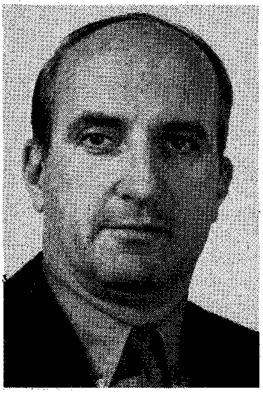
Developing Structural Integrity in Bearing Wall Buildings



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The prestressed concrete industry provides large volumes of precast floor and wall units for use in bearing wall buildings. In addition to the wide interest in precast panel bearing wall buildings, the industry has a vital interest in brick and concrete masonry bearing wall buildings which often utilize precast prestressed concrete floor units.

For decades, relatively little engineering research and development attention was paid to bearing wall construction. It is heartening to note that the Prestressed Concrete Institute has taken a lead in this important field. This progress is reflected in the work of PCI Committees and published reports and papers appearing in the PCI JOURNAL (see list of references at end of paper). Other useful information on the subject is contained in the current *PCI Design Handbook*, the *PCI Manual for Structural Design of Architectural Precast Concrete*, and some early issues of *PCItems*.

Recently, extensive research programs at the Portland Cement Association and at the Massachusetts Institute of Technology have helped systematize and extend our knowledge of behavior and design requirements for panel structures. This work has been summarized by Mark Fintel, and James Becker and Peter Mueller at the recent PCI Seminar on "Advanced Design Concepts in Precast Pre-

NOTE: This article is an expanded and updated version of a paper presented at the PCI Seminar on "Advanced Design Concepts in Precast Prestressed Concrete," held in conjunction with PCI's 25th Anniversary Convention in Dallas, Texas, October 17-18, 1979.

Summarizes the current direction of design and construction to resist progressive collapse. A positive approach requiring the development of improved structural integrity with adequate continuity and ductility is much more practical than provision of alternate path or specific resistance criteria. General procedures for implementing improved structural integrity in bearing wall structures are discussed and specific recommendations are made.

Fig. 1. Geneva Towers, Lake Geneva, Wisconsin. This nine-story condominium and office building used 245 6-in. (152 mm) thick loadbearing wall panels with sizes up to 10 ft high x 42 ft long (3.05 x 12.81 m). Double-tees were used for the floor and roof members.

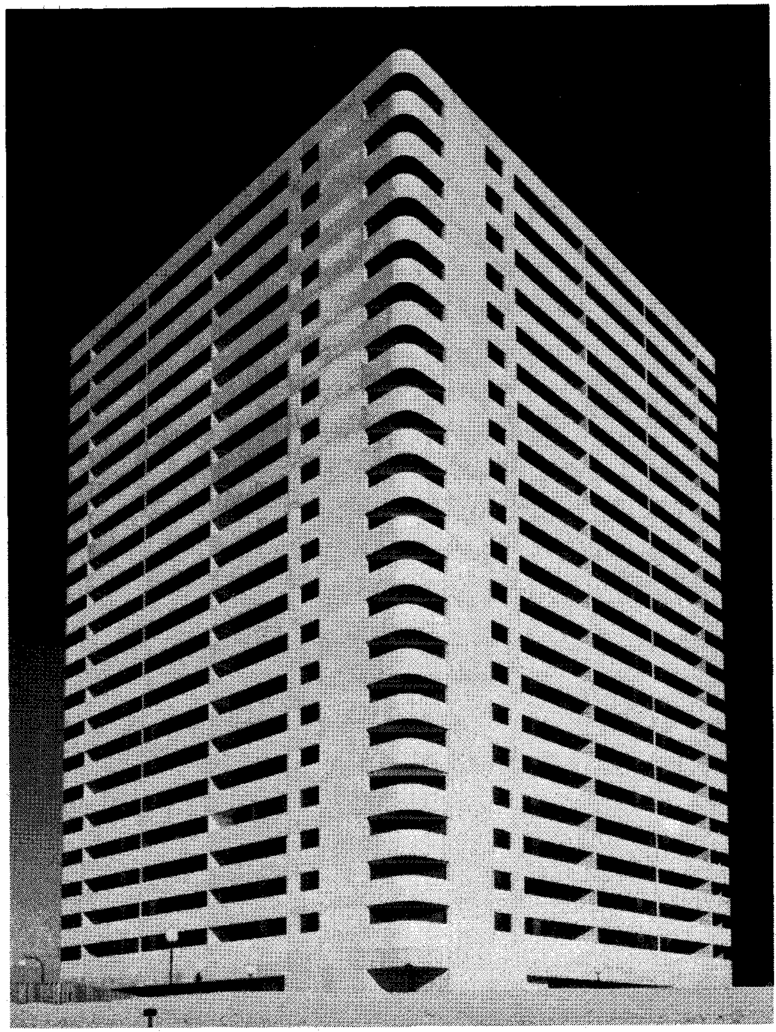


Fig. 2a. Atlantis Condominiums, Ocean City, Maryland. This 22-story building is comprised of precast floor slabs, wall panels, spandrels, and stair and elevator towers.

stressed Concrete" held in Dallas, Texas, October 17-18, 1979. Undoubtedly, these and other studies will lead to improved detailing practices for bearing wall systems.

Much of bearing wall panel development has taken place in living laboratories scattered over North America, where innovative design engineers and precasters have worked together on the economic and safe solution of the many design, production, and erection problems involved with this type of construction.

The bearing wall structure is simple in concept, but extremely careful attention is required in the connection details to ensure safe buildings. When properly designed, the resulting structure will be attractive, economi-

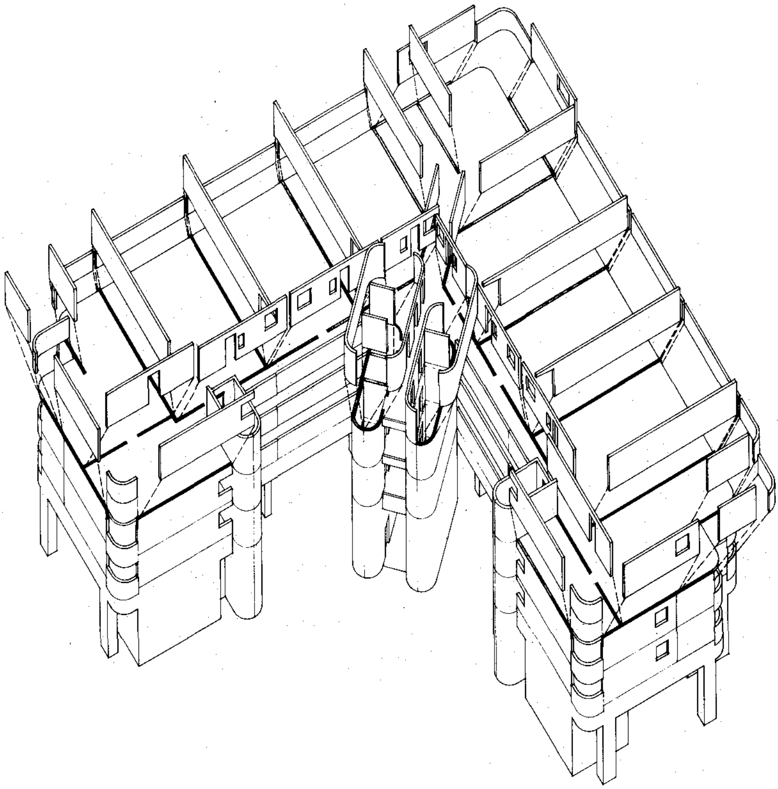


Fig. 2b. Isometric view of Atlantis Condominiums showing the arrangement of the various precast elements.

cal, functional, and structurally sound. Figs. 1 through 4 show various examples of buildings employing large loadbearing wall panels. In addition, Figs. 3, 4a and 4b show how hollow-core slabs can be combined very effectively with loadbearing wall panels.

In this presentation, my aim is to update you on the general direction that regulatory bodies are taking with respect to previous concerns over the potential for progressive collapse of bearing wall structures, to outline general principles for design, detailing, and construction which would greatly improve structural performance under unforeseen loadings, and to suggest sources for detailed technical solutions to carry out these principles. It is not my intent to dwell deeply on details for individual systems, but rather to suggest sources for guidance on such details.

Background

In 1968, a dramatic chain reaction collapse following a localized gas explosion on the eighteenth floor totally destroyed one quadrant of the 22-story precast concrete panel construction Ronan Point apartment building in England (see Figs. 5a and 5b).[1] Such widespread propagation of

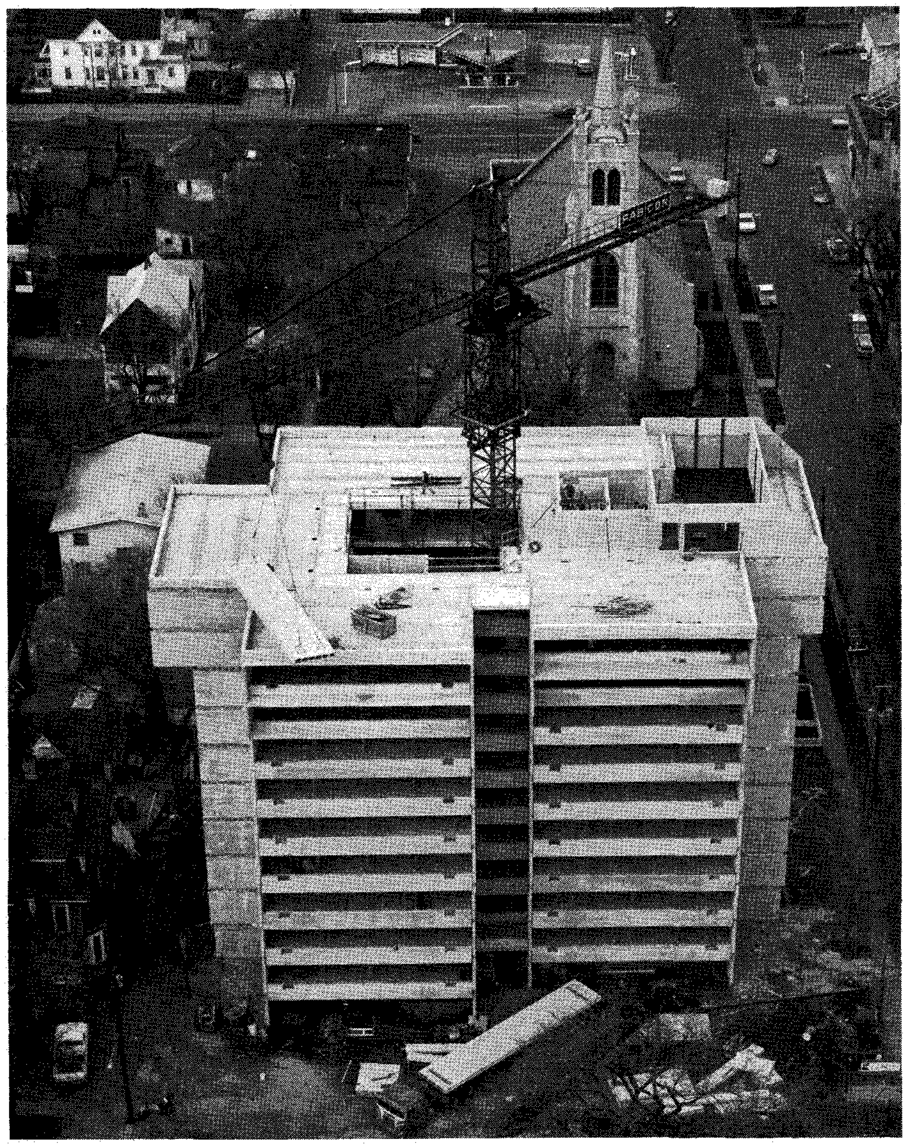


Fig. 3. Phillips Tower, Minneapolis, Minnesota. This ten-story building for the elderly consists of 328—8 x 24 to 26-ft x 8-in. solid precast wall panels; 316—8 x 42-ft x 12-in. precast hollow-core floor slabs; 54—4 x 42-ft x 8-in. solid raked precast spandrels; and 21—4 x 8-ft x 4-in. precast hollow-core floor panels. (Note: 1 ft = 0.305 m; 1 in. = 25.4 mm.)

Fig. 4a. The 20-story Roberts Plaza Apartment Building in Regina, Saskatchewan, is comprised of some 2000 precast prestressed hollow-core slabs, loadbearing wall panels and other precast elements.

Fig. 4b. Closeup of Roberts Plaza Apartment Building showing texture of loadbearing wall panels and hollow-core slab being swung into position.

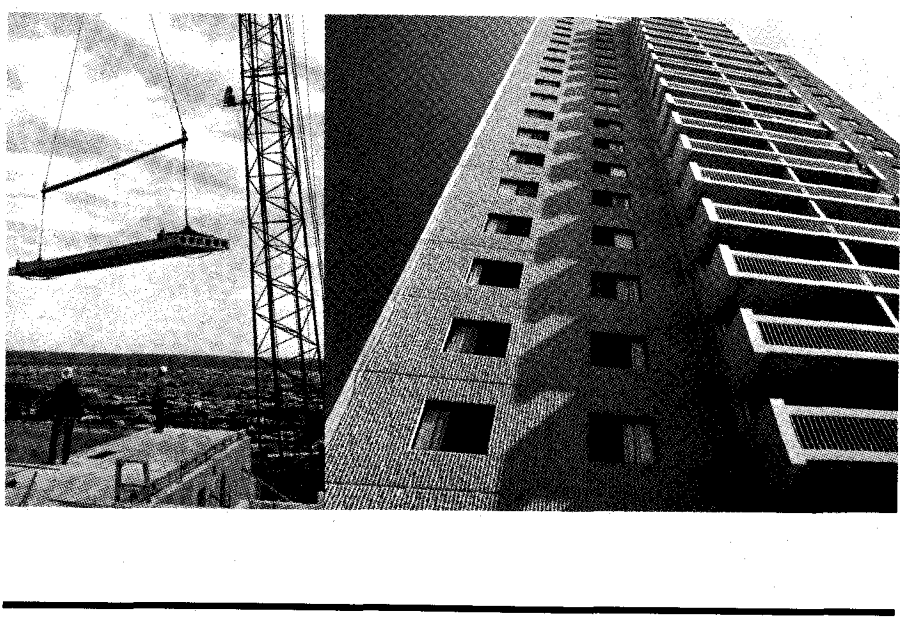


Fig. 5a. Ronan Point Apartment Building after collapse, with a second identical building in background. (Courtesy: London Express News and Feature Services.)

failure following damage to a relatively small portion of the structure has been termed "progressive collapse."

Today, the term progressive collapse has come to signify an incremental type of failure, where the total damage done is considered out of proportion to the initial cause. Definition as an *incremental* type of failure eliminates consideration of the total collapse of statically determinate structures, such as a truss, upon loss of a single member.

While the Ronan Point collapse drew attention to this failure mode, it was certainly not a unique occurrence. A large number of progressive collapses have been documented in the engineering literature, many occurring before 1968.[2] There are examples of vertical collapse propagation as in Ronan Point and Bailey's Crossroads (see Figs. 6a and 6b), horizontal pro-

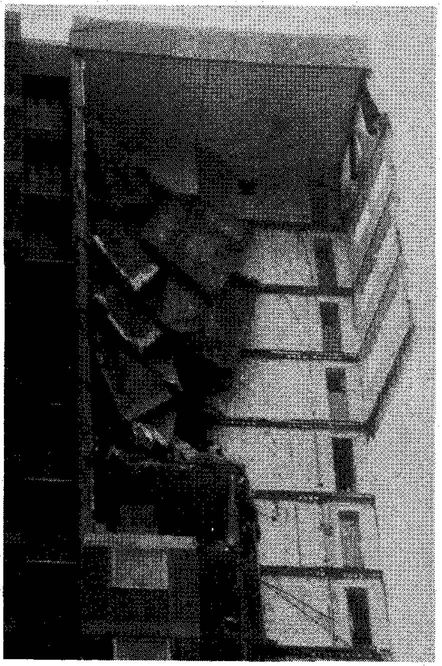


Fig. 5b. Closeup of damage at top portion of Ronan Point Apartment Building (Reference 1).

Fig. 6a. Cast-in-place concrete apartment collapse (Bailey's Crossroads, Virginia). Vertical propagation occurred from a shoring error on the upper stories of building.

Fig. 6b. Debris at base of tower due to building collapse (Bailey's Crossroads, Virginia).

pagation (see Fig. 7), and combinations of the two.

The particular type of joint detail used in the Ronan Point apartment building relied heavily on joint friction between elements. This resulted in a structure which has been termed a "house of cards." This occurrence indicated that structures with similar joint characteristics were particularly susceptible to progressive collapse.

Indeed, occurrences have happened in all major modern construction materials (reinforced and prestressed concrete, steel, wood, and masonry). Many types of structures are most susceptible to this type of collapse while under construction; however, examples exist of several completed structures which have also undergone progressive collapse.

Because the Ronan Point collapse was initiated by an explosion, which is a loading condition not generally considered in the design of buildings, it was termed an "abnormal loading." Several studies have been undertaken to predict the frequency and magnitude of similar loading conditions. In some studies this category has been extended to include faulty practice, such as design and construction errors.

In the years following Ronan Point, literally hundreds of engineering articles and reports on these subjects have been published. An extensive annotated bibliography on abnormal loading and progressive collapse has been published by the National Bureau of Standards as Building Science Series No. 67.[2]

Most European countries and Canada adopted some form of regulatory standard to minimize the risk of progressive collapse resulting from abnormal loading. These standards were difficult to meet in some bearing wall systems. However, very few building codes in the United States

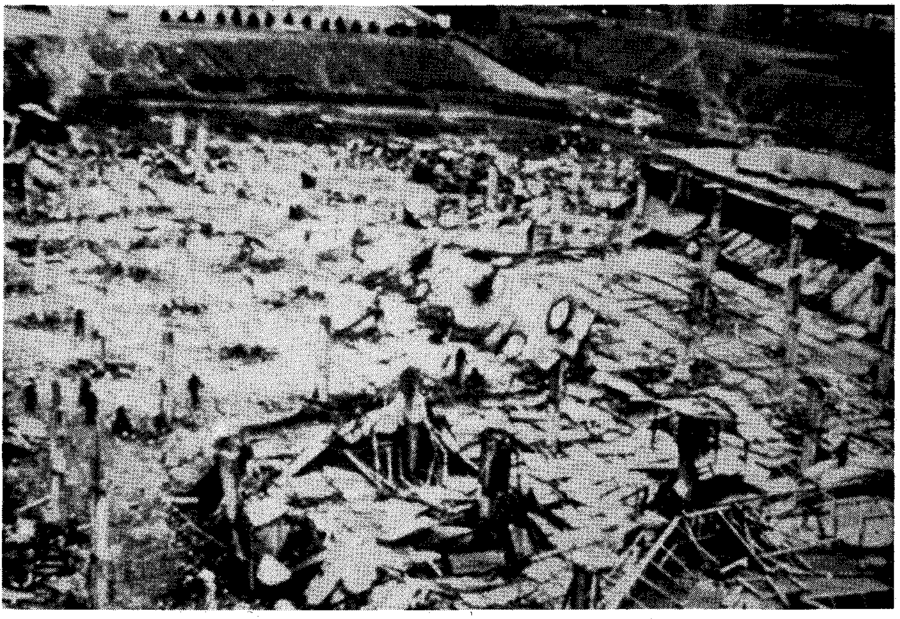


Fig. 7. Collapse of cast-in-place post-tensioned parking garage which was struck at one edge by falling tower crane and debris (Bailey's Crossroads, Virginia).

have taken specific action to include progressive collapse regulations although a number of possible procedures have been discussed.

Since the Ronan Point collapse, the Department of Housing and Urban Development has been a major stimulus for improvement in resistance to progressive collapse in industrialized construction. HUD stimulated research and developed regulation proposals both as a part of Operation Breakthrough and in connection with a major study undertaken at the Portland Cement Association.

While these studies have indicated that the types of construction suited for United States markets differed markedly from industrialized construction in Europe, only a narrow segment of the design and construction community showed an interest in undertaking serious studies of requirements in this area. Recent programs aimed at improving the resistance of bearing wall structures to seismic loads have contributed extensively to a parallel resistance to progressive collapse.

A few consulting engineers[3-5] who studied the progressive collapse problem indicated that they felt the joint details used in Ronan Point would be unacceptable in North America, and that American practice would be considerably more conservative than that used in the design of Ronan Point. They argued for adoption of design principles with respect to jointing and continuity similar to those used for earthquake design to minimize the danger of progressive collapse in precast systems. Most importantly, they urged that evaluation of the danger of progressive collapse and the method of resistance to this incremental type of failure should be made in the United States against the background of American practice and building regulations, rather than a simple copying or immediate adoption of European codes and regulations.

Despite this counsel, a study of the recommendations adopted by the New York City Building Code[6] indicated that this advice was often disregarded and that the European approach was adopted in some cases almost verbatim, at least as an interim measure.

The Aftermath of "Ronan Point"

The report of the Ronan Point Commission of Inquiry[1] revealed several deficiencies in existing British codes and standards, particularly as they applied to multistory construction. The Commission focused on the lack of redundancy or "alternate paths" in the structure. As an offshoot of the investigation, the British building regulations[7,8] were changed (The Fifth Amendment) to require that multistory structures be designed either to provide an "alternate path" in case of loss of a critical member or to have sufficient local resistance so as to withstand the effects of a gas-type explosion.

Implementation of these recommendations produced a great deal of both controversy and uncertainty. Continental authorities[9-11] were quick to point out that the 1967 CEB Recommendations[12] had spoken of panel structures as a "house of cards" and had called for mechanically continuous networks of reinforcement to prevent progressive collapse. Regrets were expressed that the official Ronan Point inquiry report did not specifically point out that the collapsed building violated the CEB principles.[9] It has been claimed that if these principles had been followed, the progressive collapse would not have occurred.

The Ronan Point report urged improved detailing to toughen bearing

Fig. 8. Algerian bearing wall concrete building which survived a major explosion (Reference 1).

wall buildings and cited successful examples such as an Algerian apartment building which effectively contained the spread of damage from a major explosion which destroyed loadbearing panels on the ground and first floors (see Fig. 8). Several engineers have indicated that the 5 psi (720 psf) (34.5 kPa) pressure required in the specific local resistance design procedures was excessive and that the alternate path method of design was confusing and complex.

The reactions of design professionals in Great Britain to the aftermath of the Ronan Point collapse were mixed.[13-16] Generally, the design professionals recognized that there were substantial social implications inherent in the problem of progressive collapse if an occurrence in one dwelling space of a high-rise structure could jeopardize the lives and property of occupants of far-removed dwelling spaces.

The general concepts of risk analysis which led to the conclusion that there was a high probability of an explosion within a high-rise structure were not attacked. However, the specific implementations of the "Fifth Amendment" to the building regulations met a great deal of criticism. Very little factual data were available as to the economic consequences. The regulations were basically criticized because of the lack of existing knowledge of the effect of gas and other type explosions on buildings. In particular, substantial questions were raised as to the dynamic characteristics of typical explosions and the response of various types of structures to such internal explosions.

The entire concept of designing for redundancy in the construction of buildings was questioned because of the long history of statically determinate structures used in civil engineering construction. Much criticism had to do with the conflict between building regulations and the personal responsibility of the designer.

Particular criticism was raised concerning provisions which stated that traditional forms of construction such as steel and concrete framed structures which met existing building codes were "deemed to satisfy" the special progressive collapse provisions. It was pointed out that no thorough study had been made in this area and it was certainly possible to design statically determinate structures in conformance with existing codes and regulations.

Reaction to the Ronan Point incident and the amendments to the British building regulations which followed quickly spread worldwide. The precast concrete industry became the target of jokes (mostly unjustified) such as the cartoon (Fig. 9), which appeared in *Punch*, a British humor magazine. Unfortunately, undue caution and too many regulations set industrialized construction back.

Similar code provisions were adopted in many countries and in the United States the Department of Housing and Urban Development[17] circulated for comment a draft document to implement such standards in construction under its mortgage insurance programs. The City of New York[6] amended its building code to require resistance to progressive collapse by either the alternate path method or the specific local resistance to 720 psf (34.5 kPa) method.

A National Bureau of Standards program [18-21] documented the frequencies of occurrence and risk analysis of abnormal loadings. A Portland Cement Association program[22,23] suggested design and construction considerations for large concrete panel buildings and was a moving force in the development of an overall philosophy to reduce the risk of progressive collapse by incorporating improved overall structural integrity in the large panel structures. Both programs have emphasized the American aspects of the problem presented by U.S. characteristics in loads, spans, building layouts, and construction practices.

Abnormal Loadings

The term "abnormal loading" has been used to indicate any loading condition not generally considered in the design of a building. For traditional construction and typical American codes and standards, designers usually consider dead load, live load, snow load, wind load, earthquake load, soil load, hydrostatic load, and effects of temperature and dimensional changes.

Abnormal loadings would be loads which have generally been considered to have such a low probability of occurrence as to warrant neglect in design, such as:

1. Violent Change in Air Pressure
   (a) High explosive detonation (sabotage, suicide)
   (b) Service system explosion (gas unit or gas system leaks)

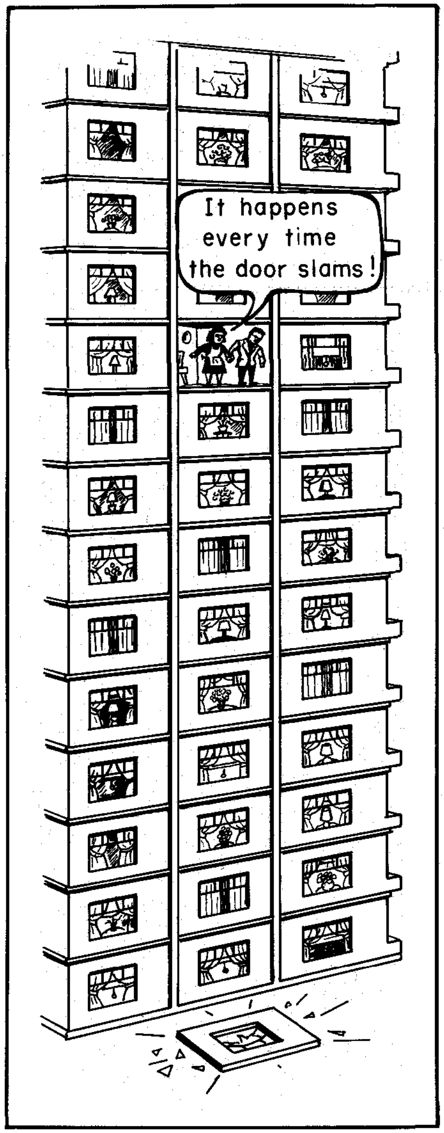


Fig. 9. British cartoon poking fun at prefabricated apartment buildings (Courtesy: *Punch*).

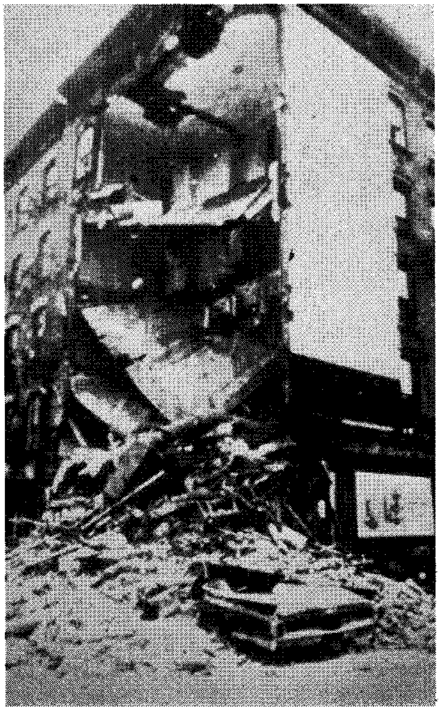


Fig. 10. The driver turned the corner of this building too tightly.

2. Accidental Impact
   (a) Automotive (Fig. 10)
   (b) Crane (Fig. 11)
   (c) Airplane
3. Loads Due to Faulty Practice
   (a) Construction error (Figs. 12 and 13)
   (b) Unauthorized alteration
   (c) Lack of maintenance
4. Fire
5. Flood
6. Tornado

A major part of the initial program in progressive collapse research at the National Bureau of Standards was to determine the frequency of occurrence of various abnormal loadings for residential and commercial type structures in the United States.[18-21] The results of these studies established the probability of occurrence for the various abnormal loadings that while not precise seems to give at least an order of magnitude assessment.

In general, the results agree fa-

Fig. 11. Parking garage struck by falling crane (Cleveland, Ohio).

Fig. 12. Egyptian apartment building after loss of a foundation. Note that building is identical to one in background.

vorably with similar studies completed in Canada, England, and The Netherlands. There appears little likelihood of developing significantly better information on frequencies and probabilities for the United States until a more comprehensive data bank is established.

In all the studies, the probability of occurrence of a relatively severe gas explosion is the highest. Next in probability is a high explosive (bomb) explosion. This type of explosion seems to be on the increase and is extremely serious. One major problem in all of the frequency studies of abnormal loads is that the reporting information concerning the structural consequence or the magnitude of the load is so scanty that it is difficult to determine whether any structural significance should be attached to the given reported incidents.

Specific information on the magnitude of abnormal loads that might be expected can only be characterized as marginal. The 5-psi (34.5 kPa) pressure used in the specific resistance method of calculation in the United Kingdom's standards following Ronan Point was based on theoretical calculations of probable pressures in a gas explosion in that size unit and on examination of the damaged appliances and piping. A pressure of 5 psi

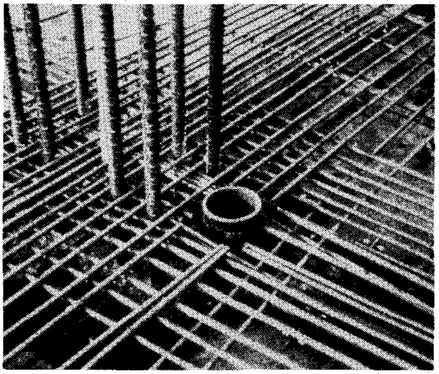


Fig. 13. The plumber came late.

(34.5 kPa) was believed relatively representative of the pressure that occurred in the Ronan Point incident.

There is substantial argument concerning the magnitude of pressure that might be expected in a typical incident in an American living unit because of size differences, venting characteristics, and other factors. Conclusions drawn from some studies have indicated that the combination of interior partition venting (if effective) and the general layout of American apartments could perhaps combine to reduce explosive pressures to a level closer to 1 psi (17.2 kPa) rather than 5 psi (34.5 kPa). This conclusion has been vigorously denied.

There is an interrelationship between abnormal loads and progressive collapse. This interrelationship has significance in connection with risk. Risks are the products of the probabilities of an occurrence and the consequences of that occurrence. Progressive collapse has higher consequences because it is usually accompanied by an increase in injuries and loss of life or property than would accompany a local collapse. This means an increase in the risk to individuals or to society. It is thus necessary to consider the scale effect.

There are other disasters that can cause large damage, such as tornados, hurricanes, floods, and earthquakes. These are termed "Acts of God." They cannot be prevented but the consequences could be reduced and there is some attempt to do that with our building codes. However, accidents that lead to large damage usually lead to rather immediate reaction from the public and from the regulatory agencies and they can and usually are prevented or steps are taken to prevent them by means of regulatory actions. Because progressive collapse has higher consequences, loadings of lower probability must be considered in order to keep the risk constant, as compared to a failure that does not involve progressive collapse.

The entire area of abnormal loading should be substantially deemphasized. There is some need for development of frequency statistics for the purpose of risk analysis and regulatory provision justification. In a few isolated cases provision of a loading force or pressure would be useful in designing a specific element in an unusual structure. However, in most cases the level of attention paid to assessing the magnitude of possible abnormal loadings is not justified for the most feasible methods of handling the overall problem of progressive collapse.

The British experience indicated that the 5-psi (34.5 kPa) loading adopted in the specific resistance method led to costly solutions without necessarily ensuring the desired safety. The only area of application where a pressure loading might be helpful for some engineers was in selecting values for proper tie forces or calculating wall rupture loads. This does not really justify a large research effort in the area of abnormal loadings.

More effort should be spent in defining the level of damage which is acceptable or is to be contained than in assessing a magnitude of load. Acceptable damage can consider factors such as occupancy, area, volume, cost, and use.

Progressive collapses can be triggered by other causes than explosive loads. The magnitude of the abnormal loads becomes less important if the design approach taken is to provide overall structural integrity which will bridge and contain local damage. A positive emphasis on improved structural integrity to limit the propagation of damage is far more desirable than using the specific resistance method to try to withstand some arbitrary load. As a result, the importance of abnormal load magnitude is greatly diminished, except as an indicator of the amount of initial damage which might be expected.

Generally speaking, improved structural integrity is obtained by provision of integral ties throughout the structure. The amount of ties can be determined from considerations of debris loading and the amount of damage to be tolerated without using the magnitude of the explosion or other abnormal load.

The futility of pursuing the subject of abnormal loading in depth is best illustrated by some recent remarks of an experienced designer. He believed that a substantial number of structural engineers would be extremely uncomfortable with a 20-story bearing wall building with no ties even if gas systems were prohibited, the building was erected in the center of a golf course miles from any adjacent structure, and armed guards were on duty at all hours of night and day in the lobby!

Panel and Bearing Wall Structures

The risk of a progressive collapse in large panel and bearing wall structures is greater than the risk in traditional cast-in-place structures. The increased susceptibility to progressive collapse is due to a combination of the use of relatively brittle materials and the general lack of ductility and continuity in the overall structure because of the details used in assembling the pieces. Structures such as the Ronan Point apartment building, utilizing basically friction connections for resistance to lateral forces, are completely unsatisfactory as a structural type.

The addition of suitable ties to develop continuity could have probably made that structure resistant to the consequences of the abnormal loading to which it was subjected. A number of solutions are available to the problem of increasing the structural integrity of large panel and bearing wall buildings. However, a great deal of design ingenuity is required to translate these into efficient, economical construction.

The susceptibility towards progressive collapse is further increased in American practice because the buildings tend to contain fewer intermediate walls and supports than in European practice. This reduces the chance of redundancy and development of an alternate path. An examination should be made of the general configuration and the joint details of early bearing wall buildings to determine their susceptibility to progressive collapse. Remedial action should be taken to develop improved continuity in any buildings adjudged to be particularly susceptible to progressive collapse.

The direct applicability of a large body of the research findings and design recommendations developed for European practice is reduced because of the very large differences which exist in building layouts between American and European residential structures. European structures typically have bearing walls surrounding almost every room, slabs cast to room size for a particular job with reinforcing steel protruding from the slabs on all peripheries, and thus the development of continuity is much easier with interlacing of the protruding reinforcement and a cast-in-place (wet) joint.

American architecture calls for a very different structural layout. While European walls are often spaced at 15 to 20 ft (4.6 to 6.1 m), typical spans in the United States vary from 22 to 40 ft (6.7 to 12.2 m). Intermediate non-loadbearing partitions are used to subdivide living units up into rooms. The proportion of walls to slabs in American construction is frequently as low as one-third the ratio found in European buildings. In addition, hollow-core slabs produced on long casting beds and cut to length according to the needs of the job are prominent in the United States. With this form of floor slab construction, no protruding reinforcement is available at the ends of the slabs to develop continuity. Thus, the continuity details must be considerably different.

Because of the pattern of labor organization in the United States, large concrete panels are often connected by dry joints utilizing bolting or welding, so that the iron worker does not have to wait for the masonry or concrete craft to complete the joint before proceeding with further erection. Efforts to import the types of building systems prevalent in Europe have been generally unsuccessful, both because of economic factors and because of a general unacceptability of the architectural layout prevalent in Europe.

The extensive research program on large panel concrete structures at the Portland Cement Association concentrated on building arrangements, panel connections, and span proportions typical of conditions in the United States. The generally reduced number of vertical elements in American structures means that the problem of the overall stability of the damaged structure is probably more severe than occurs in European structures. Thus, the design philosophy implemented in the United States should pay careful attention to the need of ensuring stability in the overall structure after the loss of one or more elements.

Design Philosophy to Resist Progressive Collapse

Any design and construction requirements imposed to reduce the probability of a progressive collapse must follow a consistent overall philosophy to insure effectiveness. The consideration of the possibility of a progressive collapse assumes that, due to some overload or weakness of the structure, a local failure has already occurred. Supposedly, normal factors of safety have been set to reduce the possibility of such a local failure to a generally acceptable level. Design to resist progressive collapse recognizes that a local failure cannot be prohibited absolutely. The general philosophy is that a structure should be stable under that local damage and be able to bridge over the damaged area without complete collapse of the structure.

A fundamental problem in implementing this design philosophy is that it is difficult to quantify the volume of damage which the structure must be capable of sustaining without progressive collapse. Studies are needed to define socially and technically acceptable volumes of damage as related to use, occupancy, and stability.

The various regulations issued in Europe,[7,8] Canada,[24] and in draft form by HUD,[17] give some indication of the extent of damage being considered. Generally, the structure must be able to take a reasonable amount of debris load and resist the loss of a principal bearing member. Practical implementation of such regulations in the design and construction of structures has indicated that specification of an abnormal load such as a gas pressure is relatively meaningless. An indication of the volume and type of damage which must be contained is far more effective for designers seeking to develop effective resistance in unique situations.

There appears to be a general consensus that it would be desirable for panel and bearing wall structures to have the same degree of resistance to progressive collapse as traditional monolithic construction, such as found in typical cast-in-place concrete con-

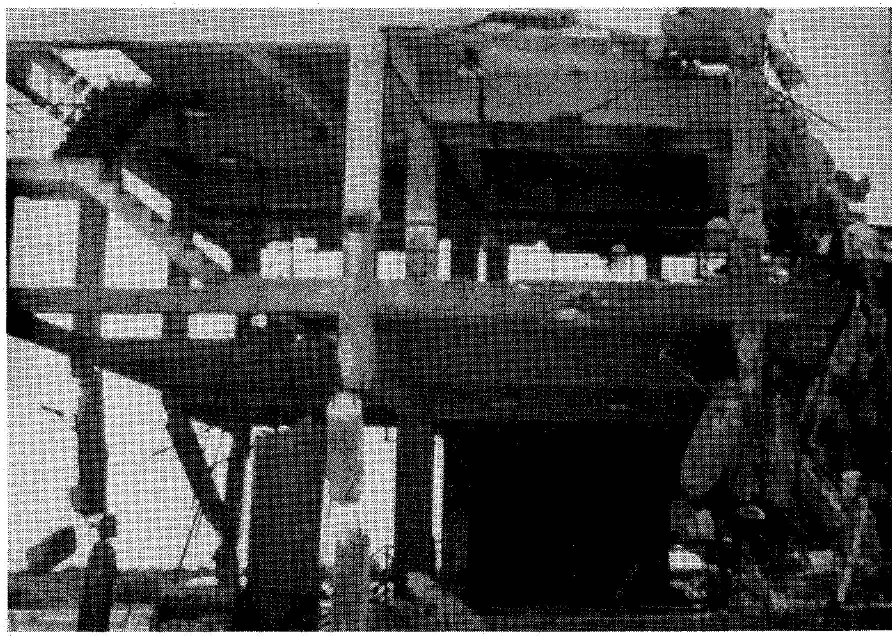


Fig. 14. General structural integrity of a cast-in-place frame structure with 75 percent of the columns on one story destroyed during a demolition attempt. Incredibly, the structure is still standing.

struction. Detailing practices for cast-in-place structures requiring arbitrary percentages of reinforcement to be brought to the supports, minimum tension capacity in columns, and overall development of reinforcement impart substantial toughness in a structure. This capability is demonstrated in Fig. 14 which shows a condemned building resisting the "attacks" of a demolition contractor!

This does mean that the design and construction procedures for precast panel and bearing wall structures should try to develop overall stability, ductility, and redundancy similar to what might be expected in cast-in-place construction.

In some unusual forms of construction it may be necessary to prevent the propagation of collapse laterally by provision of frequent expansion joints which will isolate the structure into acceptable segments. In general, there are so many different types of structures, methods of construction, and possibilities of initial damage that regulatory groups have found it very difficult to quantify the functional requirements for resistance to progressive collapse. It has been far easier to rely on the ingenuity of the designer after assurance that he has developed a sound understanding of the overall design philosophy.

Design to resist progressive collapse is in essence an advanced limit state, since it already assumes that a local portion of the structure has failed. Because of this, calculations are usually made using load factors approaching unity for foreseeable debris load and partial live load (usually one-third to one-half of design live and wind load). The structure must be stable under this load to allow for evacuation and emergency operations and permit temporary support or repair. More specific guidance needs to be provided as to appropriate safety factors under this condition and appropriate strength reduction factors for assessing member strength.

For satisfactory resistance to progressive collapse, the structure must maintain a stable configuration after the extensive damage to or loss of a major member. Design of a structure so that it can develop a substitute load-carrying configuration which will permit it to remain stable upon the loss of a support makes a substantial contribution towards eliminating the danger of progressive collapse. The very large tributary areas carried by individual bearing walls in United States construction impose substantial problems in engineering stability upon loss of two walls. Preliminary studies indicate that in many panel buildings typical of American practice the structure can be made to bridge over a missing bearing wall at relatively little additional cost. Extremely large additional costs would be involved if the structure had to bridge two or more successive missing walls.

It is necessary to ensure stability by provision of suitable compression struts and tension ties to allow the structure to bridge over the damaged area, as indicated in Fig. 15. Any structure suspected to be susceptible to progressive collapse should be carefully investigated for stability under reasonable local damage. This check may be done indirectly by providing improved overall structural integrity by ensuring that proper details are used to develop continuity and redundancy and that an overall stable structure results. In some very special cases, it may be necessary to actually check a structure under several different configurations corresponding to removal of key supports.

Much can be done to improve overall stability in the initial architectural layout and arrangement of bearing walls. This can be achieved by bringing the structural engineer in at an early stage of the design. Certain patterns of wall layout will make it much easier to develop bridging. Overall provision of ductility and continuity at the joints can assist in developing resistance to progressive collapse equivalent to that of frame-type structures.

The concept of limiting damage propagation means that careful consideration must be given to resisting the debris which might follow the failure of an adjacent member. In European practice, designers have tried to develop an inherent load-carrying capacity in a floor, so that it can resist a debris loading equivalent to the weight of the floor above plus 30 percent live load imposed with a substantial impact factor. This debris must be carried as a superimposed live load, although it may be carried at exceptionally large deformations and with substantial damage to the member.

With the long spans of American practice, this imposes a substantial load requirement for a member. One possibility for reduction is to develop tie forces and details which will prevent a large part of the debris from falling on the member below when a failure occurs.

In any case, any debris loading that is imposed must be carried in shear as well as in flexure at the limit state. Failures which have occurred in some American large panel structures indicate that the weakest point of the section under debris loading was the shear strength at the supports.

If floor systems are designed to ensure effective membrane or catenary action upon loss of an intermediate support, the initially damaged member must carry its own debris load. This can generally be done effectively by developing proper tie

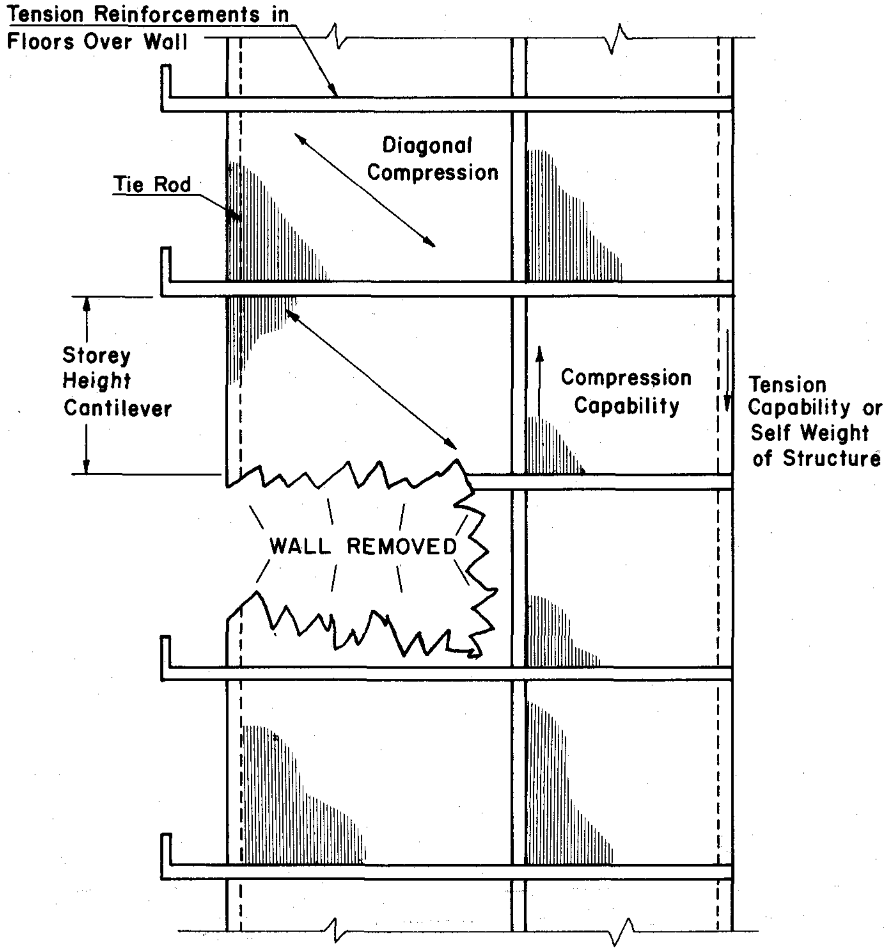


Fig. 15. Bridging in a bearing wall panel structure.

forces at the ends and ensuring that bottom reinforcement in the member at the point of the missing support is fully effective as tension reinforcement by proper anchorage or designed as a splice. In order to provide a suitable gap to ensure that the membrane or catenary action is effective in preventing the debris from bearing on the floor below, the limiting deflection under failure conditions should not exceed approximately one-half the clear story height.

Design of all members in the structure should include details to ensure that proper compression and tensile resistance is possible, so that the structure can bridge over a missing outer support, as shown in Fig. 15. Effective vertical ties are required as tiebacks and compression struts are required for fulcrums.

Careful attention must be paid to develop continuity of vertical wall and floor panels, so that shear transfer is obtained to allow the bridging wall to act as a deep cantilever beam. Substantial experimental programs and design ingenuity are needed to study various arrangements of bridging members and to determine the most efficient ways to provide the tensile, compressive, and shear resistance in the panels and joints.

The most important method to minimize the risk of progressive collapse in panel and bearing wall structures is to provide adequate horizontal, vertical, and peripheral ties between all structural elements to develop improved structural integrity. Emphasis on ductility and continuity similar to that used for seismic and wind design is the most useful technique for minimizing the risk of progressive collapse.

Most structures designed and detailed to resist seismic loads in UBC's Seismic Zones 2 and 3 in the United States would have a low susceptibility to progressive collapse. However, buildings in low wind and low seismic areas might be quite susceptible to progressive collapse. The PCI Committee report on precast bearing wall buildings[25] recommends that all bearing wall structures be designed for a minimum lateral total design force equal to 2 percent of the service dead load.

In structures which are designed for substantial lateral forces, the designers are used to the concept of developing diaphragm action in the floor and wall elements, in order to provide flow paths horizontally and vertically for the lateral forces. However, in some regions of the United States, designers are not accustomed to developing diaphragm action. They are principally concerned with gravity loads and not as attentive to the needs of tieing all of the elements together (Figs. 16 and 17). The use of panels with small bearing areas, no protruding reinforcement, and generally inadequate connections is common in low seismic and low wind load areas.

Fig. 16. Two-story tiltup bearing wall building with grossly inadequate connection details (Baton Rouge, Louisiana).

These buildings will be the most susceptible to progressive collapse.

Both design practices in the United States and the general direction of the Portland Cement Association large panel project emphasize the provisions of adequate longitudinal, transverse, vertical, and peripheral ties, as shown in Fig. 18. The function of the longitudinal ties placed in the floor system (often in joints between floor planks) is to ensure that the floor can develop membrane or catenary action and restrict debris loading from impacting on the floor below. The function of the transverse ties (often placed in the joint above the wall panel) is to create cantilever action in case the wall panel is removed. This cantilever action is a major element in ensuring stability by bridging. The function of the peripheral ties is to ensure overall diaphragm action of the floor and to provide adequate anchorage for the longitudinal and transverse ties. The peripheral tie can also create an edge member to assist membrane action in case a corner loses support.

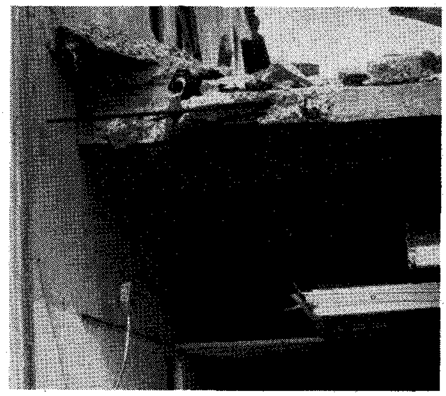


Fig. 17. "Friction Connection" of a lateral beam to a tiltup bearing wall. Note that the angle beneath slab was added subsequent to original construction.

Vertical ties are provided to act as the tension tieback for cantilever action and to help string the wall panels together to reduce the chance of a panel being knocked out. All ties must be designed and detailed so that the connections will hold through load reversals and resist impact loads.

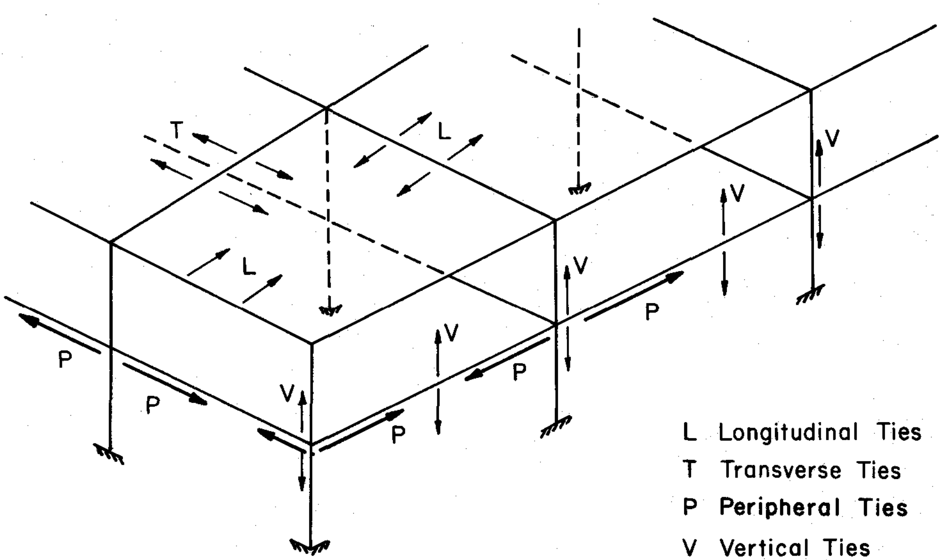


Fig. 18. Tie forces needed to develop overall structural integrity.

Panel and Bearing-Wall Construction Systems

The predominant construction materials used in panel and bearing wall structures in the United States are precast prestressed concrete floor and wall panels, reinforced concrete floor slab and wall panels, and masonry walls with concrete, bar joist, or wood floor systems.

The Prestressed Concrete Institute has responded to the problem of progressive collapse very responsibly. A comprehensive report on design considerations for precast concrete bearing wall buildings to withstand abnormal loads was developed by a PCI Committee and published in the March-April 1976 PCI JOURNAL.[25]

This report recommended development of a degree of continuity and ductility which can develop reasonable resistance to progressive collapse without undue economic penalties. The report emphasizes that historically there has been insufficient attention towards developing overall structural integrity in panel structures and recommends that the structure be tied together in all directions. The level of tie forces specified is roughly equivalent to current construction practices.*

The American Concrete Institute investigated design and construction requirements to minimize the risk of progressive collapse through parallel activities in ACI Committee 356 (Industrialized Concrete Construction) and through a Task Force on Progressive Collapse of Committee 318 (Standard Building Code). Since the ACI Building Code does not contain loads but depends on a general building code for specific load requirements, it appears more appropriate that it concentrate on factors involving details and development of appropriate continuity and ductility rather than including performance or functional statements regarding the resistance to progressive collapse. The major changes from the provisions for prestressed concrete would appear to be that tie forces could be reduced since the span lengths are generally significantly shorter.

A substantial area of concern is the current state-of-the-art concerning the resistance of masonry structures to progressive collapse. The initial reaction of the masonry industry to the HUD draft criteria on increasing resistance of buildings to progressive collapse was to essentially consider the problem as not applicable to masonry structures. There seems to be a growing awareness that this is a potential problem for masonry structures as well, which must be met.

ACI Committee 531 (Masonry Structures) has recently reported new design rules which should improve structural integrity.[26] While substantial research programs are underway in the masonry area to improve overall structural integrity for resistance of seismic loads, no formal consideration of provisions to minimize the risk of progressive collapse has been generally indicated.

One of the fundamental problems in masonry design is the lack of a comprehensive ultimate strength design procedure. Since calculations for tie forces and bridging assume a limit state wherein the structure is already partially collapsed and the attempt is to confine the spread of that collapse, the elements are working at very near their ultimate strength. It will be difficult to include block, brick and composite masonry until a fundamental

*Edge ties on the periphery must develop the diaphragm force but not less than 16 kips. Horizontal ties should be provided at right angles. Those across the floor span must develop at least 1500 lb/ft while in the direction of the span it should develop 2½ percent of the wall service load but not less than 1500 lb/ft. Vertical ties should be provided in buildings over two stories to develop any tension but not less than 3000 lb/ft.

understanding of masonry behavior is developed.

Review of a number of actual masonry designs indicated large potential susceptibility to progressive collapse.[27] Engineers who have reviewed several masonry building designs expressed disappointment at the level of engineering, particularly in non-reinforced masonry design. While designers utilizing masonry in high seismic zones seem attentive to details to develop diaphragm action, large regions of the country use high-rise masonry structures with essentially gravity loads analysis. The resulting designs and details may be extremely susceptible to progressive collapse.

There is a substantial amount of test data available on the performance of reinforced and unreinforced masonry elements, such as individual panels. However, there seems to be relatively little information on the behavior of overall structures and representative wall-floor joints. The large amount of masonry construction in the United States and the increasing use of masonry in high rise (greater than three stories) construction indicates that this is a major area needing research. Generally, the manufacturers of masonry have not been attuned to the need for research on overall structural behavior.

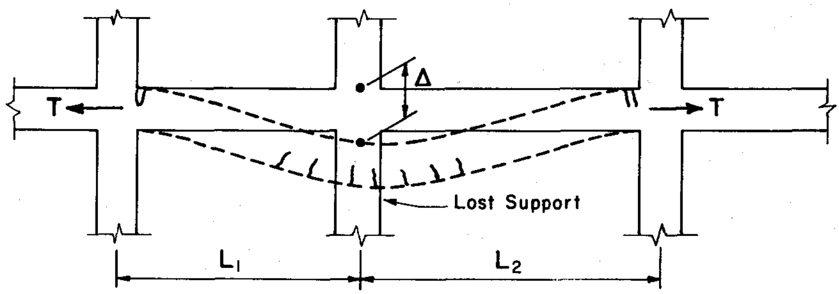
Much research work has been done on masonry structure resistance to progressive collapse in Europe.[28-30] As in the previous discussions, the structural layouts are for relatively short spans as compared to American practice. A great deal can be learned from a review of this work but substantial additional input is required to determine the resistance of typical American masonry buildings and floor systems.

Properly constructed masonry is very rigid and the field assembly gives the opportunity for development of continuity, if proper panel reinforcement and jointing details are included. The large variety of mixed construction (masonry-concrete-masonry-steel, etc.) indicates that a larger variety of details will have to be examined. Particularly with non-reinforced masonry, wall capacity near collapse may be weaker than the joint capacity. Masonry structures appear to need a more extensive testing program than reinforced concrete and prestressed concrete panel construction, since the wall panels themselves and not just the joints may be potential failure locations. For this type of failure analysis, it is doubtful that the tension or shear capacity of the masonry should be used in calculations of the shear and tensile forces needed to provide adequate ties. This indicates that substantial changes in practice will be required.

Membrane or Catenary Action

By the provision of adequate horizontal ties it is possible to develop a membrane or catenary action in the floor slab above the origin of the disaster. This can serve to arrest progressive collapse of the overall structure by ensuring that the damaged floor slabs are held together and do not add to the debris load on floors below. An extremely large deflection in the slab can be tolerated under this extreme condition.

Typical calculation of the magnitude of horizontal tie forces required to ensure membrane or catenary action is illustrated in Fig. 19. British practice has been generally based on spans of approximately 17 ft (5.2 m) and the assumption that two-way membrane action will be available, so that the tie force in either direction may be cut in half. Typical recommendations in the British Code



Catenary Action

$$T \cdot \Delta = \frac{W(L_1 + L_2)^2}{8}$$

$$T = \frac{W(L_1 + L_2)^2}{8\Delta}$$

For $L_1 = 20$ ft.
$L_2 = 24$ ft.
$W = .2$ K/Ft.
$\Delta = 4$ ft.
$T = 12.1$ K/Ft.
Grade 60 – # 4@12"

2 Way Action – Cut in Half | Grade 60 – #4@24"

Fig. 19. Determination of tie forces assuming two-way catenary action.

of Practice call for ties with a capacity of 4 kips per ft (58.5 kN/m).

Tests on typical European structures indicate that floor slabs with ties actually behave somewhat better than the catenary analysis indicates. The assumption of two-way action with one-half of the load carried in each direction seems to be borne out by these tests and work from the Portland Cement Association. There is a need for further testing with longer spans and substantial variations in length-to-width ratios to determine the correctness of the assumption as to longitudinal and transverse distribution. Typical North American application with longer spans generally results in substantial increases in tie forces.

The catenary analysis is again a limit state analysis for dead load and partial live loads. While the British calculations have been based on a deflection of 15 percent of the span, it seems reasonable that a maximum limit such as one-half of the clear story height could be adopted.

There are several recommendations for the magnitude of the horizontal tie forces. Many of these are based on a catenary analysis for relatively short spans. Others are based on simply providing tie forces equivalent to what commercial fabricators are now providing in buildings. A substantial amount of research is needed to investigate procedures for proper calculation of tie forces, and basic criteria for loads to be carried by the floor system should be codified.

Provision of horizontal tie forces implies proper anchorage of the ties at all supports. It will be necessary to prevent the tie from pulling out at the edge support and to act effectively as a splice at the central support which might be destroyed. This will require substantial improvement of tie details.

Cantilever/Bridging Action

The flank and corner walls in a panel or bearing wall structure are both the most vulnerable to many types of abnormal loads and are the hardest to replace with an alternate load path. Relatively small edge stiffeners and effective longitudinal, transverse, and peripheral ties greatly increase the capacity of a corner slab to cantilever over missing supports. Relatively light partitions can provide strong points to assist the cross walls in supporting cantilever action. Dry wall partitions commonly used in the United States may not be effective in this regard.

Proper design of the wall and tie system will allow the structure above the damaged zone to cantilever over the missing support. Comprehensive studies are required to determine what level of ties and shear connectors are required to fully develop such cantilever action in all forms of panel and bearing wall construction.

Tests on large concrete panel construction at the Portland Cement Association confirm that shear along the horizontal joints can be critical. Proper details and analysis for shear capacity are required to ensure that brittle shear failures will be avoided. The transverse ties must work with the vertical ties to provide effective clamping action at the joint.

The vertical ties can be extremely important in acting as suspender rods to carry the floors and walls immediately over the damaged area. If this is envisioned in design, appropriate anchorage must be provided at each story level for this function.

Diaphragm Action

The peripheral ties are necessary to establish an edge beam around the structure at the level of each floor to provide proper anchorage for the longitudinal and transverse ties, to improve cantilever membrane action at the corners, and to develop diaphragm action in the floor slab for the entire story.

While most authorities have recommended that the peripheral tie forces be similar in magnitude to the tie force within the edge strip of the adjacent floor, there is no clear rationale for determination of the appropriate level of force in the general case.

Joint and Details

There is a very clear consensus that panel and bearing wall structures should be designed and constructed to have joints with adequate strength, stiffness, and ductility. Specific provisions which will ensure the strength, stiffness, and ductility of joints are still very uncertain and need substantial additional research. The recommendations of Reference 25 provide interim guidance.

The details which will ensure ductility at the joints and continuity with the wall in various systems must be developed and verified. Designers must be given insight and analytical tools to indicate not only the magnitude of the tie forces but the most efficient location and method of placement of the tie elements concerning both ductility and strength.

Design procedures will of necessity be based on observed behavior of various tie systems in laboratory or prototype tests. It is essential that these tests have realistic anchorage details and be conducted with the very large deformations which are associated with collapse level loadings. In general, it is necessary to determine anchorage and splice behavior at levels far beyond the point where previous testing which has formed the basis for most current design rules has been discontinued.

Anchorage details should carefully simulate prototype applications. Such a factor as loss of ductility due to the notch effect in threaded connections may prevent formation of the catenary action.

The large rotations often associated with collapse level loads can cause the point of load applications on supporting walls to shift and give substantial eccentricities. Overall strength of the wall as well as the behavior of the joint will be affected by this movement.

Very little is known on the proper detailing to develop the catenary force system in structures typical of practice in the United States. Effects of confinement and variations in steel overlap should be studied under extreme deformation conditions. The focus of the studies should be the development of efficient details, and design for efficiency at large deformations rather than simply an evaluation of traditional detail patterns. In the interim, the designer must provide ingenuity to determine acceptable solutions.

As an example of a practical tie application, the large concrete panel research project at the Portland Cement Association studied the use of unstressed tendons as a highly efficient way of developing resistance to catastrophic loads. One of the principal advantages of these tendons is that they can be embedded in joints and are flexible enough to allow for normal tolerances in abutting the lengths of the precast units forming the joint.

Further experimental programs are needed to study the requirements for strength, stiffness, and rotation of the joints in typical panel and bearing wall structures after a local collapse. Such studies will provide a measure of the strength and ductility requirements for the joints, which will be a basic factor in assessing the adequacy of any joint. Actual joint tests should model the complex state of load which exists on the joint as a three-dimensional body. This indicates that some work will need to be performed at essentially full scale in the laboratory and not on models until complete correlations are obtained between prototype and model behavior.

Joint Classification

Because of the wide range of jointing techniques for the large number of types of panel and bearing wall structures, a classification system should be established to ensure that representative joint types are evaluated. Substantial development of such classification systems is underway in NSF-sponsored studies in the RANN program in earthquake engineering.[31]

A large number of systems utilized in the United States use dry or semidry joints in contrast to the prevalent wet joint in Europe. The utilization of interlocking hooked bars is much less in the United States than in Europe. As a result, relatively little of European literature on performance of joint systems is applicable to American practice. However, behavioral ideas established in European tests can be of substantial benefit in assessing the rotational needs and the factors which contribute substantially to such rotational development.

The common use of saw-cut extruded floor slabs in the United States places emphasis on the determination of the importance of grouting in and between panels and on the adequacy of grouting ties into the cores. Tests will be required to ensure sufficient bond at the level of deformations expected.

Wide use of tack-welded connections raises questions concerning the brittleness of the connections with improper quality control. In connection with the notch sensitivity of threaded connections, this may produce zones of weakness under impact-type loads.

In some cases tie forces may depend on erection and manufacturing tolerances. This is particularly true in lateral forces due to out-of-plumb members and in floor plank members where relatively inadequate bearing exists under the ends even before large deformations take place. A survey is needed of actual field tolerances for all types of construction. Throughout the study on details in joints, it is important to get input on the way that details will affect construction practice. Success of any research studies in this area will depend on active involvement of designers and constructors as well as competent research professionals.

Joints in Precast Panel Structures

In addition to the general questions regarding joint strength, stiffness, and ductility, field experience has indicated that the shear strength of hollow-core slabs immediately adjacent to the joints can be a critical factor in collapse conditions.

Attention should be paid to the possibility of development of additional shear strength by filling the voids with mortar or by applying shear reinforcement in the webs. The longitudinal and transverse tie systems can be used to resist the shear forces and the section can be checked by shear-friction theory.

Masonry Construction

The state-of-the-art in design and detailing of joints in masonry structures for strength, stiffness, and ductility is far behind the state-of-the-art in large concrete panel construction.

One of the main weaknesses in assessing the behavior of an unreinforced or reinforced masonry structure at collapse load levels is the fundamental lack of a behavioral oriented strength design method for masonry. The absence of an accepted basic relation between axial compression and flexure in the presence of strain gradient precludes the development of basic strength and deformation theories for masonry, which are needed before general analytical treatment at limit state conditions can be carried out.

Masonry differs substantially from reinforced concrete in fundamental behavior, in planes of weakness, and in stiffness distributions. The construction process makes it easy to develop horizontal ties and it is possible that the basic pattern for providing an alternate path for loads in masonry structures might depend more on suspending cables from horizontal cantilever sections than is generally done with concrete structures.

Specific details for effective jointing in masonry must consider construction sequence and type of inspection to be provided. The type of joints selected may greatly influence the cost of the structure if it hampers the productivity of the workmen.

The use of unreinforced or partially reinforced masonry in high-rise construction seems to be contrary to the idea of improved overall structural integrity. In addition, the question of openings and connections of lightly reinforced lintels in otherwise heavily reinforced wall panels provides substantial planes of weakness. Inspection of damage after the San Fernando earthquake indicated variations in efficiency of grouting the reinforcement into the masonry cells. Certain types of masonry bonded well to the grouted bars while others did not.

One of the characteristics of masonry construction is that all types of floor slabs are possible and are used. Precast and cast-in-place concrete slabs, bar joists, corrugated metal decking, and wood all have been utilized for flooring systems. The development of alternate load paths and sufficient tie forces in these wide varieties of structural systems needs extensive evaluation. Some construction systems may be shown to be undesirable for this application.

The present status of information on the behavior at large deformations of the wide variety of joints possible is extremely meager. There is a very low possibility of being able to assess the actual effectiveness of typical ties and grouting systems for extremely large rotations in development of catenary actions from test results currently reported in the United States. While a number of research programs in masonry have been directed towards seismic resistance, the joint details are generally not typical of the types of details widely used throughout the United States. The majority of data concerns joints which are much heavier reinforced and have substantially more effective grouting than found in practice.

Research work in this area can undoubtedly benefit from close coordination with research programs underway to improve the seismic resistance of masonry structures at several agencies and universities on the West Coast.

Masonry structures have a great deal of potential for developing resistance to progressive collapse. Placement of reinforcement between wall and slab units should be easier than in precast units. Provision of reinforced bond beams and tie beams should substantially increase the resistance to progressive collapse. However, it is necessary to take a broad look at masonry structures and consider not only ideal behavioral characteristics, but effective quality assurance programs.

Economics

Engineers experienced in design and construction of panel and bearing wall structures which have tie forces deemed sufficient to greatly improve the overall structural integrity have indicated that the effort to obtain required toughness and ductility is not very expensive. Designers report that on the first project in which they are providing extra tie forces, there is some extra design cost. However, this quickly diminishes on subsequent projects as personnel become familiar with the concept and typical details are repeated.

Construction costs could increase from 0 to 10 percent when tie forces and detailing are provided to improve overall structural integrity. There is generally a very small cost if the design starts with the premise of providing suitable tie forces, while the cost is substantially higher if an existing system which was designed with very inefficient or insufficient joints has to be converted to meet the new requirements.

Future Directions

After extensive discussion of various strategies for minimizing the risk of progressive collapse in panel and bearing wall structures, the attendees at a major national workshop on the problem[32] felt that the most practical procedure was to adopt a positive requirement that would encourage the designer to develop an integral three-dimensional structural system for carrying gravity and lateral loads. Such an integral system would rely on proper longitudinal, transverse, vertical, and peripheral ties, to ensure that all members interacted and that a high level of ductility and continuity was obtained.

After substantial discussion and in response to a specific request for a sense of the meeting as to what an appropriate direction for the ACI Building Code to take might be, the workshop passed the following resolution with an approximately 85 percent affirmative vote:

"With regard to large panel structures, it is agreed that:

1. Satisfactory control over progressive collapse can be provided by embodying in ACI 318 requirements for horizontal and vertical ties.
2. These Code requirements can be of a qualitative nature.
3. Commentary provisions can be quantitative either specifically or by reference.
4. No reference need be made to 'progressive collapse' either in the Code or Commentary."

In discussing the motion it was brought out that this positive approach emphasizing the beneficial effect of tie forces and providing guidance to the designer was extremely appropriate for a material code specification when there is no general code requirement in an overall building code which applies to all materials. It was the consensus of the group that an action of this sort would greatly reduce the danger of a progressive collapse in a large concrete panel structure and prevent the occurrence of a "Ronan Point" in the United States. It was recognized that substantial effort was required to provide the quantitative values for tie forces for the Commentary.

This appears to be the way the industry and profession is moving. The general principles are clear. The details are fuzzy. We *must* provide overall structural integrity in our bearing wall structures. It is now up to our ingenuity to come up with details that do this dependably and economically.

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NOTE: A list of references on bearing wall buildings is provided on the next couple of pages.

Discussion of this paper is invited. Please forward your comments to PCI Headquarters by July 1, 1980.

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