Treatment of Progressive Collaps 
in US Codes and Standards

Treatment of the topic of progressive collapse is still evolving in U.S. codes and standards, as indicated in this section of the present article that represents part 2 of two articles about accidental loading and progressive collapse. Part 1/2, published in CPI 03-2014, presented an introduction and described the phenomenon as such. Additionally, proposals for the design of precast structures to withstand accidental loading were provided. Since both articles belong together, the numbering of paragraphs, figures and tables is continued here, following part 1/2. The two articles continue a series of papers written by authors from fib and PCI, presenting different views of both associations on certain topics that are of importance to the precast industry. The goal is to disseminate common knowledge, and to increase awareness of similarities and differences of both approaches.

7. Introduction

The commentary to ASCE 7-10 Minimum Design Loads for Buildings and Other Structures [5] defines progressive collapse as "the spread of an initial local failure from element to element resulting eventually in the collapse of an entire structure or a disproportionately large part of it." The onset of progressive collapse can be triggered by unintentional overload, misuse of the facility, or an abnormal loading (such as an accidental explosion or act of terrorism) not considered in the design. Progressive collapse increases the likelihood of human casualties and trapped survivors.

8. History of US Codes and Standards

A requirement for progressive collapse due to "local failure caused by severe overloads" was first introduced in ANSI Standard A58.1-1972 [6], the first edition following the 1968 Ronan Point collapse that was already mentioned in part 1/2, published in CPI 03-2014. No commentary or other guidance was provided. ANSI Standard A58.1-1982 contained a more comprehensive performance statement under the altered title of General Structural Integrity and provided a greatly expanded commentary section and references for guidance. ANSI Standard A58.1 became ASCE Standard 7 [5], starting with the 1988 edition of the standard. Several structural system layouts that would lead to development of alternate load paths were illustrated in ASCE 7-88 and ASCE 7-93. Section 1.4 of ASCE 7-95 retained the performance requirement that a building be designed to sustain local damage, with the structural system as a whole remaining stable. However, the commentary was shortened, keeping the discussion of general design approaches to structural integrity but eliminating the figures and other specific guidance. At the same time, a new Section 2.5 was added that required a check of strength and stability of structural systems under low-probability events, where required by the authority having jurisdiction (AHJ).

Structural integrity requirements for cast-in-place concrete structures were first added to ACI 318 Building Code Requirements for Structural Concrete in the 1989 edition of the standard (ACI 318-89) [7]. Structural integrity requirements for precast concrete structures followed one edition later in ACI 318-95 and have remained essentially unchanged through ACI 318-11.

After the malevolent attack on and partial collapse of the Alfred P. Murrah Federal Building in April 1995, overall structural integrity of buildings and the mitigation of progressive collapse received national attention. From visual observation and analysis, an emergency FEMA team estimated that progressive collapse significantly increased the direct damage incurred by the bomb blast. It was estimated that up to 90 percent of the 168 fatalities were the result of crushing caused by falling debris [8].

Later, the collapse of the World Trade Center Towers in 2001 generated further national discussions about progressive collapse. Could progressive collapse have been avoided (or minimized) in the Murrah Federal Building and the World Trade Center? Are current building codes and national standards adequate to address structural performance under catastrophic events, or are more rigorous methods needed?

Recommendations in the report on the official investigation of the World Trade Center collapse [9, 10] led directly to the NIST Best Practices document [11] and to the inclusion of Section 1614, Structural Integrity, in the 2009 International Building Code (IBC) [12]. The same requirements are in Section 1615 of the 2012 IBC.

For some time now, the U.S. Federal Government has been developing approaches to addressing progressive collapse prevention in building design. Two major building owners, the General Services Administration (GSA) and the Department of Defense (DoD), require engineers to consider progressive collapse as a design criterion. The design guidance provided by these two organizations represents the most comprehensive information in the U.S. currently available on this topic.

8.1 ACI 318

Major research was conducted in the 1970s under the sponsorship of the U.S. Department of Housing and Urban Development (HUD) by the Portland Cement Association (PCA) into the structural integrity and progressive collapse resistance of large-panel precast buildings. This research led to the publication of six reports and three supplemental reports on the analysis
and design of large-panel concrete structures between 1975 and 1979 [13, 14]. The PCI Committee on Precast Bearing Walls formulated recommendations [15] largely based on this research. Those recommendations eventually led to the inclusion of structural integrity requirements for precast concrete structures in ACI 318-95.

ACI 318-11 [7] requires tension ties in the form of reinforcement and connection hardware to be used in all precast concrete structures to

![Image of MBK machine](image-url)

**Characteristics:**
- Customized solution through modular plant system.
- Compact plant design requires little space (35 x 12 x 4 mtr / L x W x H).
- Performance capacity up to 400 m² per hour.
- Higher productivity on demand.
- CAD based mesh welding with windows and door openings.
- Reinforcement meshes for floors and walls welded on the same machine equipment.
- No subsequent rotation required.
- Rigid construction of the pallet circulation. Low maintenance required.
- Subsequent system expansion possible.
- Extendable with bending equipment.
- Nr. 5 Ø wire diameters from 3 to 6.
- Wire diameters standard from 6 - 12 and optional to 16 mm.
- Center to center 50, optional 25 mm.
- Shortest bar 400, optional 200 mm.
- Shortest spacing 100, optional 50mm.
- Base execution with 2 welding heads, extendable to 4.

**Fig. 12:** Location of tensile ties in large-panel structures [16]
achieve integrity of structures. For precast concrete construction, tension ties are required to be provided in the transverse, longitudinal, and vertical directions and around the perimeter of the structure, in order to tie elements together effectively (see details in Figure 12). The overall integrity of a structure can be substantially enhanced by minor changes in the amount, location, and detailing of member reinforcement and in the detailing of connection hardware; however, connection details that rely solely on friction caused by gravity forces are not permitted.

Through the detailing of the minimum ties, a level of acceptable integrity, established from research, is achieved. This provision of general structural integrity eliminates the need to design for any particular abnormal load.

The rationale for the horizontal, vertical and peripheral ties, as discussed in [16], is reproduced below. It is important to note that the minimum provisions of ACI 16.5.2.1 through 16.5.2.5 apply only to precast concrete bearing wall structures three or more stories in height.

Transverse Ties to Develop Cantilever and Beam Action in Wall Panels

The provisions for transverse ties are prescribed in ACI 16.5.2.1: “Longitudinal and transverse ties shall be provided in floor and roof systems to provide a nominal strength of 4.5 kN per meter of width or length. Ties shall be provided over interior wall supports and between members and exterior walls. Ties shall be positioned in or within 0.6 m of the plane of the floor or roof system.” Additionally, ACI 16.5.2.3 requires that “transverse ties perpendicular to floor or roof slab spans shall be spaced not greater than the bearing wall spacing.”

These provisions essentially locate the transverse ties in the horizontal joints between wall panels at the level of the floor. The intent of these transverse ties is to create cantilever action in the wall stack in the event of severe damage to or loss of a load-bearing wall. This cantilever action will transfer vertical shear from the walls above the damage to adjacent walls in the line of the damaged wall. To accomplish this load transfer, there must be tensile continuity in the horizontal connections. The detailing of the floor and the inclusion of transverse walls aid in maintaining the load path. The shear strength in the horizontal joint must be sufficient to prevent horizontal panels from sliding at the joint. The tie force has been determined empirically from the load tests conducted as part of the research so that this cantilever action can be mobilized by either a stack of cantilevered walls or by individual floor cantilevers. This cantilever action is the primary mechanism that creates a load path around a damaged wall.

Longitudinal Ties for Membrane Action in Floor

ACI 16.5.2.1 also provides for longitudinal ties, which run in the direction of the floor component span. ACI 16.5.2.2 adds, “longitudinal ties parallel to floor or roof slab spans shall be spaced no more than 3 m on centers. Provisions shall be made to transfer forces around openings.” These longitudinal ties are proportioned to act as catenaries between walls on either side of the ineffective wall only to the extent of supporting loads from the local debris. The empirical forces used are not sufficient for the catenary to carry the wall above the floor and floor loads to those adjacent walls. The research found that provision of that level of force was not feasible and was not required.

Vertical Ties to Develop Suspension Action

Vertical ties are prescribed by ACI 16.5.1.3 and 16.5.2.5. Vertical tension tie requirements are applied to all vertical structural members, except cladding. Connections are required to be provided at horizontal joints. For precast columns, the nominal strength in tension is required to be greater than 1.4Ag KN (Ag in mm²), where Ag is the area of the cross section of the column. A reduced area Ag is permitted if the area of cross section is larger than required to resist the loads, but not less than one-half the total area. For precast wall panels, a minimum of two ties is required per panel, with a nominal tensile strength not less than 45 KN/tie. The ties are permitted to be anchored into an appropriately reinforced concrete floor slab on ground, when design forces result in no tension at the base.

The vertical ties provide for the vertical suspension of ineffective walls to limit debris load. Because the alternative support mechanism in bearing wall structures develops through the cantilever action of walls above, these ties need only hold the damaged wall and floor that it supported to limit the loads on lower walls and floors. They provide resistance against a wall or column kicking out, preventing total removal of a wall or column. They also provide clamping action for shear-friction capacity in the joint. Walls and columns on the perimeter are more vulnerable and should be detailed with more ties. These vertical ties may also be needed for overturning strength in resisting system loads.

Peripheral Ties to Develop Diaphragm Action

ACI 16.5.2.4 adds provisions for perimeter ties: “Ties around the perimeter of each floor and roof, within 1.2 m of the edge, shall provide a nominal strength in tension of not less than 71 KN.” These perimeter ties provide for a minimum strength in the floor diaphragm.

8.2 ASCE 7-10

Provisions

The provisions in Section 1.4 of ASCE 7-98, ASCE 7-02, and ASCE 7-05 were essentially the same as in the 1995 edition mentioned above and read as follows:

“Buildings and other structures shall be designed to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage. This shall be achieved through an arrangement of the structural elements that provides stability to the entire structural system by transferring loads from any locally damaged region to adjacent regions capable of resisting those loads without collapse. This shall be accomplished by providing sufficient continuity, redundancy, or energy-dissipating capacity (ductility), or a combination thereof, in the members of the structure.”

The above performance language has been replaced in ASCE 7-10 with Sections 1.4.1 through 1.4.4, which present minimum strength criteria intended to ensure minimum interconnectivity of structural elements and the existence of a complete lateral force-resisting system with sufficient strength to provide for stability under gravity loads and nominal lateral forces that are independent of design wind, seismic, or other anticipated loads. Conformance with
these criteria will provide structural integrity for normal service and minor unanticipated events that may reasonably be expected to occur throughout their lifetimes. For many structures meant to accommodate large numbers of persons, or housing functions necessary to protect public safety, or with occupancies that may provoke intentional sabotage or attack, more rigorous protection should be incorporated into designs than provided by Section 1.4. For such structures, additional precautions can and should be taken in the design of structures to limit the effects of local collapse and to prevent or minimize progressive collapse in accordance with the procedures of Section 2.5, as charged by Section 1.4.5.

The Commentary to Section 2.5, Load Combinations for Extraordinary Events, in ASCE 7-98, 02, and 05 recommended the following load combination for checking the ability of a damaged structure to maintain its overall stability for a short time following an abnormal load event:

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

in which \(D\), \(L\), \(W\), and \(S\) are specified dead, live, wind, and snow loads determined in accordance with ASCE 7 provisions. This check suggests the notional removal of selected (presumably damaged) load-bearing elements at the discretion of the engineer without stipulating tolerable damage.

If certain key elements in the structural system must be designed to withstand the effects of the accident (perhaps to allow the development of alternate load paths), they should be designed using the combination:

\[(0.9 \text{ or } 1.2) \times D + A_k + (0.5 \text{ L or } 0.2 S)\]

in which \(A_k\) is the postulated action due to the abnormal load. Normally, only the main load-bearing structure would be checked using these equations.

**Commentary**

Two design alternatives for progressive collapse resistance are specified in the Commentary to the ASCE 7 section on General Structural Integrity: indirect design and direct design. Indirect design implicitly considers resistance to progressive collapse by providing minimum levels of strength, redundancy, continuity, and ductility. One form of indirect design involves the concept of tie forces. In this approach, ties capable of resisting a minimum level of force are provided to enhance the structure’s redundancy, continuity, and ductility. It is envisioned that the system of ties will help keep the structure from collapsing in the event of a catastrophic event.

The second design alternative, direct design, involves explicit consideration of resistance to progressive collapse and includes approaches such as the alternate path method and the specific local resistance method. In the alternate path method, the design allows local failure to occur, but seeks to prevent major collapse by providing alternate load paths. Typically, a specified initial local failure is assumed (such as a missing column) and the structure is designed to be able to redistribute the load around this area. In the specific local resistance method, the design seeks to provide elements with sufficient strength to prevent failure under an extraordinary loading event. This method assumes an overload condition (such as a blast pressure and impulse), and then designs the members to withstand this loading. Specific suggestions for the implementation of each of the defined methods are contained in [17].

The ASCE 7-10 Commentary includes recommendations for incorporating general structural integrity into building design. These guidelines discuss several methods of providing sufficient integrity to distribute loads around severely damaged structural members. Unlike documents developed by the DoD and GSA, however, ASCE 7-10 is general in nature and does not include any quantifiable or enforceable requirements. The recommendations found in ASCE 7-10 are formulated around the following design concepts and details:

1. Good plan layout,
2. Integral system of ties,
3. Returns on walls,
4. Changing span directions of floor slabs,
5. Load-bearing interior partitions,
6. Catenary action of floor slabs,
7. Beam action of walls,
8. Redundant structural systems,
9. Ductile detailing,
10. Additional reinforcement to resist blast and local reversal when blast loads are considered,

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### 8.3 International Building Code (IBC)

The structural integrity requirements of the 2009 and 2012 IBC apply to high-rise buildings that are assigned to Risk Category III or IV. A high-rise building is defined by the IBC as a building with an occupied floor located more than 23m above the lowest level of fire department vehicle access.

Table 2: UFC 4-023-03 Occupancy and Design Requirements [20]

<table>
<thead>
<tr>
<th>Occupancy Category</th>
<th>Nature of Occupancy</th>
<th>Design Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Low occupancy: Low hazard to human life in the event of failure</td>
<td>No Specific Requirements</td>
</tr>
<tr>
<td>II</td>
<td>Inhabited buildings with less than 50 personnel, primary gathering buildings, billeting, and high occupancy family housing; Buildings and other structures except those listed in Categories I, III, and IV</td>
<td>Option 1: TF and ELR or Option 2: AP See UFC for details.</td>
</tr>
<tr>
<td>III</td>
<td>Buildings and other structures that represent a substantial hazard to human life or represent significant economic loss in the event of failure.</td>
<td></td>
</tr>
<tr>
<td>IV</td>
<td>Buildings and other structures designed as essential facilities; Facilities designed as national strategic military assets</td>
<td></td>
</tr>
</tbody>
</table>

Risk Category III structures are defined in ASCE 7-10 (adopted by reference by the 2012 IBC) as follows:

Buildings and other structures, the failure of which could pose a substantial risk to human life.

Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure. Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.

Risk Category IV structures are defined in ASCE 7-10 as follows:

Buildings and other structures designated as essential facilities.

Buildings and other structures, the failure of which could pose a substantial hazard to the community.

Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released and is sufficient to pose a threat to the public if released.

Buildings and other structures required to maintain the functionality of other Risk Category IV structures.

For precast concrete structures, the IBC simply refers to ACI 318 requirements.

### 8.4 U.S. Department of Defense (DoD) Standard

Incorporation of progressive collapse mitigation considerations into the design and construction of Military facilities is a relatively recent development. The first published document to address the issue was the Interim Department of Defense Antiterrorism/Force Protection Construction Standards (16 December 1999). As the name suggests, the interim standards served as a bridging document prior to the development of the final standard. The final standard, UFC 4-010-01, DoD Minimum Antiterrorism Standards for Buildings (8 October 2003), is part of the Unified Facilities Criteria (UFC) system. The UFC system provides planning, design, construction, sustainability, restoration, and modernization criteria for all DoD construction projects. The current version of UFC 4-010-01 is dated 9 February 2012 [18].

The minimum standards not only address structural issues, but also cover a broad range of security aspects from site planning to mass notification systems. In general, the DoD understands that comprehensive protection against the range of possible threats for all facilities is cost-prohibitive. However, an appropriate level of protection can be provided to lessen the risk of mass casualties for all DoD personnel at a reasonable cost.

Progressive collapse is currently addressed in Standard 6 (Section B-2.1 of Appendix B) of UFC 4-010-01. Detailed guidance is provided in UFC 4-023-03, Design of Buildings to Resist Progressive Collapse - first version was dated 25 January 2005, current version is dated 14 July 2009 [19].

The requirements of UFC 4-023-03 are nicely summarized in [20]. The summary provided here is largely taken from that source. The design approach is dependent on the use or occupancy of the building structure. Based on the level of occupancy or Occupancy Category (OC), three design approaches are used. They include the tie force (TF), enhanced local resistance (ELR), and alternate load path (AP) methods. The first is an indirect and the latter two are direct methods of design. For high levels of occupancy and criticality, all three methods may be required; while for low levels of criticality, none of the methods may be needed.

### Tie Force Method

The tie force method requires that the tensile force capacity of the floor or roof system be adequate to allow the transfer of load from a damaged portion of the structure to an undamaged portion. The approach does not specifically remove any vertical elements but instead requires a minimum horizontal tensile strength in the floor or roof diaphragm. According to UFC 4-023-03, three types of horizontal ties are required to provide integrity to the floor and roof diaphragms. They include longitudinal, transverse and peripheral ties, which are the same as in ACI 318-11.

Tension ties in reinforced (including precast) concrete structures typically consist of continuous reinforcing steel in beams, columns, slabs, and walls. Reinforcement required for tension ties can be provided in whole, or in part, by steel already sized to...
resist other actions, such as shear or flexure. For example, consider a beam that needs an area of 1400 mm$^2$ of steel to satisfy the tie force criteria. Also assume that this beam has two 20-mm-diameter continuous longitudinal bars provided for flexure at top and bottom. The total area of flexural steel (1260 mm$^2$) is assumed to act as part of the tension tie, in order to fully comply with the tie requirement, a total of 140 mm$^2$ of steel should be added to the beam. This can be accomplished by either adding more bars or increasing the size of the existing bars. One option is to increase the size of the two 20-mm-diameter bars to 22-mm-diameter bars, for a total steel area of 1520 mm$^2$.

In many cases, the quantity of steel provided to resist gravity and lateral forces in typical reinforced (including precast) concrete structures is also sufficient to develop the necessary tie forces. It is, therefore, reasonable to simply check compliance with tie force requirements after a structure is initially designed for gravity and lateral loading. Ties must be properly spliced and adequately anchored at each end in order to develop their full capacity and ultimately perform as anticipated.

Reinforcing steel used as tension ties must be lapped, welded, or spliced with Type 1 or Type 2 mechanical splices per ACI 318-11. In addition, the DoD stipulates that splices should be staggered and located away from joints and regions of high stress. No provisions are included in the standards that specify acceptable splice locations or minimum stagger distance.

Seismic detailing is required to be used to anchor ties to other ties or at points of termination, such as at the perimeter of a building. This includes providing seismic hooks, as defined in Chapter 21 of ACI 318-11, and using seismic development lengths, as specified in Section 21.7.S of ACI 318-11. Anchorage is critical to the performance of ties and must be carefully assessed, particularly in cases of where building layout may be atypical.

The following specific statement about precast concrete floor and roof systems is included in UBC 10-023-04: “For precast concrete floor and roof systems, the rebar within the precast slabs may be used to provide the internal tie forces, provided the rebar is continuous across the structure and properly anchored; this may be difficult to accomplish in the short direction of the slab. Also, the rebar may be placed within a concrete topping in this case, provide positive mechanical engagement between the reinforcement and the precast floor system, with sufficient strength to ensure that the precast units do not separate from the topping and fall to the space below. Do not rely on bond strength between the topping and precast units, as the bond can be disrupted by the large deformations associated with catenary behavior. This attachment between the rebar in the concrete topping and the precast slabs may be accomplished with hooks, loops or other mechanical attachments that are embedded in the precast floor units.”

Alternate Path (AP) Method
Under this approach, the building must bridge across a removed element, so it is a direct method for progressive collapse analysis.
and design. Especially, if a corner column is specified as the removed element location in a ten-story building with a column splice at the third story, one AP analysis is performed after removing the ground story corner column; another AP analysis is performed after removing the corner column at the tenth story; a third AP analysis is performed with the fifth story corner column removed (mid-height story) and a final AP analysis is performed with the fourth story corner column removed (story above the column splice).

The Alternate Path method employed in UFC 4-023-03 follows the general philosophy of the standard LRFD (Load and Resistance Factor Design) approach, but with modifications to facilitate the integration of ASCE 41 procedures. For LRFD, the design strength is taken as the product of the strength reduction factor ϕ and the nominal strength Rn calculated in accordance with the requirements and assumptions of applicable material specific codes. The design strength must be greater than or equal to the required strength:

\[ \phi R_n \geq R_d \]

where ϕRn = Design strength
ϕ = Strength reduction factor
Rn = Nominal strength
Rd = Required strength
γl = Load factor
Q = Load effect

While ASCE 41 requires that all ϕ factors be taken as unity, UFC 4-023-03 requires that strength reduction factors, ϕ, be used as specified in the appropriate material specific code, for the action or limit state under consideration.

ASCE 41 uses the term “action” in the way LRFD uses “required strength”. ASCE 41 further differentiates actions into “deformation-controlled” (bending moment, axial force) and “force-controlled” (shear force).

In UFC 4-023-03, the LRFD “nominal strength” is defined as either the “expected strength” when deformation-controlled actions are being considered or the “lower-bound strength” for force-controlled actions; ASCE 41 sets all ϕ factors to 1 and, therefore, the expected and lower bound strengths are the nominal strengths in that document.

Both UFC 4-023-03 and ASCE 41 employ the same “overstrength factors” to translate lower-bound material properties to material properties that would yield expected strength. The overstrength factors are provided in ASCE 41.

Methods for calculation of individual component strengths and deformation capacities comply with the requirements in the individual ASCE 41 material chapters. The load increase factors for deformation-controlled and force-controlled actions for column and wall removal are provided in UFC 4-023-04 Table 3-4.

To model, analyze, and evaluate a building, a three-dimensional assembly of elements is created. Linear Static Procedure (LSP) is applied for regular structures or irregular structures with

\[ \text{Fig. 13: GSA Alternate Path Method flowchart} \]
DCR≤2.0, where DCR is the demand-capacity ratio. The LSP model includes all the primary components except the removed component; it is optional to include secondary components. When modeling the building by the LSP procedure, the column considered to have failed is removed before the factored load is applied on the considered region of the structure. For LSP, component capacities for deformation-controlled actions are defined as the product of m-factors (see ASCE 41 for m-factors) and expected strengths, $Q_{CE}$, multiplied by the appropriate strength reduction factor $\varphi$. Capacities for force-controlled actions are defined as lower-bound strengths, $Q_{CL}$, multiplied by the appropriate strength reduction factor $\varphi$. In comparison with the GSA method, the linear static procedure in UFC 4-023-03 requires increased loads to be applied only over the areas above the removed column, while GSA requires the increased factored load, which is equal to 2 times dead load plus 0.5 times live load, to be applied at each floor level over the whole structure.

Nonlinear Static Procedure (NSP) and Nonlinear Dynamic Procedure (NDP) have no limitations on applicability, based on structural regularity. For nonlinear procedures, component capacities for deformation-controlled actions are taken as permissible inelastic deformation limits, and component capacities for force-controlled actions are taken as lower-bound strengths, $Q_{CL}$, multiplied by the appropriate strength reduction factor $\varphi$.

For all three analysis types (LS, NS, and ND), the building is structurally adequate if none of the primary and secondary elements, components, or connections exceeds the acceptance criteria, in UFC 4-023-03 Sections 3-2.11.7, 3-2.12.7, and 3-2.13.6, as appropriate. If the analysis predicts that any element, component, or connection does not meet these acceptance criteria, the building does not satisfy the progressive collapse requirements and must be redesigned or retrofitted to eliminate the non-conforming element.

Enhanced Local Resistance (ELR)
Enhanced Local Resistance (ELR) is required in three cases: OC II Option 1 (Tie Forces and ELR), OC III (Alternate Path and ELR), and OC IV (Tie Forces, Alternate Path, and ELR) - see Table 2 above. All three cases contain the same objective, which is to ensure that a ductile failure mechanism can form when the column or wall is loaded laterally to failure. To meet this objective, the column or wall must not fail in shear prior to the development of the maximum flexural strength.

Two components must meet the ELR requirement: 1. The column or wall, and 2. The connections between the end of the column or wall and the lateral supports (floor slab, base plate, etc.).

Note that design for ELR is not required if the wall or column has been designed for a specific design basis threat, providing that the design basis threat was developed with a risk assessment approach that was approved by the building owner, government agency or other responsible entity.

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Table 3: Summary of Progressive Collapse Requirements in Major U.S. Codes, Standards, and Resource Documents

<table>
<thead>
<tr>
<th>Standard or Agency</th>
<th>Document</th>
<th>Requirements</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>American Concrete Institute (ACI)</td>
<td>ACI 318, Building Code Requirements for Structural Concrete (2011) [7]</td>
<td>Although progressive collapse is not explicitly considered, structural integrity reinforcement is required, to improve redundancy and ductility.</td>
<td>For example, minimum requirements are given for longitudinal, transverse, peripheral, and vertical ties in precast concrete bearing wall construction.</td>
</tr>
<tr>
<td>American Society of Civil Engineers (ASCE)</td>
<td>ASCE 7-10, Minimum Design Loads for Buildings and Other Structures (2010) [5]</td>
<td>Basic load path and continuity requirements are specified in the standard. Commentary contains general discussion on reducing the potential for progressive collapse. No quantifiable or enforceable requirements for progressive collapse prevention are provided.</td>
<td>Discusses two design alternatives for progressive collapse resistance: direct design (alternate load path or specific local resistance) and indirect design.</td>
</tr>
<tr>
<td>International Code Council</td>
<td>International Building Code (IBC) (2012) [12]</td>
<td>Structural integrity requirements are provided for high-rise buildings that are ASCE 7-10 Risk Category III or IV.</td>
<td>For reinforced including precast concrete structures, reference is made to ACI 318 requirements.</td>
</tr>
<tr>
<td>U.S. Department of Defense (DoD)</td>
<td>UFC 4-010-01, DoD Minimum Antiterrorism Standards for Buildings (9 February 2012) [18] &amp; UFC 4-023-03, Design of Buildings to Resist Progressive Collapse (14 July 2009) [19]</td>
<td>For all new DoD construction and major renovations: progressive collapse must be considered for buildings of three or more stories.</td>
<td>The design approach is dependent on the use or occupancy of the building structure. Based on the level of occupancy or Occupancy Category (OC), three design approaches are used. They include the tie force (TF), enhanced local resistance (ELR), and alternate load path (AP) methods. For high levels of occupancy and criticality, all three methods may be required, while for low levels of criticality, none of the methods may be needed.</td>
</tr>
</tbody>
</table>

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8.5 U.S. General Services Administration (GSA) Guidelines
The GSA’s current progressive collapse criteria are found in Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects (June 2003) [22]. The purpose of the Guidelines is to “ensure that the potential for progressive collapse is addressed in the design, planning and construction of new Federal buildings and major renovation projects.” The first version of the Guidelines was released in November 2000.

The GSA Guidelines prescribe a threat-independent methodology for reducing the likelihood of progressive collapse in new and upgraded buildings and for assessing the potential for progressive collapse in existing facilities. The intent of the Guidelines is to prevent widespread collapse after an assumed local failure has occurred. This is accomplished by providing sufficient continuity, ductility, and redundancy within the system to allow the structure to bridge over severely damaged members.

The primary method of analysis presented in the GSA Guidelines is the static, linear elastic approach. Linear procedures are used for low- to medium-rise structures, with ten or fewer stories above grade and typical structural configurations. When analyzing structures with more than 10 stories and/or atypical structural configurations, the Guidelines recommend that the use of non-linear procedures be considered.

The minimum requirement of the GSA Guidelines is to ensure that a building can withstand loss of one primary exterior vertical load bearing member without experiencing progressive collapse. Consideration of interior element removal is required only if an interior threat, such as presented by underground parking areas and/or uncontrolled public ground floor areas, exists. For an exterior threat, it is required to remove members, one at a time, at the building’s perimeter and evaluate the response.

Element removal is controlled by several factors including: location of the threat (i.e., exterior vs. interior), type of structural system, and building layout. For most typical structures, the number of columns that need to be evaluated is a small fraction of the total number of columns (or load-bearing walls) in the building. The GSA Guidelines require the removal of significantly lesser number of structural elements as compared to the DoD requirements. While, the DoD standards require elements at critical locations to be removed from every story of the building, one at a time, GSA Guidelines require the same elements to be removed only from the first story.

The GSA provides two load cases, the use of which is dependent on the chosen analysis method. If a static analysis is performed, the factored vertical load case is as follows:

\[ \text{Load} = 2(D + 0.25L) \]

If a dynamic analysis is performed, the factored vertical load case is as follows:

\[ \text{Load} = D + 0.25L \]

Material strengths used for the design of reinforced concrete members may be increased by an over-strength factor to reflect the expected material strength. Overstrength factors account for rapid strain rate effects and the fact that actual material strengths typically are larger than specified design strengths.

The GSA, on the other hand, specifies a limit on the maximum amount of damage that can be sustained under removal of a single load-bearing element. If results indicate that the collapsed area extends beyond these limitations, the structure is considered non-compliant and the design must be revised before the analysis can continue. Maximum allowable extent of collapse resulting from removal of a primary support is based on location of the removed element.
To evaluate the results of a linear elastic analysis, the magnitude and distribution of predicted demands are expressed as Demand-Capacity-Ratios (DCRs), defined as:

$$ \text{DCR} = \frac{Q_{UD}}{Q_{CE}} $$

where,

- $Q_{UD}$ = Demand in component or connection/joint (i.e. moment, axial force, shear, and possible combined forces)
- $Q_{CE}$ = Expected, ultimate, unfactored capacity of the component or connection/joint (i.e. moment, axial force, shear, and possible combined forces)

$Q_{CE}$ represents the calculated flexural, shear, and axial capacities using expected material strength (see Section 3.6) and no capacity reduction factor (i.e., $\phi = 1.0$).

A DCR value greater than 1.0 indicates that the member or connection has exceeded its ultimate capacity at that location. This alone, however, does not signify failure. The flexural DCR along a beam, for example, may exceed 1.0 as long as the beam is capable of moment redistribution.

Structural components and connections that exceed the allowable DCR values are considered severely damaged or collapsed. Allowable DCR values are as follows:

- **DCR ≤ 2.0** for typical structural configurations (as defined in GSA Guidelines Section 3.4.1).
- **DCR ≤ 1.5** for atypical structural configurations (as defined in GSA Guidelines Section 3.4.2).

A flowchart outlining the steps for performing the static linear elastic alternate path analysis is provided in Figure 13.

### 8.6 Concluding Remarks

Section 7 of this article provides a brief overview of provisions related to structural integrity and prevention of progressive collapse in U.S. codes (IBC, adopted by most of cities, counties, and states for their legal building codes), standards (ACI 318, ASCE 7), and resource documents DoD Standards, GSA Guidelines). The requirements are summarized in Table 3.

### References