Reinforced Concrete and Secure Buildings: Progressive Collapse

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Introduction

With the tragic attacks on the Murrah Federal Building and the World Trade Center towers, attention in the engineering community is again being focused on general structural integrity and design strategies for the prevention of progressive collapse.

There has been much debate over what changes, if any, should be made to building codes and what types of buildings should be addressed. There is agreement, however, that engineers, as a matter of standard practice, should consider the "load paths" within a structure and focus on tying individual structural members together to improve a building's overall integrity.

In light of recent world events, two major building owners, the General Services Administration (GSA) and the Department of Defense (DoD) are requiring engineers to consider building security as another design criterion. The Department of Homeland Security is mandating greater security considerations for structures housing critical infrastructure. Even private sector owners and developers of high-profile buildings are taking a closer look at security risks as their buildings may be considered "targets" of both domestic and foreign terrorists.

Most engineers are experienced in designing structures to resist natural disasters such as hurricanes and earthquakes, but few are knowledgeable in designing structures to meet requirements known as Antiterrorism/Force Protection (AT/FP) standards. AT/FP covers a very broad range of disciplines from site planning to mass notification systems. Of most interest to those involved with the structural design of buildings are the prevention of progressive collapse and blast resistance.

Blast resistant design is a highly specialized field concerned with minimizing the hazards associated with intentional or accidental explosions. The general objective is to design a building to resist specified blast overpressures within required performance limits. For antiterrorism considerations, the design explosion parameters are based on terrorist threat scenarios. Typical threat scenarios include means of delivery (i.e. vehicle bomb, place bomb, mail bomb, etc.), weight of explosive and standoff distance (i.e. distance from building to detonation). Unfortunately, the exact location, size and nature of terrorist threats are unpredictable.

Because it is impossible to design for every conceivable threat, the prevailing design philosophy is to allow local damage, but to mitigate the occurrence of progressive collapse. Progressive collapse is a condition where local failure leads to resulting damage that is disproportionate to the damage that initiated the collapse. Prevention of progressive collapse is achieved by providing sufficient continuity, redundancy, and/or energy dissipation capacity in the structural members and connections. Even where security concerns are not an issue, these concepts are essential to sound engineering practice.

There is a growing need to better educate the practicing engineer (and others in the construction industry) on the basic design principles associated with blast resistance and the prevention of progressive collapse. Although both of these design scenarios concern the structural response to abnormal loading, their incorporation into the design process present unique challenges. For this reason, and because of the considerable amount of information involved, it makes sense to discuss these topics separately rather than combine them.

In the authors' opinion, the mitigation of progressive collapse is a better starting point, because it is applicable to a greater number of buildings in both the public and private sector. Furthermore, the fundamental concepts involved with progressive collapse mitigation are valuable to the design of almost all structures. This Structural Bulletin, therefore, focuses on progressive collapse mitigation in reinforced concrete structures. The first section of the Bulletin provides a general discussion of the current approaches used to design for progressive collapse resistance. The middle portion includes specific projects designed by the authors where reinforced concrete was used to mitigate the potential for progressive collapse. At the end of the Bulletin, there is a discussion on reinforcing steel detailing requirements.
What is Progressive Collapse?

ASCE 7-02 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.”

Progressive collapse increases the likelihood of greater human casualties and trapped survivors as a result of the collapse. One of the more infamous examples of progressive collapse is the Alfred P. Murrah Federal Building in Oklahoma City (See Figure 1) where approximately 70% of the building experienced dramatic collapse. It was determined that most of the devastation, including the loss of 168 lives was the result of progressive collapse, not the direct effects of the explosion.

On the other hand, a good example of a building that experienced extensive local damage, but did not experience progressive collapse, is the Khobar Towers in Saudi Arabia. The Khobar Towers lost a significant area of its exterior framing due to damage sustained from a large terrorist truck bomb attack. As is illustrated in Figure 2, there was significant local damage, but no progressive collapse. This is a good example of a building that had incorporated many of the design concepts listed in ASCE 7-02’s “Guidelines for the Provision of General Structural Integrity”.

Limitations of Current Design Approaches

As a starting point, considering continuity, redundancy and ductility in structural design and detailing will lead to a more robust and resilient structure.

These are basic principles, not driven by formulas or computers, that engineers should conceptualize and consider in the process of designing any building. The American Concrete Institute recognized these principles with the addition of Section 7.13 “Requirements For Structural Integrity” to ACI 318-89.

This recent section provided guidance on minor changes in detailing of reinforcement that would enhance the overall performance of the structure by improving its redundancy and ductility. While not requiring an explicit analysis, the intent is to increase the survivability of a structure in the event of damage to a major supporting element from an abnormal loading.

To actually analyze and design a building to prevent specific progressive collapse scenarios is a more complicated issue. Although defined in ASCE 7, there is still much debate over what constitutes progressive collapse. For example, was the initiating event at the Murrah Building truly a “local” failure? The truck bomb parked next to the building contained approximately 4,000 pounds of fertilizer-based ANFO (ammonium nitrate/fuel oil) explosive. The blast wave disintegrated one of the perimeter columns and caused brittle failure of two adjacent columns — approximately 120 linear feet of building perimeter was supported by these columns. In order to design for prevention of progressive collapse, the concepts of “initial local failure” and “disproportionate collapse” must be more clearly defined.

The design approach to mitigate the potential for progressive collapse as a result of abnormal loading is not standardized in the United States. In general, current national building codes and standards either address progressive collapse in qualitative terms (i.e., by requiring designs to provide general structural integrity) or they do not broach the subject matter at all. Government agencies, however, tend to be more direct in handling progressive collapse. The GSA and the DoD have developed criteria that do provide engineers with a prescriptive procedure to assess the susceptibility of a building to progressive collapse.

It should be emphasized that the GSA and DoD procedures are not predicated on any specific abnormal loading or threat scenario. The “analytical” removal of a single column or beam from a building in a computer analysis does not necessarily represent a “real world scenario”. Instead, these analytical approaches are a method of providing sufficient redundancy, continuity, and/or energy dissipation capacity in the members and connections of a structure to allow alternate load paths to develop if local damage occurs. These procedures typically focus on the post-event capacity, ductility, and robustness of the individual members and connections compared with just key element resistance.

Designing for the mitigation of progressive collapse should not be confused with blast design or blast hardening of a structure. Blast design is a complicated field that requires the knowledge of a specific threat. The goal in blast hardening is to prevent the blast from breaching the building’s envelope. Blast design can include analysis of not only the primary structural components, but also the building’s windows, doors and facade.
Existing Progressive Collapse Guidelines
A number of building codes, standards (both national and international) and design guidelines include discussion on the prevention of progressive collapse. The level of detail in these documents, however, varies considerably and can be difficult for a design engineer to explicitly incorporate into the structural design process.

Table 1:
SUMMARY OF PROGRESSIVE COLLAPSE REQUIREMENTS IN MAJOR CODES AND STANDARDS

<table>
<thead>
<tr>
<th>Standard or Agency</th>
<th>Document</th>
<th>Requirements</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Department of Defense (DoD)</td>
<td>Unified Facilities Criteria (UFC) 4-010-012,</td>
<td>For all new DoD construction and major renovations, progressive collapse shall be considered for buildings of three or more stories.</td>
<td>Reference: “DoD Interim AT/FP Construction Standards: Guidance on Structural Requirements” (5 March 01) &amp; UFC 4-012-06, “Design of Buildings to Resist Progressive Collapse” (Draft 25 Feb 04).</td>
</tr>
<tr>
<td>American Society of Civil Engineers (ASCE)</td>
<td>ASCE 7-98, “American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures” (2002)</td>
<td>General discussion on reducing the potential for progressive collapse. No quantifiable or enforceable requirements provided.</td>
<td>Discusses two design alternatives for progressive collapse resistance: direct design (alternative load path, specific load resistance) and indirect design.</td>
</tr>
<tr>
<td>American Concrete Institute (ACI)</td>
<td>ACI 318. “Building Code Requirements for Structural Concrete” (2002)</td>
<td>Although progressive collapse is not explicitly considered, structural integrity reinforcement is required to improve redundancy and ductility.</td>
<td>For example, in cast-in-place construction provisions require a minimum amount of continuous top and bottom reinforcement in perimeter beams.</td>
</tr>
<tr>
<td>Stationary Office, England</td>
<td>“The Building Regulations” United Kingdom (UK) (1992)</td>
<td>Disproportionate collapse is required to be applied to all buildings of five or more stories.</td>
<td>Provides a three-tiered design approach that includes: providing effective tie forces, alternative load path method and specific local resistance.</td>
</tr>
</tbody>
</table>

In the U.S. the approach to preventing progressive collapse is not clearly defined. For example, the ASCE 7-02 provides a general discussion of progressive collapse in its commentary, but does not include any quantifiable or enforceable requirements.

Several government agencies such as the Department of Defense and the General Services Administration, on the other hand, have developed their own independent criteria. U.S. government facilities present highly visible targets for domestic or international terrorists, increasing the likelihood of a potential attack with explosives or other damaging means. For this reason, the DoD and the GSA have realized the need to address progressive collapse mitigation in a more comprehensive manner. Although some of the particular details are different, many of the concepts used in the DoD’s standards are similar to those of the GSA.

It is not the intent of this Structural Bulletin to provide an in-depth discussion of the design procedures presented in either the DoD standards or the GSA guidelines. For further information, the reader can refer to the documents listed in Table 1. The authors do, however, wish to highlight a number of the key ideas and general design philosophies from these documents.
Progressive Collapse Design & Analysis Procedures

The ASCE 7-02 standard describes two design approaches for dealing with progressive collapse: direct design and indirect design.

Design Applications

In the direct design approach, there is an explicit consideration of resistance to progressive collapse. Two methods representing the direct design approach are presented in the ASCE document: (1) the alternate path method, "a method that allows local failure to occur but seeks to provide alternate load paths so that the damage is absorbed and major collapse is averted"; and (2) the specific local resistance method, "a method that seeks to provide sufficient strength to resist failure from accidents or misuse." In the indirect design approach, progressive collapse mitigation is provided by ensuring minimum levels of strength, continuity, and ductility within the structural members and connections.

Currently, direct design, or specifically the alternate path method, is the most widely used approach for explicitly addressing progressive collapse. Both the DoD standard and the GSA guidelines are based on this procedure. It is interesting to note, however, that the DoD approach appears to be changing quite significantly. The draft of UFC 4-012-06, "Design of Buildings to Resist Progressive Collapse," uses an approach similar to the British Standard. In the British Standard, a combined approach of both direct and indirect design is used. Under the proposed new UFC, a combination of indirect design, the alternate path method, and the specific local resistance method would be used. The specific analysis method would ultimately depend on the level of protection desired and whether it is new or existing construction. The examples provided in this bulletin were designed using the alternate path method.

Aside from the more prescriptive design approaches mentioned, the ASCE commentary also provides discussion about "good" design practices that will result in a more robust structure. Examples include: good plan layout, returns on walls, changing directions of the floor slab span, load-bearing interior partitions, catenary action of floor slab, and beam action of walls. Consideration of these design practices encourages engineers to better understand how a structure will likely behave and predict the performance under an extreme event.

There is no consensus among those in the engineering community on the "best" approach for dealing with progressive collapse. The limited number of actual progressive collapse failures has made some engineers question if a national codified standard is even necessary. Most will agree, however, that incorporating "good" design practices, as described above, will enhance a building's performance under abnormal loading. Typically, these modifications can be achieved in cast-in-place monolithic reinforced concrete (RC) buildings with little impact on overall cost.

Removal of Structural Members

The minimum requirement of the DoD standards and GSA guidelines is to ensure a building can withstand the loss of one primary exterior vertical or horizontal load-bearing member without progressive collapse. Interior member removal is only required if a specific threat or risk exists—such as a parking garage being located beneath a building. The designer is required to go around the exterior perimeter of the building envelope removing a single member at a time and evaluating the effects. Selection of the appropriate members is dependent on the structural framing system used and can include columns, beams, bearing walls, or slabs.

At first, this process may appear overwhelming since most buildings have hundreds (if not thousands) of individual structural members (i.e., beams, columns, walls, etc.). However, for most typical structures, the number of cases that actually need to be evaluated is a small fraction of the total number of members. Careful selection of critical members and a good understanding of anticipated alternate load paths will help to greatly reduce the number of cases to consider. For example, on relatively simple layouts, the GSA limits the number of cases that must be checked. At each floor, the loss of a column shall be considered at the following locations: (1) near the middle of the building’s short side; (2) near the middle of the building’s long side; and (3) at the corner.

Although the speed at which a member is removed has no impact on a static analysis, it may have a significant impact on a dynamic analysis. The vertical member that is removed should be removed instantaneously, which in the case of a dynamic analysis should occur over a time period that is no more than 1/10 of the period associated with the structural response mode for the vertical member removal. Based on the GSA guidelines, columns are to be analytically removed directly below the joint (i.e., without damaging the joint) as illustrated in Figure 3. The initial damage assumed by this approach is a simplifying assumption that does not correspond to any specific threat (or abnormal load case).
Loading Criteria and Materials Properties

To avoid an overly conservative design, it is recognized that full live load is unlikely to occur at the same time as an extraordinary event. For this reason, the factored load combinations for design typically represent a realistic prediction of the actual loads on a structure.

The DoD standard, for example, requires that the following factored load combination be applied to the entire structure.

\[ P = 1.0D + 0.5L + 0.2W \]

where
- \( D \) = Dead Load
- \( L \) = Live Load
- \( W \) = Wind Load

On the resistance side of the equation, it is also important to model material strength as accurately as possible. In order to account for material over-strength as well as strain hardening, an over-strength factor is usually permitted. For RC structures the GSA document allows an increase factor of 1.25 for both concrete compressive strength and reinforcing steel ultimate tensile and yield strength. Although previous versions of the DoD standard only permitted a 10% increase for these properties, the draft version of UFC 4-012-06 (26 February 2004) follows the GSA guidelines.

Acceptance Criteria

The acceptance criteria specified in the draft proposal of the UFC 4-012-06 specifies that in determining the design strength (i.e., \( f_{c} \) and \( f_{v} \)) for RC members the nominal strengths shall be calculated in accordance with ACI 318-02. All \( f \)-factors are equal to 1.0 except for shear where \( f = 0.75 \). Maintaining \( f = 0.75 \) for shear is stipulated because shear failures are known to be sudden and occur in a brittle manner.

To limit the possibility of collapse, the DoD provides maximum allowable ductility and/or rotation limits for most structural members. These empirically determined damage criteria are for typical elements in conventional construction (i.e., construction that has not been hardened to resist abnormal loading). As illustrated in Table 2, the response limits for RC construction are dependent on the type of member being analyzed and the detailing of the reinforcing steel.

Once the model is analyzed with the proper member removed, the remaining members should be checked for both strength and deformation. The exact procedure will depend on whether a linear elastic or non-linear method is used for the analysis. In the DoD approach, if the shear capacity (or response limit) of a member is exceeded, the member is considered a failed member. Any dead and live loads associated with a failed member should be distributed to other members in the same story or to members in the story below. Where the load from the failed member is imparted to members in the story below, it must be increased to account for the dynamic impact of the falling debris.

The general philosophy for the GSA guidelines is similar to that of the DoD. One main difference, however, for linear elastic methods, is that the GSA uses the Demand-Capacity Ratio (DCR) concept found in seismic rehabilitation to assess the likelihood for progressive collapse. The analysis procedure prescribed in the GSA guidelines is similar to the “in-factor” method currently used in FEMA 273\(^{2}\) elastic methods.

Limits of Damage

As indicated by ASCE’s definition of progressive collapse, some “initial local failure” is acceptable. ASCE 7-02, however, does not clearly define the acceptable limit of “local failure.” Both the GSA guidelines and the DoD standard attempt to place a quantifiable limit on the maximum amount of damage that can be sustained under the instantaneous removal of a single load-bearing member. If the analysis indicates that the collapsed area will extend beyond these limitations, the structure is considered non-compliant and the design must be revised and the analysis repeated. The maximum allowable extent of collapse for an exterior threat shall be confined to the smaller of the following.

GSA requirements:
1. The structural bays directly associated with the removed vertical member in the floor directly above the removed member.
2. 167 m\(^{2} \) (1,800 ft\(^{2} \)) at the floor level directly above the removed vertical member.

![Figure 4: GSA Redesign Approach to Mitigate Progressive Collapse](image)

Original Design

Redesign - Option 1
(improvements distributed evenly over the entire height)

Redesign - Option 2
(improvements concentrated on the 2nd and 3rd floor level)
DoD requirements:
1. 70 m² (750 ft²) at the floor area directly above and directly below the removed member.
2. 15% at the floor area directly above and directly below the removed member.
3. In no case shall the predicted damage extend beyond the bays associated with the removed wall or column.

If the damage is analytically determined to extend beyond the above limits, the structure is considered to have a high potential for progressive collapse and must be redesigned. As indicated in Figure 4, there are a number of approaches for improving a structure’s ability to resist the required “missing beam” scenario. The most appropriate solution should also consider architectural constraints, constructability, and overall cost. Equally important, the modifications for progressive collapse should be compatible with the structural design for other hazards. For example, redesign Option 2 illustrated in Figure 4 may have a negative impact on seismic performance. If the frame in question is a moment frame, this local increase in beam size may create an undesirable “weak column-strong beam” condition. Although an analytically acceptable solution, concentrating the “progressive collapse resistance” at the bottom of a structure, as indicated in Option 2, may not be the most desirable alternative. Unlike the approach illustrated in Figure 3, damage sustained during an abnormal loading is not predictable and may propagate into the beam. If it occurs, local beam damage increases the potential for progressive collapse more significantly in the structural system illustrated in redesign Option 2 as compared to that in redesign Option 1.

Examples of Progressive Collapse Mitigation in Reinforced Concrete Structures

Designing to mitigate the potential for progressive collapse is a relatively new concept for the majority of engineers in the U.S. For any given building, there are various ways to achieve resistance to progressive collapse. The purpose of the following example projects is to illustrate different approaches for mitigating progressive collapse in reinforced concrete structures and to highlight a number of key design considerations.

Removal of Interior Load-Bearing Wall (“BASE Hanger System”)

Through extensive experience with Department of Defense projects, the authors have come to realize that the increased emphasis on creating security enhanced buildings has not come with a proportional increase in project funding. In light of this, the importance of selecting an efficient and economical structural system has become even more critical to ensuring a successful project.

In the authors’ opinion, progressive collapse should be considered as early in the design phase as possible. One excellent method of efficiently addressing the new progressive collapse requirements is through a “multi-hazard” approach. In this approach, a single structural system is selected that can resist the forces associated with various hazards (i.e., wind, seismic, progressive collapse, etc.). The following example illustrates an innovative application of this approach for a 5-story reinforced concrete dormitory.

The floor layout, typical of residential or dormitory-type construction, is shown in Figure 5. In plan, the structure is approximately 73 m (240 ft) long x 17.7 m (58 ft) wide. The building has a central corridor with living units on each side. The typical unit width and hence the distance between load-bearing walls is approximately 4.95 m (16.25 ft). The main structural system consists of 165 mm (6.5 in.) thick RC one-way and two-way slabs supported on 203 mm (8 in.) thick interior RC partition walls. Use of reinforced concrete for the primary load-resisting elements was a requirement of the Military Design of this project was required to conform to the Interim DoD AT/FP Construction Standards (16 December 1999) and the DoD Interim AT/FP Construction Standards, Progressive Collapse Design Guidance (4 April 2000). As previously discussed, the minimum progressive collapse standards are typically concerned with removal of external members. The criterion for this building, however, was even more stringent than the minimum standards. It specified the removal of all vertical load-bearing elements that are within 3 m (10 ft.) (i.e., the floor-to-floor height) from the exterior perimeter of the building. Load-bearing walls, this distance was also required to be no less than the distance to the first vertical construction joint. Progressive collapse mitigation is achieved through the use of load-bearing partition walls, referred to as the “BASE Hanger System”. The interior partition walls are divided into two distinct and separate vertical members, an “exterior” and an “interior” (See Figures 6 and 7). The separation between the two walls consists of a 60 mm (2 in.) weakened or “fuse” section of wall, 305 mm (12 in.)

<table>
<thead>
<tr>
<th>RC Beam</th>
<th>Ductility (φ)</th>
<th>Rotation (°)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC Beam One-Way Slabs (without tension membrane)</td>
<td>6°</td>
<td>6°</td>
<td></td>
</tr>
<tr>
<td>RC Beam One-Way Slabs (with tension membrane)</td>
<td>12°</td>
<td>6°</td>
<td></td>
</tr>
<tr>
<td>RC Beam Two-Way Slabs (without tension membrane)</td>
<td>6°</td>
<td>12°</td>
<td></td>
</tr>
<tr>
<td>RC Beam Two-Way Slabs (with tension membrane)</td>
<td>12°</td>
<td>12°</td>
<td></td>
</tr>
<tr>
<td>RC Columns&lt; (tension controls)</td>
<td>1</td>
<td>6°</td>
<td></td>
</tr>
<tr>
<td>RC Columns&gt; (compression controls)</td>
<td>2°</td>
<td>Sidesway ≤ H/25</td>
<td></td>
</tr>
</tbody>
</table>

Table 2:

*Provide shear stirrups per requirements of DAHSCWE Manual where rotation >2°.
of back-up wall and a physical 89 mm (3.5 in.) gap. At the top floor, however, the wall is continuous, connecting the exterior and interior elements. This top portion of wall is designed to act as a “hanger” and carry the walls below if the wall at any level is lost due to an explosion. The system is analogous to the stick-line support structure used in the word game “Hangman.”

Each building cross-section is composed of two identical hangers that are symmetrical about the corridor opening at the center of the building. Under normal service loads the “Hanger System” behaves as an ordinary RC load bearing wall with a vertical opening running from the foundation to the fifth floor. At the fifth floor the wall is solid, bridging the gap between the interior and exterior portions of wall. The walls are designed to support the gravity loads transmitted from the slabs as well as the lateral forces due to wind and earthquake loading. This structural configuration is fairly typical for these types of buildings.

The basic behavior of the “BASE Hanger System” is illustrated in Figures 7 and 8. Because the wall at the first level is removed, the vertical loads at the exterior of the building can no longer take the same path into the foundation. Instead, the loads are supported from the solid wall at the fifth level and follow the load path shown in Figure 7. Typical wall reinforcement consists of two layers of #13 @ 406 mm (#4 @ 16 in.) o.c. each way. Splices for the vertical
wall reinforcement are staggered at 1219 mm (4 ft.) o.c., and continuous hanger reinforcement (Dywidag Threadbars) and tie-off plates are added to help support the slabs and distribute the tension stresses introduced into the wall. At the very top of the wall an additional (2) #19 (#6) horizontal bars are added to resist the flexural stresses in the deep cantilever member.

In order to fully separate the interior and exterior portions of the wall, a physical 76 mm (3 in.) gap runs the height of the wall up to the fifth floor. In addition to the gap, a 610mm (2 ft.) long by 89mm (3.5 in.) wide section of expanded polystyrene is sandwiched within the concrete. The polystyrene is added to act as a “fuse” that helps dissipate the energy from the blast load before it reaches the interior wall (See Figures 9 and 10).

Although not required by the DoD Standards, the BASE Hanger System remains within the elastic range of behavior. A maximum vertical deflection of approximately 13 mm (0.5 in.) was calculated using cracked section properties of the concrete. In addition, the inherent redundancy in the “BASE Hanger System” actually allows more than one wall to be removed without progressive collapse. It is important to note that the security provided by the “BASE Hanger System” was achieved with a mere premium of roughly 3% of the overall construction cost of the structure. Furthermore, the “BASE Hanger System” did not have any impact on the architectural integrity of the project.
Removal of Exterior Member (RC Slab Strengthening)

The building structure for this second example is fairly similar to the dormitory building discussed in the previous section. For a number of reasons, however, the progressive collapse analysis and mitigation approach is significantly different in this example. First, based on the request for proposal (RFP) criteria, only an external explosive threat needed to be considered. Unlike with the “BASE Hanger System”, no portion of the interior transverse walls needed to be removed. This removal criterion is consistent with the latest version of the DoD Minimum AT/FP Standards (UFC 4-010-01). Secondly, because of architectural constraints, the walls at the front entrance to the building could not be made continuous to the foundation. These discontinuous walls are therefore supported on exterior columns (or narrow widths of wall). The progressive collapse review included an independent analysis for the removal of each of these columns.

Similar to the previous example, this dormitory structure is also five stories in height with overall plan dimensions of 56m (184 ft.) x 17m (56 ft.) (See Figure 11). The typical floor framing consists of 152mm (6 in.) thick cast-in-place RC slabs supported by 152mm (6 in.) thick load-bearing concrete partition walls.

At the ends of the building (last two bays), the slab thickness is increased to 165mm (6.5 in.) for progressive collapse mitigation. The main partition walls run in the transverse direction and are spaced at approximately 3.7 m (12 ft.). The RC walls in the longitudinal direction are located primarily along the corridor. Because of the modular layout of the units, the RC slab behaves primarily as a one-way system between the transverse walls and more as a two-way system at the central corridor.

The unique layout and architectural features of this building required the removal of (1) several columns at the front entrance and (2) portions of the load-bearing exterior walls at both ends of the structure. Because the bay spans in this building are relatively short, the floorslabs are capable of providing an alternate load path primarily through bending. As indicated in Figure 11, the amount of reinforcing steel in the end bays of the building needed to be increased in specific areas. For practical purposes, the slab thickness was also increased by 13 mm (0.5 in.) in these locations to accommodate the additional reinforcing steel.

After removal of an external column, the floor slabs are designed to span over the removed member without experiencing collapse (See Figures 12 and 13). The enhancements to the building are distributed uniformly over the height of the structure, similar to re design Option 1 in Figure 4. For the particular building geometry and progressive collapse criteria, this proved a very efficient solution. Yes, the floor slabs needed to be thickened and additional slab reinforcing steel added. However, this work was limited to isolated areas at the ends of the building and had negligible impact on the overall cost of the structure.

The typical slab reinforcement is #13 @ 203 mm (#4 @ 12 in.) o.c. each way top and bottom. The reinforcing steel is continuous in both directions and lap splices are staggered a minimum of 1217 mm (4 ft.). Continuity of the slab reinforcing steel is important for a number of reasons.

First, with continuous (or properly spliced) reinforcing steel and adequate anchorage into supports or adjacent members tensile membrane behavior can develop. It has been shown that the ultimate capacity and the response limit of RC slabs can be greatly enhanced by membrane action. Although tensile membrane behavior will most probably be accompanied by severe damage, this level of performance is acceptable if collapse is prevented.
Secondly, since an explosion produces a shock wave that propagates outward in all directions, floor slabs may be subject to an upward pressure due to blast loading. These upward forces load the slab in the opposite direction of typical gravity loads (See call-out box titled “Reinforcement Detailing for Progressive Collapse” on page 15). In order to account for this phenomenon, the current UFC 4-010-01 requires that all floors be designed “to withstand a net uplift equal to the dead load plus one-half the live load.”

Figures 14 and 15 are photographs of the additional slab reinforcing steel required at the ends of the building to satisfy the progressive collapse criteria. At the front of the building, the reinforcing steel is increased to #13 @ 203 mm (#4 @ 8 in.) o.c. each way, top and bottom. For the other three locations at the end bays, only the top bars perpendicular to the interior concrete walls are modified. In these locations the reinforcing steel is increased to #16 @ 178 mm (#5 @ 7 in.) o.c. over a width of approximately 6.1 m (20 ft.). The increased area of top steel is required to resist negative bending stresses that develop when the exterior end wall is removed and the floor slab must act as a cantilevered member.

Figure 13:
2-DIMENSIONAL ANALYSIS
MODEL OF SLAB BEHAVIOR

Figure 14:
CONTINUOUS SLAB REINFORCEMENT
FOR MITIGATION OF PROGRESSIVE
COLLAPSE AND LOAD REVERSAL DUE
TO BLAST

Figure 15:
ADDED SLAB REINFORCEMENT
FOR PREVENTION OF
PROGRESSIVE COLLAPSE
Existing Mid-Rise Building (Progressive Collapse and Seismic Design)

Buildings designed with an adequate level of continuity, redundancy, and/or energy dissipation capacity — typical of buildings in seismic regions — have an inherent ability to develop alternative load paths and better mitigate progressive collapse.

To demonstrate this point, the authors performed a study of a 12-story reinforced concrete moment frame building that was designed to the seismic provisions of the 1991 Uniform Building Code (UBC). It was demonstrated that based on the GSA criteria, progressive collapse is unlikely to occur with the prescribed column removal. The seismic detailing of the reinforcing steel in the exterior moment frames allows for load reversal in the beams spanning over the removed column. For a more detailed discussion, see the article entitled, “Preventing Progressive Collapse in Concrete Buildings,” Concrete International (November 2003).

An efficient and cost-effective structure can often be achieved by designing one system to simultaneously provide resistance to multiple hazards (i.e., seismic, wind, and progressive collapse). This concept is clearly illustrated in the aforementioned Concrete International article.

Another project completed by the authors, the major renovation of a 16-story mid-rise dormitory for the DoD, demonstrates how good seismic detailing in RC structures can also help mitigate progressive collapse. One of the requirements of UFC 4-010-01 is that the implementation of the AT/FP standards, including the mitigation of progressive collapse, is mandatory for “all DoD building renovations, modifications, repairs, and restorations where those costs exceed 50% of the replacement cost of the building.” As with seismic retrofits, upgrading an existing building for progressive collapse mitigation can be both a technically challenging and costly process. For this reason, it is important to model the as-built conditions of the existing structure as completely and accurately as possible. In many instances, a more refined analysis can be the difference between finding that a building meets the criteria versus recommending a costly and unnecessary retrofit.

A preliminary security analysis of the existing structure revealed a number of locations that appeared susceptible to progressive collapse. One area of concern was a narrow section of exterior load-bearing wall that occurs in approximately ten locations around the perimeter of the building (See Figure 16). In the case that this wall is removed, an alternate load path needs to be provided for the gravity loads previously supported by this member. One potential alternate load path that is readily apparent is through the RC floor slabs. If the RC slabs at each floor level can support the load over the removed wall as a cantilevered member; progressive collapse can be prevented. Unfortunately, the floor slabs are only designed to resist gravity loads and, therefore, do not have adequate strength or the proper detailing required to act as cantilevered members.

Upon review of these results, it appeared that some type of strengthening would be required to meet the progressive collapse
criteria. One proposal, as indicated in Figure 16, was to infill the existing opening between the exterior walls and the interior load-bearing walls with concrete. The new RC wall would be cast-in-place and tied to the existing concrete with epoxy dowels. In the case that a portion of the exterior wall is lost during an extreme event, the new concrete wall behaves as a deep cantilever member transferring any unsupported loads back into the interior wall. Although this is a technically sound option, the existing conditions and the magnitude of the retrofit made it a very difficult and costly solution.

As with most retrofit projects, one of the most difficult tasks is determining the details of the existing structural framing. A comprehensive review of the original structural drawings indicated a deep cast-in-place roof parapet around the perimeter of the building. Interestingly, in certain locations this beam was fairly large, 610mm (2 ft.) wide x 1829mm (6 ft.) deep, and heavily reinforced. Upon further investigation it was determined that this parapet was actually a "link" beam used to tie together adjacent concrete walls for lateral load resistance. Although not originally intended to resist progressive collapse, it was demonstrated through a 3-dimensional finite element analysis that this "link" beam provides an adequate alternate load path if the wall in question is removed (See Figure 16).

This project indirectly illustrates both the importance of "good" seismic detailing and the potential benefits of having a structural system that is capable of resisting multiple hazards. True, a "multi-hazard" approach that included progressive collapse was not considered for the original design of this building. However, the current progressive collapse analysis demonstrates the efficiency that can be achieved if the "multi-hazard" approach is used. Without the deep upturned roof beam at the perimeter of the building, an expensive and potentially intrusive retrofit would have been required to meet the progressive collapse criteria.

<table>
<thead>
<tr>
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</thead>
<tbody>
<tr>
<td>Mitigation Approach:</td>
<td>Option 1: Add interior walls solely for progressive collapse mitigation.</td>
</tr>
<tr>
<td></td>
<td>Option 2: Existing seismic link beam provides alternate load path.</td>
</tr>
<tr>
<td>Key Design Considerations:</td>
<td>Option 1: Labor intensive process to infill existing openings.</td>
</tr>
<tr>
<td>Cost/Aesthetic Impact:</td>
<td>Option 2: Define existing conditions and provide refined analysis.</td>
</tr>
<tr>
<td></td>
<td>Option 1: Roughly $750,000/infill existing openings.</td>
</tr>
<tr>
<td></td>
<td>Option 2: No cost impact/ No aesthetic impact.</td>
</tr>
</tbody>
</table>

**Figure 17:** EXISTING ALTERNATE LOAD PATH FOR MISSING WALL SCENARIO
### Table 3:

<table>
<thead>
<tr>
<th>Member</th>
<th>Section</th>
<th>Reinforcement Requirements</th>
<th>Splice Location and Anchorage Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joist</td>
<td>7.13.2.1</td>
<td>One continuous bottom bar.</td>
<td>Splice steel at or near support. Provide standard hook at non-continuous support.</td>
</tr>
<tr>
<td>Perimeter Beam</td>
<td>7.13.2.2 &amp; 7.13.2.3</td>
<td>Continuous top bars; 1/6 of steel required for negative moment at support (2 bars min.)</td>
<td>Splice steel at mid-span. Provide standard hook at non-continuous support.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Continuous bottom bars; 1/4 of steel required for positive moment at mid-span (2 bars min.)</td>
<td>Splice steel at or near support. Provide standard hook at non-continuous support and at change in beam depth.</td>
</tr>
<tr>
<td></td>
<td>7.13.2.4</td>
<td>All continuity steel shall be enclosed by U-stirrups or one-piece closed stirrups.</td>
<td>Splice steel at or near support. Provide standard hook at non-continuous support and at change in beam depth.</td>
</tr>
<tr>
<td></td>
<td>7.13.2.5</td>
<td>All bottom bars with column strip in ea. direction shall be continuous. At least 2 bars shall pass through column.</td>
<td>Splice steel at or near support per Fig 13.3.8. Provide standard hook at non-continuous support.</td>
</tr>
</tbody>
</table>

*All splices to provide continuity shall be Class A tension lap splices, or mechanical or welded splices satisfying 12.14.3.*

### Detailing for Structural Integrity

For the design of any RC structure, engineers should be aware of the concepts of redundancy, continuity, and energy dissipation.

Although the examples presented in this Bulletin describe Department of Defense projects, it is important to note that the general design philosophies are applicable to all buildings. For the design of any RC structure, engineers should be aware of the concepts of redundancy, continuity, and energy dissipation. Although not as explicit as in the DoD standard or the GSA guidelines, ACI 318 does address the prevention of progressive collapse.

ACI 318 includes provisions for enhancing the overall integrity of a structure under unforeseen loading conditions. Section 7.13 - Requirements for structural integrity of ACI 318-02 represents an in direct approach to dealing with progressive collapse. This is emphasized in the Commentary to Section 7.13 that states, “It is the intent of this section of the code to improve the redundancy and ductility in structures so that in the event of damage to a major supporting element or an abnormal loading event, the resulting damage may be confined to a relatively small area...”

The provisions of Section 7.13 provide a number of rather simple detailing requirements for the reinforcing steel that help tie the structure together to provide resistance to progressive collapse. In the 2002 code, the use of mechanical or welded splices for splicing reinforcement is now permitted to meet the structural integrity requirements. Also in 2002 the provisions for the stirrups enclosing the continuity steel were revised. The code requires either U-stirrups with no less than 135-deg hooks around the continuous top steel or one-piece closed stirrups. This change was adopted to eliminate the use of two-piece closed stirrups because the top crosstie is ineffective in restraining the top continuous bars from pulling out of the top of the beam. Table 3 and Figure 18 provide a summary of the ACI requirements for structural integrity.
Reinforcement Detailing for Progressive Collapse

One key aspect in designing for progressive collapse resistance in reinforced concrete structures is the proper structural detailing of reinforcing steel. Since this design deals with extreme events, it is often assumed that the structure will deform well into the inelastic range. Special consideration must be given to reinforcement continuity, lap slices and anchorage so that the assumed alternate load paths are indeed provided. Continuity of slab reinforcement is an important aspect not only when designing for progressive collapse, but also when designing for the upward pressures occurring due to a blast scenario.

**Detail for Gravity Load**

- **Gravity Loads**
  - Neg. Moment Region
  - Pos. Moment Region
  - Placement of long rebar follows moment distribution

  *Note: Structural integrity steel as may be req’d by ACI 318 not shown.

**Detail for Continuity and Progressive Collapse**

- Neg. Moment Region
- Closely spaced stirrups
- Pos. Moment Region
- Cont. top & bot. rebar

Alternate Load Path **Not Provided**

- Neg. Moment Region
- Increased neg. moment as member must span 2 bays
- Loss of primary support
- Lack of cont. Bot. rebar
- Potential for sudden, brittle failure

Alternate Load Path **Provided**

- Neg. Moment Region
- Closely spaced stirrups enhance ductility
- Pos. Moment Region
- Loss of primary support
- Cont. Bot. rebar provides pos. moment capacity

**Blast Loading**

- Neg. Moment Region
- No rebar to resist load reversal
- Blast

Member **Cannot** Resist Upward Forces

- Neg. Moment Region
- Cont. rebar resists load reversal

Member **Can** Resist Upward Forces

- Neg. Moment Region
- Closely spaced stirrups enhance ductility
- Blast

- Blast

- Blast
Conclusions

The existing standards and guidelines for the mitigation of progressive collapse discussed in this Bulletin provide good tools for dealing with typical, fairly "regular" buildings. Unfortunately, many of today's buildings are not very "regular" and may be difficult to directly analyze with these documents. Furthermore, there may be some disagreement among members of the engineering community as to the "exact" methodology that should be adopted for progressive collapse. With regard to differing opinion, no matter what approach is used, basic concepts such as continuity, redundancy, and energy dissipation are key properties that will enhance the performance of any building. Understanding these concepts will help designers develop a confidence for how a building behaves. Although the authors understand the importance of sophisticated computer software, they also believe that the most powerful computer is no substitute for having a "good engineering understanding" of structural behavior.

As illustrated in the referenced examples, the monolithic nature of cast-in-place reinforced concrete structures makes the material inherently well suited for preventing progressive collapse. Typically, enhanced continuity, redundancy, and energy dissipation can all be readily achieved with simple modifications and attention to structural detailing of reinforcement. Progressive collapse is a design criterion that should be considered as early as possible in the design process.

A good understanding of concepts, such as alternate load paths, will help in the selection and configuration of the structural system. If properly selected, the mitigation of progressive collapse in RC structures does not significantly impact the overall cost of a project.

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ACI 318, Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, Farmington Hills, MI, 1989.

Unified Facilities Criteria (UFC) 4-010-01, Department of Defense Minimum Antiterrorism Standards for Buildings, Department of Defense, October 2003.


ACI 318, Building Code Requirements for Structural Concrete, American Concrete Institute, Farmington Hills, MI, 2002.


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