in compliance with Section 1604.9.4.4.

(**UK 5.1e**) **Section 1604.9.3.4 Class 4 buildings**. Class 4 buildings shall comply with the requirements for Class 3 buildings as per section 1604.9.3.3 and a systematic risk assessment of the building shall be undertaken taking into account all the normal hazards that may be reasonably foreseen, together with any abnormal hazard. Critical situations for design shall be selected that reflect the conditions that can reasonably be foreseen as possible during the life of the building.

(**UFC Chapter 6**)**Section 1604.9.4 Building Design Requirements**. The details of the effective anchorage, horizontal and vertical ties, together with the design approaches for checking the integrity of the building following the removal of vertical members and the design of key elements, shall be in accordance with 1604.9.4.1 through 1604.9.4.4:

> (**UK 5.3**) **Section 1604.9.4.1. Structural use of reinforced and unreinforced masonry.** Design against disproportionate collapse for unreinforced masonry construction shall be in accordance with 1604.9.4.1.1 through 1604.9.4.1.8 For internal masonry walls, the distance between lateral supports that are subject to a maximum length shall not exceed 2.25 times the height of the wall. For an external masonry wall, the length shall be measured between vertical lateral supports.

(**UFC Intro**) **1604.9.4.1.1 General.** For composite construction, such as masonry load-bearing walls with other materials for the floor and roof systems, the application of both the requirements of this section and those provided for the other materials are required. The masonry walls shall comply with the tie (vertical, peripheral, and wall) requirements or Alternate Path requirements. Peripheral, internal, and column or wall ties shall be provided at each floor level and at roof level, except where the roof is of lightweight construction, no such ties need be provided at that level. Horizontal ties shall be provided by structural members or by reinforcement that is provided for other purposes.

(**UFC 6-2.4**) **1604.9.4.1.2 Tie Force Design requirements.** Load bearing walls shall be tied from the lowest to the highest level. Reinforcement that is provided for other purposes and shall be regarded as forming part or whole of the required ties. Splices in longitudinal reinforcing bars that provide tie forces shall be lapped, welded or mechanically joined. Splices are not to be located near connections or mid-span. Tie reinforcing bars that provide tie forces at right angle to other reinforcing bars shall used 135 degree hooks with six-diameter, but not less than 3 inches, extension. Use the strength reduction factors φ for development and splices of reinforcement and for anchor bolts as specified in Section 3-1 of Building Code Requirements for Masonry Structures in ACI 530

(**UFC 6-2.5.1**) **1604.9.4.1.3 Internal ties.** Internal ties shall be anchored to peripheral ties at each end, or must continue as wall or column ties. Internal ties shall be straight and continuous through the entire length of the slab, beam or girder. Internal ties can be arranged in accordance with one of the following:

1. Uniformly throughout the floor or roof width, or
2. concentrated, with a 20 foot maximum horizontal tie spacing, or
3. within walls no more than 20 inches above or below the floor or roof and at 20 foot maximum horizontal spacing (in addition to peripheral ties spaced evenly in the perimeter zone).

(**UFC 6-2.5.2**) **1604.9.4.1.3.1 Two-way spans.** For two-way spans the internal ties shall be design to resist a required tie strengths equal to the greater of:

a) $(1.0D + 1.0L)L_aF_t/(8475)$ (Kips/ft)

or

b) $1.0F_t/3.3$ (Kips/ft)

Where: D = Dead load (psf)

L = Live load (psf)

L_a = Lesser of: i) the greatest distance in the direction of the tied between the centers of columns or other vertical load-bearing members where this distance is spanned by a single slab or by a system of beams and slabs, or ii) 5h (ft).

h = Clear story height (ft).

F_t = "Basic Strength" = Lesser of $4.5 + 0.9 N_s$) or 13.5.

N_s = Number of stories including basement(s)

(**UFC 6-2.5.3**) **1604.9.4.1.3.2 One-way spans.** For one-way spans the internal ties shall be design to resist a required tie strengths greater than specified in a) and b) above. In the direction perpendicular to the span, the internal ties shall resist a required tie strength of F_t.

(UFC 6-2.6) **1604.9.4.1.4 Peripheral ties.** Peripheral ties shall have a required tie strength of $1.0F_t$. Peripheral ties shall be 4 feet from the edge of a floor or roof or in the perimeter wall and anchor at re-entrant corners or changes of construction.

(UFC 6-2.7) **1604.9.4.1.5 Horizontal ties to external columns and walls.** Each external column and every 3.33 feet length of external load-bearing wall shall be anchored or tied horizontally into the structure at each floor and roof level with a design tie strength equal to:

$2.0F_t$ or $(h/8.2)F_t$, whichever is smaller (kips)

Where: h = Clear story height (ft)
F_t = "Basic Strength" = Lesser of $(4.5 + 0.9N_s)$ or 13.5
N_s = Number of stories including basement(s)

The tie connection to masonry shall be in accordance with ACI 530. Tie corner columns in both directions. Space wall ties, where required, uniformly along the length of the wall or concentrated at centers not more than 16.5 feet on center and not more than 8.25 feet from the end of the wall. External column and wall ties can be provided partly or wholly by the same reinforcement as peripheral and internal ties.

(UFC 6-2.8) **1604.9.4.1.6 Vertical Ties.** Vertical ties shall be in accordance with this 1604.9.4.1.6.1 through 1604.9.4.1.6.3.

(UFC 6-2.8.1) **1604.9.4.1.6.1 Wall requirements.** Columns and load-bearing walls shall have vertical ties as required by Table 1604.9.4.1.6.1. Vertical ties shall be spaced at a maximum of 16.5 feet on center along the wall, and a maximum of 8.25 feet from any free end of any wall. Vertical ties shall extend from the roof level to the foundation. Vertical ties fully anchored at each end and at each floor level. All joints shall be design to transmit the required tensile forces. The wall shall be constrained between concrete surfaces or other similar construction capable of providing resistance to lateral movement and rotation across the full width of the wall. Vertical ties shall be designed to resist a horizontal tensile force of F_t (kips) per 3.33 feet width.

(UFC 6-2.8.2) **1604.9.4.1.6.2 Columns.** A column or every 3.33 feet length of a load-bearing wall that complies with the minimum requirements of Table C4.2, shall provide a required tie strength equal to:

$6.2 \times 10^{-4} A(h_a/t)^2$ or 22.5 whichever is larger. (kips)

Where: A = Horizontal cross sectional area of the column or wall including piers, but excluding the non-load-bearing wythe, if any of an external wall for cavity construction (ft).
h_a = Clear height of a column or wall between restraining surfaces (ft).
t = Wall thickness or column dimension (ft).

(UFC Table 6-1)

Table 1604.9.4.1.6.1 Minimum Properties for Masonry Walls with Vertical Ties

Property	Requirement
Minimum thickness of a solid wall or one load-bearing wythe of a cavity wall.	6 inches
Minimum characteristic compressive strength of masonry	725 psi
Maximum ratio h_a/t	20
Allowable mortar designations	S, N

(UFC 6-2.9) **1604.9.4.1.6.3 Load-Bearing Walls and Columns with Deficient Vertical Tie Forces.** Load-bearing elements that do not comply with the required vertical tie strength, shall be design in accordance with Section 1604.9.4.1.8, the Alternate Path method. Each deficient element from the structure shall be removed, one at a time, and performed an Alternate Path analysis to verify that the structure can bridge over the missing element. The required number of elements to be removed from the structure is given in Table 1604.9.4.1.6.3.

(UFC Table 6-2)

Table 1604.9.4.1.6.3 Removal of Deficient Masonry Vertical Tie Elements

Vertical Load-bearing Element Type	**Definition of Element**	**Extent of Structure to Remove if Deficient**
Column	Primary structural support member acting alone	Clear height between lateral restraints
Wall Incorporating One or More Lateral Supports[a]	All external and internal load-bearing walls	Length between lateral supports or length between a lateral support and the end of the wall. Remove clear height between lateral restraints.
Wall Without Lateral Supports	All external and internal load-bearing walls	For internal walls: length not exceeding 2.25H, anywhere along the wall where H is the clear height of the wall. For external walls: Full length.

		For both wall types: clear height between lateral restraints.

a. Using the definition of F_t, lateral supports shall be provided by the following:

1) An intersecting or return wall tied to a wall to which it affords support, with connections capable of resisting a force of F_t in $0.45F_t$ in kips per foot height of wall, having a length without openings of net less than H/2 at right angles to the supported wall and having an average weight of not less than 70 psf.
2) A pier or stiffened section of the wall not exceeding 3.33 feet in length, capable of resisting a horizontal force of $0.45F_t$ in kips per foot height of wall.
3) A substantial partition at right angles to the wall having average weight of not less than 31 psf, tied with connections capable of resisting a force of $0.15F_t$ in kips per foot height of wall, and having a length without openings of not less than H at right angles to the supported wall.

(**UFC Appendix E**) **1604.9.4.1.7 Detailed connections for tie forces.** Reinforced masonry connections and joints shall be ductile. Unreinforced masonry connections and joints shall have continuous reinforcement to insure ductile behavior.

(**UFC 6-3**) **1604.9.4.1.8 Alternate Path Method Design requirements.** Alternate path method is used to verify that the structure can bridge over removed elements. The design strengths shall be determined from ACI 530. If the design strengths are less than those in Table 1604.9.4.1.8, then compliance shall be in accordance with the Alternate Path model subsection.

(**UFC Table 6-3**)

Table 1604.9.4.1.8 Acceptability Criteria and Subsequent Action for Masonry

Structural Behavior	**Acceptability Criteria**	**Subsequent Action for Alternate Method Model**
Element Flexure	φM_n[a]	Section 1604.9.4.1.8.1
Element Axial	φP_n[a]	Section 1604.9.4.1.8.2
Element Shear	φVnA	Section 1604.9.4.1.8.3
Connections	Connection Design Strength[a]	Section 1604.9.4.1.8.4
Deformation	Deformation Limits, defined in Table 1604.9.4.1.8.1.8	Section 1604.9.4.1.8.5

a. Nominal strengths are calculated with the appropriate material properties and over-strength factor Ω; all φ factors are defined per Chapter 3 of ACI 530.

(UFC 6-3.1.1) **1604.9.4.1.8.1 Flexural Resistance of Masonry.** The flexural design strength shall be equal to the nominal flexural strength multiplied by the strength reduction factor φ. The nominal flexural strength shall be determined in accordance with ACI 530.

(UFC 6-3.1.1) **1604.9.4.1.8.2 Linear Static Analysis.** An effective plastic hinge shall be added to the model by inserting a discrete hinge into the member at an offset from the member end if the required moment exceeds the flexural design strength and if the reinforcement layout is sufficient for a plastic hinge to form and undergo significant rotation. The location of the hinge is determined through engineering analysis.

(UFC 3-2.4.3) **1604.9.4.1.8.3 Non-linear Static Analysis.** For non-linear static analysis shall be model to represent post-peak flexural behavior. Flexural design strength must develop before shear failure occurs.

(UFC 3-2.4.3) **1604.9.4.1.8.1.4.** The structural element shall be removed when the required moment exceeds the flexural design strength and shall redistributed as per Section 1604.9.4.1.8.1.9, if the structural element is not able to develop a constant moment while undergoing continued deformation.

(UFC 6-3.1.2) **1604.9.4.1.8.1.5 Axial Resistance of Masonry.** The axial design strength with the applicable strength reduction factor φ shall be determined in accordance with Chapter 3 of ACI 530. If the connection exceeds the design strengths of Table 1604.9.4.1.8, remove the connection from the model. If the connections at each end of an element fail, remove the element and redistribute the loads in accordance with Section 1604.9.4.1.8.1.9.

(UFC 6-3.1.3) **1604.9.4.1.8.1.6 Shear Resistance of Masonry.** The shear design strength of the cross-section with the applicable strength reduction factor φ is determined in accordance with ACI 530. If the connection exceeds the design strengths of Table 1604.9.4.1.8, remove the connection from the model. If the connections at each end of an element fail, remove the element and redistribute the loads in accordance with Section 1604.9.4.1.8.1.9.

(UFC 6-3.1.4) **1604.9.4.1.8.1.7 Connections.** The connections design strength with the applicable strength reduction factor φ is determined in accordance with ACI 530. If the connection exceeds the design strengths of Table 1604.9.4.1.8, remove the connection from the model. If the connections at each end of an element fail, remove the element and redistribute the loads in accordance with Section 1604.9.4.1.8.1.9.

(UFC 6-3.2) **1604.9.4.1.8.1.8 Deformation Limits for Masonry.** Deformation limits shall be applied to structural members as per Table 1604.9.4.1.8.1.8.

(UFC Table 6-4)

Table 1604.9.4.1.8.1.8 Deformation Limits for Masonry

	Class 2 and 3 buildings		Class 4 buildings	
Component	**Ductility** υ	**Rotation, Degrees** θ	**Ductility** υ	**Rotation, Degrees** θ
Unreinforced Masonry[a]	-	2	-	1
Reinforced Masonry[b]	-	7	-	2

a. Response of unreinforced masonry walls is also limited by D/t, the maximum member displacement to thickness ratio. This ratio is limited to 0.75. Compare this limit, with the rotation limits and use the most restrictive condition.

b. The ultimate resistance is based on the moment capacity using 90% of F_y for reinforcement.

(UFC 3-2.4.3) **1604.9.4.1.8.1.9 Loads Associated with Failed Elements.** Nonlinear Dynamic, and Linear or Nonlinear Static Analysis shall be in accordance with Section 1604.9.4.1.8.1.9.1 through 1604.9.4.1.8.1.9.3.

(UFC 3-2.4.3) **1604.9.4.1.8.1.9.1 Nonlinear Dynamic.** For a Nonlinear Dynamic analysis, double the loads from the failed element to account for impact and apply them instantaneously to the section of the structure directly below the failed element, before the analysis continues. Apply the loads from the area supported by the failed element to an area equal to or smaller than the area from which they originated.

(UFC 3-2.4.3) **1604.9.4.1.8.1.9.2 Linear or Nonlinear Static Analysis.** For a Linear or Nonlinear Static analysis, if the loads on the failed element are already doubled as shown in Section 1604.9.4.2.4.7.3, then the loads from the failed element are applied to the section of the structure directly below the failed element, before the analysis is re-run or continued. If the loads on the failed element are not doubled, then double them and apply them to the section of the structure directly below the failed element, before the analysis is re-run or continued. In both cases, apply the loads from the area supported by the failed element to an area equal to and smaller than the area from which they originated.

(UFC 3-2.4.2) **1604.9.4.1.8.1.9.3 Linear and Nonlinear Static Analysis Load Case.** Linear and nonlinear static analysis shall have a factored load combination applied to the immediate adjacent bays and at all the floors above the removed element, using the following formula.

$$2.0[(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)] + 0.2W$$

Where: D = Dead load (psf)

L = Live load (psf)

S = Snow load (psf)
W = Wind load (psf)

The adjacent bay for load-bearing wall systems shall be defined as the plan area that spans between the removed wall and the nearest load-bearing wall.

(UFC 6-4) **1604.9.4.1.8.1.10 Loading.** Perimeter ground floor columns and load-bearing walls shall be designed so that the lateral uniform load, which defines the shear capacity, is greater than the load associated with the flexural capacity.

(UFC Chapter 5) **Section 1604.9.4.2. Structural use of steel.** Design against disproportionate collapse for structural steel shall be in accordance with sections 1604.9.4.2.1 through 1604.9.4.2.4.

(UFC Chapter 5) **1604.9.4.2.1 General.** For composite construction, such as concrete deck slabs on steel beams, sheet steel decking with an integral slab, and columns reinforced with structural steel shapes, the application of both the requirements of this section and those provided for reinforced concrete in ACI 318-02 are required. For a concrete deck slab on steel beam in which the slab is used to provide internal tie capacity, the floor system and roof system shall comply the internal tie requirements of ACI 318-02, while the steel frame shall comply the other tie requirements (vertical, peripheral, and external column) and the Alternate Path requirements of this section.

(UFC 5.1) **1604.9.4.2.2 Material Properties.** The over-strength factor specified in Table 1604.9.4.2.2 shall be applied to calculations of the design strength for both Tie Forces and Alternate Path method.

(UFC Table 5-1)

Table 1604.9.4.2.2 Over-Strength Factors for Structural Steel

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

Plates	1.05	1.1
All other products	1.05	1.1

(UFC 5-2.1) **1604.9.4.2.3 Steel Tie Force Requirements.** All buildings shall be effectively tied together at each principal floor level. Each column shall 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 shall similarly be provided at the roof level, except where the steelwork only supports cladding that weighs not more than 14.6 psf and that carries only imposed roof loads and wind loads. Ties shall be effectively straight. Arrange continuous lines of ties as close as practical to the edges of the floor or roof and to each column line. At re-entrant corners, anchor the tie members nearest to the edge into the steel framework.

(UFC 5-2.2) **1604.9.4.2.3.1. Strength Reduction Factor Φ for Steel Tie Forces**. For the steel members and connections that provide the design tie strengths, use the applicable 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 (AISC LRFD.

(UFC 5-2.3) **1604.9.4.2.3.2. 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 in accordance with ACI 318-02 and meeting the continuity and anchorage requirements of Sections 1604.9.4.2.3.2.1.

(UFC 4-2.4) **1604.9.4.2.3.2.1. Continuity and Anchorage of Ties**. Ties shall comply with Section 1604.9.4.2.3.2.1.1 through 1604.9.4.2.3.2.1.2.

(UFC 4-2.4) **1604.9.4.2.3.2.1.1.** Splices in longitudinal steel reinforcement used to provide the design tie strength shall be lapped, welded or mechanically joined with Type 1 or Type 2 mechanical splices, in accordance with ACI 318-02. Locate splices away from joints or regions of high stress and shall be staggered.

(ACI 21.5.4) **1604.9.4.2.3.2.1.2.** Use seismic hooks, as defined in Chapter 21 of ACI 318-02, and seismic development lengths, as specified in Section 21.5.4 of ACI 318-02, to anchor ties to other ties. At re-entrant corners or at substantial changes in construction, ties shall be adequately developed.

(UFC 5-2.4) **1604.9.4.2.3.3. Internal Ties**. Design steel members acting as internal ties and their end connections shall be capable of resisting the following required tie strength, which need not be considered as additive to other loads.

The required tie strength is calculated as follows:

$0.5(1.2D + 1.6L)s_tL_l$ but not less than 16.9 kips

Where:
- D = Dead load (psf)
- L = Live load (psf)
- L_l = Span (ft.)
- s_t = Mean transverse spacing of the ties adjacent to the ties being checked (ft.)

(UFC 5-2.5) **1604.9.4.2.3.4 Peripheral Ties.** Peripheral ties shall be capable of resisting the following load:

$0.25(1.2D + 1.6L)s_tL_l$ but not less than 8.4 kips

Where:
- D = Dead load (psf)
- L = Live load (psf)
- L_l = Span (ft.)
- s_t = Mean transverse spacing of the ties adjacent to the ties being checked (ft.)

(UFC 5-2.6) **1604.9.4.2.3.5 Tying of External Columns.** The required tie strength for horizontal ties anchoring the column nearest to the edges of a floor or roof and acting perpendicular to the edge is equal to the greater of the load calculated in Section 1604.9.4.2.3.3 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.

(UFC 5-2.7) **1604.9.4.2.3.6 Vertical Ties.** All columns shall be continuous through each beam-to-column connection. All column splices shall provide a design tie strength equal to the largest factored vertical dead and live load reaction (from all load combinations used in the design) applied to the column at any single floor level located between that column splice and the next column splice down or the base of the column.

(UFC 5-2.8) **1604.9.4.2.3.7 Columns with Deficient Vertical Tie Forces.** The Alternate Path method shall be used in each deficient column, where it is not possible to provide the vertical required tie strength. Remove each deficient column from the structure, one at a time, and perform an Alternate Path analysis to verify that the structure can bridge over the missing column.

(UFC 5-3 and 5-3.1) **1604.9.4.2.4 Alternate Path Method Design requirements.** Alternate path method is used to verify that the structure can bridge over removed elements. The design strengths shall be determined in accordance with AISC LRFD. If the design strengths are less than those in Table 1604.9.4.2.4.1, then compliance shall be in accordance with the Alternate Path model subsection.

(UFC Table 5-2)

Table 1604.9.4.2.4.1. Acceptability Criteria and Subsequent Action for Structural Steel

Structural Behavior	**Acceptability Criteria**	**Subsequent Action for Violation of Criteria**
Element Flexure	φM_n [a]	Section 1604.9.4.2.4.1
Element Combined Axial and Bending	AISC LRFD Chapter H Interaction Equations[a]	Section 1604.9.4.2.4.2
Element Shear	φV_n [a]	Section 1604.9.4.2.4.3
Connections	Connection Design Strength[a]	Section 1604.9.4.2.4.4
Deformation	Deformation Limits, defined in Table 1604.9.4.2.5(1)	Section 1604.9.4.2.4.5

a. Nominal strengths are calculated with the appropriate material properties and over-strength factors Ω_y and Ω_u depending upon the limit state; all Φ factors are defined per AISC LRFD.

(UFC 5-3.1.1) **1604.9.4.2.4.1. Flexural Resistance of Structural 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. 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. Distribute the loads associated with the element in accordance with Section 1604.9.4.2.4.7. 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.

Verify that deformation of primary members will not cause premature failure in secondary members, due to geometric interference; for instance, torsional rotation of a girder should not cause excessive deformation and stresses in any beam that frames into the girder with a simple shear tab connection.

(UFC 5-3.1.1.1) **1604.9.4.2.4.1.1 Formation of Plastic Hinge.** If hinge formation, i.e. material non-linearity, is included in the Alternate Path analysis, the requirements of Section A5.1 of the AISC LRFD for plastic design shall be met. AISC LRFD 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, 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. 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 shall consider the influence of axial or shear force when appropriate. Further information on plastic design is provided in The Plastic Methods of Structural Analysis (Neal 1963) and Plastic Design of Steel Frames (Beedle 1958).

(UFC 5-3.1.1.2) **1604.9.4.2.4.1.2. Modeling of a Plastic Hinge**. Plastic hinges shall be modeled as per Sections 1604.9.4.2.4.1.2.1 through 1604.9.4.2.4.1.2.2.

(UFC 5-3.1.1.2) **1604.9.4.2.4.1.2.1. Linear Static Analysis.** For Linear Static analyses, if the calculated moment exceeds the nominal moment strength and it is determined that the element is capable of forming a plastic hinge, insert an "equivalent" plastic hinge into the model by inserting 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. Determine the location of the plastic hinge through engineering analysis and judgment or with the guidance provided for seismic connections in FEMA 350, Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings and AISC 341, Seismic Provisions for Structural Steel Buildings.

(UFC 5-3.1.1.2) **1604.9.4.2.4.1.2.2. Nonlinear Static and Dynamic Analysis.** For Nonlinear Static and Dynamic Analysis, use software capable of representing post-peak flexural behavior and considering interaction effects of axial loads and moment. Ensure that shear failure will not occur prior to developing the full flexural design strength.

(UFC 5-3.1.2) **1604.9.4.2.4.2. 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). The response is determined by the magnitude of the axial force, cross sectional properties, magnitude/direction of moments, and the slenderness of the element. 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 $P_y = A_gF_y$) 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 1604.9.4.2.4.1. 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.

1. 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, 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 1604.9.4.2.4.6.

2. 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. 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). 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 1604.9.4.2.5(1).

(UFC 5-3.1.3) **1604.9.4.2.4.3 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, multiplied by the strength reduction factor Φ and the over-strength factor Ω. If the element exceeds the design strengths of Table 1604.9.4.2.4.1, remove the element and redistribute the loads associated with the element per Section 1604.9.4.2.4.6.

(UFC 5-3.1.4) **1604.9.4.2.4.4. Connections.** All connections shall meet the requirements of AISC LRFD; employ the applicable strength reduction factor Φ for each limit state and over-strength factor Ω. As detailed in AISC LRFD, consider multiple limit states for the connections. If a connection exceeds the design strengths of Table 1604.9.4.2.4.1, remove it from the model. If the connections at each end of an element fail, remove the element and redistribute the loads associated with the element per Section 1604.9.4.2.4.6.

(UFC 5-3.2) **1604.9.4.2.4.5. Deformation Limits for Structural Steel.** The Deformation Limits are given in Table 1604.9.4.2.5(1). Fully Restrained and Partially Restrained connections are given in Table 1604.9.4.2.5(2). Note that testing in accordance with Appendix S of AISC 341 can be used to verify and quantify the rotational capacities of connections that are not listed in Table 1604.9.4.2.5(2).

(UFC 3-2.4.3) **1604.9.4.2.4.6 Loads Associated with Failed Elements.** Nonlinear Dynamic, and Linear or Nonlinear Static Analysis shall be in accordance with Section 1604.9.4.2.4.6.1through 1604.9.4.2.4.6.2.

(UFC 3-2.4.3) **1604.9.4.2.4.6.1. Nonlinear Dynamic.** For a Nonlinear Dynamic analysis, double the loads from the failed element to account for impact and apply them instantaneously to the section of the structure directly below the failed element, before the analysis continues. Apply the loads from the area supported by the failed element to an area equal to or smaller than the area from which they originated.

(UFC 3-2.4.3) **1604.9.4.2.4.6.2 Linear or Nonlinear Static Analysis.** For a Linear or Nonlinear Static analysis, if the loads on the failed element are already doubled as shown in Section 1604.9.4.2.4.6.3, then the loads from the failed element are applied to the section of the structure directly below the failed element before the analysis is re-run or continued. If the loads on the failed element are not doubled, then double them and apply them to the section of the structure directly below the failed element, before the analysis is re-run or continued. In both cases, apply the loads from the area supported by the failed element to an area equal to or smaller than the area from which they originated.

(UFC 3-2.4.2) **1604.9.4.2.4.6.3 Linear and Nonlinear Static Analysis Load Case.** Linear and nonlinear static analysis shall have a factored load combination applied to the immediate adjacent bays and at all the floors above the removed element, using the following formula.

$$2.0[(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)] + 0.2W$$

Where: D = Dead load (psf)

L = Live load (psf)
S = Snow load (psf)
W = Wind load (psf)

(UFC Table 5-3)

Table 1604.9.4.2.5(1) Deformation Limits for Structural Steel

	Class 2 and 3 buildings		**Class 4 buildings**	
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)		2.0		1.5
Reduced Beam Section		2.6		2
Steel Frame Connections; Partially Restrained				

Limit State governed by rivet shear or flexural yielding of plate, angle or T-section		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		1.3		0.9

a. As defined in AISC 341.

(UFC Table B-1)

Table 1604.9.4.2.5(2) Steel Moment Frame Connection Types

Connection	**Description**	**Type**
Strong Axis		
Welded Unreinforced Flange	Full penetration welds between beams and columns, flanges, bolted or welded web.	FR
Welded Flange Plates	Flange plate with full-penetration weld at column and fillet welded to beam flange.	FR
Welded Cover-Plated Flanges	Beam flange and cover-plate are welded to column flange.	FR
Bolted Flanges Plates	Flange plate with full-penetration weld at column and field bolted to beam flange.	FR or PR
Improved Welded Unreinforced Flange – Bolted Web	Full-penetration welds between beam and column flanges, bolted web.	FR
Improved Welded Unreinforced Flange – Welded Web	Full-penetration welds between beam and column flanges, welded web.	FR
Free Flange	Web is coped at ends of beam to separate flanges; welded web tap resists shear and bending moment due to eccentricity due to coped web.	FR
Welded Top and Bottom Haunches	Haunched connection at top and bottom flanges.	FR
Reduced Beam Section	Connection in which net area of beam flange is reduced to force plastic hinging away from column face.	FR
Top and Bottom Clip Angles	Clip angle bolted or riveted to beam flange and column flange.	PR

Double Split Tee	Split tees bolted or riveted to beam flange and column flange.	PR
Composite Top and Clip Angle Bottom	Clip angle bolted or riveted to column flange and beam bottom flange with composite slab.	PR
Bolted Flange Plates	Flange plate with full-penetration weld at column and bolted to beam flange.	PR
Bolted End Plates	Stiffened or unstiffened end plate welded to beam and bolted to column flange.	PR
Shear Connection with or without Slab	Simple connection with shear tab, may have composite slab.	PR
Weak Axis		
Fully Restrained	Full-penetration welds between beams and columns, flanges, bolted or welded web.	FR
Shear Connection	Simple connection with shear tab.	PR

Note: PR = Partially Restrained Connections
FR = Fully Restrained Connections

(**ACI Section 7.13**) **Section 1604.9.4.3. Structural use of plain, reinforced and prestressed concrete.** Design against disproportionate collapse for concrete shall be in accordance with American Concrete Institute (ACI) 318 or 1604.9.4.3.1. For a reinforced concrete wall, the distance between lateral supports that are subject to a maximum length shall not exceed 2.25 times the height of the wall. For composite construction, such as concrete deck slabs on steel beams, sheet steel decking with an integral slab, and columns reinforced with structural steel shapes, the application of both the requirements of this section and those provided for structural steel in Section 1604.9.4.2 are required. For a concrete deck slab on steel beam in which the slab is used to provide internal tie capacity, the floor system and roof system shall comply the internal tie requirements of ACI 318, while the steel frame shall comply the other tie requirements (vertical, peripheral, and external column).

(**UFC 4-3**) **1604.9.4.3.1 Alternate Path Method Design requirements.** Alternate path method is used to verify that the structure can bridge over removed elements. The design strengths shall be determined in accordance with ACI 318. If the design strengths are less than those in Table 1604.9.4.3.1, then compliance shall be in accordance with the Alternate Path model subsection.

(**UFC Table 4-3**)

Table 1604.9.4.3.1 Acceptability Criteria and Subsequent Action for Reinforced Concrete

Structural Behavior	**Acceptability Criteria**	**Subsequent Action for Violation of Criteria**
Element Flexure	φM_n[a]	Section 1604.9.4.3.1.2
Element Combined Axial and Bending	ACI 318 Chapter 10 Provisions[a]	Section 1604.9.4.3.1.3
Element Shear	φV_n[a]	Section 1604.9.4.3.1.4
Connections	Connection Design Strength[a]	Section 1604.9.4.3.1.5
Deformation	Deformation Limits, defined in Table 1604.9.4.3.1.6	Section 1604.9.4.3.1.6

Nominal strengths are calculated with the appropriate material properties and over-strength factors Ω_y and Ω_u depending upon the limit state; all Φ factors are defined per ACI 318.

(UFC 4-1) **1604.9.4.3.1.1 Over-Strength Factors for Reinforced Concrete.** The applicable over-strength factor shall be applied to calculations of the design strength Alternate Path method. The over-strength factors are given in Table 1604.9.4.3.1.1.

(**UFC Table 4-1**)

Table 1604.9.4.3.1.1 Over-Strength Factors for Reinforced Concrete

Reinforced Concrete	**Over-Strength Factor, Ω**
Concrete Compressive Strength	1.25
Reinforcing Steel (ultimate and yield strength)	1.25

(UFC 4-3.1.1) **1604.9.4.3.1.2 Flexural Resistance of Reinforced Concrete.** The flexural design strength shall be equal to the nominal flexural strength calculated with the appropriate material properties and over-strength factor Ω, multiplied by the strength reduction factor φ of 0.75. The nominal flexural strength shall be calculated in accordance with ACI 318.

(UFC 4-3.1.1) **1604.9.4.3.1.2.1 Linear Static Analysis.** For linear static analysis when the required moment exceeds the flexural design strength and if the reinforcement layout is sufficient for a plastic hinge to form and undergo significant rotation, add an equivalent plastic hinge to the model, by inserting a discrete hinge at the correct location within the member. The location of the hinge shall be determined through engineering analysis, but shall be less than ½ the depth of the member from the face of the column. Apply two constant moments, one at each side of the new hinge, in the appropriate direction of the acting moment.

(UFC 4-3.1.1) **1604.9.4.3.1.2.2 Non-linear Static and Dynamic Analysis.** For non-linear static and dynamic analysis shall be model to represent post-peak flexural behavior. Flexural design strength must develop before shear failure occurs.

(**UFC 4-3.1.1**) **1604.9.4.3.1.2.3.** The structural element shall be removed when the required moment exceeds the flexural design strength and shall redistributed as per Section 1604.9.4.3.2, if the structural element is not able to develop a constant moment while undergoing continued deformation.

(**UFC 4-3.1.1**) **1604.9.4.3.1.2.4.** The structural element shall be removed when the required moment exceeds the flexural design strength and shall redistributed as per Section 1604.9.4.3.2, if the structural element is not able to develop a constant moment while undergoing continued deformation.

(**UFC 4-3.1.2**) **1604.9.4.3.1.3 Combined Axial and Bending Resistance of Reinforced Concrete.** The acceptability criteria for elements undergoing combined axial and bending loads are based on the provisions given in Chapter 10 of ACI 318, including the appropriate strength reduction factor Φ and the over-strength factor Ω. If the combination of axial load and flexure in an element exceeds the design strength and the un-factored axial load is greater than the nominal axial load strength at balanced strain P_b, remove the element and redistribute the loads associated with the element per Section 1604.9.4.3.2. If the un-factored axial load is less than P_b, then insert an equivalent plastic hinge into the column, per the procedure discussed in Section 1604.9.4.3.3.1.

(**UFC 4-3.1.3**) **1604.9.4.3.1.4 Shear Resistance of Reinforced Concrete.** The acceptability criteria for shear are based on the shear design strength of the cross-section, per ACI 318, using the appropriate 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 1604.9.4.3.2.

(**UFC 4-3.1.4**) **1604.9.4.3.1.5 Connections.** The connections design strength with the applicable strength reduction factor φ shall be determined in accordance with ACI 318. The effects of embedment length, reinforcement continuity, and confinement of reinforcement in the joint shall be considered when determining the joint design strength. If the connection exceeds the design strengths of Table 1604.9.4.3.1, remove it from the model. If the connections at each end of an element fail, remove the element and redistribute the loads associated with the element in accordance with section 1604.9.4.3.2 .

(**UFC 4-3.2**) **1604.9.4.3.1.6 Deformation Limits for Reinforced Concrete.** If the element or the connections at each end of an element exceed the a deformation limit in Table 1604.9.4.3.1.6, remove the element and redistribute the loads associated with the element in accordance with section 1604.9.4.3.2**.** Deformation limits are only applied to the structural elements, not to the connections.

(UFC Table 4-4)

Table 1604.9. 4.3.1.6 Deformation Limits for Reinforced Concrete

	Class 2 & 3 Buildings		Class 4 Buildings	
Component	**Ductility** υ	**Rotation, Degrees** θ	**Ductility** υ	**Rotation, Degrees** θ
Slab and Beam Without Tension Membrame[a]				
Single-Reinforced or Double-Reinforced without Shear Reinforcing[b]	-	3	-	2
Double-Reinforced with Shear Reinforcing[c]	-	6	-	4
Slab and Beam with Tension Membrame[a]				
Normal Proportions ($L/h \geq 5$)	-	20	-	12
Deep Proportions ($L/h < 5$)	-	12	-	8
Compression Members				
Walls and Seismic Columns[d,e]	3	-	2	-
Non-Seismic Columns[e]	1	-	0.9	-

[a] The tension membrane effect is an extension of the yield line theory of slabs and it increases the ultimate resistance. It cannot be developed when the slab has a free edge.

[b] Single-reinforced members have flexural bars in one face or mid-depth only. Double-reinforced members have flexural reinforcing in both faces.

[c] Stirrups or ties meeting ACI 318 minimums must enclose the flexural bars in both faces, otherwise use the response limits for Double-Reinforced without shear reinforcing.

[d] Seismic columns have ties or spirals in accordance with ACI 318 Chapter 21 seismic design provisions for special moment frames.

[e] Ductility of compression members is the ratio of total axial shortening to axial shortening at the elastic limit.

(UFC 3-2.4.3) **1604.9.4.3.2 Loads Associated with Failed Elements.** The following procedure shall be met for Nonlinear Dynamic, and Linear or Nonlinear Static Analysis.

(UFC 3-2.4.3) **1604.9.4.3.2.1 Nonlinear Dynamic.** For a Nonlinear Dynamic analysis, double the loads from the failed element to account for impact and apply them instantaneously to the section of the structure directly below the failed element, before the analysis continues. Apply the loads from the area supported by the failed element to an area equal to or smaller than the area from which they originated.

(UFC 3-2.4.3) **1604.9.4.3.2.2 Linear or Nonlinear Static Analysis.** For a Linear or Nonlinear Static analysis, if the loads on the failed element are already doubled as shown in Section 1604.9.4.2.4.7.3, then the loads from the failed element are applied to the section of the structure directly below the failed element, before the analysis is re-run or continued. If the loads on the failed element are not doubled, then double them and apply them to the section of the structure directly below the failed element, before the analysis is re-run or continued. In both cases, apply the loads from the area supported by the failed element to an area equal to and smaller than the area from which they originated.

(UFC 3-2.4.2) **1604.9.4.3.2.3 Linear and Nonlinear Static Analysis Load Case.** Linear and nonlinear static analysis shall have a factored load combination applied to the immediate adjacent bays and at all the floors above the removed element, using the following formula.

2.0[(0.9 or 1.2)D + (0.5L or 0.2S)] + 0.2W

Where: D = Dead load (psf)

L = Live load (psf)
S = Snow load (psf)
W = Wind load (psf)

Section 1604.9.4.4 Key Elements analysis. When applying the Alternate Path Method Design requirements from sections 1604.9.4.1.8, 1604.9.4.2.4 or 1604.9.4.3.1 and the removal of columns and lengths of walls result in a disproportionate collapse, then such element shall be designed as a key element.

(ASCE C2.5) **Section 1604.9.4.4.1 Load Combinations.** The following load combinations shall be used in addition to the accidental design loading in the key element analysis:

$$1.2D + A_k + (0.5L \text{ or } 0.2S)$$

$$(0.9 \text{ or } 1.2)D + A_k + 0.2W$$

As per the definition of key element, A_k = 700 psf.

(UK 5.4) **Section 1604.9.5 Alternate Approach.** In lieu of compliance with Sections 1604.9.4 and 1604.9.5, an alternative design approach may be used. The alternative design approach shall be approved upon a finding by the building official that the proposed design is satisfactory and complies with the intent of the provisions of this section and that the design is at least the equivalent of that prescribed herein in quality, strength, effectiveness, durability and safety.