Design of load-bearing precast concrete buildings to resist progressive collapse

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- This paper summarizes the PCI report "Design of Precast Buildings to Resist Progressive Collapse: Load-Bearing Buildings," which describes design requirements by U.S. government agencies to resist progressive collapse with an illustrative example design for a building with precast concrete load-bearing components.
- The PCI report provides guidance for designing precast concrete buildings to resist progressive collapse. Such guidance, or example problems, are not provided in current U.S. government documents for design to resist progressive collapse.
- The example uses both the tie force method and the alternate path method for progressive-collapse-resistant design.

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Progressive collapse occurs when the localized failure of one component in a building initiates cascading failure of adjacent components, potentially resulting in the collapse of the entire structure or at least a disproportionately large part of the structure. Design to resist progressive collapse, also referred to as disproportionate collapse, requires improved connectivity and continuity between components so that the components in a building that surround a failed supporting component can redistribute loads without failing themselves, though they may sustain heavy damage in doing so.

Government agencies in the United States have developed detailed design criteria intended to prevent progressive collapse in building structures, particularly those occupied by federal, military, or other high-profile tenants. Design to resist progressive collapse is briefly addressed in U.S. and international building design codes; however, the pertinent discussion in these documents is only part of the "conventional" design and does not include separate specified requirements like those published by U.S. government agencies. The U.S. Department of Defense (DoD), U.S. Department of Veterans Affairs, and General Services Administration (GSA) have developed similar, but not identical, design requirements that are written into the specifications for new buildings and building retrofits that are owned and/or occupied by these agencies. These design guidance documents pertain to the design of new buildings as well as buildings undergoing upgrades to resist progressive collapse. Most design guidance and supporting research has focused on cast-in-place reinforced concrete and structural-steel-framed buildings that can inherently provide continuity between components with relatively minor changes to conventional detailing and connections (for example, lap splicing reinforcement and welded moment connections).

On the other hand, precast concrete structures, which typically consist of discrete components with limited connectivity, have rarely been used for buildings that are required to resist progressive collapse; however, momentum has been building toward their implementation in this market area over the past decade. For example, recent research efforts have examined progressive-collapse-resistant design solutions for segmented load-bearing concrete wall systems[1–3] as well as precast concrete frames[4–8] and floor systems.[9] In practice, segmented tilt-up load-bearing concrete walls were recently used to meet progressive collapse requirements for several low-rise government office buildings at lower cost and greater construction efficiency compared with other design options.[1,10] Despite these positive developments, little formalized design guidance has been established to date for the design of collapse-resistant precast concrete structures. Currently, the implementation of precast concrete for progressive collapse resistance relies on customized solutions that require higher levels of engineering design effort and special detailing. To improve design guidance for progressive collapse resistance in precast concrete structures, PCI recently funded a study to develop design guidance that will enable the use of precast concrete in this growing segment of the building construction industry. The resulting PCI report,[11] "Design of Precast Buildings to Resist Progressive Collapse: Load-Bearing Buildings," presents an overview of the current U.S. government criteria to design buildings to resist progressive collapse and provides a design example for a building with load-bearing precast concrete walls and framing. This paper provides a summary of the PCI report and highlights its design guidance for resisting progressive collapse.

Design methodologies

The three most prevalent design methods that are used in practice to resist progressive collapse are described in **Table 1**: tie forces, alternate path, and enhanced local resistance. The tie force and enhanced local resistance methods are indirect approaches that enhance the continuity, ductility, and structural redundancy of a structure such that it has an increased resistance to collapse without the consideration of any defined damage scenarios. The tie force method was primarily developed in response to the partial progressive collapse of an apartment building in 1968 at Ronan Point in the United Kingdom.[12] This method requires that the elements of the structure be mechanically tied together with tensile steel reinforcement designed to resist specified tie forces. The enhanced local resistance method ensures ductile response of key load-bearing components by requiring that they have adequate connection and shear capacity to withstand the reactions caused by a plastic flexural mechanism. Therefore these components are not expected to undergo a brittle shear or connection failure. The alternate path method is a direct design approach that involves targeted removal of load-bearing elements that are assumed to be damaged or failed due to an extreme load. It is based on methodologies first proposed by Ellingwood and Leyendecker,[13] among others, in response to the progressive collapse of the Alfred P. Murrah Federal Building following the 1995 Oklahoma City bombing.[14] When using this method, the structure is subjected to the instantaneous removal of a column or a section of load-bearing wall at a single floor level, one at a time at several critical locations, and the subsequent response is then calculated to determine whether the structure can bridge across the removed element and avoid collapse.

Table 2 summarizes the use of these methods to resist progressive collapse per design criteria issued by several U.S. government agencies: the U.S. Department of Veteran's Affairs' *Physical Security Design Manual for VA Mission Critical Facilities*[15] and *Physical Security Design Manual for VA Life-Safety Protected Facilities,*[16] the GSA's *Alternate Path*

Table 1. Blast design methods to resist progressive collapse

Design method	Description
Tie forces	The floor and roof slabs must have continuous tension tie elements in both in-plane directions and around their perimeter (as well as the perimeter of large openings), which are designed to resist prescribed tension forces. The orthogonal and perimeter tie elements must be well connected to transfer tension forces. Also, the longitudinal steel in the columns must resist prescribed tension forces and the floor slabs must be connected so they can hang from columns or beams above if they lose support from below.
Alternate path	The building is explicitly designed so that no part of the building will collapse if a single load-bearing component is removed. The prescribed extent of removal (columns, load-bearing walls, and so forth, at selected story heights) can vary between criteria documents.
Enhanced local resistance	Key load-bearing components (such as first-floor columns near corners of buildings) are designed to be ductile such that their shear capacity is adequate to develop a fully plastic flexural response against lateral loads (such as blast or impact). This approach assumes that the postyield behavior of the component is controlled by flexural rather than shear response.

Table 2. Overview of progressive collapse design methods for selected government agencies

Government agency	Design documents	Overview of design methods*
U.S. Department of Defense	UFC 4-023-03	• Risk factors are established per UFC 3-301-01. • No progressive collapse design for lowest risk level. • Tie force or alternate path for buildings with intermediate risk factors. • Both tie force and alternate path for buildings with highest risk factor. • Enhanced local resistance for critical components at all risk levels requiring progressive collapse design.
U.S. Department of Veterans Affairs	*Physical Security Design Manual for VA Mission Critical Facilities* and *Physical Security Design Manual for VA Life-Safety Protected Facilities*	• Progressive collapse resistance required only for mission critical and life-safety protected buildings. • Blast-resistant design always required for load-bearing components on exterior and in unscreened areas of mission critical and life-safety protected buildings. • Tie force required for all life-safety protected buildings and two-story mission critical buildings. • Tie force, alternate path, and enhanced local resistance required for mission critical buildings with three or more floors.
GSA	*Alternate Path Analysis & Design Guidelines for Progressive Collapse Resistance*	• GSA buildings are assigned an FSL. • Progressive collapse design is not required for FSL I and II buildings and FSL III and IV buildings with fewer than four floors. • Otherwise, alternate path design is required. All FSL V buildings require alternate path design.†

Note: FSL = facility security level; GSA = General Services Administration.

* Progressive collapse design is generally required for key exterior loading-bearing components and interior loading-bearing components in unscreened areas. Buildings with the highest risk or importance also generally require progressive collapse design for all interior columns.

† The GSA requires redundancy design procedures in conjunction with alternate path design for all FSL V buildings. Redundancy design procedures are satisfied if each collapse-resisting floor of a building is designed to resist its own floor loads plus a maximum of loads from two (or in some cases three) additional floors over the removed column in the alternate path design.

Analysis & Design Guidelines for Progressive Collapse Resistance,[17] and the DoD's *Unified Facilities Criteria (UFC): Design of Buildings to Resist Progressive Collapse* (UFC 4-023-03).[18] Blast-resistant design of key components may also be required for high-risk unscreened areas of buildings (for example, spaces accessible to the public or areas that receive packages prior to screening with a metal detector or similar device), such as lobbies, loading docks, and mailrooms in important buildings. The extent to which these design methods are applied to a building can vary based on the importance, risk, and occupancy categories of the building.

Example building

The example building was a five-story precast concrete structure with load-bearing precast concrete exterior walls, hollow-core precast concrete floor planks, and interior precast concrete framing (beams and columns). This example building was based on information and drawings provided by Enterprise Properties Inc. in Omaha, Neb. **Figure 1** shows an isometric elevation of the building, and **Fig. 2** shows the basic floor plan. The clear floor height was conservatively taken as 12 ft (3.7 m) based on the maximum floor height in the building. The building in the PCI report[11] was designed to resist progressive collapse per UFC 4-023-03[18] (that is, as a DoD facility) assuming Risk Category II, which is the most common risk category for multistory DoD buildings. Either tie force or alternate path design could be used for this risk category. Enhanced local resistance was also required for critical load-bearing components, particularly the load-bearing walls at the corners and at the bays directly adjacent to the corners. The building did not have any interior unscreened areas. In the PCI report,[11] the building

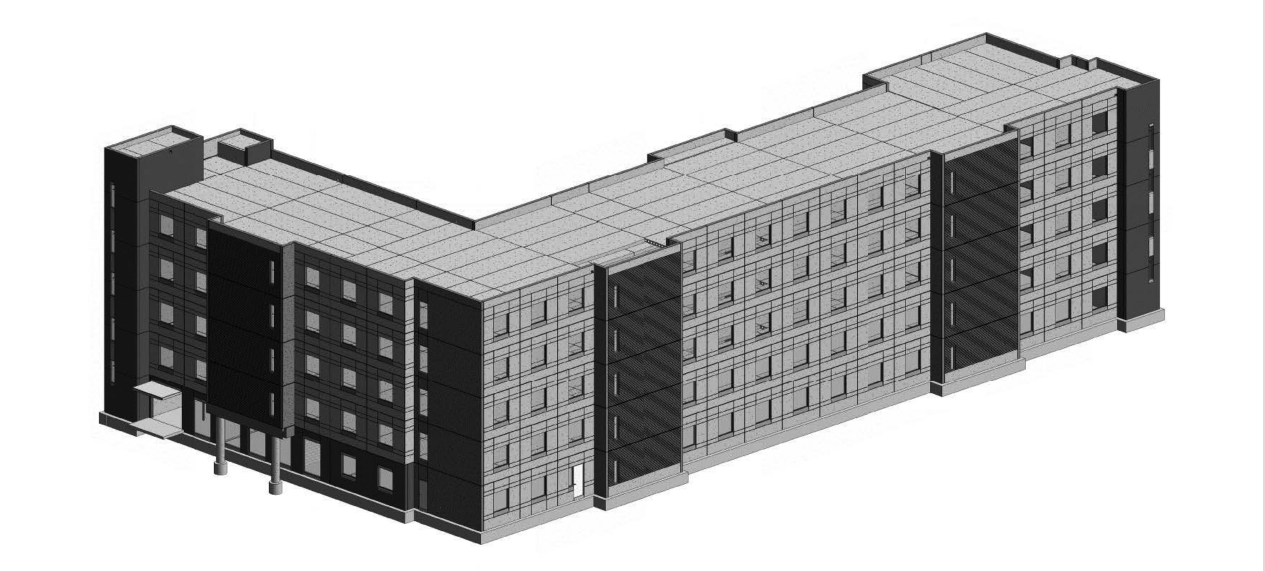

Figure 1. Isometric elevation of example building.

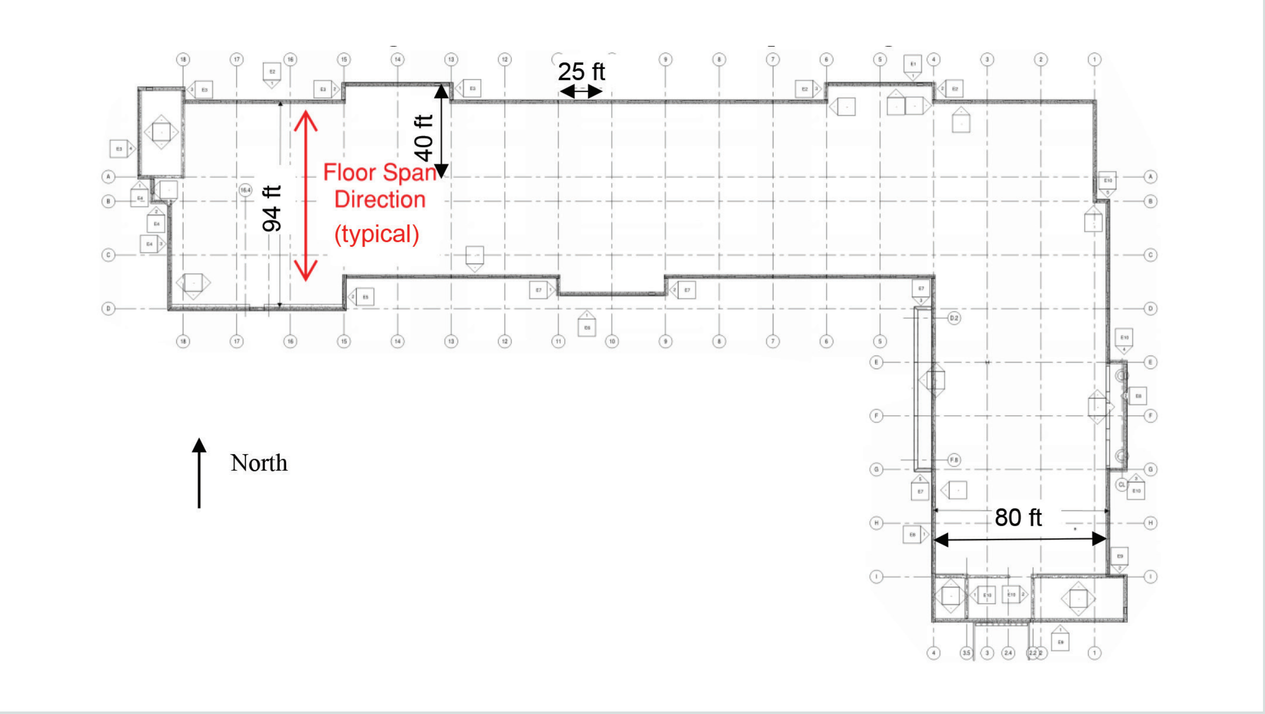

Figure 2. Plan view of example building. Note: 1 ft = 0.305 m.

was designed using all three methods. Only significant parts of the design are included in this paper with limited discussion to provide an overview of the design methods. The PCI report includes a more complete discussion of requirements for each progressive collapse design method, as well as detailed design equations for the example building.

Tie force design

Tie force design is an indirect design approach that enhances continuity, ductility, and structural redundancy of a building by requiring that continuous, well-anchored tensile elements called ties be placed between the structural elements. This

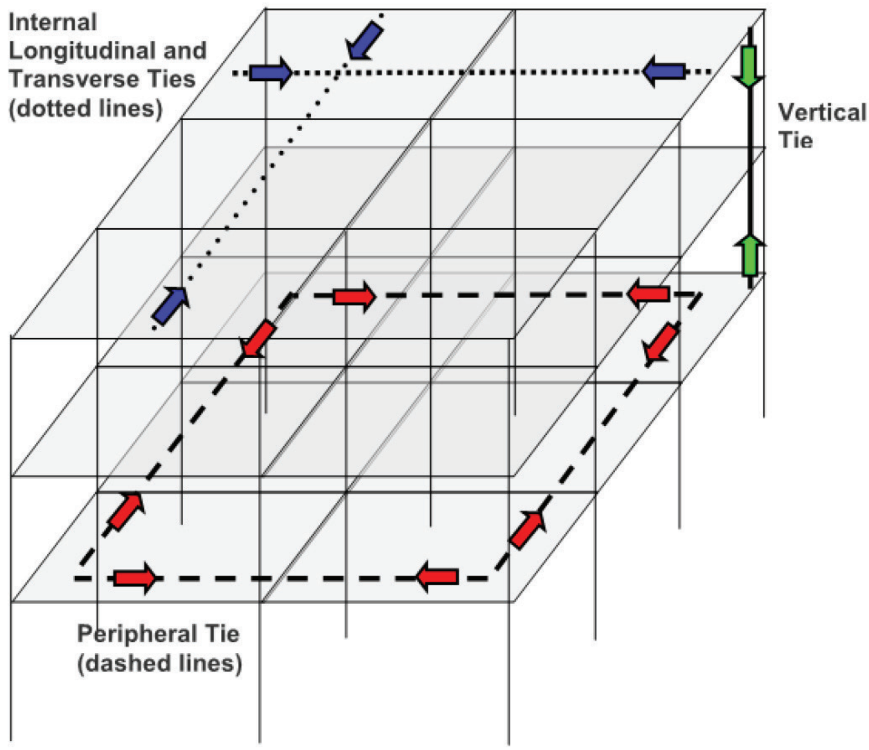

Figure 3. Typical tie types considered in tie force design. Source: Reproduced from Department of Defense (2016, Fig. 3-1).

provides alternate load paths into surrounding load-bearing components for gravity loads that were carried by the failed load-bearing component. There are generally three types of ties in a tie force design: internal ties, peripheral ties, and vertical ties (**Fig. 3**). As a first step in this process, the example structure must meet the following two requirements to allow the use of tie force methods to meet the progressive collapse design requirements per the UFC (otherwise, only the alternate path method can be used):

- For one-way load-bearing wall structures, the number of bays in the one-way span direction must be four or greater, where a bay is defined as the square or rectangular floor area with boundaries demarked by vertical load-bearing elements, such as columns at the corners or load-bearing walls along the edges.
- The length of load-bearing walls in the direction transverse to the one-way span direction must be at least $4h_w$, where h_w is equal to the structure's clear story height.

The example building meets these requirements. Note that in many cases, such as enlisted barracks, small office buildings, and other similar structures, there are typically only three bays in the one-way spanning direction due to supports at the exterior walls as well as on each side of a single interior corridor. These structures must therefore be designed to resist progressive collapse with the alternate path method.

All three types of ties illustrated in Fig. 3 are designed to resist tension tie forces that are calculated based on the design floor load w_F and the span lengths between supporting vertical load-bearing components. The internal longitudinal ties in both directions and peripheral ties are intended to carry floor loads over a failed load-bearing component to adjacent load-bearing components. Vertical ties are intended to vertically distribute loads to floors above to alleviate excessive redistribution within the same floor level.

The tie forces for the internal and peripheral ties were calculated using the equations shown in **Table 3** for each type of tie force. These equations include an empirical factor that enables the tie elements to develop sufficient tension membrane forces to resist the design floor loads with an increased span length if a supporting load-bearing component fails. The ties must have an area of reinforcement (or other ductile material) with a nominal strength per load-and-resistance-factor-based design to resist the design tie forces in Table 3. The nominal strength may be increased with a material overstrength factor from the American Society of Civil Engineers' *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE/SEI [Structural Engineering Institute] 41-13),[19] which for reinforcing bars is 1.25. Including a strength reduction factor of 0.75 per UFC 4-023-03,[18] the design yield strength for Grade 60 reinforcing steel is 56 ksi. The calculated areas of reinforcement required for each type of tie force in the example building are shown

Tie force	Design equation	Required reinforcing steel area	Comment
Internal longitudinal tie force F_L	$F_L = 3w_F L_L$	0.31 in.^{2}/ft	L_L = maximum longitudinal span length of floor panels between supports; different parts of the building may have different values of L_L and F_L = 40 ft (maximum)
Internal transverse tie force F_T	$F_T = 3w_F L_T$	0.46 in.^{2}/ft	L_T = transverse span length of floor panels, equal to the lesser of total building width in transverse direction or $5h_w$ for internal ties in the transverse direction = 60 ft
Perimeter peripheral tie force in longitudinal direction F_{pL}	$F_{pL} = 6w_F L_1 L_p + 3(W_{cL})$	4.3 in.^{2}	L_1 = the greatest distance between the centers of supports in the longitudinal direction = 40 ft W_{cL} = self-weight of panels in longitudinal direction = 0 because panels in longitudinal direction are not load-bearing (that is, are not removed) and support their own weight
Perimeter peripheral tie force in transverse direction F_{pL}	$F_{pL} = 6w_F L_2 L_p + 3(W_{cT})$	2.7 in.^{2}	L_2 = span length for peripheral ties over load-bearing walls in transverse direction for floors with one-way spans = $2h_w$ = 25 ft L_p = width of perimeter tie = 3.3 ft W_{cT} = self-weight of panels in transverse direction = 80 lb/ft because perimeter force must support panel weight due to removed panel below
Vertical tie force F_p	$F_p = w_F A_T$	1.8 in.^{2}	A_T = tributary area supported by a load-bearing wall panel with tension tie = 500 ft^2 (maximum)

Note: h_w = clear floor height = 12 ft (maximum); w_F = design floor load for tie force method;. 1 ft = 0.305 m; 1 in.^{2} = 645.2 mm^2; 1 ft^2 = 0.093 m^2; 1 lb/ft = 14.6 N/m.

in Table 3 for the design floor load w_F, which is calculated using the following equation.[20]

$$w_F = 1.2D + 0.5L = 1.2(100 \text{ lb/ft}^2) + 0.5(50 \text{ lb/ft}^2)$$

$$= 145 \text{ lb/ft}^2 \ (6.9 \text{ kN/m}^2)$$

where

D = dead load = 100 lb/ft^2 (4.8 kN/m^2)

L = floor live load = 50 lb/ft^2 (2.4 kN/m^2)

Note that the live load may be multiplied by a live-load reduction factor, which is conservatively neglected here because it only provides a 5% reduction in the total design load for this building.

The internal longitudinal and transverse ties and peripheral ties that provide the required steel areas of reinforcement in Table 3 must be continuous within the floor and roof system. This includes full tension splices of reinforcement and a positive connection between internal reinforcing bar ties in both directions and the peripheral reinforcing bar ties. The structural design to resist conventional loads can also be used to meet the design requirements to resist progressive collapse as long as these detailing requirements are met. The wall panels and interior columns must have a continuous system of vertical ties in the wall panels from the roof down to the second floor. These ties can be provided with a combination of reinforcement in the wall panels and discrete connections between the wall panels and the floor/roof systems. Vertical tie forces for the interior of the building are typically resisted by the reinforcement in the columns, which must have an adequate connection to the floor system.

Figure 4 shows peripheral ties in a floor system as green lines and internal ties as red lines. The internal ties must be continuous, satisfy restrictions for allowable splices, meet maximum spacing requirements, and be capable of transferring load to peripheral ties. Type 2 mechanical splices may be used at any location for reinforcement resisting tie forces. Peripheral ties are required around all large openings in the floor system, such as openings for stairs and elevator shafts, as illustrated in Fig. 4. The peripheral ties should be placed within a 3.3 ft (1.0 m) width near the perimeter of the floor system, but not directly over floor beams unless those elements are shown to be ductile per testing or analysis. Therefore, the reinforcement acting as peripheral ties is typically concentrated in a 3.3 ft

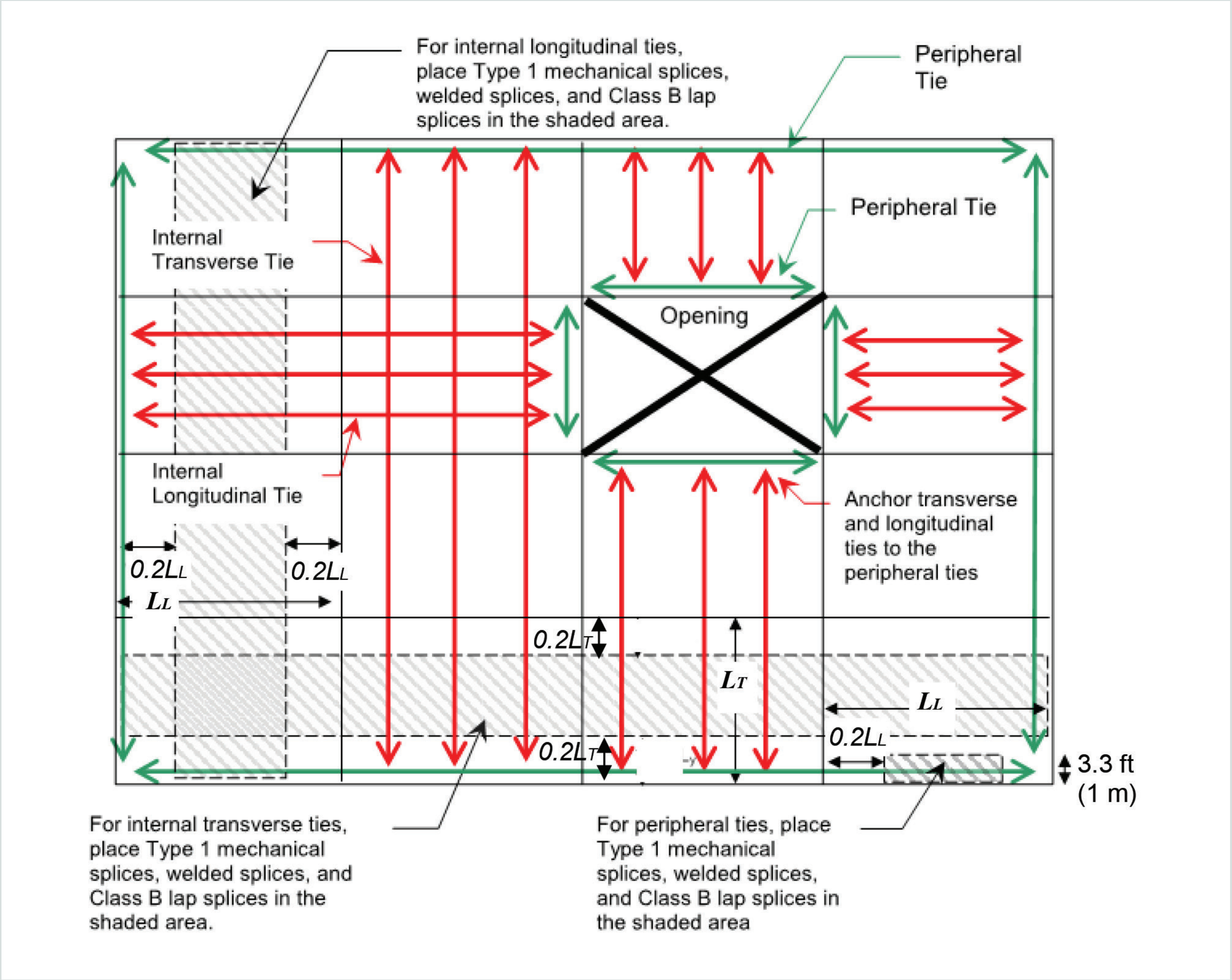

Figure 4. Internal and peripheral ties in a floor system with a large opening.
Source: Adapted from Department of Defense (2016, Fig. 3-2).
Note: Shaded areas show permissible areas for splices if needed. L_L = greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the longitudinal direction; L_T = greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the transverse direction.

width adjacent to the perimeter floor beams. At reentrant corners, or at substantial changes in construction such as large openings in the slab for stairwells, special attention to detailing is required to ensure that the transverse, longitudinal, and peripheral ties are adequately anchored and developed.

Precast concrete floor and roof systems pose a challenge for meeting the continuity and splicing requirements for internal ties. Reinforcement within precast concrete planks may be used to provide the internal tension ties only if the reinforcement is continuous across the whole floor system in both directions and properly anchored to the peripheral ties. The left side of **Fig. 5** shows a concept to provide internal tension tie reinforcement in both directions within a cast-in-place concrete topping with positive mechanical engagement required between the topping reinforcement and the precast concrete floor system. The mechanical attachment of the precast concrete to the topping must have sufficient strength to ensure that the precast concrete units do not separate from the topping and fall into the space below when engaged to resist local damage. The bond strength between the topping and precast concrete units is not sufficient because it can be disrupted by the large deformations associated with tension membrane response. Ties cannot be cast into the floor planks with short cutout sections for lap splice bars because this approach does not typically enable contact splices between continuous reinforcement as required for internal tie reinforcement. The internal tension tie elements in the floor system must be fully developed into the peripheral ties around the perimeter of the floor system. For reinforcing bars, this consists of a seismic hook, as shown in the right side of Fig. 5, or a similar connection. **Figure 6** shows concepts for developing continuity of vertical ties in precast concrete wall panels.

Enhanced local resistance

The tie force method is based on the development of catenary action in building floor and roof systems, which is less likely

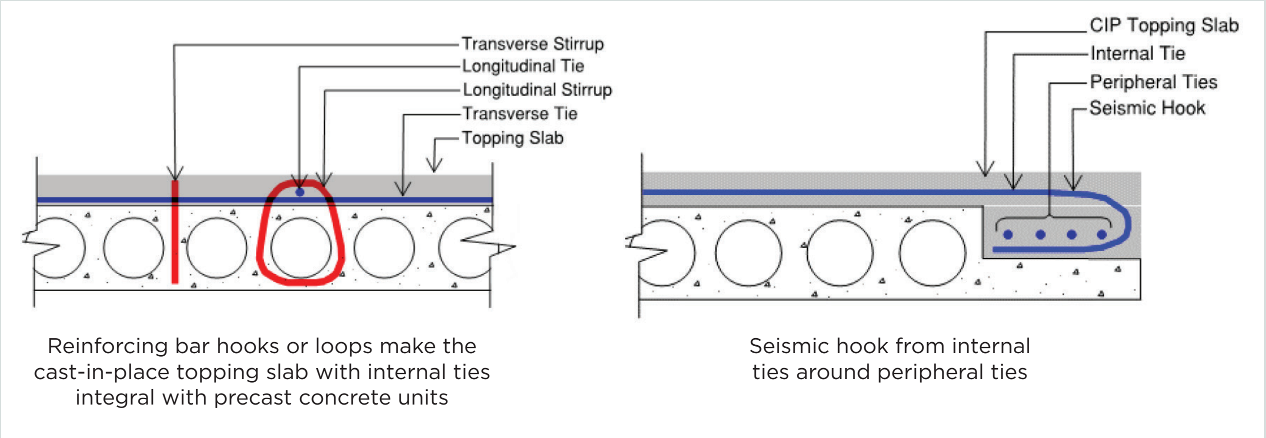

Figure 5. Concepts for placement of internal tension tie reinforcing bar and peripheral ties in precast concrete floor and roof systems. Note: CIP = cast-in-place.

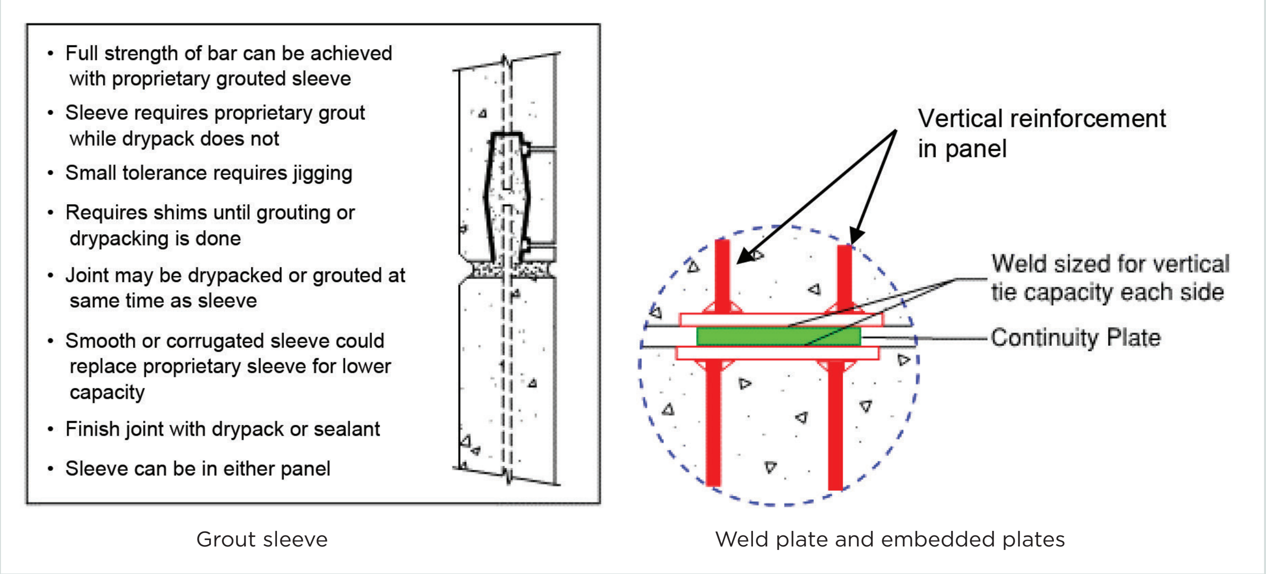

Figure 6. Continuous vertical ties in precast concrete wall panels.

to develop if a load-bearing component at or near the corner of a building fails. Therefore, DoD and U.S. Department of Veteran's Affairs criteria require that designated critical first-floor load-bearing components be designed to have enhanced local resistance. For the example building, the critical components are the load-bearing walls at the corner and directly adjacent to the corner. Enhanced local resistance design mandates that these critical components have a ductile response mechanism if overloaded by lateral, out-of-plane loading perpendicular to the building perimeter facade. They must be designed to yield at all maximum moment regions in flexure (that is, form a mechanism) before they fail in shear when subjected to a uniformly applied lateral load. The uniform lateral load that causes the flexural mechanism w_{lf} (that is, the lateral failure load) is based on the expected ultimate moment capacity of the critical member; this value should conservatively include the effect of axial load, material overstrength factors, the plastic section modulus, and boundary conditions. The shear capacity is based on a load- and resistance-factor design (LRFD) response that includes strength reduction factors and expected material strengths (that is, minimum specified strengths multiplied by material overstrength factors from ASCE/SEI 41-13[19]).

The calculated ultimate moment capacity of the typical corner wall panel at the first floor against uniform lateral load is approximately 15 kip-ft/ft (68 kN-m/m). This allows for a w_{lf} equal to 10 kip/ft (146 kN/m) of width along the 12 ft (3.7 m) tall panel. Assuming symmetric boundary conditions at the top and bottom of the wall, this force induces a 5 kip/ft (73 kN/m) reaction along each supported edge. The shear capacity of the wall panel is 10 kip/ft per ACI 318-14[21] considering axial effects and the expected concrete strength. Therefore, no additional reinforcement or special detailing of the first-floor corner and adjacent wall panels is required for compliance with the enhanced local resistance requirements. However, the connections of these panels to the foundation and to the second-floor diaphragm must be designed to resist the 5 kip/ft reaction.

Alternate path design

Alternate path design, also referred to as alternate load path analysis, requires that the building be designed to bridge over removed critical load-bearing elements, such as columns and prescribed widths of load-bearing walls. Critical locations for removal of load-bearing components in plan view include the exterior of the building, particularly corners, and interior locations of the building that house unscreened space. Load-bearing components are removed over their clear vertical span between lateral supports. Load-bearing walls are removed over a width equal to two times the clear floor height $2h_w$. At corners, a width equal to the clear floor height h_w is removed in each direction. Additional cases could be required for reentrant corners and other atypical areas. In elevation view, the DoD design guidelines[18,22,23] require that load-bearing components be removed at each floor level in the following list for Risk Category II:

- first story above grade
- story directly below roof
- story at midheight
- story above the location of a change in wall size

The intent of these requirements is to preclude a design where all the floors are supported by very strong components at only one floor. In general, engineering judgment should be exercised to determine the number of load-bearing-component removal scenarios that are required.

There are four typical methods for alternate path analysis, and they require varying levels of computational complexity.[24]

- rational analysis (may be based on hand calculations or spreadsheet calculations)
- linear static finite element analysis modeling
- nonlinear static finite element analysis modeling
- nonlinear dynamic finite element analysis modeling

Generally, nonlinear dynamic analysis will result in the most economic design, especially for more complex structures, and require the largest effort and finite element analysis modeling expertise. Linear static finite element analysis modeling and rational analysis may only be used if the building is not irregular per criteria defined in the controlling progressive collapse design method, which is similar to that for earthquake-resistant design. All analysis methods that use finite element analyses must use three-dimensional models of the building. For simple frame structures and walled structures with uniform and regular load-bearing wall layouts, design for progressive collapse using rational analysis with hand calculations or spreadsheet applications may be appropriate and more efficient than finite element analysis modeling. The rational analysis can include tension membrane response of properly detailed and laterally supported beams and/or slab elements over a removed column. Also, rational analysis may require an independent third-party review.

For the example building, wall panels at each floor above the removed wall panel are designed as beams spanning over the removed section of the wall panel below. The worst-case scenario for the panel design with this approach occurs when the fourth-floor wall panel is removed and the fifth-floor wall panel above it must support the tributary loads from both the fifth floor and the roof. In another scenario where the fifth floor wall panel is removed, a new beam must be added to support the roof. Contributions from the new beam are conservatively neglected for any case of wall panel removal except at the fifth floor because the additional beam has much lower flexural stiffness compared with the in-plane flexural stiffness of the wall panels.

The components in the building are divided into primary and secondary components for alternate path design. Primary components directly resist collapse when a vertical load-bearing element is removed from the building. Secondary components do not contribute to this capacity. Also, each type of action (for example, shear, moment, axial response, and others) and interaction of these actions (such as moment–axial load interaction) that occur in primary components (such as columns, beams, walls, and connections) are classified as either deformation controlled or force controlled. The component action type is based on the force-deflection Q-Δ curve for the given component action. Generally speaking, a component action is deformation controlled if it has a Q-Δ curve with a ductility ratio of 2 or greater before failure. Otherwise, it is generally force controlled. For example, the flexural response of beams is classified as a deformation-controlled action. Examples of force-controlled actions include connection response, shear response, and axial load response. It is conservative for linear static alternate path analysis to assume all actions are force controlled. Section 3-2.5 in UFC 4-023-03[18] has additional information.

The design loads for the floor and roof G_{LD} for alternate path design of the example building are calculated using Eq. (1) and the following inputs, along with values of Ω_{LD} in **Table 4**.

$$G_{LD} = \Omega_{LD} \times [1.2(D_f A_{trib} + D_w) + (0.5LA_{trib}LLR \text{ or } 0.2SA_{trib})] \quad (1)$$

Table 4. Design loads for linear static alternate path analysis of the example building

Case*	Design case	Ω_{LD}†	Design load, kip/ft	*m* factor‡	Comment
1	Wall panel acting as an in-plane beam to support floor load over removed load-bearing panel below	2	7.8	1	Actions include flexure (deformation controlled), shear, and connections (both force controlled). In this example all actions of wall panels acting as beams are conservatively considered force-controlled actions.
2	New steel roof beam supporting roof load over removed load-bearing panel at the fifth floor	6.5	8.1	6	Values for Ω_{LD} and m correspond to flexural (deformation controlled) design for this beam. The value of Ω_{LD} is offset by the high m factor in accordance with governing criteria.
3	Connections for new steel roof beam to roof plank it supports, shear in new roof beam	2	2.5	1	Actions include shear and connections (both force controlled). This is the design load for the roof slab.

Note: m = component demand modifier; Ω_{LD} = load increase factor applied to gravity loads for rational analysis depending on component action. 1 kip/ft = 14.6 kN/m.

* Add cases 1 and 3 for design load on wall panel at top floor of building because this wall panel is much stiffer than new roof beam and therefore supports both fifth floor and roof.

† Load increase factor in Eq. (1).

‡ m factor used in Eq. (2).

where

G_{LD} = design gravity load supported by the removed load-bearing component

Ω_{LD} = load increase factor applied to gravity loads for rational analysis depending on component action

D_f = dead load of floor system = 100 lb/ft² (4.8 kN/m²) for floors, 50 lb/ft² (2.4 kN/m²) for roof

A_{trib} = worst-case tributary floor area for the removed load-bearing component = 20 ft (6.1 m)

D_w = dead load of wall panel = 80 lb/ft² (3.8 kN/m²) × 12 ft (3.7 m) height = 960 lb/ft (14 kN/m)

L = floor live load = 50 lb/ft² (2.4 kN/m²)

S = snow load = 7 lb/ft² (0.3 kN/m²) (used only for roof loading)

LLR = live load reduction factor per ASCE/SEI 7-16[20] = 0.73

No lateral design loads are included in Eq. (1) because current alternate path design approaches are focused on gravity loads. This equation is based on the static load combination for extreme loading events in ASCE/SEI 7-16[20] and is amplified by a dynamic load increase factor Ω_{LD}, which accounts for suddenness of the element removal and the downward inertial effects of the structure above it. Equation (2) shows that each undamaged primary component must possess adequate strength to withstand the applied forces Q_U of each action (for example, flexural moment, shear load, or connection loads) caused by the design load.

$$\phi m Q_C \geq Q_U \tag{2}$$

where

ϕ = strength reduction factor from an applicable LRFD method

m = component demand modifier ≥ 1 for deformation-controlled actions based on material and component type per chapters 4 through 8 of UFC 4-023-03[18] or ASCE/SEI 7-16[20] = 1 for all force-controlled actions

Q_C = minimum specified strength (if force-controlled action) or expected strength (if deformation-controlled action) of the component to resist applied force

Q_U = applied force on the component using the corresponding load from Eq. (1)

The component demand modifier m is based on the expected ductility of the action, with minimum strength for force-controlled actions and larger expected strength for deformation-controlled actions. Table 4 shows the calculated design loads for relevant actions of the primary components of the example building (that is, for wall panels acting as beams and for the new steel roof beam) and the values of Ω_{LD} and m.

Values for Ω_{LD} and m are taken from seismic design guidance in ASCE/SEI 41-13.[19] These values depend on the type of alternate path design (such as rational analysis or life-safety protected finite element design), the component material type, and the structural action. Ω_{LD} and m for force-controlled and deformation-controlled actions in all four types of alternate path design are provided in UFC 4-023-03 for reinforced concrete, wood, and steel components.[18] ASCE/SEI 41-13[19] can also be used to determine these values, if necessary. Force-controlled primary components typically have values of Ω_{LD} and m equal to 2.0 and 1.0, respectively, for linear static alternate path design. A given primary component can have two design loads calculated with Eq. (1): one load for its deformation-controlled actions (for example, flexural response) and a separate load for its force-controlled actions (for example, shear response and connections), based on different values of Ω_{LD}. Conservatively, all actions can be designed as force-controlled actions. This approach was used for the wall panels in the example building that respond as beams to bridge over wall-panel removals below.

Prior to designing the wall panels to resist progressive collapse, there are several improvements to the connections between precast concrete floor planks and the load-bearing walls that are needed to maintain structural integrity at large deformations. Nearly all precast concrete floor systems are composed of simple-span members that support and transfer only gravity loads. These members are typically supported at their ends by connections, such as corbels, that are within the clear span between floors (that is, within the area that is removed for progressive collapse design). **Figure 7** shows how the connections of the wall panels to the floor system in the example building were improved so that the wall panels above can support the design load from the floor system, assuming removal of the shaded area with the corbel. The design load for the improved connections between the wall panel and the floor system is 5.5 kip/ft (80 kN/m), which is the 7.8 kip/ft (114 kN/m) design load for case 1 in Table 4 minus the self-weight of the wall panel. The capacity of the originally designed connection in Fig. 7 must be increased by approximately 40% to resist the progressive collapse design load. Any prying action, or other more complex effects, in these connections must be considered in the connection design.

Figure 8 shows the simplified progressive collapse design approach for a typical wall panel that supports floor load (and the roof load at the fifth floor) over a removed 25 ft (7.6 m) span of wall panel below. The wall panel at the top floor has the highest progressive collapse design load—10.3 kip/ft (150 kN/m) per the first note in Table 4—and therefore controls the design. Within the wall panel in Fig. 8, the two spans that are designated as beams are assumed to deflect together noncompositely. Each beam resists a design load w_B

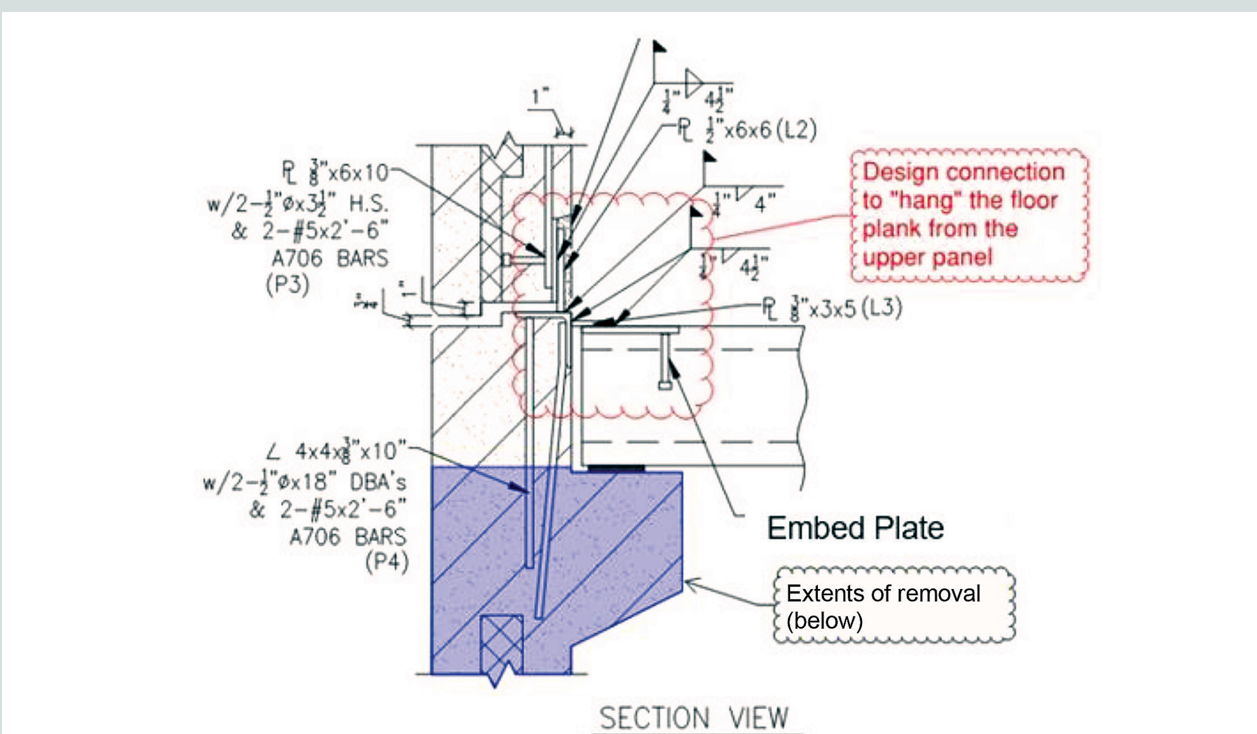

Figure 7. Improved connection design between precast concrete panels and floor planks for alternate path design.
Note: ∠ = angle; # = number; DBA = deformed bar anchor; H.S. = headed stud; w/ = with; ϕ = strength reduction factor from an applicable load- and resistance-factor design method; L2, L3, P3, P4, PL = part identifiers. 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.305 m.

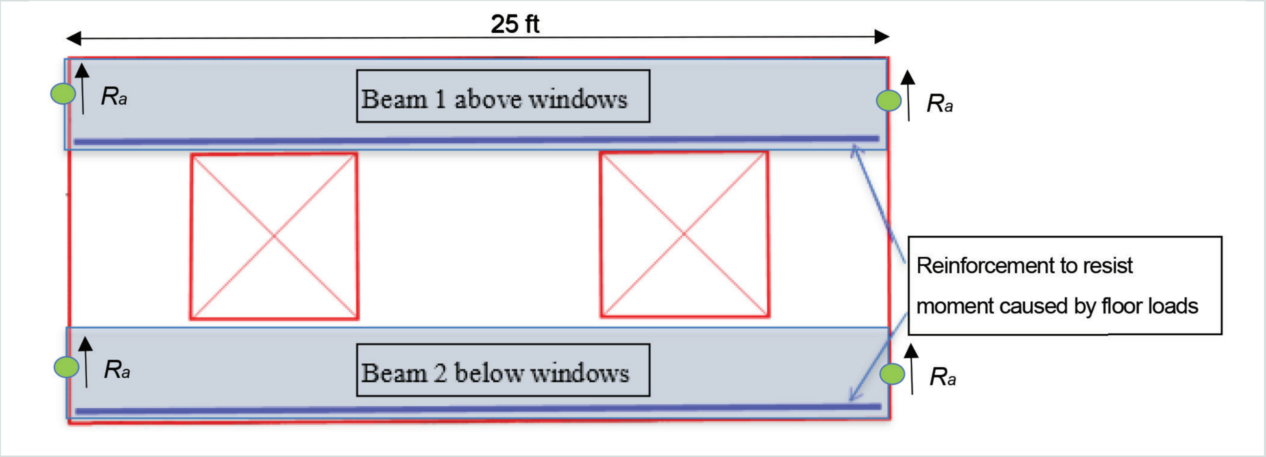

Figure 8. Beam action in wall panel for removal of perimeter load-bearing panel below. Note: R_a = panel-to-panel reaction force. 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.305 m.

of 5.15 kip/ft, which causes a design moment of 400 kip-ft (542 kN-m). The original design for the typical wall panel has two no. 4 (13M) reinforcing bars along the edges and above the windows, providing 0.40 in.^{2} (258 mm^2) of reinforcing steel. This is not sufficient to resist the applied design moment in the beams using values in Eq. (2) for force-controlled action. The reinforcement must be increased so that the total reinforcement in this beam section has an area not less than 1.25 in.^{2} (806 mm^2). This can be conservatively achieved by adding two no. 8 (25M) reinforcing bars (with a total area of 1.57 in.^{2} [1013 mm^2]) along the bottom edge of the panels and above the windows (Fig. 8). The additional reinforcing bars should be continuous from end to end of the panel with no splices.

Next, the shear strength ϕV_n of the two beams designed into the wall panel must be checked against the shear load V_u caused by the design load of 5.15 kip/ft in each beam at the critical shear section above and below the window edge that is nearest to the wall panel support. This check assumes that shear response is a brittle, and therefore force-controlled, action that must be resisted per Eq. (2) with a lower bound strength. Q_C is calculated per LRFD in ACI 318-14[21] using a strength reduction factor ϕ of 0.75 with the minimum specified material strength for the concrete. The value of m in Eq. (2) would be 1.0. This design check shows that stirrups are required. Single-leg no. 3 (10M) shear ties in each wythe at 12 in. (300 mm) spacing will provide adequate additional shear capacity. These stirrups are only required in the critical shear areas. If there is significant torsion in the wall panel from the applied floor load, this reaction must be added to the shear caused by flexural response.

Next, the connections at the ends of the panels shown in Figure 8, must be designed to resist the reaction force at the end of the panel R_a from the design load w_B of 5.15 kip/ft in each beam. Assuming there will be two panel-to-panel connections at each end of the wall panel, the shear design load for each connection is 64 kip (285 kN). This is a force-controlled action that again must be resisted per Eq. (2) with a lower-bound strength of the connections calculated using LRFD. The connections are not designed in the example problem in the PCI report[11] because it is assumed that the designer would perform these calculations in accordance with the pertinent design standards using available hardware. The number of panel-to-panel connections can be increased to reduce the design load per connection.

The aforementioned wall panel removal case assumes that all wall panels are aligned vertically over the height of the building (with in-line joints) and that a single wall panel was wholly removed. Therefore, the panels above the removed panel could be designed as a simple beam. Another removal case that should be considered is that of a $2h_w$ removal of 25 ft (7.6 m) from two adjacent panels (that is, half of each neighboring panel is removed). In this scenario (shown in **Fig. 9**), two panels in each floor above the removal must work together to support the same design floor load per beam within the panel w_B over the removed wall panel span. The typical cantilever span for these panels in this case is 12 ft (3.7 m) plus the distance to the shim d_{shim} that is nearest to the removed panel area. Additional shims can be used in the wall panel design to minimize the cantilevered span length. The PCI report shows that this design case will require two no. 9 (29M) negative moment reinforcing steel bars placed near the top of beam 1 and underneath the window openings in beam 2 (using the beam notation shown in Fig. 8). Also, the shear force in each beam is slightly higher than the scenario in Fig. 8 so that the shear reinforcement must be increased to single-leg no. 4 (13M) shear ties in each wythe at 12 in. (300 mm) spacing in the critical shear areas. The panel-to-panel reaction forces R_a in Fig. 9 are in the opposite direction from these reaction forces in Fig. 8 and with a lower shear load of 50 kip (220 kN). The compressive reaction force in the shim R_b in Fig. 9 is 360 kip (1600 kN). A shim that can resist this compression force must be provided between all panels at the location shown in Fig. 9.

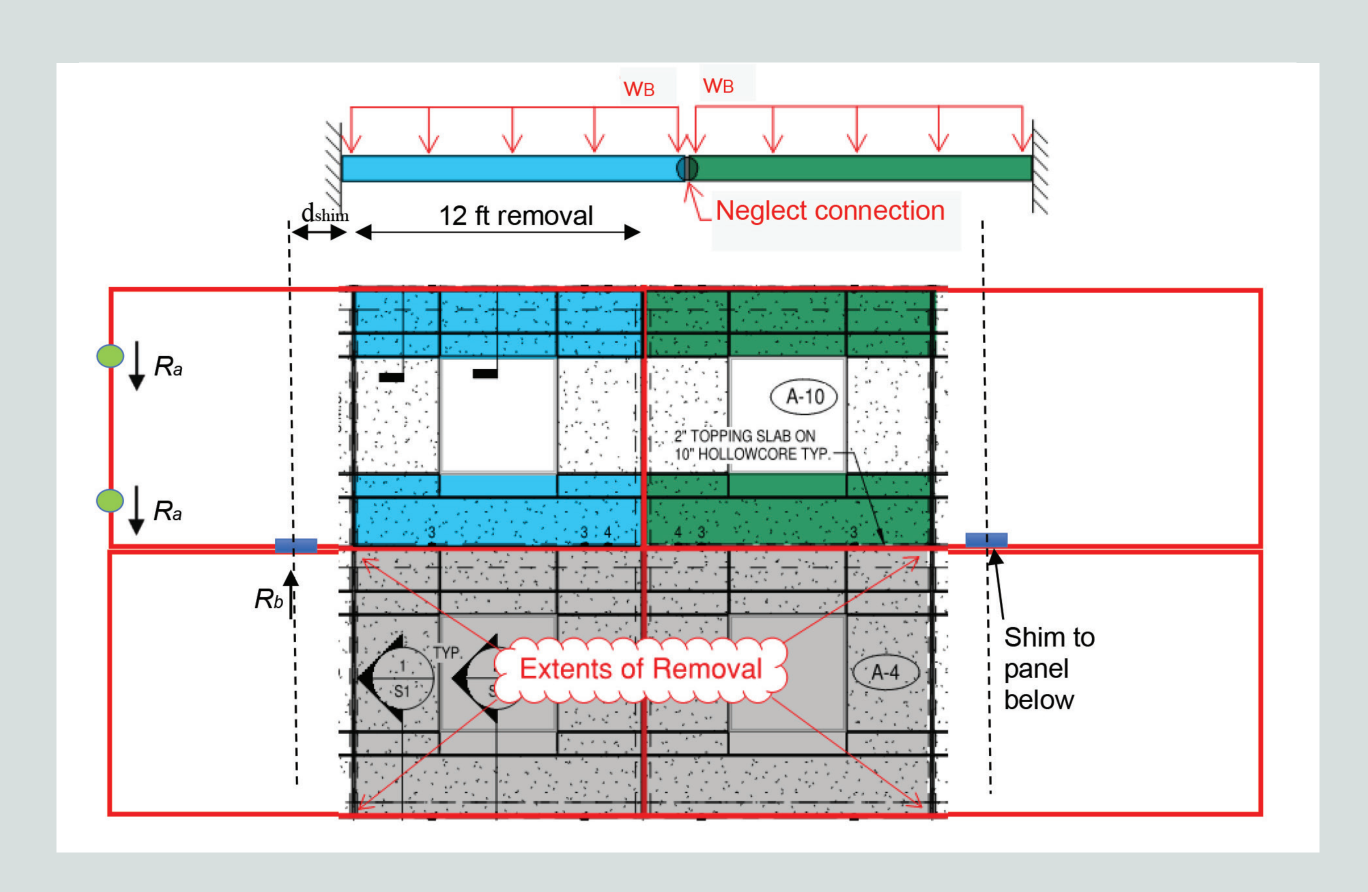

Figure 9. Removal of 25 ft panel width from two adjacent panels. Note: Red lines show the limits of each panel. d_{shim} = distance to the shim; R_a = panel-to-panel reaction force; R_b = compressive reaction force in the shim; TYP. = typical; w_B = design floor load for alternate path method for each beam designed into wall panel. 1 ft = 0.305 m. 1″ = 1 in. = 25.4 mm.

Because the removed wall panel width may create the case in Fig. 8 or Fig. 9, all wall panels except corner panels should be designed with reinforcing steel, connections, and shims that can resist design loads from both cases. For a corner removal, **Fig. 10** shows a 12 ft (3.7 m) load-bearing panel width removed at the corner along each face of the building. This case has the largest panel-to-panel connection design loads.

As described previously, a steel beam is needed to support the roof planks for the design case where a 25 ft (7.6 m) width of wall panel is removed from the top (fifth) floor. This exterior beam is attached to the roof planks with positive connections that resist 2.5 kip/ft (36 kN/m) per Table 4 based on their minimum strength from Eq. (2). The steel beam is designed to resist the moment (that is, a deformation-based action) applied by a higher design load of 8.1 kip/ft (118 kN/m) per Table 4. However, the design strength of the moment capacity includes m equal to 6.5 with the expected yield strength and the full plastic section modulus (that is, the expected moment capacity that develops at a large ductile deflection) per Eq. (2). The PCI report[11] shows that the calculated size for the new steel beam is an 8 × 6 × ½ (200 × 150 × 13 mm) hollow structural section. This tubular cross section has adequate shear capacity (based on the lower-bound material strength) that exceeds the shear load calculated using a design load of 2.5 kip/ft (36 kN/m) from Table 4 for a force-controlled action. This steel beam size is also sufficient for a design case where a 12 ft (3.7 m) width of the top floor wall panels are removed at the corner of the building and the beam acts as a cantilever to support roof planks. The roof planks must be attached to the wall panels in the top floor with a positive connection that resists uplift for this design case.

It is worth noting the ratio of Ω_{LD}/m in Table 4 for the steel roof beam acting in flexure is equal to 1.1, implying that the steel beam is designed so that its expected moment capacity (that is, expected yield strength acting with full plastic section modulus) resists 1.1 times the floor load (that is, the load from Eq. [1] excluding Ω_{LD}). By contrast, the other cases in Table 4 are designed so that their minimum capacity resists two times the floor load. This demonstrates the advantage of ductile actions for progressive collapse design, which can absorb the additional energy caused by a suddenly applied load (that is, sudden loss of a load-bearing component) with postyield strain energy of a ductile component, compared with brittle actions, which can only absorb applied energy with a much stiffer elastic response.

Alternate path design also requires that secondary members maintain their structural integrity during the progressive-collapse-resisting event. The secondary components are checked for combined stress from the design floor load in Eq. (1) with Ω_{LD} equal to 1.0 (that is, stresses before load-bearing component removal) and stress caused by the deflections of primary

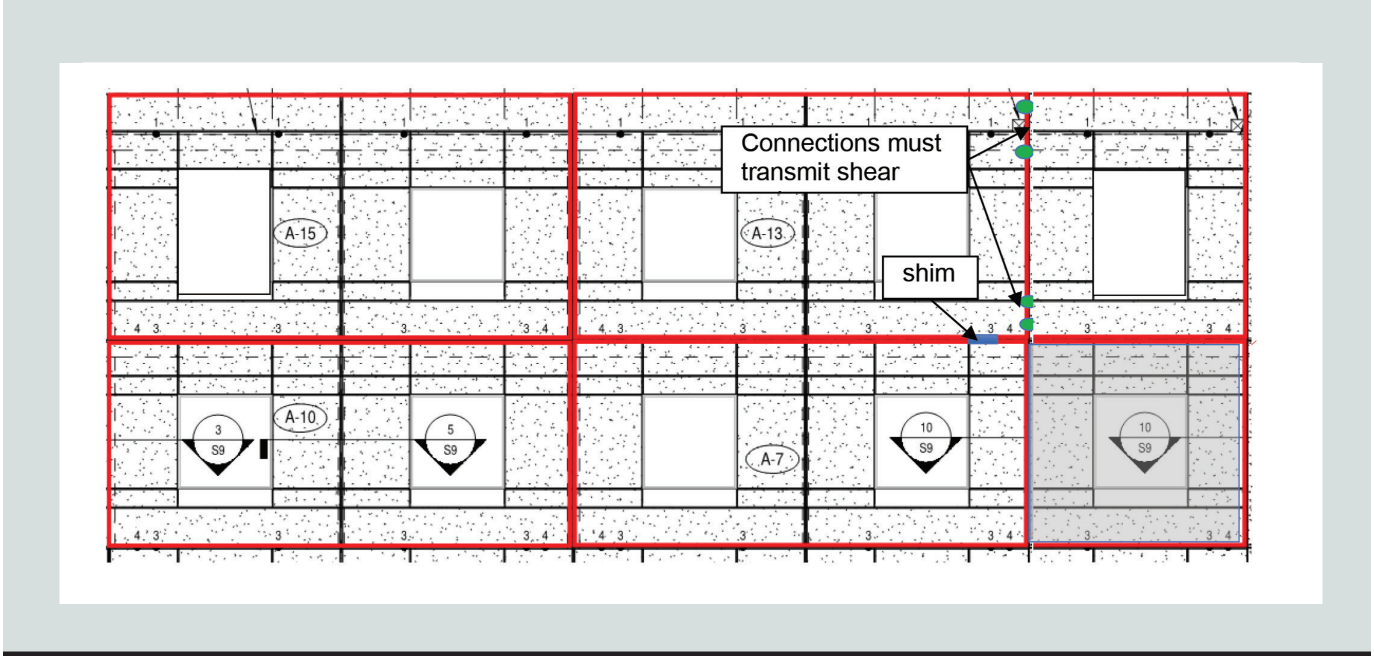

Figure 10. Removal of 12 ft panel width at corner of building. Note: Red lines show the limits of each panel. 1 ft = 0.305 m.

components after removal of the load-bearing component. For example, a connection of the floor panel in Fig. 7 to its interior support, which is a secondary component, may have significant additional stresses due to prying action if there is a large deflection or torsional twist in the wall panel. Because deflections of primary components are not calculated explicitly in a linear static analysis, this part of the alternate path design may involve engineering judgment (that is, the primary components are quite stiff even after the load-bearing component removal) or a separate calculation of the primary component deflections and then an analysis of the effect of these deflections on the secondary components, including their connections.

Finally, design modifications required to resist progressive collapse may have an effect on the design of the building to resist conventional loads. For example, added connections between the precast concrete wall panels and floor systems for the alternate path design generally increase the shear stiffness and rigidity of the wall system, and this may adversely affect serviceability and/or seismic requirements. The designer should perform calculations to confirm that excessive cracking due to restrained thermal effects will be avoided and account for any increase in the base shear of the building due to increases in lateral stiffness from the additional connections.

Conclusion

This design example illustrates the upgrades needed for a typical five-story building with exterior precast concrete load-bearing walls to meet progressive collapse design requirements per the alternate path and tie force design methods. Due to the prescriptive nature of the tie force design requirements, the designer cannot use very much ingenuity to accommodate the resulting design enhancements (for example, reinforcement continuity requirements for internal and peripheral ties) for a given project. The alternate path design method allows more flexibility with performance-based design requirements but at an increase in analytical complexity compared with tie force design. Therefore, the engineering associated with an alternate path design can be more costly and require more time than a corresponding tie force design.

This paper also illustrates that the two design approaches can result in significantly different designs. For the example building, the alternate path design required more connections between load-bearing wall panels as well as from the panels to the floor and roof systems than the tie force method; however, the alternate path approach avoids the tie force design requirement for continuous reinforcement in both directions of the floor and roof system as well as around the floor and roof perimeter. The additional connection requirements of the alternate path design can be accommodated with an increased quantity of typical precast concrete connections or with enhanced connections with increased capacity. Either would avoid major modifications to typical precast concrete construction or erection. On the other hand, the continuous reinforcement required in the topping slab by the tie force method (especially at cut-out sections of the precast concrete floor and roof) must typically have a positive connection to the precast concrete component, which is not a typical construction detail. Reinforcing bars or steel inserts that project out of the precast concrete must be provided to form the positive connection with the cast-in-place topping around the continuous reinforcement. This additional detailing can complicate shipping and handling for the precast concrete components.

This paper demonstrates that precast concrete load-bearing wall structures can, in fact, be designed and detailed to resist progressive collapse per current U.S. government guidelines. This design space presents an untapped market opportunity for the precast concrete industry that to date has mostly relied on structural steel and cast-in-place concrete design solutions.

Acknowledgments

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Notation

A_{trib} = worst-case tributary floor area for the removed load-bearing component

A_T = tributary area supported by a load-bearing wall panel with tension tie

d_{shim} = distance to the shim

D = dead load

D_f = dead load of floor system

D_w = dead load of wall panel

F_L = internal longitudinal tie force

F_p = vertical tie force

F_{pL} = perimeter longitudinal tie force

F_{pT} = perimeter transverse tie force

F_T = internal transverse tie force

G_{LD} = design gravity load supported by the removed load-bearing component

h_w = clear floor height

L = live load

L_1 = the greatest distance between the centers of supports in the longitudinal direction

L_2 = span length for peripheral ties over load-bearing walls in transverse direction for floors with one-way spans

L_L = greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the longitudinal direction

L_p = width of perimeter tie

L_T = greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the transverse direction

LLR = live load reduction factor

m = component demand modifier

Q = force

Q_C = minimum specified strength (if force-controlled action) or expected strength (if deformation-controlled action) of the component to resist applied force

Q_U = applied force on the component using the corresponding load

R_a = panel-to-panel reaction force

R_b = compressive reaction force in the shim

S = snow load

V_u = shear load caused by lateral failure load

w_B = design floor load for alternate path method for each beam designed into wall panel

w_F = design floor load for tie force method

w_{lf} = lateral failure load

W_{cL} = self-weight of panels in longitudinal direction

W_{cT} = self-weight of panels in transverse direction

Δ = deflection

ϕ = strength reduction factor from an applicable load- and resistance-factor design method

ϕV_n = shear strength

Ω_{LD} = load increase factor applied to gravity loads for rational analysis depending on component action

About the authors

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Abstract

Building designs that can resist progressive collapse (also referred to as disproportionate collapse) require enhanced connectivity and continuity between structural elements so that the elements surrounding a damaged supporting component can redistribute loads without failing themselves, though they may sustain heavy damage in doing so. The design space for progressive-collapse-resistant buildings presents an untapped market opportunity for the precast concrete industry that to date has mostly relied on structural steel and cast-in-place concrete design solutions. To help unlock these opportunities, a recent PCI-funded project has developed an illustrative example for a progressive-collapse-resistant precast concrete building design that meets the requirements of current U.S. government standards. The example summarized in this paper illustrates the upgrades needed for a typical five-story office building with exterior precast concrete load-bearing walls to meet progressive-collapse-resistance requirements per the alternate path and tie force design methods. The resulting designs per each method have substantial differences due to their varying strategies for developing continuity between structural elements.

Keywords

Alternate path method, load-bearing wall, precast concrete, progressive collapse, tie forces method.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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