

1604.11 Disproportionate collapse. Design for structural integrity to protect against disproportionate collapse shall be in accordance with Section 1605.

1605 DISPROPORTIONATE COLLAPSE

1605.1 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.

1605.2 DEFINITIONS.

ALTERNATE LOAD PATH METHOD. A design approach that assumes that a local failure occurs, but demonstrates an alternate load path so that damage is absorbed and spread of collapse arrested.

DISPROPORTIONATE COLLAPSE. The spread of damage from an initiating event from element to element resulting in the collapse of an entire structure or a disproportionately large portion of it.

LOAD-BEARING CONSTRUCTION. Load-bearing construction shall include masonry cross-wall construction and walls of lightweight steel Section studs.

KEY ELEMENT. A structural element essential to the integrity and stability of the structure that resists abnormal loading without failure.

STRUCTURAL FRAME. The columns and other structural members including the girders, beams, trusses, and spandrels having direct connections to the columns and bracing members designed to carry gravity loads, together with their connections.

TIES. Structural elements that mechanically connect the building components to enhance continuity, ductility and redundancy.

1605.3 Building class. Buildings shall be classified in accordance with Table 1605.3. Buildings with occupancy groups within more than one classification shall be designed as the higher class.

**TABLE 1605.3
BUILDING CLASS**

CLASS	BUILDING TYPE AND OCCUPANCY
<u>1</u>	<u>Group I-1, R-3 or R-4 not exceeding 4 stories</u> <u>Agricultural buildings</u> <u>Unoccupied buildings that are separated from other buildings by a distance of 1.5 times the buildings height.</u> <u>Group E not exceeding 2 stories</u>
<u>2</u>	<u>Group I-3</u> <u>Group R-3 not exceeding 5 stories</u> <u>Group R-1 not exceeding 4 stories</u> <u>Group R-2 not exceeding 4 stories</u> <u>Group B not exceeding 4 stories</u> <u>Group F not exceeding 3 stories</u> <u>Group M not exceeding 3 stories of less than 21,500 square feet floor area in each story.</u> <u>All buildings of Group A not exceeding 2 stories which contain floor areas not exceeding 21,500 square feet at each story.</u> <u>Group S buildings not exceeding 6 stories</u>
<u>3</u>	<u>Group R-1 and R-2 buildings greater than 4 stories but not exceeding 15 stories</u> <u>Group E buildings greater than 2 stories but not exceeding 15 stories</u> <u>Group M buildings greater than 3 stories but not exceeding 15 stories.</u> <u>Group I-2 buildings not exceeding 3 stories.</u> <u>Group B buildings greater than 4 stories but not exceeding 15 stories.</u> <u>Group A buildings which contain floors of more than 21,500 square feet but less than 54,000 square feet per floor.</u>
<u>4.</u>	<u>All buildings that exceed the limits on area or number of stories for class 1-3.</u> <u>Grandstands accommodating more than 5000 spectators.</u> <u>Building containing hazardous substances and/or processes – Groups H-1, H-2, H-3, H-4, and H-5.</u>

1605.4 Performance design approach: Design to protect against disproportionate collapse shall be in accordance with Section 1605.5. Alternative design approaches may be used provided that it is demonstrated that the alternative(s) chosen result in a level of structural robustness at least equivalent to that specified in Section 1605.5. For all collapse resistance approaches, verification of acceptable damage to the remaining structure outside of the collapse extent shall be determined by an analysis that allows a comparison of residual inelastic capacity to initial capacity (or a similar metric.) In every case, post-event stability of the structural system shall be verified.

1605.5 Prescriptive design approach. Design of new buildings to protect against disproportionate collapse shall be in accordance with the requirements specified below for each building class.

1605.5.1 Class 1 buildings. Class 1 buildings are not required to comply with this section.

1605.5.2 Class 2 buildings. Class 2 buildings shall be provided with horizontal ties in accordance with Section 1605.5.2.1 or with anchorage in accordance with Section 1605.5.2.2.

1605.5.2.1 Class 2 horizontal ties. Horizontal ties shall be provided in accordance with Sections 1605.6.1, 1605.6.2, and 1605.6.3, as applicable.

1605.5.2.2 Class 2 anchorage. Anchorage of suspended floors to walls shall be provided in accordance with Sections 1605.6.1, 1605.6.2, and 1605.6.3, as applicable, for load-bearing construction.

1605.5.3 Class 3 buildings. Class 3 buildings shall be provided with horizontal ties, in accordance with Section 1605.5.3.1, anchorage in accordance with Section 1605.5.3.2, and vertical ties in accordance with Section 1605.5.3.3 or shall be designed utilizing alternate load path analysis in accordance with Section 1605.5.3.4.

1605.5.3.1 Class 3 horizontal ties. Horizontal ties shall be provided in accordance with Sections 1605.6.1, 1605.6.2, and 1605.6.3, as applicable.

FOUO -- DCA Progressive Collapse Design Methodology -- FOUO

1605.5.3.2 Class 3 anchorage. Anchorage of suspended floors to walls shall be provided in accordance with Sections 1605.6.1, 1605.6.2, and 1605.6.3, as applicable, for load-bearing construction.

1605.5.3.3 Class 3 vertical ties. Vertical ties shall be provided in accordance with Sections 1605.6.1, 1605.6.2, and 1605.6.3, as applicable.

1605.5.3.4 Class 3 alternate load path analysis. An alternate load path analysis shall be performed in accordance with Sections 1605.6.1.8, 1605.6.2.4, 1605.6.3.1, as applicable.

1605.5.3.4.1 Class 3 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 smaller, or extends further than the immediate adjacent story.

1605.5.3.4.2 Class 3 key element analysis. Where the removal of columns and lengths of walls would result in an extent of damage in excess of the limit established in 1605.5.3.4.1, then such elements shall be designed as “key elements” in compliance with Section 1605.6.4.

1605.5.4 Class 4 buildings. Class 4 buildings shall comply with the requirements for Class 3 buildings in accordance with Section 1605.5.3 and a systematic risk assessment of the building shall be undertaken. Critical situations identified by the risk assessment shall be accounted for in the design. A peer review of the risk assessment and of the design shall be submitted.

1605.6 Prescriptive 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 Section 1605.6.1 through Section 1605.6.4:

1605.6.1 Structural use of reinforced and unreinforced masonry. Design to protect against disproportionate collapse for unreinforced masonry construction shall be in accordance with 1605.6.1.1 through 1605.6.1.8 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.

1605.6.1.1 Masonry 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 load 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.

1605.6.1.2 Masonry 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 in accordance ACI 318. Splices are not to be located near connections or mid-span. Tie reinforcing bars that provide tie forces at right angles to other reinforcing bars shall use 135 degree hooks with six-diameter extension, but not less than 3 inch, extension. Use the strength reduction factors ϕ for development and splices of reinforcement and for anchor bolts as specified in Section 3-1 of ACI 530

1605.6.1.3 Masonry 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).

FOUO -- DCA Progressive Collapse Design Methodology -- FOUO

1605.6.1.3.1 Masonry two-way spans. For two-way spans the internal ties shall be design to resist a required tie strengths equal to the greater of:

1. $(1.0D + 1.0L)L_a F_t / (8475)$ (Kips/ft)
or
2. $1.0F_t / 3.3$ (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)

1605.6.1.3.2 Masonry one-way spans. For one-way spans the internal ties shall be designed to resist a required tie strengths greater than specified in Section 1605.6.1.3.1. In the direction perpendicular to the span, the internal ties shall resist a required tie strength of F_t .

1605.6.1.4 Masonry 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.

1605.6.1.5 Masonry 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:

$$2.0F_t \text{ or } (h/8.2)F_t, \text{ whichever is smaller (kips)}$$

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.

1605.6.1.6 Masonry vertical ties. Vertical ties shall be in accordance with this 1605.6.1.6.1 through 1605.6.1.6.3.

1605.6.1.6.1 Masonry wall requirements. Columns and load-bearing walls shall have vertical ties as required by Table 1605.6.1.6.1. 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 shall be 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.

1605.6.1.6.2 Masonry columns. A column or every 3.33 feet length of a load-bearing wall that complies with the minimum requirements of Table 1605.6.1.6.1, shall provide a required tie strength equal to:

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

Where:

A = Horizontal cross sectional area of the column or wall including piers, but excluding the non-load-bearing width, 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).

**TABLE 1605.6.1.6.1
MINIMUM PROPERTIES FOR MASONRY WALLS WITH VERTICAL TIES**

<u>PROPERTY</u>	<u>REQUIREMENTS</u>
<u>Minimum thickness of a solid wall or one load-bearing wythe of a cavity wall.</u>	<u>6 inches</u>
<u>Minimum characteristic compressive strength of masonry</u>	<u>725 psi</u>
<u>Maximum ratio h_p/t</u>	<u>20</u>
<u>Allowable mortar designations</u>	<u>S, N</u>

1605.6.1.6.3 Masonry 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 designed in accordance with Section 1605.6.1.8, the alternate load path method. Each deficient element from the structure shall be removed, one at a time, and an alternate load path analysis shall be performed 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 1605.6.1.6.3.

**TABLE 1605.6.1.6.3
REMOVAL OF DEFICIENT MASONRY VERTICAL TIE ELEMENTS**

<u>VERTICAL LOAD-BEARING ELEMENT TYPE</u>	<u>DEFINITION OF ELEMENT</u>	<u>EXTENT OF STRUCTURE TO REMOVE IF DEFICIENT</u>
<u>Column</u>	<u>Primary structural support member acting alone</u>	<u>Clear height between lateral restraints</u>
<u>Wall Incorporating One or More Lateral Supports^a</u>	<u>All external and internal load-bearing walls</u>	<u>Length between lateral supports or length between a lateral support and the end of the wall.</u> <u>Remove clear height between lateral restraints.</u>
<u>Wall Without Lateral Supports</u>	<u>All external and internal load-bearing walls</u>	<u>For internal walls: length not exceeding 2.25H, anywhere along the wall where H is the clear height of the wall.</u> <u>For external walls: Full length.</u> <u>For both wall types: clear height between lateral restraints.</u>

a. 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.

1605.6.1.7 Masonry detailed connections for tie forces. Reinforced masonry connections and joints shall be ductile. Unreinforced masonry connections and joints shall have continuous reinforcement to ensure ductile behavior.

1605.6.1.8 Masonry alternate load path method design requirements. Alternate load 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 1605.6.1.8, then compliance shall be in accordance with the alternate load path Section 1605.6.1.8.1 through 1605.6.1.8.8.

**TABLE 1605.6.1.8
ACCEPTABILITY CRITERIA AND SUBSEQUENT ACTION FOR MASONRY**

<u>STRUCTURAL BEHAVIOR</u>	<u>ACCEPTABILITY CRITERIA</u>	<u>SUBSEQUENT ACTION FOR ALTERNATE METHOD MODEL</u>
Element Flexure	ϕM_n^a	Section 1605.6.1.8.1
Element Axial	ϕP_n^a	Section 1605.6.1.8.2
Element Shear	$\phi V_n A$	Section 1605.6.1.8.3
Connections	Connection Design Strength ^a	Section 1605.6.1.8.4
Deformation	Deformation Limits, defined in Table 1605.6.1.8.1.8	Section 1605.6.1.8.5

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.

1605.6.1.8.1 Masonry 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.

1605.6.1.8.2 Masonry 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.

1605.6.1.8.3 Masonry non-linear static analysis. Non-linear static analysis shall be modeled to represent post-peak flexural behavior. Flexural design strength must develop before shear failure occurs.

1605.6.1.8.4 Flexural design strength. The structural element shall be removed when the required moment exceeds the flexural design strength and shall be redistributed in accordance with Section 1605.6.1.8.1.9, if the structural element is not able to develop a constant moment while undergoing continued deformation.

1605.6.1.8.5 Masonry 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 1605.6.1.8, 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 1605.6.1.8.1.9.

1605.6.1.8.6 Masonry 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 1605.6.1.8, 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 1605.6.1.8.1.9.

1605.6.1.8.7 Masonry 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 1605.6.1.8, 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 1605.6.1.8.1.9.

1605.6.1.8.8 Masonry deformation limits for masonry. Deformation limits shall be applied to structural members in accordance with Table 1605.6.1.8.1.8.

**TABLE 1605.6.1.8.1.8
DEFORMATION LIMITS FOR MASONRY**

<u>Component</u>	<u>CLASS 2 AND 3 BUILDINGS</u>		<u>CLASS 4 BUILDINGS</u>	
	<u>Ductility</u> <u>\underline{v}</u>	<u>Rotation, Degrees</u> <u>$\underline{\theta}$</u>	<u>Ductility</u> <u>\underline{v}</u>	<u>Rotation, Degrees</u> <u>$\underline{\theta}$</u>
Unreinforced Masonry ^a	-	<u>2</u>	-	<u>1</u>
Reinforced Masonry ^b	-	<u>7</u>	-	<u>2</u>

- 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.

1605.6.1.8.9 Masonry loads associated with failed elements. Nonlinear Dynamic, and Linear or Nonlinear Static Analysis shall be in accordance with Section 1605.6.1.8.1.9.1 through 1605.6.1.8.1.9.3.

1605.6.1.8.9.1 Masonry 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.

1605.6.1.8.9.2 Masonry 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 1605.6.1.8.9.3, 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.

1605.6.1.8.9.3 Masonry 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.

1605.6.1.8.10 Masonry 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.

1605.6.2 Structural use of steel. Design against disproportionate collapse for structural steel shall be in accordance with Sections 1605.6.2.1 through 1605.6.2.4.

1605.6.2.1 Steel 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 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 with the internal tie requirements of ACI 318, while the steel frame shall comply with the other tie requirements (vertical, peripheral, and external column) and the alternate load path requirements of this section.

1605.6.2.2 Steel material properties. The over-strength factor specified in Table 1605.6.2.2 shall be applied to calculations of the design strength for both tie forces and alternate load path method.

TABLE 1605.6.2.2
OVER-STRENGTH FACTORS FOR STRUCTURAL STEEL

<u>STRUCTURAL STEEL</u>	<u>ULTIMATE OVER-STRENGTH FACTOR, Ω_u</u>	<u>YIELD OVER-STRENGTH FACTOR, Ω_v</u>
Hot-Rolled Structural Shapes and Bars	1.05	
ASTM A36/A36M	1.05	1.5
ASTM A572/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

1605.6.2.3 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.

1605.6.2.3.1 Steel 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 AISC 360.

1605.6.2.3.2 Steel 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 and meeting the continuity and anchorage requirements of Section 1605.6.2.3.2.1.

1605.6.2.3.2.1 Steel continuity and anchorage of ties. Ties shall comply with Section 1605.6.2.3.2.1.1 through 1605.6.2.3.2.1.2.

1605.6.2.3.2.1.1 Splices. 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. Splices shall be located away from joints or regions of high stress and shall be staggered.

1605.6.2.3.2.1.2 Hooks. Use seismic hooks, as defined in Chapter 21 of ACI 318, and seismic development lengths, as specified in Section 21.5.4 of ACI 318, to anchor ties to other ties. At re-entrant corners or at substantial changes in construction, ties shall be adequately developed.

1605.6.2.3.3 Steel 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:

$$0.5(1.2D + 1.6L)s_tL_t \text{ but not less than 16.9 kips}$$

Where:

D = Dead load (psf)

L = Live load (psf)

L_t = Span (ft.)

s_t = Mean transverse spacing of the ties adjacent to the ties being checked (ft.)

1605.6.2.3.4 Steel peripheral ties. Peripheral ties shall be capable of resisting the following load:
 $0.25(1.2D + 1.6L)_s L_t$ but not less than 8.4 kips

Where:

D = Dead load (psf)

L = Live load (psf)

L_t = Span (ft.)

S_t = Mean transverse spacing of the ties adjacent to the ties being checked (ft.)

1605.6.2.3.5 Steel 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 1605.6.2.3.3 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.

1605.6.2.3.6 Steel 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.

1605.6.2.3.7 Steel columns with deficient vertical tie forces. The alternate load 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 load path analysis to verify that the structure can bridge over the missing column.

1605.6.2.4 Steel alternate load path method design requirements. Alternate load path method is used to verify that the structure can bridge over removed elements. The design strengths shall be determined in accordance with AISC 360. If the design strengths are less than those in Table 1605.6.2.4.1, then compliance shall be in accordance with the alternate load path model Sections 1605.6.2.4.1 through 1605.6.2.4.5.

**TABLE 1605.6.2.4.1
 ACCEPTABILITY CRITERIA AND SUBSEQUENT ACTION FOR STRUCTURAL STEEL**

STRUCTURAL BEHAVIOR	ACCEPTABILITY CRITERIA	SUBSEQUENT ACTION FOR VIOLATION OF CRITERIA
<u>Element Flexure</u>	ϕM_n^a	<u>Section 1605.6.2.4.1</u>
<u>Element Combined Axial and Bending</u>	<u>AISC LRFD Chapter H Interaction Equations^a</u>	<u>Section 1605.6.2.4.2</u>
<u>Element Shear</u>	ϕV_n^a	<u>Section 1605.6.2.4.3</u>
<u>Connections</u>	<u>Connection Design Strength^a</u>	<u>Section 1605.6.2.4.4</u>
<u>Deformation</u>	<u>Deformation Limits, defined in Table 1605.6.2.5(1)</u>	<u>Section 1605.6.2.4.5</u>

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 360.

1605.6.2.4.1 Steel 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 360. 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 1605.6.2.4.1.1. 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.

Deformation of primary members shall not cause premature failure in secondary members, due to geometric interference. Torsional rotation of a girder shall not cause excessive deformation and stresses in any beam that frames into the girder with a simple shear tab connection.

1605.6.2.4.1.1 Steel formation of plastic hinge. If hinge formation, i.e. material non-linearity, is included in the alternate load path analysis, the requirements of Section A5.1 of the AISC 360 for plastic design shall be met. AISC 360 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 that is assumed to form a plastic hinge is capable of not only forming the hinge, but is 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).

1605.6.2.4.1.2 Steel modeling of a plastic hinge. Plastic hinges shall be modeled in accordance with Sections 1605.6.2.4.1.2.1 through 1605.6.2.4.1.2.2.

1605.6.2.4.1.2.1 Steel linear static analysis. For Linear Static analyses, when 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.

1605.6.2.4.1.2.2 Steel 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.

1605.6.2.4.2 Steel combined axial and flexural resistance of structural steel. The response of an element under combined axial force and flexural moment can be force controlled (i.e. nonductile) 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 $P_y = A_g F_y$) 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 1605.6.2.4.1. 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. When the controlling action for the element is force controlled, evaluate the strength of the element using the interaction equations in Chapter H of AISC 360, 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 in accordance with Section 1605.6.2.4.6.
2. When 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 (in accordance with 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 1605.6.2.5(1).

1605.6.2.4.3 Shear resistance of structural steel. The acceptability criteria for shear of structural steel is based on the nominal shear strength of the cross-section, in accordance with AISC 360, multiplied by the strength reduction factor Φ and the over-strength factor Ω . If the element exceeds the design strengths of Table 1605.6.2.4.1, remove the element and redistribute the loads associated with the element in accordance with Section 1605.6.2.4.6.

1605.6.2.4.4 Steel connections. All connections shall meet the requirements of AISC 360; employ the applicable strength reduction factor Φ for each limit state and over-strength factor Ω . If a connection exceeds the design strengths of Table 1605.6.2.4.1, 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 in accordance with Section 1605.6.2.4.6.

FOUO -- DCA Progressive Collapse Design Methodology -- FOUO

1605.6.2.4.5 Deformation limits for structural steel. The Deformation Limits are given in Table 1605.6.2.5(1). Fully Restrained and Partially Restrained connections are given in Table 1605.6.2.5(2). Verify and quantify the rotational capacities of connections that are not listed in Table 1605.6.2.5(2) in accordance with the testing requirements of Appendix S of AISC 341.

1605.6.2.4.6 Steel loads associated with failed elements. Nonlinear Dynamic, and Linear or Nonlinear Static Analysis shall be in accordance with Section 1605.6.2.4.6.1 through 1605.6.2.4.6.2.

1605.6.2.4.6.1 Steel 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.

1605.6.2.4.6.2 Steel 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 1605.6.2.4.6.3, 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.

1605.6.2.4.6.3 Steel 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)

**TABLE 1605.6.2.5(1)
DEFORMATION LIMITS FOR STRUCTURAL STEEL**

<u>Component</u>	<u>CLASS 2 AND 3 BUILDINGS</u>		<u>CLASS 4 BUILDINGS</u>	
	<u>Ductility</u> μ	<u>Rotation,</u> <u>Degrees</u> θ	<u>Ductility</u> μ	<u>Rotation,</u> <u>Degrees</u> θ
Beams – Seismic Section ^a	<u>20</u>	<u>12</u>	<u>10</u>	<u>6</u>
Beams – Compact Section ^a	<u>5</u>		<u>3</u>	
Beams – Non-Compact Section ^a	<u>1.2</u>		<u>1</u>	
Plates	<u>40</u>	<u>12</u>	<u>20</u>	<u>6</u>
Columns and Beam-Columns	<u>3</u>		<u>2</u>	
Steel Frame Connections; Fully Restrained				
Welded Beam Flange or Coverplated (all types)		<u>2.0</u>		<u>1.5</u>
Reduced Beam Section		<u>2.6</u>		<u>2</u>
Steel Frame Connections; Partially Restrained				
Limit State governed by rivet shear or flexural yielding of plate, angle or T-section		<u>2.0</u>		<u>1.5</u>
Limit State governed by high strength bolt shear, tension failure of rivet or bolt, or tension failure of plate, angle or T-section		<u>1.3</u>		<u>0.9</u>

a. As defined in AISC 341.

TABLE 1605.6.2.5(2)
STEEL MOMENT FRAME CONNECTION TYPES CONNECTION DESCRIPTION TYPE

<u>CONNECTION</u>	<u>DESCRIPTION</u>	<u>TYPE</u>
<u>Strong Axis</u>		
<u>Welded Unreinforced Flange</u>	<u>Full penetration welds between beams and columns, flanges, bolted or welded web.</u>	<u>FR</u>
<u>Welded Flange Plates</u>	<u>Flange plate with full-penetration weld at column and fillet welded to beam flange.</u>	<u>FR</u>
<u>Welded Cover-Plated Flanges</u>	<u>Beam flange and cover-plate are welded to column flange.</u>	<u>FR</u>
<u>Bolted Flanges Plates</u>	<u>Flange plate with full-penetration weld at column and field bolted to beam flange.</u>	<u>FR or PR</u>
<u>Improved Welded Unreinforced Flange – Bolted Web</u>	<u>Full-penetration welds between beam and column flanges, bolted web.</u>	<u>FR</u>
<u>Improved Welded Unreinforced Flange – Welded Web</u>	<u>Full-penetration welds between beam and column flanges, welded web.</u>	<u>FR</u>
<u>Free Flange</u>	<u>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.</u>	<u>FR</u>
<u>Welded Top and Bottom Haunches</u>	<u>Haunched connection at top and bottom flanges.</u>	<u>FR</u>
<u>Reduced Beam Section</u>	<u>Connection in which net area of beam flange is reduced to force plastic hinging away from column face.</u>	<u>FR</u>
<u>Top and Bottom Clip Angles</u>	<u>Clip angle bolted or riveted to beam flange and column flange.</u>	<u>PR</u>
<u>Double Split Tee</u>	<u>Split tees bolted or riveted to beam flange and column flange.</u>	<u>PR</u>
<u>Composite Top and Clip Angle Bottom</u>	<u>Clip angle bolted or riveted to column flange and beam bottom flange with composite slab.</u>	<u>PR</u>
<u>Bolted Flange Plates</u>	<u>Flange plate with full-penetration weld at column and bolted to beam flange.</u>	<u>PR</u>
<u>Bolted End Plates</u>	<u>Stiffened or unstiffened end plate welded to beam and bolted to column flange.</u>	<u>PR</u>
<u>Shear Connection with or without Slab</u>	<u>Simple connection with shear tab, may have composite slab.</u>	<u>PR</u>
<u>Weak Axis</u>		
<u>Fully Restrained</u>	<u>Full-penetration welds between beams and columns, flanges, bolted or welded web.</u>	<u>FR</u>
<u>Shear Connection</u>	<u>Simple connection with shear tab.</u>	<u>PR</u>

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

1605.6.3 Structural use of plain, reinforced and prestressed concrete. Design against disproportionate collapse for concrete shall be in accordance with ACI 318 or 1605.6.3.1. 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 1605.6.2 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).

1605.6.3.1 Concrete alternate load path method design requirements. Alternate load 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 1605.6.3.1, then compliance shall be in accordance with the alternate load path model Sections 1605.6.3.1.1 through 1605.6.3.1.6.

**TABLE 1605.6.3.1
ACCEPTABILITY CRITERIA AND SUBSEQUENT ACTION FOR REINFORCED CONCRETE**

<u>STRUCTURAL BEHAVIOR</u>	<u>ACCEPTABILITY CRITERIA</u>	<u>SUBSEQUENT ACTION FOR VIOLATION OF CRITERIA</u>
Element Flexure	ϕM_n^a	Section 1605.6.3.1.2
Element Combined Axial and Bending	ACI 318 Chapter 10 Provisions ^a	Section 1605.6.3.1.3
Element Shear	ϕV_n^a	Section 1605.6.3.1.4
Connections	Connection Design Strength ^a	Section 1605.6.3.1.5
Deformation	Deformation Limits, defined in Table 1605.6.3.1.6	Section 1605.6.3.1.6

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 in accordance with ACI 318.

1605.6.3.1.1 Over-strength factors for reinforced concrete. The applicable over-strength factor shall be applied to calculations of the design strength alternate load path method. The over-strength factors are given in Table 1605.6.3.1.1.

**TABLE 1605.6.3.1.1
OVER-STRENGTH FACTORS FOR REINFORCED CONCRETE**

<u>REINFORCED CONCRETE</u>	<u>OVER-STRENGTH FACTOR, Ω</u>
Concrete Compressive Strength	1.25
Reinforcing Steel (ultimate and yield strength)	1.25

1605.6.3.1.2 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.

1605.6.3.1.2.1 Concrete linear static analysis. For linear static analysis when the required moment exceeds the flexural design strength and when the reinforcement layout is sufficient for a plastic hinge to form and undergo significant rotation, an equivalent plastic hinge shall be added 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 $\frac{1}{2}$ 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.

1605.6.3.1.2.2 Concrete 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.

1605.6.3.1.2.3 Flexural design strength. The structural element shall be removed when the required moment exceeds the flexural design strength and shall be redistributed in accordance with Section 1605.6.3.2, when the structural element is not able to develop a constant moment while undergoing continued deformation.

1605.6.3.1.3 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 unfactored 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 in accordance with Section 1605.6.3.2. If the unfactored axial load is less than P_b , then insert an equivalent plastic hinge into the column, in accordance with the procedure in Section 1605.6.3.1.2.

1605.6.3.1.4 Shear resistance of reinforced concrete. The acceptability criteria for shear are based on the shear design strength of the cross-section, in accordance with ACI 318, using the appropriate strength reduction factor Φ and the over-strength factor Ω . When the element violates the shear criteria, remove the element and redistribute the loads associated with the element in accordance with Section 1605.6.3.2.

1605.6.3.1.5 Concrete 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. When the connection exceeds the design strengths of Table 1605.6.3.1, remove it from the model. When the connections at each end of an element fail, remove the element and redistribute the loads associated with the element in accordance with Section 1605.6.3.2.

1605.6.3.1.6 Deformation limits for reinforced concrete. Deformation limits shall be applied to structural members in accordance with Table 1605.6.3.1.6. When the element or the connections at each end of an element exceed the deformation limit in Table 1605.6.3.1.6, remove the element and redistribute the loads associated with the element in accordance with Section 1605.6.3.2. Deformation limits are applied only to the structural elements, not to the connections.

**TABLE 1605.6.3.1.6
DEFORMATION LIMITS FOR REINFORCED CONCRETE**

<u>Component</u>	<u>CLASS 2 & 3 BUILDINGS</u>		<u>CLASS 4 BUILDINGS</u>	
	<u>Ductility</u> ν	<u>Rotation,</u> <u>Degrees</u> θ	<u>Ductility</u> ν	<u>Rotation,</u> <u>Degrees</u> θ
Slab and Beam Without Tension Membrane ^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 Membrane ^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	-

- 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.
- Single-reinforced members have flexural bars in one face or mid-depth only. Double-reinforced members have flexural reinforcing in both faces.
- 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.
- Seismic columns have ties or spirals in accordance with ACI 318 Chapter 21 seismic design provisions for special moment frames.
- Ductility of compression members is the ratio of total axial shortening to axial shortening at the elastic limit.

1605.6.3.2 Concrete loads associated with failed elements. The following procedure shall be met for Nonlinear Dynamic, and Linear or Nonlinear Static Analysis.

1605.6.3.2.1 Concrete 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.

1605.6.3.2.2 Concrete linear or nonlinear static analysis. For a Linear or Nonlinear Static analysis, when the loads on the failed element are already doubled as shown in Section 1605.6.2.4.7.3, 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. When 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 rerun 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.

FOUO -- **DCA Progressive Collapse Design Methodology** -- FOUO

1605.6.3.2.3 Concrete 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)

1605.6.4 Key elements analysis. When applying the alternate load path method design requirements from Sections 1605.6.1.8, 1605.6.2.4 or 1605.6.3.1 and the removal of columns and lengths of walls result in a disproportionate collapse, then such element shall be designed to withstand 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.

1605.6.4.1 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.