Although large-concrete-panel construction is economical for hotels, motels, and multifamily housing units, a disastrous gas explosion in 1968 in an apartment building focused attention on the need to address abnormal loads in the design of large-concrete-panel systems.

This paper contains a review of structural integrity and progressive collapse in large-panel precast concrete systems in the context of 21st-century reevaluation of acceptable risk and abnormal loads.

To learn more about the design of precast concrete for seismic loads, see PCI’s *Seismic Design of Precast/Prestressed Concrete*.

Multistory buildings using large-concrete-panel construction have been the subject of considerable interest and research in the past. These large-panel systems combine shallow, hollow-core floor slabs with large precast concrete wall panels to form a complete system that is economical for hotels, motels, and multifamily housing units.¹

The general arrangement of the load-bearing walls may be cross walls, where the load-bearing walls are perpendicular to the longitudinal axis of the building, or they may be spine walls, where the load-bearing walls are parallel to the longitudinal axis of the structure. Figure 1 illustrates these systems. Mixed systems combine cross-wall and spine-wall systems. As complete precast concrete systems, they provide many benefits to the owners and occupants.

In the early development of multistory, large-panel concrete buildings, the primary loads considered in the design of structures were gravity and wind. The sustained gravity loads of these structures often provided ample resistance to the wind loads and small eccentricities of the floor loads. Detailing of these structures often relied on friction capacity alone for load transfer in grouted joints. However, in 1968 a disastrous gas explosion at Ronan Point, a London, U.K., apartment building, focused attention on the need to address abnormal loads in the design of these structural systems.²
Conventional structural analysis and design are based on the application of traditional loads: dead load, live load, temperature effect and other volume changes, soil and hydrostatic pressure, snow load, wind load, and earthquake load. Where these loads involve complex behavior or application, the behavior or application has usually been simplified to permit safe and economical design using well-understood principles of structural mechanics.

Traditional loads defined in U.S. building codes do not address all possible conditions to which a structure may be subjected. Historically, abnormal loads include tornadoes, flood load, service-system explosions, aircraft collisions, external explosions from accidents, ground vehicle impact, and sabotage. Changes in societal experience and expectations have caused a change in the level of acceptable risk and protection against violent actions, which might include vehicle impact, external blast, internal blast, and demolition blast (shape charge).

This change in acceptable risk coincides with an increasingly competitive environment in our economy that tends to prefer buildings that are designed as cost efficiently as possible to meet code requirements. These buildings may, therefore, be more susceptible to abnormal loads. The trend toward limited ductility and continuity that was prevalent in the late 20th century, however, has been moderated or reversed today by the advent of more universal application of earthquake-design principles.

In 1968, the behavior of large-panel precast concrete buildings subjected to abnormal loading became a heightened concern due to the gas explosion of the Ronan Point apartment building. The initial reaction of the design community was to define a system explosion load of 5 psi (34 kPa) that could be used during design to allow the structure to resist the event. When this severe criterion was applied, however, the resulting designs were not feasible for most of the structures used in the housing market.

Research evaluated which levels of pressure might be more realistic. The PCA reports indicated that “studies have suggested that pressures can be kept relatively low, compared with 5 psi, by appropriate venting.” It was proposed that the amount and nature of venting should be considered to relieve pressure. Formulas for the maximum equivalent static wall pressure \( P_{max} \) were proposed:

\[
P_{max} = \text{greater of } \left\{ \begin{align*}
0.44 + 0.5P_o & \text{ or } \\
0.44 + 0.5P_o + 0.00054 / \psi^2
\end{align*} \right. 
\]

where

\( P_o \) = pressure at which the vent blows out
$\psi$ = ratio of venting area to the volume of the room

This approach has its own problems.

“Assuming the glazed areas of the structure as the only available venting leads to unnecessarily high estimates of the maximum pressure obtained.”

In any case, the focus on service-system explosions as a defined load did not address other sources of abnormal and low-occurrence events.

Today, the focus of society is on explosive bombings, which were also addressed in the PCA report.

“In explosive bombings, the pressure generated during detonation varies over a large range, depending on the size of the charge and the type of explosive. Therefore, a deterministic evaluation for this loading condition may never be made. It can be argued, however, that consideration of other more prevalent abnormal loading conditions will provide a sufficient level of structural resistance to account for most explosive bombings.”

The approach ultimately chosen did not try to define specific load criteria for abnormal events.

**Philosophy of structural response to abnormal loads**

Structures designed to resist only traditional loads may lack ductility and redundancy, a concern that is only partially addressed in the minimum seismic requirements for a complete load path from any structural element to the foundation. The problem in the design of these structures has generally been conceived as vulnerability to progressive collapse.

“Progressive collapse may be defined as a chain reaction of failures following damage to only a small portion of the structure.”

It is difficult to determine how "a small portion" should be defined. Buildings designed and detailed without continuity between elements and without ductility can be extremely brittle and susceptible to progressive collapse. A structure designed for specifically defined damage may be susceptible to a larger area of damage that would otherwise not result in a disproportional loss of structure. A progressive collapse can be initiated by any form of abnormal load.

The research into progressive collapse in large-panel concrete structures dealt with two modes: the inability to form an alternate load path to bridge local damage and the insurmountable debris load. Three approaches were considered to address them:

- eliminate the hazard
- design for the hazard with robust elements
- allow local failure but design to ensure an alternate load path

In the future, acceptable solutions will likely combine all three approaches.

In the 1970s, the efficacy of using primarily the first or second approach was questionable. Our ability to eliminate the hazards today is limited, even with increased security standards. Designs that are sufficiently robust to withstand all abnormal loads are likely to be prohibitively expensive, as well as dysfunctional for their primary purposes. Therefore, the third approach was emphasized in the research.

In order to develop quantitative design guidelines for alternate load paths, “a limiting (maximum) damage volume or area must be defined in some way.”

Given a damage volume or area, it is possible to design an alternate path by rational procedure to establish necessary integrity. The General Services Administration’s progressive-collapse criteria use the approach of notional removal of first-floor columns at selected locations. Applying this approach, called direct design, to every area-supporting element in a structure individually would be unduly cumbersome and time consuming.

As an alternative, the general configuration of the conventional assembly of large-panel structures was evaluated to establish alternate load-path mechanisms for this class of building. These alternate mechanisms were subjected to design and experimental verification. From this research, it was possible to develop a minimum detailing practice to establish a suitable degree of continuity and ductility, ensuring integrity through indirect design. Indirect design has significant advantages:

- Prescriptive rules of indirect design can be evaluated in plan review.
- Design engineers are not required to directly consider the effects of abnormal loads for one form of construction and not another.
- The complexity of the definition of the size of local damage as a design rule for detailed calculation is avoided.
- Experience has shown that minimum detailing requirements based on sound engineering judgment can establish an adequate degree of structural integrity for concrete structures.
Rationale for general structural integrity in large-panel concrete structures

It is not possible to minimize the probability of abnormal loading to an acceptable level. Since the initial research, this has become even more difficult due to societal demand for lower risk and the emergence of terrorism. The option of designing for greater strength against abnormal loads poses two significant problems.

The first is the definition of the load, which depends on the source, location, and magnitude of the abnormal-load event. The second problem is the considerable expense of designing every element in the building for a significant, nonconventional overload. Therefore, the third approach is the one that has been pursued. This indirect approach of providing minimum detailing to keep a local event from becoming an overall collapse is more feasible in practice.

The rationale for the provision of general structural integrity in large-panel concrete systems recognizes the strengths and weaknesses of jointed construction. It is not difficult to establish tension continuity across dry or grouted joints, but it can be difficult to create moment continuity. A successful system of alternate load paths must recognize this in order to establish continuity across connections and ductility within them.

It is important to recognize the realistic nature of damage from abnormal loads. It has been suggested that the rules for design for abnormal loads should include notional removal of any structural element. With large-panel concrete structures, as with any precast concrete structure designed under the provisions of ACI 318-95 chapter 16, there are requirements for transverse, longitudinal, vertical, and peripheral ties. With the continuity and ductility in the form of element characteristics and vertical and horizontal ties, complete removal of any element is not likely. An approach that considers the effects of damaged elements is more appropriate than the complete notional removal of wall elements.

Experimental investigations of general structural integrity

The research conducted on structural integrity in the 1970s was a combination of experimental and analytical work. The experimental analysis focused on the cantilever behavior of walls with removal of a panel, the slab suspension mechanism, and the behavior of the horizontal joint.

Cantilever behavior of the wall assembly is the primary mechanism to develop an alternate load path for integrity.

“The objective of the cantilever action tests was to investigate stability of panel structures in the event of partial loss of a supporting wall. Panels above the ineffective panel were intended to act as cantilevers from the adjacent undamaged structure.”

The cantilever-action tests used 1/3-scale models of multistory walls of representative buildings. The test models included six stories, with five stories cantilevered above a missing base wall. Three stories replicated interior- and end-wall conditions as well as various corridor or wall-joint conditions at the interior ends of the walls. Although the tests were developed with only single wall planes and partial floor construction, the loading was conducted to replicate a 30-ft-span (9.1 m) floor with 30-ft-long walls. The loading included 55 lb/ft² (2.6 kPa) floor dead load, 8 lb/ft² (0.4 kPa) partition dead load, and 13 lb/ft² (0.62 kPa) live load, which was 1/3 of the design live load for the catastrophic condition.

The cantilever wall tests showed that the cantilevered resisting moment depended on tension in the connecting ties at each floor and a compression force at the lowest level.

Slab suspension provided an additional alternate load path, where tension ties across the joints above a crippled wall acted in a catenary fashion to provide partial support for load.

“The objective of the slab suspension tests was to develop suitable tie details in a floor system to ensure continuity after a supporting wall panel becomes ineffective.”

Four tests were conducted with two spans of full-sized floor slabs tied at the center. The center support was removed to represent loss of support. Other tests were conducted to establish tie development and slip. Full suspension of the slabs was not achieved in the tests because the large deflection required for the suspension system to become operative resulted in premature fracture of well-anchored ties. Controlled slippage of the tie anchorage is required to allow large displacements, but it is also important to maintain some continuity in the slab system to reduce debris loading.

Extensive testing was conducted on horizontal joints constructed with platform framing to investigate the transmission of vertical loads and the potential for wall splitting. Figure 2 illustrates the typical joint details. The variables that were included in the tests were strength of grout, amount of transverse wall reinforcement, filled or unfilled slab cores, and applied floor moment and rotation. This investigation provided design guidance for the joints and qualified these details as part of the overall system.

Structural integrity in large-panel systems

The approach of minimum detailing is predicated on defining an acceptable limit of damage and alternate load paths that are effective in keeping damage within this limit. This
approach prescribes the minimum horizontal and vertical ties that must be incorporated into the structure. The rationale for the horizontal and vertical ties is discussed in the following sections.

**Transverse ties to develop cantilever and beam action in wall panels**

ACI 318 section 16.5.2.1 prescribes the provision for transverse ties.

“Longitudinal and transverse ties shall be provided in floor and roof systems to provide a nominal strength of 1500 lb per foot of width or length. Ties shall be provided over interior wall supports and between members and exterior walls. Ties shall be positioned in or within 2 ft of the plane of the floor or roof system.”

In addition, section 16.5.2.3 requires that “transverse ties perpendicular to floor or roof slab spans shall be spaced not greater than the bearing wall spacing.”

These provisions essentially locate the transverse ties in the horizontal joint between wall panels at the level of the floor. The intent of the transverse ties is to create cantilever action in the wall stack in the event of the crippling or loss of a load-bearing wall. This cantilever action will transfer vertical shear from the walls above the damage to adjacent walls in the line of the damaged wall. There must be tensile continuity in the horizontal connections to accomplish this load transfer.

The detailing of the floor and the inclusion of transverse walls aid in maintaining the load path. The shear strength in the horizontal joints must be sufficient to prevent horizontal panels from sliding at the joints. The tie force has been determined empirically from the load tests conducted as part of the research so that this cantilever action can be mobilized by either a stack of cantilevered walls or by individual-floor cantilevers.

**Longitudinal ties for membrane action in floor**

ACI 318 section 16.5.2.1 also provides guidance for longitudinal ties, which run in the direction of the floor component spans. Section 16.5.2.2 adds that “Longitudinal ties parallel to floor or roof slab spans shall be spaced not more than 10 ft on centers. Provisions shall be made to transfer forces around openings.”

These longitudinal ties are proportioned to act as catenary between walls on either side of the crippled wall only to the extent of supporting loads from the local debris. The forces prescribed for this design are not sufficient for the catenary to carry the wall above the floor and floor loads to those adjacent walls. That level of force for that function was found not to be feasible and was not required.

**Vertical ties to develop suspension action**

ACI 318 section 16.5.2.5 prescribes vertical ties to develop suspension action.

“Vertical tension ties shall be provided in all walls and shall be continuous over the height of the building. They shall provide a nominal tensile strength not less than 3000 lb per horizontal foot of wall. Not less than two ties shall be provided for each precast panel.”

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**Figure 2.** These diagrams illustrate the typical platform framing details for the testing that investigated the transmission of vertical loads and the potential for wall splitting. Note: 1 in. = 1” = 25.4 mm.
These ties provide for the vertical suspension of ineffective walls to limit debris load. Because the alternative support mechanism develops through the cantilever action of adjacent walls, these ties need only hold the damaged wall and floor that it supported to limit the loads on lower walls and floors. They provide resistance against a wall kicking out, thereby preventing total removal of a wall. They also provide clamping action for shear-friction capacity in the joint. Walls on the perimeter are more vulnerable. Additional ties add strength and limit out-of-plane movement. These vertical ties may also be needed for overturning strength in resisting lateral-system loads.

**Peripheral ties to develop diaphragm action**

ACI 318 section 16.5.2.4 adds provisions for perimeter ties that provide for a minimum strength in the floor diaphragm.

“Ties around the perimeter of each floor and roof, within 4 ft of the edge, shall provide a nominal strength in tension of not less than 16,000 lb.”

The detailing of minimum ties achieves a level of acceptable integrity, which was established from the research.

“This provision of General Structural Integrity eliminates the need to design for any particular abnormal load.”

In earthquake design, the anticipated damage is from the post-elastic behavior of the structure. Elements in the system must have integrity in the event of such damage, including the potential for the partial loss of load-bearing function. However, ACI 318 chapter 21 contains additional requirements for the seismic design of precast concrete walls. Design for structural integrity alone may not be sufficient for seismic events.

**Conclusion**

The issue of structural integrity and progressive collapse is not new for large-panel precast concrete systems. As a result of catastrophic collapse in 1968, PCA conducted more than 10 years of focused research sponsored by the U.S. government and industry to develop an indirect design approach, which is now incorporated as part of ACI 318. This approach was broadened to all forms of jointed precast concrete construction by requirements in ACI 318 with general provisions for integrity ties.

This effort provided important lessons that are still relevant today:

- The typical initial response to a catastrophe is to impose extremely conservative solutions that are ultimately determined to be unnecessary.
- Long-term, feasible solutions generally follow the initial extreme and unnecessary ones. They are based on the characteristics and strengths of the affected systems, eliminating the imposition of needless constraints that might impose unfavorable unintended consequences.
- The process of developing, testing, and codifying solutions takes years of focused effort. There is never an effective quick fix.

**References**


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**Notation**

\[ P_{\text{max}} = \text{maximum equivalent static wall pressure} \]

\[ P_o = \text{pressure at which the vent blows out} \]

\[ \psi = \text{ratio of venting area to the volume of the room} \]

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**Synopsis**

Multistory buildings using large-panel concrete construction have been the subject of considerable interest and research in the past. These systems combine economical, shallow, hollow-core floor slabs with large precast concrete wall panels to form a complete system that shares the markets for hotels, motels, and multifamily housing. The general arrangement of the load-bearing walls may be cross walls or spine walls.

In the early development of multistory, large-panel concrete buildings, the primary loads considered in the design were gravity and wind. With the sustained gravity loads often providing ample resistance to the wind loads and the ability to accommodate the small eccentricities of the floor loads, detailing often relied on friction capacity alone for load transfer in grouted joints. However, a gas explosion in 1968 in an apartment building at Ronan Point in London, U.K., focused attention on the need to address abnormal loads in the design of these systems. The Portland Cement Association conducted research in the 1970s under the sponsorship of the Department of Housing and Urban Development on the structural integrity and progressive collapse resistance of large-panel buildings. The overall program objective was to develop minimum criteria for the design and construction of large-panel concrete structures.

This paper is written as a review of structural integrity and progressive collapse in large-panel precast concrete systems in the context of 21st-century reevaluation of acceptable risk and abnormal loads.

**Keywords**

Abnormal load, large panel, longitudinal ties, multistory structure, peripheral ties, progressive collapse, transverse ties.

**Review policy**

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

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