A philosophy for structural integrity of large panel buildings

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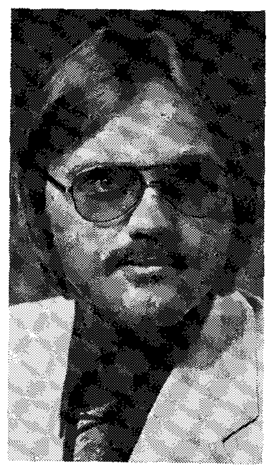

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When a local failure is not confined to the area of initial distress, but spreads either horizontally or vertically through the structure, it is termed a progressive collapse.

More accurately, it may be defined as a phenomenon in which the spread of a local failure eventually results in the collapse of a whole building, or a disproportionately large part of it.

Abnormal loading conditions and effects which can be structurally significant[1-3] include:

- Service system (gas) explosions

- Explosive bombings
- External explosions from accidents involving transportaton of hazardous materials
- Ground vehicular collisions with buildings
- Aircraft collisions with buildings
- Tornados
- Flooding
- Foundation settlements, and
- Errors in design and construction.

Traditionally, it was assumed by the profession that if established practice and codes were followed, the resulting building would automatically have a degree of structural integrity which would assure an alternate path for the load when a load-carrying element failed (Fig. 1).

This belief was justified for some traditional forms of construction which have continuity and ductility; however, it was not valid for every form of construction.[4-12]

In 1968, attention of the engineering community was focused on the problem of lack of structural integrity in certain precast structures in a tragic way. In May of that year a progressive collapse of an apartment building occurred at Ronan Point in London (Figs. 2 and 3).

A gas explosion in an apartment on the 18th floor caused an exterior panel to be blown out (Fig. 4a); this initiated a progressive collapse upwards to the roof, and then almost down to the ground, as debris fell on succeeding floors (Fig. 4b).

In terms of hazard, the initial damage was of minor consequence. The progressive collapse which occurred, however, was the result of the inability of the structure to bridge over the local failure, that is, its lack of integrity.

Therefore, it is not merely the hazard of abnormal loading—it is also the susceptibility of a structure to progressive collapse which presents the real risk.

Synopsis

The paper reviews the various methods to reduce risk from abnormal loads.

To limit the occurrence of progressive collapse in large panel residential structures, a philosophy for establishing General Structural Integrity is developed to assure bridging of local damage while maintaining overall stability, thus eliminating the need to design for any particular abnormal load.

In this approach, tensile continuity and ductility of the elements and their connections, as well as of the overall structure, is stressed.

The rationale for a minimum tie system consisting of transversal, longitudinal, vertical and peripheral ties to establish this General Structural Integrity is developed.

The objective of this approach is not to afford absolute safety in regard to every exceptional event in any part of every building; rather, the intention is to limit and substantially reduce the general risk of collapse, as compared to that existing if no such measures were taken.

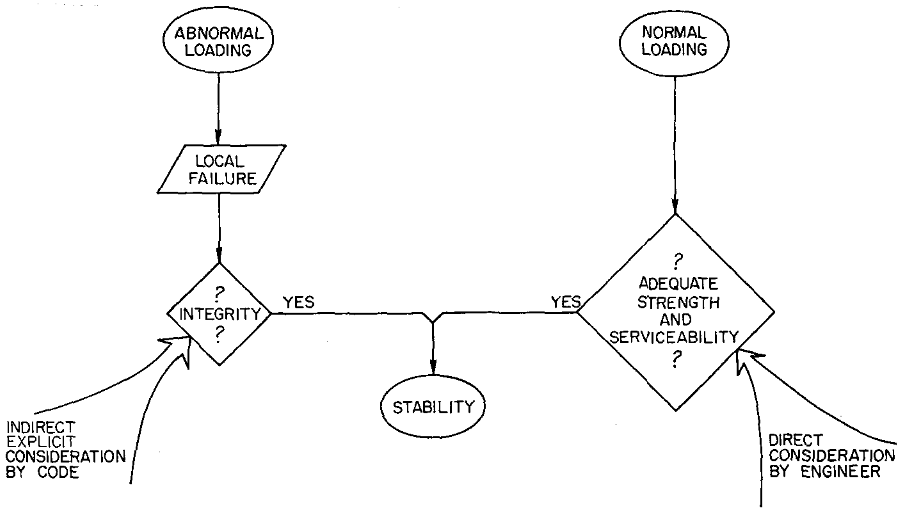


Fig. 1. Schematic diagram of design process with adequate detailing standards.

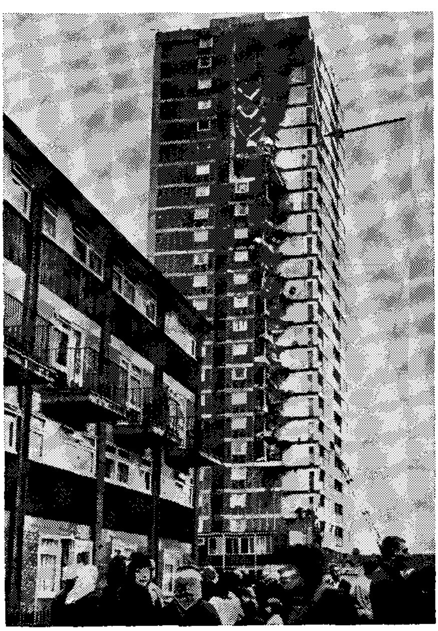
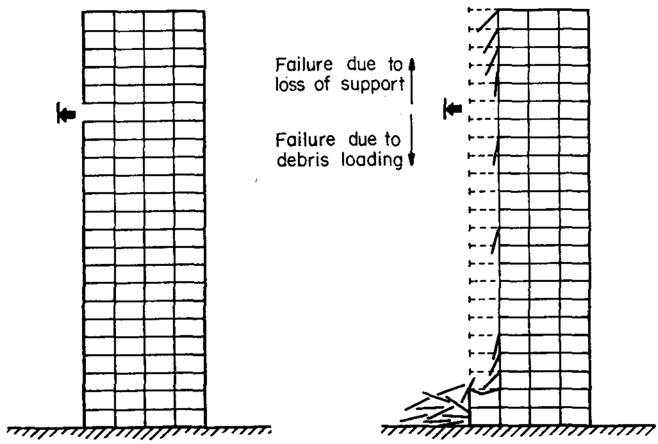


Fig. 2. Collapse at Ronan Point.

Fig. 3. Collapse at Ronan Point.



(a) Immediate local damage (b) Progressive collapse

Fig. 4. Failure modes of Ronan Point collapse.

Reducing the Risk of Progressive Collapse

Three alternative approaches, widely varying in concept, can be used to reduce the risk of progressive collapse:[13]

1. Eliminate the hazards which cause local failures (e.g., elimination of gas installations in multistory buildings, as has been done in France).

2. Design the structure so that the hazard does not cause any local failure (e.g., 5 psi gas explosion load requirement initiated in Great Britain).

3. Allow the local failure to occur but design the structure so that progressive collapse does not occur (ensure an alternate path for the load).

Elimination of hazards

Regarding the first approach, it can be assumed that certain abnormal loadings could actually be elminated: gas installations could be disallowed in large panel buildings; barriers could make vehicular collision avoidable; and construction in flood zones could be regulated.

However, with the above exceptions, abnormal loads can hardly be eliminated; also, most of them defy quantitative determination. Since any realistic solution must deal with *all* abnormal loading conditions to some extent, this method of eliminating the hazards cannot be deemed an overall solution.

Local resistance

In the second approach, providing local resistance, explicit loading requirements are established in an effort to provide sufficient strength to resist any local failure. In lieu of a probabilistic approach, it has been suggested that a minimum design load of 5 psi for gas explosions be used.

However, the approach of providing resistance to one specific abnormal load has been challenged on a number of grounds. It is not reasonable to assume that adequate resistance for one abnormal loading condition will necessarily provide sufficient strength to resist all other abnormal loading conditions.

From an economic viewpoint, additional costs to provide certain safety against local failure may not be justifiable in light of the small number of cases where progressive collapse of large panel structures has occurred.

Alternate path through structural integrity

Since the nature and magnitude of most abnormal loads are unpredictable, the third approach of allowing local failure to occur but providing an alternate path within the structure to avoid an overall collapse appears to be a sound concept. In this approach, a local failure is accepted as an inevitable consequence.

When a structure has the ability to bridge over local failure it has what will be termed in this article General Structural Integrity (GSI), the principal elements of which are continuity and ductility of members and connections and of the structure as a whole.

The General Structural Integrity approach can be implemented in two distinct manners: the design engineer can be required to apply a rational procedure to establish the necessary integrity; or, alternatively, code writers can develop the necessary minimum detailing practice (as has been done for other forms of construction) to establish a degree of continuity and ductility. In either case it is necessary to identify the special sensitivity to abnormal loadings that the particular structural form possesses.[4-10]

Minimum detailing as code requirements is recommended as the most viable alternative.* Although it is sometimes more complex, and by its nature, more general, it has several advantages:

1. Code writers and researchers have a general responsibility to evaluate details to assure the safety of structures.

2. Design engineers should not be required to directly consider the effects of abnormal loads for one form of construction and not for another.

3. Experience in other building forms has shown that minimum detailing requirements based on good engineering judgment can establish an adequate degree of structural integrity.

While, by its very nature, LP construction does not lend itself easily to a design with moment continuity, it is possible to develop force continuity and ductility of the connections as well as of the overall system through minimum details specified as recommended practice.

Since such minimum requirements cannot possibly encompass all situations, substantial reduction in the risk of progressive collapse in all future designs could be achieved if the detailing requirements are supplemented with an educational program for the engineering profession.

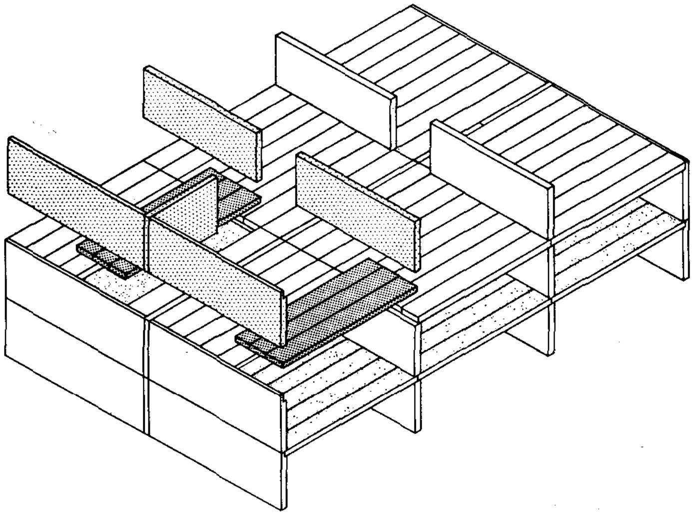
Prototype Structure

The cross wall and spine wall type of large panel residential structures (Fig. 5) have been selected as the representative prototypes for this study since they are the most popular types used in the United States.

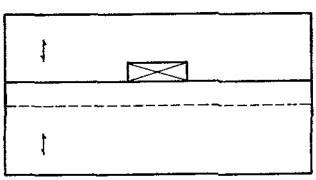
The reasons for their popularity are the economic advantages of a low wall-to-slab ratio, resulting from the comparatively large slab spans of up to 42 ft utilized until now. The large distance between the concrete bearing walls permits an open, flexible architectural layout using light partitions.

An added advantage of the cross wall type is the architectural freedom in handling the longitudinal elevations. With individual variety for each building made possible, the uniformity inherent in industrial production can be avoided.

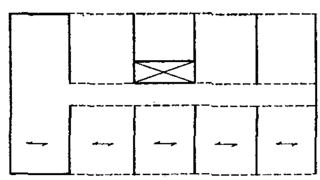
* Most recently this approach has been adopted by the PCI Committee on Precast Bearing Wall Buildings.[14]



Isometric view of typical large panel structure



Cross wall system



Spine wall system

Fig. 5. Typical large panel structures.

Rationale for General Structural Integrity

Connections between the precast elements of LP structures are recognized as the weak link in this structural system. This weakness is attributable to the lack of continuity through the connections, and to their brittle nature.

Friction-type connections can theoretically provide sufficient stability for gravity loads within an LP structure; however, such connections offer little resistance to the effects of abnormal loadings.

Recent studies and discussions [7-12,15-25] on the performance of structures under normal and abnormal loads have concluded that the integrity of a structure, regardless of the type of construction, depends on its connection characteristics; and that the strength of the elements cannot be utilized if the connections have inadequate strength.

Therefore, as stated earlier, a degree of continuity *across* the connections and ductility *within* the connections should be ensured to achieve General Structural Integrity.

Continuity is essential to develop the bridging capabilities needed for transmission and redistribution of loading (alternate paths). Ductility is necessary, not only to sustain the deformations that may be associated with conditions of partial stability (stability of the structure in the damaged state), but also to establish some measure of energy absorption under the dynamic effects of both normal and abnormal conditions.

The magnitude of required continuity can be determined by assessing the forces acting within the connections under various conditions of local failure. Because of the nature of LP structures, these forces occur only as compression, tension, and shear.* Adequate compression and shear capacity in LP connections will usually be provided by design under normal loads.

However, little tensile capacity is found between elements in LP structures; therefore, tensile capacity between elements across the connections (both horizontally and vertically) must be provided. This required tensile continuity across and within the connections can be effectively achieved by providing the ties shown in Fig. 6.

The rationale for GSI should encompass the following:

1. **Extent of damage**—An assessment of the nature and extent of local damage that is likely to occur under abnormal loadings;
2. **Alternate paths**—Evaluation of alternate structural actions which can develop as a consequence of a local failure to re-establish load flow in the remaining undamaged structure—stability analysis of the partially damaged structure; and
3. **Tie requirements**—Quantitative determination of requirements for transversal, longitudinal, vertical and peripheral ties to bridge local failures and to ensure stability of the partially damaged structure.

Extent of Damage

The extent of local damage which the structure must be capable of bridging affects the amount of ties needed to establish alternate structural action. Code writing bodies in the United Kingdom[26] and Sweden[27] have acknowledged that a certain amount of local structural damage is considered inevitable and acceptable by specifying the extent of local failure which the structure must be able to bridge. The specified amount of damage load-bearing wall is to be assumed as "notionally removed" from the structure at any location within a given building.

The main premise when using the "notional removal" approach is that an element or portion of the structure has been totally removed. In LP buildings with brittle joints which depend primarily on friction and bond under compressive loadings, the "notional removal" concept may be a realistic representation of their response to abnormal loadings. However, as vertical and horizontal ties are introduced within and between the wall elements and floor components, it becomes less likely that the elements can be *literally* removed.

Therefore, with ductility and continuity in the form of ties between structural elements specified as recommended practice in American LP construction, the "notional removal" approach becomes somewhat questionable.

With LP structures tied together horizontally and vertically it is reasonable to assume that in most instances wall panels become "ineffective" as a result of abnormal loadings. They will no longer function as load-bearing members as originally intended, but will remain in place in their damaged condition.

Whether wall panels become ineffective or are notionally removed is actually of minor consequence, since in either case the original load flow is interrupted and an alternate path must be available. In fact, differentiation between the two concepts is not required

* Development of significant moment continuity between individual elements is not considered essential technically nor easily attainable economically in usual American-type LP systems.

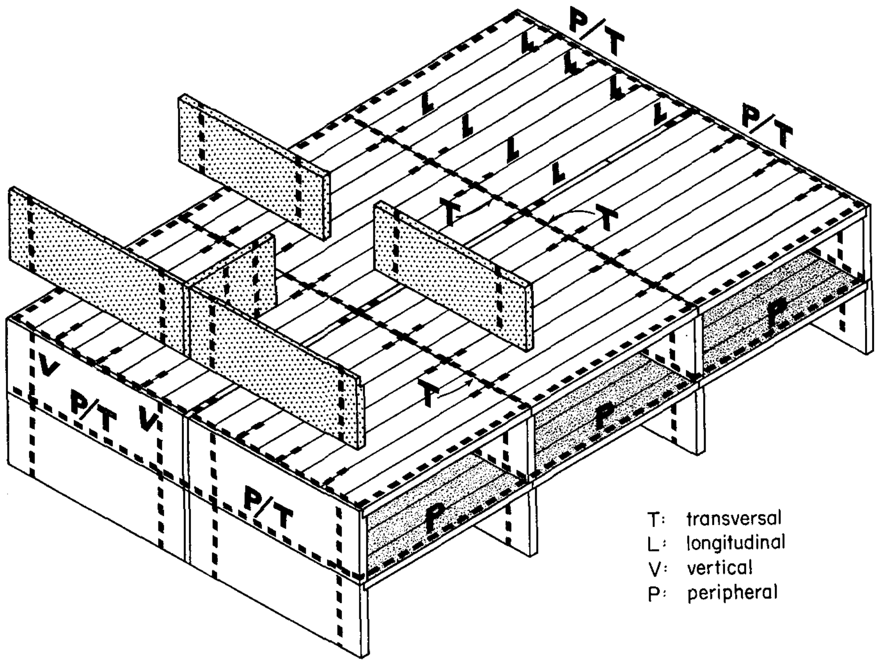


Fig. 6. Suggested system of tensile ties in large panel structure.

at all with regard to the main alternate paths available, that of cantilever and beam action.

The concept of ineffective behavior is introduced here primarily because of its effect on debris loading and slab behavior in the damaged state. Specifically, the requirements for the slabs to "hang together" over the damaged (buckled) wall differ from the requirement to develop full catenary action over the double span when the supporting wall is notionally removed. As a result, the tie requirements for the slabs using the "ineffective member" rationale will be different from those required if the "notional removal" approach is adopted.

When estimating damage extent, the following should be taken into account:

1. Probability of occurrence of an abnormal load for each element of the structure;

2. The consequence of failure of a particular element within the structure; and

3. The influence of a particular structural configuration and layout of the walls.

Element Vulnerability

Since the probable damage caused by abnormal loads differs in various parts of a building,[1] damage criteria which do not delineate the damage location, if applicable in one area, may not apply in other portions of the building. As a result, it is appropriate to define an element's vulnerability by its location within the structure, considering the consequence of its failure.

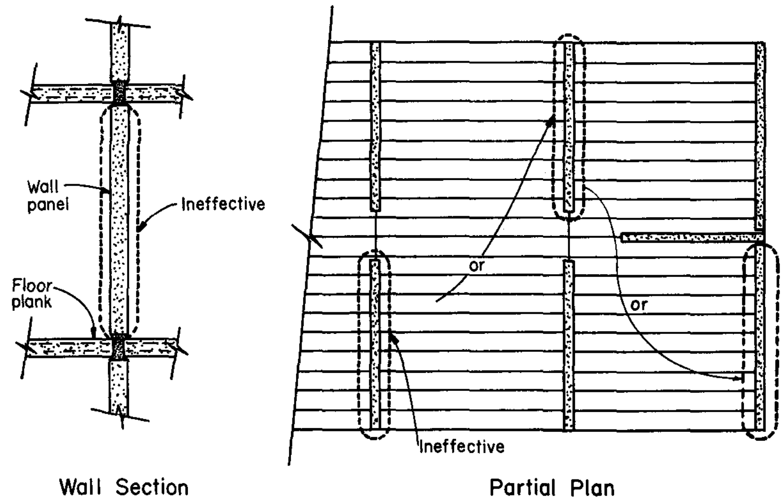


Fig. 7. Extent of assumed maximum wall damage under abnormal loads.

Exterior (flank) wall panel

The wall panels along the perimeter of an LP building are the most susceptible elements to abnormal loadings. Although the sizes of typical flank wall elements range from 25 to 45 ft in length, it is reasonable to assume that, as an outside limit, an entire unit of a flank wall assembly can become ineffective (Fig. 7).

This assumption can be somewhat relaxed, however, if measures are taken in the design to ensure that only a part of the wall panel becomes ineffective. Such measures may include "weak planes," or additional stiffening elements in the form of bracing walls or integral columns (Fig. 8).

Under the above conditions, "ineffective" indicates that the wall, although physically damaged and possibly buckled, is still "strung" in place, not allowing the slab units to drop to the level below to create impact loading.

Interior wall panel

Interior wall panels, being farther removed from the periphery of the structure than flank wall elements, are subjected to a somewhat lesser degree of risk from specific abnormal loadings. However, it is assumed as an outside limit that an entire wall panel can become ineffective as shown in Fig. 7.

As in the case of flank wall assemblies, this assumption can be relaxed if similar measures are taken in the design to ensure that the entire wall panel cannot become ineffective (Fig. 8).

Floor/roof panel

Floor/roof systems typically consist of side-by-side members (planks) up to 8 ft wide. As a result, full continuity of the system perpendicular to the span is interrupted at each joint.

It is reasonable to assume, therefore, that in case of an abnormal loading, only a few floor/roof planks would become ineffective for a full span length.

As the floor slab consists of individual planks, there is little tendency for the failure to spread beyond the planks affected by the abnormal loading.

For floor/roof elements, "ineffective" suggests that they are physically damaged and substantially weakened, and no longer capable of providing lateral support of wall units or of participating in the required diaphragm action. However, they are still strung together and capable of supporting their own dead loads within their original spans.

The most critical condition for floor/roof panels occurs when they lose their stability due to partial loss of one of their end supports, i.e., a wall panel. The loss of such support would be due to the ineffectiveness of the wall element below, resulting in large displacements at the original support. Such conditions require adequate tensile continuity between adjacent slab spans for the slab elements to hang together in their deflected shape.

Tie Functions and Alternate Paths

Structural design for "normal" loadings (those specified in codes) begins with an assessment of the loads on the building. These loads should be safely transferred from the point of application to the final resisting point through a logical load flow.

If a panel becomes ineffective as a consequence of an abnormal loading, a new load flow must be established in the remaining undamaged structure, i.e., an alternate path must be created.

To avoid progressive collapse which may occur if the structure is unable to

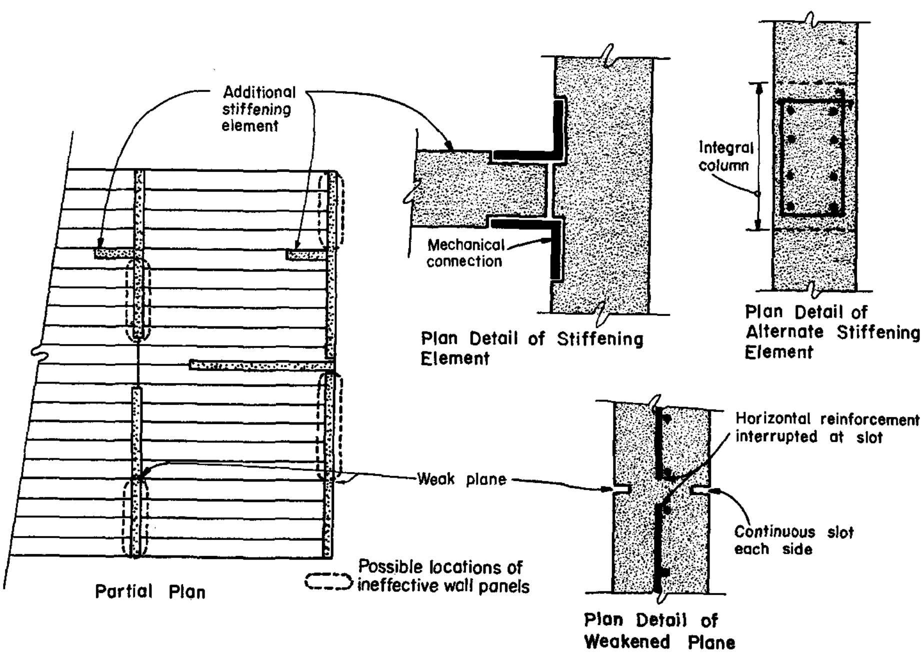


Fig. 8. Extent of assumed wall damage under abnormal loads with limiting measures taken.

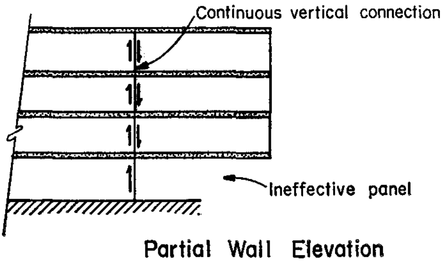


Fig. 9. Cantilever mode—Two adjacent wall panel stacks.

bridge local damage, alternate load paths should be available for any load-bearing wall or portion thereof that might become ineffective. For the slab elements such alternate paths are not essential. However, to avoid uncontrollable debris loading, provisions should be made to effectively tie (or string) the floor/roof elements together.

Tying the large panel structure together horizontally and vertically makes it possible to utilize the following structural mechanisms to bridge local failures:

(a) Cantilever action of wall panels;
(b) Beam action of wall panels;

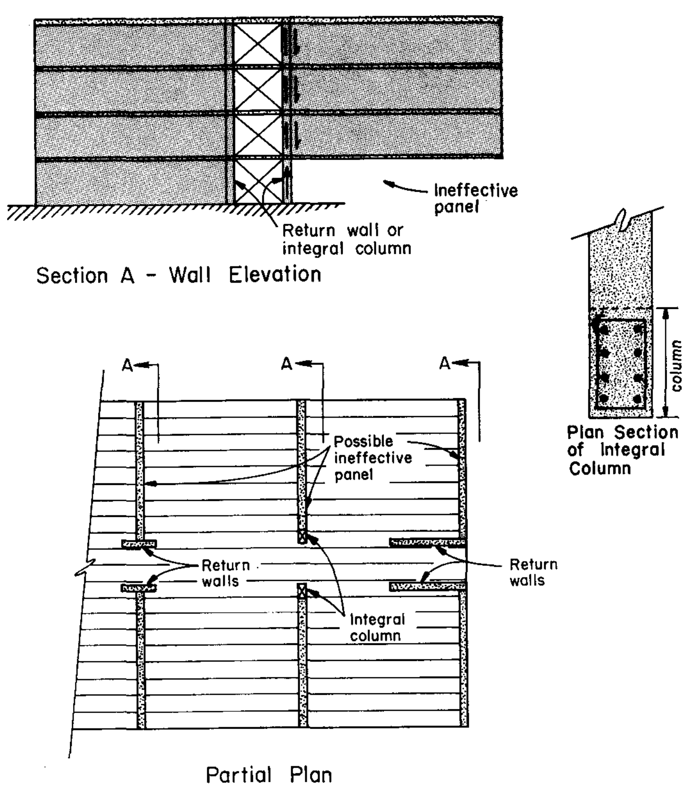


Fig. 10. Cantilever mode—Single vertical wall panel stack.

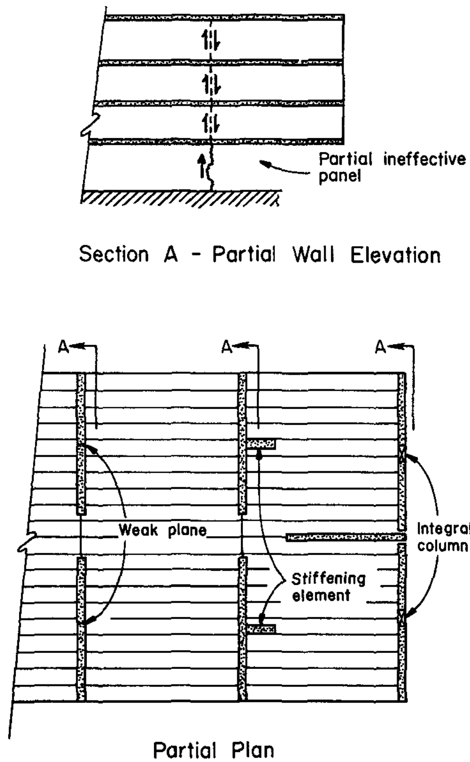


Fig. 11. Cantilever mode—Partial element ineffectiveness.

(c) Partial membrane action of successive spans of floor planks;

(d) Vertical suspension of wall panels; and

(e) Diaphragm action of the floor planks.

Transverse ties and cantilever action

When, at a given floor level a wall panel or portion thereof becomes ineffective, the wall panels above that level lose their support.

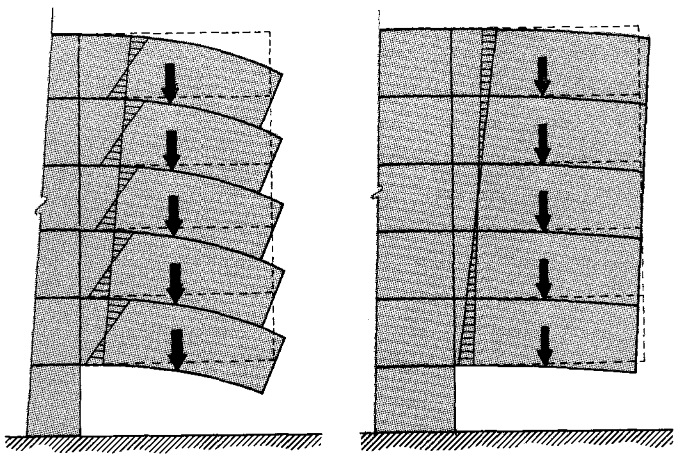
The most effective method of transferring the loads is through cantilever action of the remaining panels above. However, to obtain cantilever action, the means to transfer the vertical load at the supported end of the cantilever must exist. Vertical support at the cantilever root can be provided by:

(a) A vertical connection designed to carry the shear in each story in a wall assembly consisting of two or more adjacent vertical stacks (Fig. 9);

(b) A vertically continuous return wall or a vertically continuous integral column at the interior edge of the wall assembly consisting of a single vertical stack (Fig. 10); or

(c) The remaining portion of the wall (Fig. 11); special detailing should ensure that only a portion of the wall becomes ineffective.

Although cantilever support can be assured through proper panel design and layout, the cantilever moment can be developed only if adequate tensile continuity in the form of a reinforcing tie exists in the transverse horizontal



(a) No shear capacity in horizontal joints

(b) Adequate shear capacity in horizontal joints

Fig. 12. Modes of cantilever action.

connection between the wall panels. To assess these tie force requirements the behavioral mode must be determined.

Depending on the horizontal connection properties, two modes of cantilever action can be defined as giving upper and lower bounds to the transverse tie force requirements. In the cantilever action shown in Fig. 12a, no horizontal shear is transmitted between story-high cantilevers. The strength of this build-up cantilever is equal to the sum of the strengths of the individual cantilever panels.

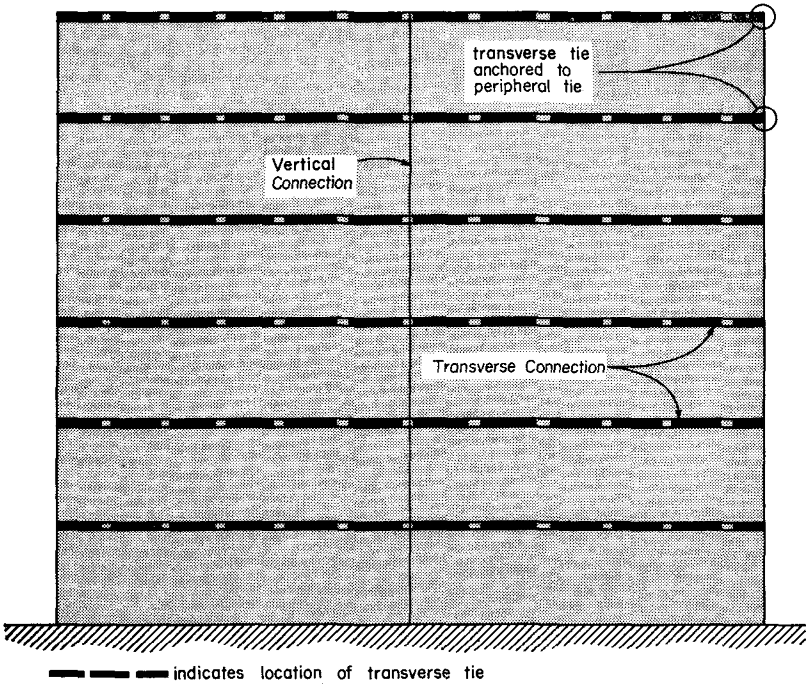
In practice, this type of action is unlikely, since the joint is filled with drypack or grout, and some vertical reinforcement (in the form of vertical ties) is used to interconnect successive wall panels. Both the concrete and the vertical ties act to provide shear resistance across the connection by shear-friction, and consequently, enhance the cantilever strength. This behavioral mode provides a lower bound in predicting cantilever strength.

If adequate shear capacity can be achieved between successive stories, the entire wall panel assembly will act as a monolithic cantilever as shown in Fig. 12b. This behavior can be achieved when there is no relative slip between stories.

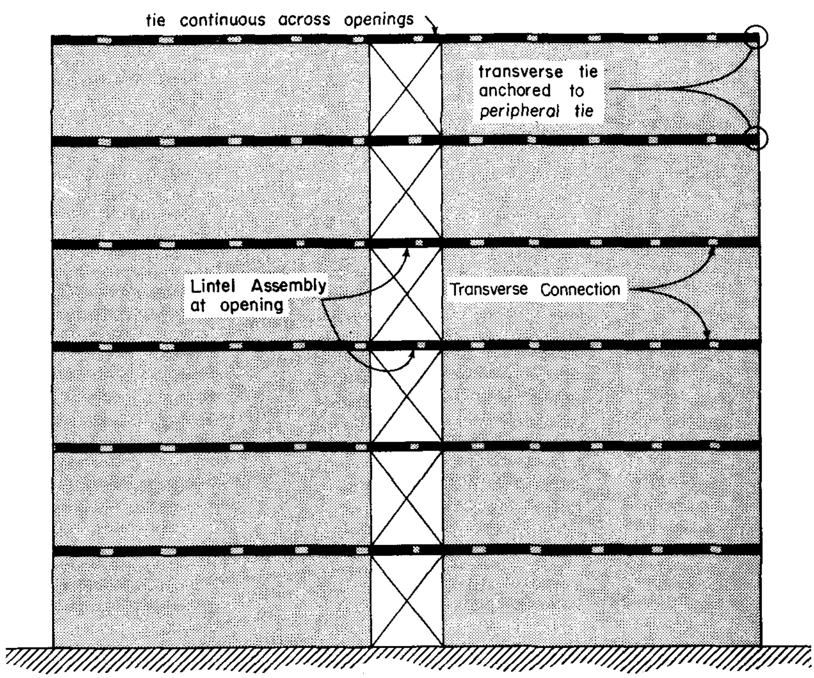
Such an idealized condition is questionable as some slip will most likely occur within the vertical tie connection details. Therefore, this behavioral mode should be considered an upper bound in predicting the cantilever strength.

In practice, the behavior will be somewhere between the two modes described above, and the tensile force requirements for the transverse tie in a multistory cantilever will depend primarily on the details affecting shear characteristics of the horizontal connections.

The transverse tie within the connection can be in the form of mild steel or unstressed prestressing strand. To ensure its effectiveness, the tie should extend the full width of the structure, and be adequately secured to the peripheral tie system (Fig. 13).



Flank wall assembly



Interior wall assembly

Fig. 13. Suggested extent and location of transverse ties.

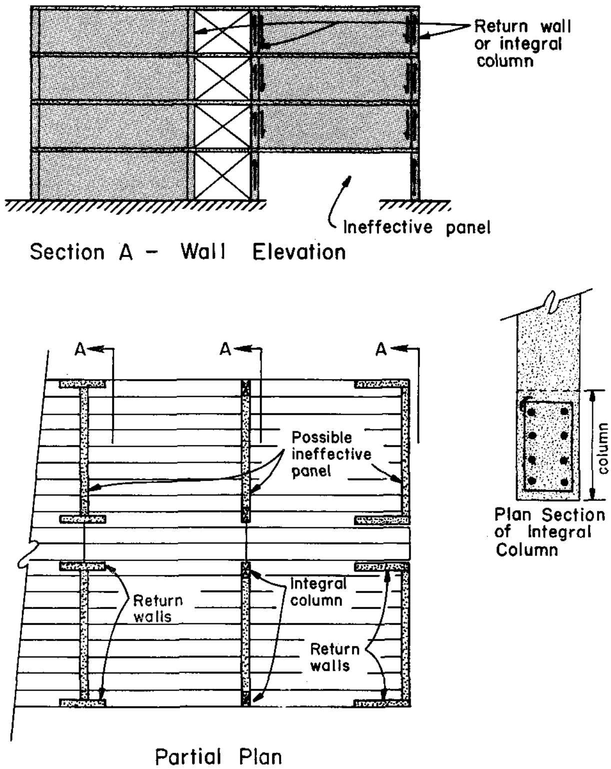


Fig. 14. Beam mode—Single vertical wall panel stack.

Transverse ties and beam action

Although cantilever action through the use of transverse ties is the single most important element in establishing alternate paths in LP buildings, in some instances of local damage the transverse ties can also be very effective in developing "beam" action of the wall panels.

However, to develop beam action, a capacity to transfer the vertical load to each of the supported ends must exist. Such supports are available in the cases shown in Fig. 14.

As in the case of cantilever action, the vertical supports for beam action are provided through proper panel design and layout. The beam's flexural resistance can be developed only if adequate tensile capacity in the form of reinforcing ties exists in the transverse horizontal connection or within the wall panels themselves.

The factors affecting the overall behavior of this alternate load flow are similar to those examined under cantilever action. Vertical load-carrying capacity at each end must be assured and, as before, the shear characteristics of the horizontal connection between successive wall panels substantially influence the behavior.

The necessary ties to resist the tensile forces from the bending moment will be less than those required under similar span cantilever action; however, this structural behavior will usually require more return walls or integral columns to assure vertical supports.

The transverse ties to effect beam action can be either of mild steel reinforcing or unstressed prestressing strand. The ties should be continuous the full width of the building and anchored to the peripheral tie system (Fig. 13).

Longitudinal ties and partial membrane action (large deflections of slabs)

Catenary action of the continuous slab spans due to loss of supports has been investigated in Europe.[28] Although full catenary action is accepted as an appropriate and functional means of reestablishing the load flow in European LP structures, in this study it is not considered a suitable method for American LP construction for the following reasons:

1. The typical slabs for European systems, which span from 10 to 18 ft, are cast for a particular building, and are joined together by interlacing loops protruding from the slabs. LP systems used in the United States have floor spans which range from 20 to 40 plus ft. The U.S. slab systems are generally prestressed hollow-core slabs produced in long beds and cut to shorter lengths according to the required span. In this fabrication technique, reinforcement protrusions at the ends are not feasible, and tensile continuity between adjacent spans must be developed by other means. Since the span of the necessary catenary in U.S. construction (double the slab span) is about twice that of its European counterpart, the problems related to physical development of the necessary longitudinal tensile forces are greatly magnified.

2. To develop the necessary catenary action, the floor systems must undergo large deflections. With 40 to 80-ft catenary spans in U.S. construction, the necessary deflection may approach a total story height. Such large deflections can be accommodated only when the panel below is totally removed; this is, in turn, contrary to the concept of walls becoming ineffective.

The major element of resistance and stability in the damaged wall-type structure is provided by cantilever and

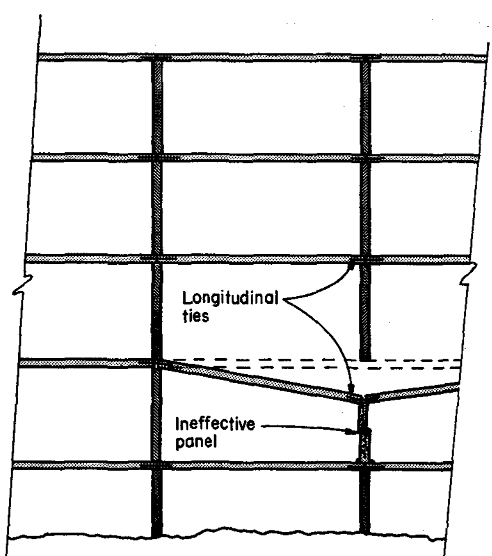
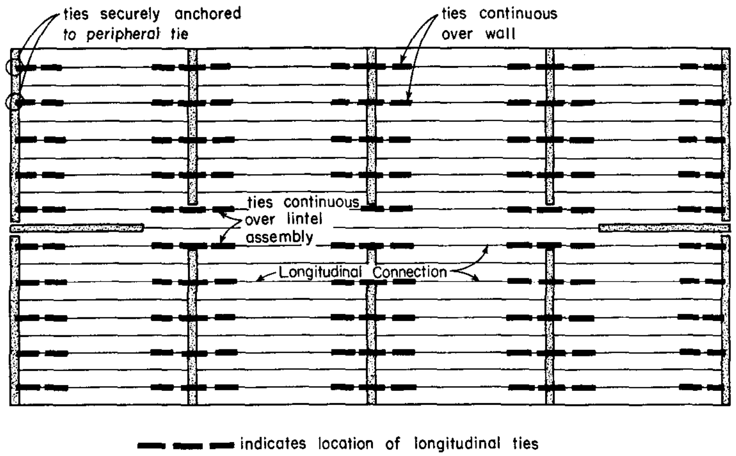


Fig. 15. Partial membrane action of floor elements at ineffective wall panel.



Typical floor/roof panel assembly

Fig. 16. Suggested extent and location of longitudinal ties.

beam action. The function of the slab is to ensure an adequate degree of partial catenary action to effect a stringing of the elements to inhibit progressive collapse from debris loading should an interior wall become ineffective. The wall's ineffectiveness is suggested as a limited displacement (Fig. 15), say up to 24 in.

Thus, to string the slab elements together in this deflected shape, adequate tensile continuity must exist between them, above the ineffective wall. Ductility of the connection must be ensured to permit the large deformations and to resist any dynamic effects that may occur.

Continuity can be developed with reinforcing placed in the longitudinal horizontal keyways between floor panels, or in the topping slab if one is used. These ties must provide tensile continuity between adjacent spans and, to ensure integral behavior, the ties of the end span should be anchored into the peripheral tie system.

Since floor elements are well reinforced, the ties need not be continuous for the full length; the tie force should be transferred by bond or mechanically from an element in one span to the one in the next span (Fig. 16).

Vertical ties and suspension action

The primary functions of vertical ties between wall elements in the damaged structure are to:

(a) Provide vertical suspension of ineffective wall panels and thus avoid debris loading;

(b) Furnish resistance against "kicking out" of the walls sideways, thus fostering "ineffective" behavior of wall elements, rather than total removal under abnormal loading; and

(c) Assure clamping and dowel action in the horizontal connections for shear-friction to resist horizontal shear and thus develop cantilever and beam action.

If an alternate path for the load transfer around an ineffective wall panel has been established either through

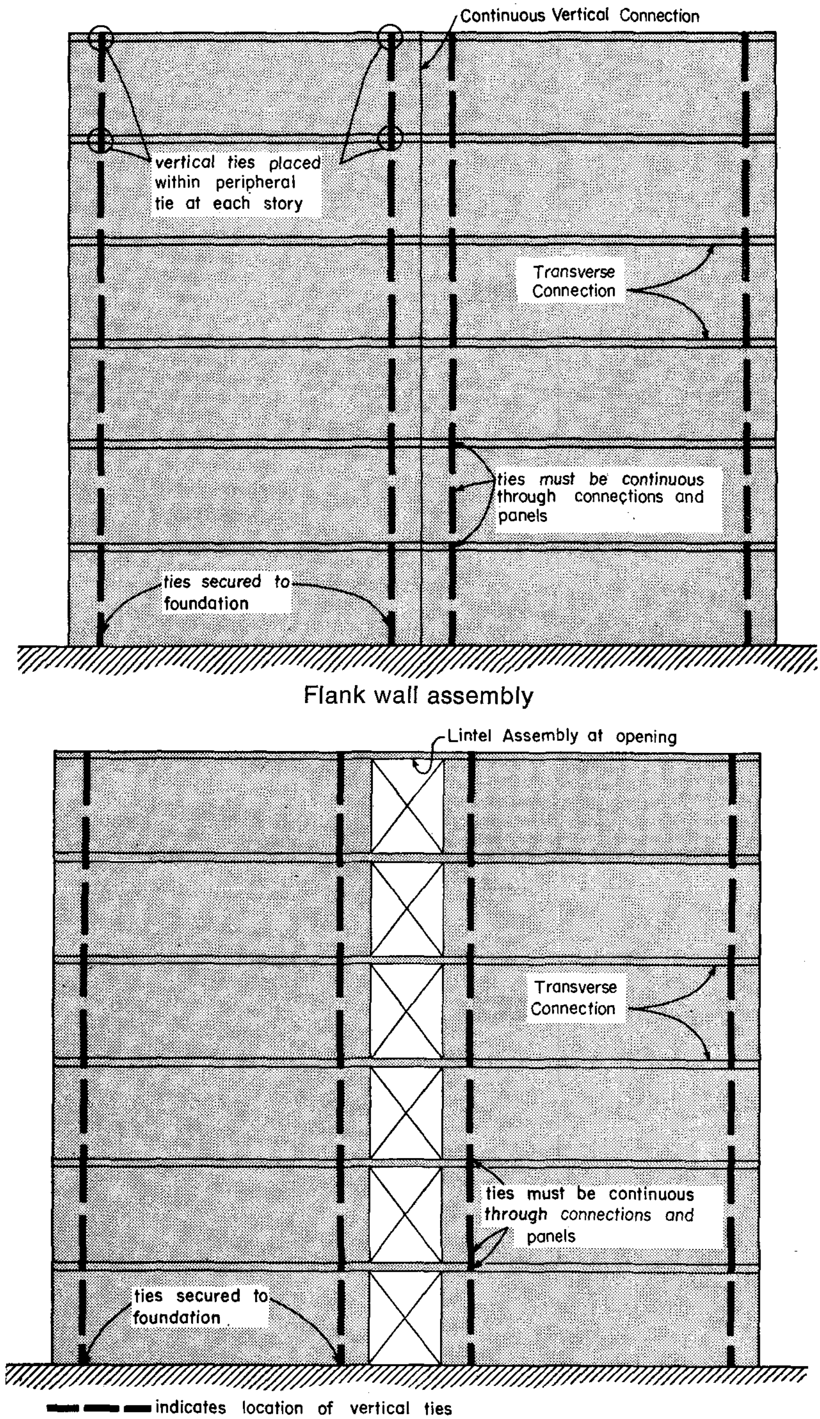


Fig. 17. Suggested extent and location of vertical ties.

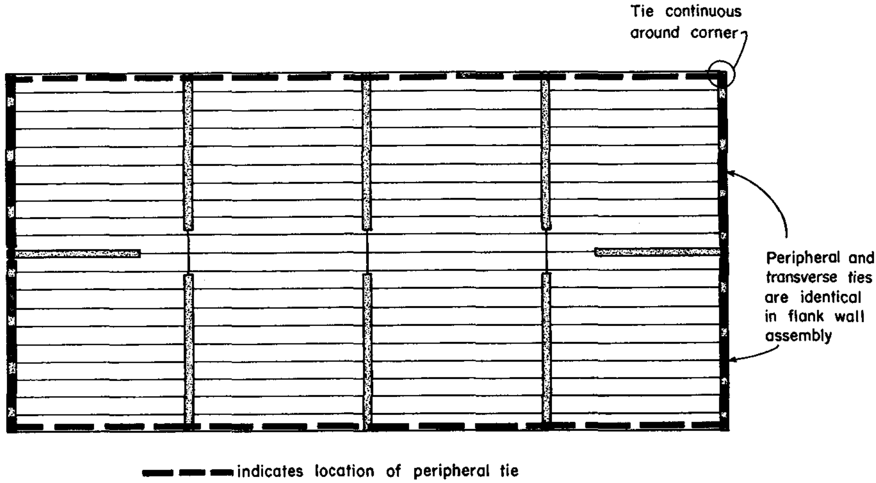


Fig. 18. Suggested peripheral tie in typical floor/roof panel assembly.

cantilever or beam action, the damaged panel should be suspended from the alternate supporting mechanism. To ensure this suspending action, vertical continuity in the form of tensile ties must exist between and through all wall panels.

Tensile continuity from foundation to the roof can be assured by providing vertical ties proportioned to resist (within each story) the dead loads of the wall plus those loads superimposed by the floor panels. In addition, the vertical tie must resist the stresses from shear-friction clamping in the horizontal connection.

Because of the increased vulnerability of exterior wall panels (as compared to interior wall panels) with regard to abnormal loadings, greater vertical continuity is desirable in flank wall assemblies in order to afford them increased protection against removal from the structure. The vertical tie of the flank walls should be placed within the peripheral tie system to improve its integral connection with the slab system.

The vertical ties may take the form of mild steel, prestressing rods, or prestressing strand, stressed or unstressed. They should be continuous for the full height of the panel assembly (Fig. 17). The ties can also function as reinforcement required for service and erection loads.

Peripheral ties and diaphragm action

The main functions of peripheral ties in the damaged building are to:

(a) Establish the necessary diaphragm action to resist the effects of wind, torsion, and unequal load distribution throughout the structure;

(b) Anchor the longitudinal and transverse tie forces at the periphery; and

(c) Help create membrane action in corner slabs when the corner support becomes ineffective.

These functions can be accomplished by employing a ring beam at the periphery of the structure at each floor and roof level.

To assess the force level required in such a ring beam in a structure is a difficult, if not impossible, task. In the damaged state the entire slab or roof may act as a membrane, with its periphery subject to large ring beam forces. It is suggested that the peripheral tie forces be based on the diaphragm action necessary to resist normal lateral loads, assuming the customary load factors and capacity reduction factors.

However, for purposes of continuity and anchorage requirements in the damaged state, the transversal tie forces required in the flank wall assemblies should be continued in the longitudinal faces (if larger than those required for diaphragm action) for the design of the complete periphery (Fig. 18).

These continuous ties, which should be located as close to the actual periphery as possible, may be of mild steel or prestressing strand or mechanical connectors between ends of planks in adjacent spans.

Loads and Safety Factors in the Damaged Structure

To determine the quantity of ties needed for stability of the damaged structure, the necessary loads to be resisted must be established. Consideration should be given only to those loads likely to occur before temporary or permanent measures are taken to repair the damaged area.

Included in this category are the full dead load and part of the design wind and live loads. It is sufficient to assume one-third of the design live load and one-third of the design wind load.

Reduced live load is based on the assumption that a building will be evacuated after an abnormal incident has caused local damage. The reduced wind loading is based on the supposition that the building will have partial stability for a limited period of time.

Since the rate of occurrence of local damage due to abnormal loadings is relatively low, and since the intent is to assure the stability of partially damaged structures, it is assumed that the above loads are resisted at a level close to the ultimate strength of the sections, say 90 per cent of ultimate capacity.

Therefore, for the determination of minimum tie forces, liberal load factors, γ, are appropriate. However, the capacity reduction factors, ϕ, which account for variations in material strengths, workmanship and dimensional errors combining to result in under-capacity, must still be used. Since the ties are utilized to resist axial tension and tension due to bending, a ϕ factor of 0.9 can be used.

Combining the loads (D, L, W) and load factor, γ the required strength equation may be expressed as:

$$U = 1.10\ (D + L/3 + W/3)$$

Incorporating the capacity reduction factor, ϕ, the safety factor of the damaged structure becomes:

$$\text{S.F.} = \frac{\gamma}{\phi} = \frac{1.10}{0.9} = 1.22$$

Specifying Tie Forces—Unstressed Strand

Specifying tie requirements in terms of forces rather than amounts of steel permits flexibility in the choice of tie type, while fostering a better understanding of the issues involved. Specifying forces allows the use of unstressed prestressing strand, which has the advantage of large tensile capacity with a relatively small diameter.

Flexible strand also offers the advantage of easy placement in misaligned connections where a normal mild steel bar cannot be placed due to its rigidity. While unstressed prestressing strand cannot be used for resistance at service load levels, since it would result in unacceptable crack widths, the large strains of the strand are not ob-

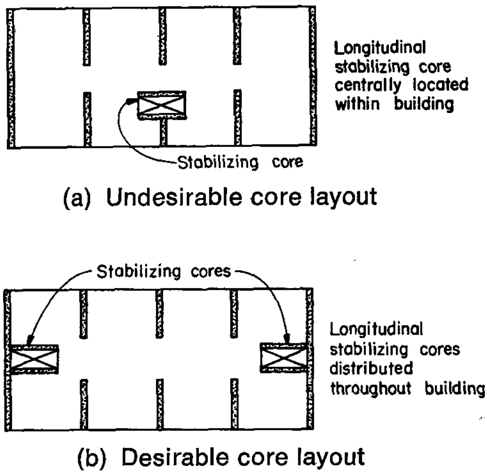


Fig. 19. Possible core layouts in typical cross wall structure.

jectionable under catastrophic conditions.

Planform

Proper structural planform, i.e., the layout of the structural walls, improves the overall rigidity and stability of the structure, thereby increasing its structural integrity. Effective planform is best accomplished by arranging walls in both orthogonal directions of the building.

Since current connection details make frame action between adjacent slab spans and between slabs and walls almost impossible, lateral resistance of LP buildings is provided economically only by cantilever action of the stacks of wall panels in their own planes.

Therefore, where possible, longitudinal spine walls or segments should be used to enhance the stability of the longitudinal direction. They will also improve the stability of individual transversal walls, while decreasing the length of wall likely to be affected by an abnormal load.

For exterior walls, returns or integral columns are desirable to enhance their resistance to abnormal loadings. If vertical stabilizing cores are the only elements for lateral stability, it is desirable to distribute them throughout the building (Fig. 19) so that if one is damaged, total stability of the structure under lateral loads is not forfeited.

Shrinkage, temperature and distribution steel placed in the topping can also be used to enable the slab to span in a perpendicular direction and prevent collapse where a support configuration is favorable for two-way action (Fig. 20).

Building Protection

Protecting the structure from abnormal effects is one of the basic methods to avoid progressive collapse.

When an abnormal load can be controlled, such control is recommended; e.g., provision of venting to relieve explosions, use of shock absorbers for crash barriers against vehicular collision, prohibiting construction in flood plains, etc.

In this way, the forces and their ef-

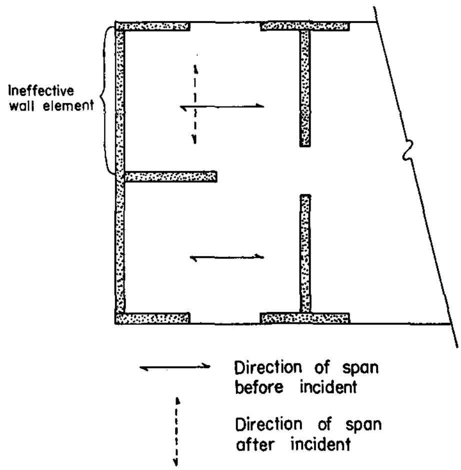


Fig. 20. Change of slab span direction under the effects of abnormal loadings.

fects due to certain abnormal events can be reduced or eliminated, thus lowering the overall risk from abnormal events.

Concluding Remarks

It is technically very difficult and economically prohibitive to design residential-type buildings for absolute safety. On the other hand, there is no justification for constructing buildings which do not afford a certain degree of safety with regard to abnormal loads.

To reduce the risk of progressive collapse of large panel residential buildings, a philosophy is developed to assure bridging of local damage while maintaining stability of the partially damaged structure by tying the components of large panel structures together horizontally and vertically.

Based on this philosophy, explicit requirements for a minimum tie system of transversal, longitudinal, vertical and peripheral ties will be developed. It is intended that these requirements establish a General Structural Integrity for the building to minimize the effects of abnormal loadings.

The provision of General Structural Integrity eliminates the need to design for any particular abnormal load, since the ability to bridge local damage is provided.

In establishing structural integrity, the need for tensile continuity and ductility of the elements, as well as of the overall structure, is recognized. This will be accomplished through a rational arrangement of tensile ties and through connection details which will assure alternate structural behavior of the damaged building.

This combination of system continuity and ductility should enable the structure to either absorb the abnormal loads with minimal damage, or bridge localized damage as a result of the abnormal load. The provision of General Structural Integrity will bring the safety of LP structures closer to that of the traditional cast-in-place reinforced concrete buildings.

The objective of this approach is not to afford absolute safety in regard to every exceptional event in any part of every building; rather, the intention is to limit and substantially reduce the general risk of collapse, as compared to that existing if no such measures were taken.

Based on experimental results performed thus far, it appears that the amount of ties required to achieve the desired level of General Structural Integrity will have a minimal effect on the economics of LP structures.

With the philosophy established, subsequent experimental tests and analytical studies will focus on development of purposeful details to optimize the effectiveness of the ties in the structure.

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References

1. Schultz, D. M., and Fintel, M., "Report 1: Loading Conditions," *Design and Construction of Large Panel Concrete Structures,* Office of Policy Development and Research, Department of Housing and Urban Development, Washington, D.C., April 1975.
2. Somes, N. F., *Abnormal Loading on Buildings and Progressive Collapse,* NTIS Com-73-11113, prepared for the Office of Policy Development and Research, Department of Housing and Urban Development, Washington, D.C., May 1973.
3. Burnett, E. F. P., "Abnormal Loading and Building Safety," *Industrialization in Concrete Building Construction,* SP-48, American Concrete Institute, Detroit, Michigan, 1975.
4. Ferahian, R. H., *Design Against Progressive Collapse,* Technical Paper 332, Ottawa, Canada, National Research Council of Canada, Division of Building Research, July 1971.
5. Ferahian, R. H., "Buildings: Design for Prevention of Progressive Collapse," *Civil Engineering,* American Society of Civil Engineers, New York, February 1972.
6. Creasy, L. R., "Partial Stability," *Design Circular,* Directorate of Civil Engineering Development, Department of the Environment, London, England, August 1971.
7. Creasy, L. R., "Stability of Modern Buildings," *The Structural Engineer,* V. 50, No. 1, London, England, January 1972.
8. Discussion of the "Stability of Modern Buildings," *The Structural Engineer,* V. 50, No. 7, London, England, July 1972.
9. Creasy, L. R., "The Art of the Possible," A Presidential Address given before the Institution of Structural Engineers, London, England, October 4, 1973.
10. *Stability of Modern Buildings,* The Institution of Structural Engineers, London, England, September 1971.
11. Popoff, A., "State-of-the-Art Report No. 3–Stability of Precast Concrete Systems Buildings," *Planning and Design of Tall Buildings,* Vol. III, American Society of Civil Engineers, New York, 1973.
12. Popoff, A., "Design Against Progressive Collapse," PCI JOURNAL, V. 20, No. 2, March-April 1975.
13. Burnett, E. F. P., *The Avoidance of Progressive Collapse: Regulatory Approaches to the Problem,* Prepared for Center for Building Technology, National Bureau of Standards, Washington, D.C., October 1975.
14. PCI Committee on Precast Bearing Wall Buildings, "Considerations for the Design of Precast Bearing Wall Buildings to Withstand Abnormal Loads," PCI JOURNAL, V. 21, No. 2, March-April 1976.
15. Granstrom, S., and Carlsson, M.,

Building Design for Avoidance of Progressive Collapse, Part II: Survey of Some Recent Collapses, Library Translation No. 1670, Building Research Station, Watford, England, April 1972.

16. "Aims of Structural Design," The Institution of Structural Engineers, London, England, August 1969.
17. Granstrom, S., "Byggnaders Stabilitet Efter Katastrofskador. Krafter i Elementfogar-Modellforsok," (Stability of Building After Accidental Damage. Forces in Element Joints-Model Tests), Statens Institut for Byggnadsforskning, *Report* R20: 1971, Sweden Symposium at University College of Cardiff, December 1970.
18. *The Stability of Precast Concrete Structures*, (Proceedings of the discussion periods following the morning and afternoon sessions at a Seminar at the Europa Hotel, London, March 27, 1973), Building Research Establishment, Departmentof the Environment, Garston, Watford, England, December 1973.
19. "Stability of Buildings and Their Design for Prevention of Progressive Collapse Due to Local Damage," *Report of Subcommittee on Large Concrete Panels*, Nordic Concrete Association Committee on Regulations, Sweden, January 29, 1970.
20. Allen, D. E., and Schriever, W. R., "Progressive Collapse, Abnormal Loads, and Building Codes," *Structural Failure: Modes, Causes, Responsibilities*, American Society of Civil Engineers, New York, 1973.
21. Smith, B. S., and Rahman, K. M. K., "A Theoretical Study of the Sequences of Failure in Precast Panel Shear Walls," *Proceedings of the Institution of Civil Engineers*, V. 55, Part 2, London, England, September 1973.
22. Armer, G. S. T., and Kumar, S., *Tests on Assemblies of Large Precast Panels*, Current Paper CP 20/72, Building Research Station, Department of the Environment, Garston, Watford, England, 1972.
23. *The Behavior of Large Panel Structures*, CIRIA Report No. 45, Construction Industry Research and Information Association, London, England, 1973.
24. Granstrom, S. A., "The Scandinavian Approach to Structural Safety," *Buildings and the Hazards of Explosion*, Proceedings of the Symposium at BRE, Garston, October 18, 1972, Building Research Establishment, Department of the Environment, Garston, Watford, England, 1974.
25. *Symposium of Progressive Collapse*, ACI Annual Convention, sponsored by the ad hoc committee on progressive collapse, Technical Activities Committee, American Concrete Institute, Boston, Massachusetts, April 1975.
26. British Standards Institution, "Part I—Design Materials and Workmanship," *The Structural Use of Concrete*, CP-110: Part 1, BSI, London, England, 1972.
27. Swedish Building Standards, "Swedish Regulations for Prevention of Progressive Collapse," SBN 22: 35, Stockholm, Sweden, 1973.
28. Wilford, M. J., and Yu, C. W., *Catenary Action in Damaged Structures—Tests on R. C. Beams and Slabs*, Building Research Establishment, Ref. 391/73, Watford, England.

Discussion of this paper is invited.
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