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BY

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Progressive Collapse

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This paper explains the nature of progressive collapse, with examples, and enlarges considerably on commentary C, supplement No. 4 of the 1975 National Building Code of Canada. Attendant philosophical problems related to code coverage and the design for progressive collapse are considered in discussions of risk, abnormal events that might initiate progressive collapse, and design procedures to reduce the risk of progressive collapse. An example illustrates some of these procedures and indicates how, in a general way, design problems may be approached.

Dans cet article, l'auteur explique la nature de l'effondrement progressif, en présente des exemples et s'appesantit sur le commentaire C du supplément, No. 4 du Code national du Bâtiment (1975). Il examine des questions de fond se rattachant, d'une part, à la conception et aux calculs tenant compte de la possibilité d'effondrement progressif, et d'autre part, aux prescriptions règlementaires correspondantes, en fonction du risque, des évènements extraordinaires qui peuvent déclencher l'effondrement progressif et des mesures à inclure dans un projet et capables de réduire le risque d'un tel effondrement. Un exemple illustre quelques-unes de ces mesures et indique, de manière générale, comment attaquer l'étude des problèmes de conception et de calcul.

[Traduit par la Revue]

Introduction

Progressive collapse is the spread of an initial local failure from element to element resulting in the collapse of a whole building or disproportionately large parts of it. Progressive collapse achieved world prominence when a corner of the Ronant Point apartment block collapsed in London, England, in 1968 (Griffiths *et al.* 1968). Examples of other cases are shown in Figs. 1 to 6 and some statistics concerning incidents in Canada and the United States in Table 1.

For the most part progressive collapses in Canada have involved low buildings and buildings with arched roofs (Morrison *et al.* 1960). Fortunately, no highrise office buildings or apartment towers have collapsed progressively during their service lives although one did collapse during construction (Fig. 6). Attempts to attain more efficient use of materials and to keep erection costs down, however, have resulted in some structures with little inherent toughness or resistance to progressive collapse. The occurrence of progressive collapse in such buildings is probable, if not inevitable, if an abnormal event occurs because no appropriate measures have been taken to prevent it.

Since 1970 there has been an article in the National Building Code of Canada (NBC) dealing with progressive collapse, and a commentary in supplement No. 4 enlarging on the problem, (article 4.1.1.7 and commentary No. 7 in the 1970 NBC, article 4.1.1.8 and commentary C in the 1975 NBC). Nevertheless in many designs there seems to be little indication that structural engineers have been observing article 4.1.1.7 and some have not even been aware of it. Where the NBC has been adopted by bylaw the designer is legally obliged to consider the article and hence should indicate in the design notes that steps have been taken to ensure that the risk of progressive collapse is indeed small enough to be acceptable.

In many structures designed to prevent progressive collapse there will be little increase in the cost of the building itself as integrity will be ensured by the use of well detailed ductile joints and reinforcement which probably would have been used in a carefully designed structure in any case, and by the proper initial layout or plan form of the structure. (These points will be illustrated later.) Design time, on the other hand, will increase initially as the

FIG. 1. Progressive collapse of Skyline Plaza parking garage, Virginia, U.S.A. due to impact from debris falling on edge of slab causing a progressive punching shear failure at the columns (Cohen *et al.* 1974; Leyendecker and Fattal 1973). (Photo courtesy of Prestressed Concrete Institute).

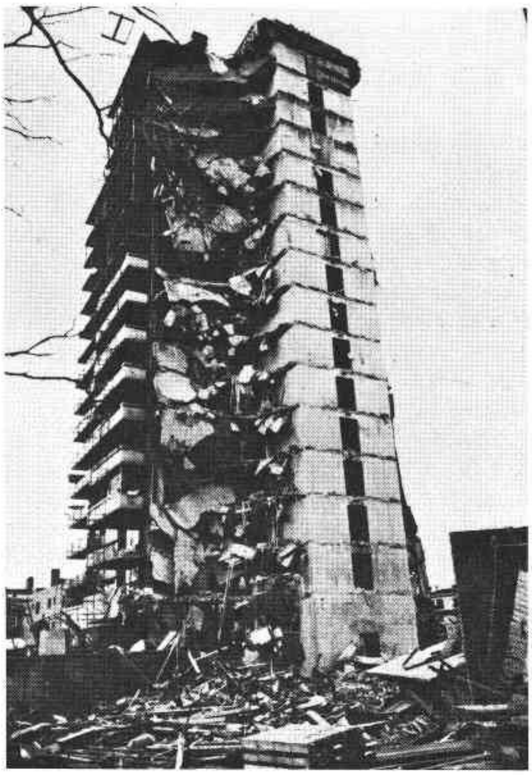

FIG. 2. Collapse of reinforced concrete building under construction, Boston, 25 Jan. 1970 (Wide World Photos) (see also Allen and Schriever 1972).

TABLE 1. Incidents involving progressive collapse gathered from the press (from Allen and Schriever 1972)

	Canada (10 years) 1962–1971	U.S.A./ Eng. News Record only (4 years) 1968–1971
During construction		
Due to impact, explosion	1	2
Formwork, bracing, or erection error	35	10
Design error	0	1
During service life		
Due to explosion	0	1
Due to impact	8	4
Design, manufacture, or construction error	22	3
During demolition, adjacent excavation	6	1
Totals involving progressive collapse	72	22
Total news incidents involving all types of collapse	495	110

engineer works out efficient ways of providing structural integrity and copes with unusual problems such as the stability of partially damaged structures.

An account of the early work on abnormal

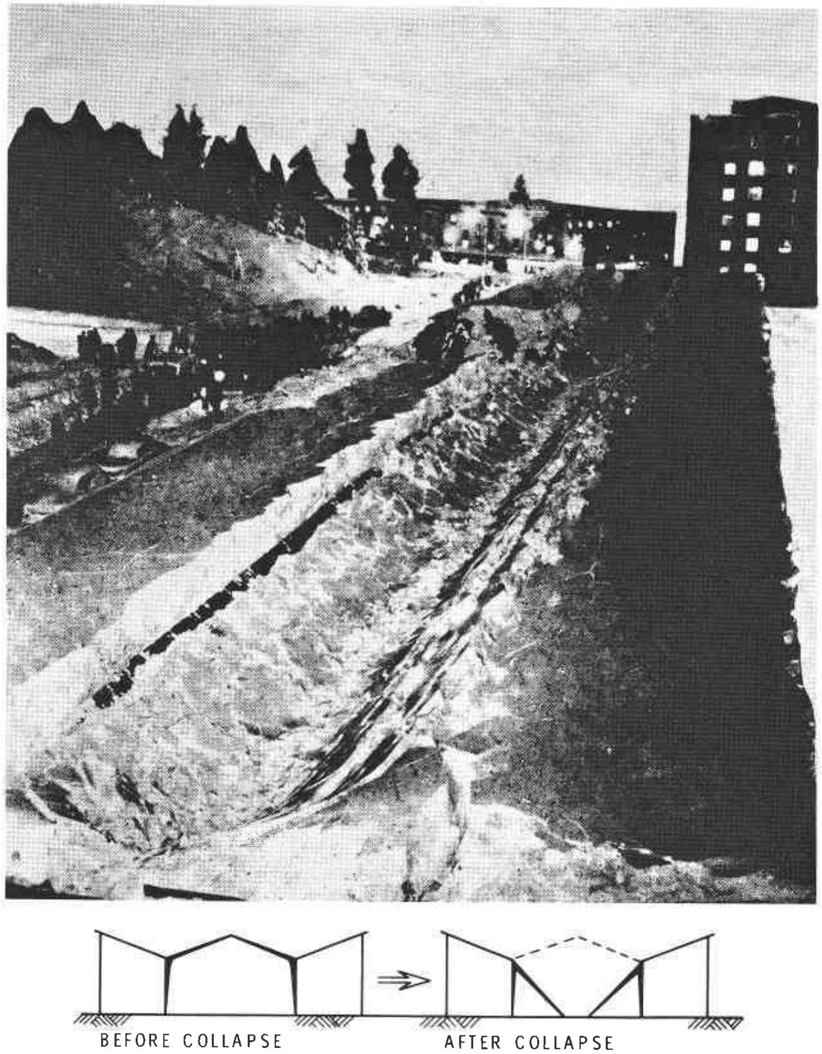

FIG. 3. Progressive collapse of garage roof. Heavy snow loads caused one frame to collapse and with the subsequent overstressing and collapse of adjacent frames the failure progressed laterally until all 20 had collapsed. One person was killed (Granström and Carlsson 1974) (published with the permission of the authors).

loads, and on building codes dealing with progressive collapse, and an appraisal of the provisions of the 1970 National Building Code of Canada has been given by Ferahian (1971).

Progressive Collapse Design Considerations

Risk Consciousness

If, at any moment, the loads applied to a structure are greater than the resistance of the structure to them a failure will occur. In the design process load factors are chosen to control the occurrence of this in order that the risk of collapse will remain below a level acceptable to society.

There is a difference between the risk of collapse and the risk of death, the latter being of even greater concern to society. Because of this it is generally thought that structures such as arenas, schools, shopping malls, and high-rises, which constitute a risk to large numbers of people if they collapse, should be safer than buildings not normally occupied by many

FIG. 4. Progressive bearing failure of roof beams at Camden School for Girls, England (Building Research Establishment 1973).

FIG. 5. Progressive collapse of arena under snow load, Listowel, Ontario, 28 Feb. 1959 (8 killed). (Morrison *et al.* 1960) (NRCC Div. Bldg. Res. photo).

people such as warehouses. Buildings that have been designed to avoid progressive collapse contribute significantly not only to a reduction of the risk to life but also to the investment dollar.

Abnormal Loads and Events

Although buildings are designed for dead, live, earthquake, wind, snow, soil, and hydraulic loads as a matter of course, there are loads and events that can cause failure that

FIG. 6. Collapse of steel frame building under construction, Toronto, 8 September 1958 (Federal News-photos (Canada) Limited) (see also Allen and Schriever 1972).

generally, but not always, occur less frequently than those considered in the regular design process. (In some parts of the world explosions may rival the occurrence of design winds.) Such loads and events are usually such that they cannot be economically considered as regular design conditions. Keeping in mind that it is not possible to design structures for absolute safety, the designer need consider only those abnormal events that have a reasonable chance of occurrence, say, of the same order of magnitude as the estimated probability of failure (10^{-4} to 10^{-6} per year (Allen 1968, 1975)) when designing to prevent progressive collapse. Such events or loads might include: explosions (gas, boiler failures, ignition of flammable liquids, bombs); vehicle impact; falling or swinging objects, usually during construction or demolition of the structure or neighboring buildings; collapse or settlement of adjacent excavations or flooding causing severe local foundation failure; defects arising from extreme construction or design errors; very high winds, tornados, or hurricanes; and sonic boom of exceptional intensity.

In addition phenomena considered as part of normal design considerations such as fire, corrosion, bearing failures, fatigue, and overloading, should not also cause progressive collapse if they occur. The implication of this is that structures should be inherently capable of limiting the spread of local failures regardless of the cause. Buildings should not be 'houses of cards' or subject to the 'domino effect.' This is not to say that they should prevent the initial damage due to the abnormal event (the loss of a wall, beam or column, for example) by having enormous excess strength, which would be prohibitively expensive, but rather that they should be able to absorb the damage without suffering progressive collapse.

Data on abnormal events are being gathered in many countries but it will be many years before reliable design decisions can be made based on these statistics. (Burnett *et al.* 1973; Ligtenberg 1969; Fribush *et al.* 1973; Somes 1973; Granström and Carlsson 1974.)

Designing to Avoid Progressive Collapse

Progressive collapse provisions apply to most structures but some types of construction appear to require more attention than others. Buildings with load-bearing walls, especially of masonry and panel construction, precast beam and column buildings, and precast beam and floor slab structures should be designed with great care. There are certain occasions, however, when progressive collapse provisions do not apply because of the nature of the structure under consideration. For example, if the structure is supported on one column only, any event that removes the column (100% of the support) would cause collapse of the entire structure. The risk of such an occurrence, a general rather than progressive collapse, could be reduced by increasing the load factor or by otherwise protecting the structure, but, of course, no 'alternate path' for carrying loads to the ground would be available. The same would apply to two-, three-, and perhaps even four-column structures where the loss of one column would remove such a large percentage of the support that general collapse would occur.

Design Considerations

Four general considerations in designing to prevent progressive collapse, (1) reduction of risk, (2) ductility, (3) design to resist abnormal loads (option A), and (4) design for alternative paths (option B), are discussed at some length.

The last two, (3) and (4), are alternatives that can in some cases be usefully combined.

(1) Reduction of Risk

In some cases the risk due to a particular abnormal event such as a gas explosion or vehicle impact on street-level walls and columns can be reduced by preventing the use of gas or storage of explosive materials on the one hand, and by providing fenders on the other. Even when this is done, however, the structure should still be resistant to progressive collapse.

Because a great many general failures and about 50% of progressive collapses occur during construction (Table 1), provision should be made to ensure integrity of the structure as it is being built, *i.e.* by the use of temporary guying and the erection of temporary and permanent bracing as soon as possible in the erection sequence. There are many examples of trusses or frames that collapsed before wind bracing was added and in many cases some of the bracing could have been connected after the first one or two frames or trusses were erected.

(2) Ductility

It is widely accepted that connections between structural members should be ductile and tough, capable of large deformations and energy absorption at ultimate loads. This requires attention to detailing. The designer must, for example, ensure that the joints are ductile without being 'slack' and that it does not become possible for beams, slabs, or wall panels to drop off their bearings (Fig. 7).

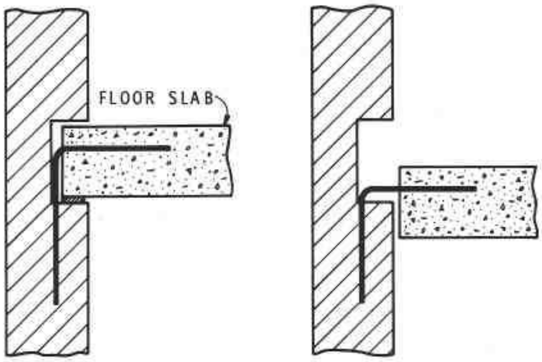

Fig. 7. 'Slack' joint resulting in beam falling off bearing seat.

On the other hand if a joint is over reinforced, failure may occur in a different and unexpected location where it is more difficult to deal with. Another danger due to over reinforcing is that a shear failure may occur, possibly with no warning, rather than a flexural failure which would probably give warning of impending collapse.

Some types of construction use joints that rely on friction due to gravity forces only. Such connections behave unpredictably and in a brittle manner at ultimate loads, and hence are usually inadequate. They have been used in some buildings with hollow-core precast slabs that bear directly on masonry or block walls and depend on friction rather than reinforcing steel to fasten them to the walls. It is revealing to consider how much more continuity and ductility would be provided in the same buildings if they were composed of monolithically cast walls and floors!

Figures 8, 9, and 10, which are diagrammatic only, show joints that can typically provide the continuity and ductility necessary, though not always sufficient in themselves, to prevent progressive collapse. Careful consideration of these details is required, however, to ensure that joints have enough grout of adequate quality to provide proper bonding of the reinforcing steel and to ensure that the elements, especially blocks or bricks to which

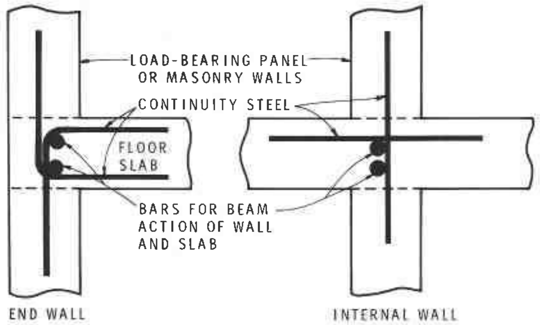

FIG. 8. Schematic detail of ductile joints showing continuity and longitudinal tying steel, diagrammatic only (see commentary C, supplement No. 4, 1975 NBC).

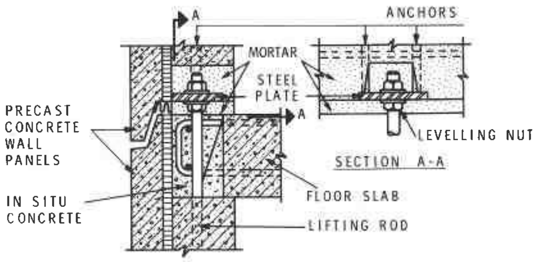

FIG. 9. Wall-to-floor joint with continuity provided through the lifting rods (Ferahian 1972). This detail can be used for internal bearing wall–floor joints (Graff 1971).

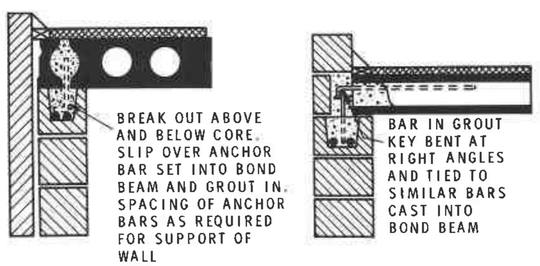

FIG. 10. Standard edge details (at roof level) (Con-Force 1974) (published with the permission of Con-Force Products Ltd.).

the reinforcing is grouted, are themselves adequately anchored to the structure. Though Fig. 8 is shown with continuity tying steel that may attract large moments, the designer may wish to position the steel within each member to reduce these moments.

(3) Design to Resist the Abnormal Loads (Option A)

To provide adequate resistance against progressive collapse, load-bearing elements must be connected by ductile joints, and continuity of tie forces provided as a first condition. The structure may then be designed to resist abnormal loads, option A (*i.e.* the load-bearing elements involved must have adequate strength or stiffness to remain standing under this ultimate load) or to accept the local damage caused by the abnormal event and carry the loads by an alternate path around the damaged area and to the ground, option B. The first is generally the less desirable, less economical approach; but in some designs it cannot be easily avoided. In other cases it can be combined with the alternate path method, such as in the design of strong points, locally strengthened areas, capable of carrying the abnormal loads, to give a successful design.

In option A, then, if the removal of a structural element by a foreseeable abnormal event would initiate progressive collapse, that element should be designed to remain functional when the abnormal event occurs. A major difficulty with this option is that all reasonably foreseeable abnormal loads have to be anticipated and assessed.

In Great Britain, where the Fifth Amendment regulations governing the design of buildings for progressive collapse have been in force since April 1, 1970, elements in residential quarters potentially subject to a gas explosion, and not designed for alternate paths, must be able to withstand a pressure of 5 p.s.i. (34 kPa) + dead load + $\frac{1}{3}$ live load + $\frac{1}{3}$ wind load with a 'safety' factor of 1.05 (Ministry of Housing and Local Government 1970). The 5 p.s.i. (34 kPa) figure was apparently obtained primarily from estimates of the average pressure that caused the destruction of the bearing walls in the Ronan Point collapse (Griffiths *et al.* 1968).

Actually gas pressures in closed containers may reach 100 p.s.i. (689 kPa) but in average residences, including highrise apartments, a large proportion of the gas escapes unburnt through doorways, ventilation shafts, and windows and walls ruptured by the initial blast (Rasbash and Stretch 1969; Alexander and Hambly 1970; Dragosavic 1973). This process, called venting, normally limits the pressure to about 5 p.s.i. (720 p.s.f.) (34 kPa) or less. Venting is not as effective in explosive bomb blasts which may be fundamentally different.

While a gas explosion involves the rapid burning of all the gas that does not escape unburnt through the vents, a bomb blast involves the instantaneous eruption of all the compact explosive material in the bomb. None escapes by venting. Indeed, unless the proportion of vented area in a room is very large, venting will not cause any significant reduction of blast pressure due to a bomb.

From the first appearance of the Fifth Amendment Regulations the Institution of Civil Engineers in Britain took issue with the 5 p.s.i. (34 kPa) values as a general figure in buildings that did not use town gas (gas manufactured from coal) and suggested 2.5 p.s.i. (17 kPa) for buildings using natural gas or no gas at all (Institution of Structural Engineers 1968). However the 2.5 p.s.i. (17 kPa) figure was not adopted by the various Code Committees in Britain. The value of 5 p.s.i. (34 kPa) was the basis for the 3000 lb/ft (44 kN/m) tie reinforcing required between walls and floors (*i.e.* 720 p.s.f. $\times \frac{1}{2}$ storey height $\simeq$ 3000 lb/ft) in the British Standard Code of Practice for large panel structures (British Standards Institution 1970). Many national codes now use a figure ranging from about 1400 to 4000 lb/ft (approximately 20 to 60 kN/m) depending on the height of the building, floor span, and floor loading (British Standards Institution 1972; Federal Housing Administration 1973; Nordic Concrete Association 1970; Lewicki 1971). However, one set of recommendations has minimum internal tie requirements as low as 350 lb/ft (5.1 kN/m) (Comité Européen du Béton 1967). In Canada no categorical minimum member or connection resistance can yet be stated and it is a matter requiring research and some dialogue among design committees and the design profession.

(4) Design for Alternative Paths (Option B)

In this option the structure is designed to bridge the region damaged by an abnormal event. There is a problem in defining the size of ruptured zone to be assumed in design and in checking the subsequent stability of the structure, so arbitrary limitations have been specified in a number of building codes. In Britain, for buildings over four storeys in height (including basements), one load-bearing element at a time is considered to have failed: *e.g.*, a beam, a column, a length of wall not greater than 2.25 times the storey height, or a load-bearing wall panel. The failure due to the removal of the component must be limited to the storey of the incident and the one above and below and in addition it must not exceed a plan area of 750 ft^2 (70 m^2) or 15% of the floor area in question (Fifth Amendment Regulations). In Sweden the building must remain stable after damage to a cubic volume with sides equal to the largest of: one storey height including two floors, $\frac{1}{10}$ the height of the building, or $\frac{1}{20}$ the length of the building where the length is defined as the length of independently stabilized building parts (Lewicki and Olesen 1974). Denmark requires that the walls and floors around a room be able to fail completely without causing the collapse of the other floors in buildings over six storeys in height (Lewicki and Olesen 1974). In Czechoslovakia the size of local failure is taken as one panel of a load-bearing wall including a gable wall corresponding to the dimensions of a room (Vyzkumny Ustav Pozemnich Staveb 1970). The U.S. Department of Housing and Urban Development is considering requirements very much like the British Fifth Amendment Regulations (Federal Housing Administration 1973).

There are many factors that must be considered when designing a structure to ensure that it has the integrity required to carry loads around a locally damaged zone. If these factors are taken into consideration at the concept stage it should be possible to arrive quickly at designs with more integrity than would ordinarily be the case. Some of these factors are presented here (Haseltine and Thomas 1969; Redland Bricks Ltd. 1971).

(a) Good floor plan—The choice of the proper plan form of the building from the basement to the roof is probably the most important measure in achieving structural integrity. In bearing-wall structures, spine walls are recommended to reduce the span of cross-walls and to enhance the stability of the cross-wall and of the building as a whole (Fig. 11). The mutual support afforded by the junction of the spine and cross-walls will reduce the length of wall likely to be damaged by a gas explosion or by vehicle impact.

(b) Returns on walls—A 'return' on a wall

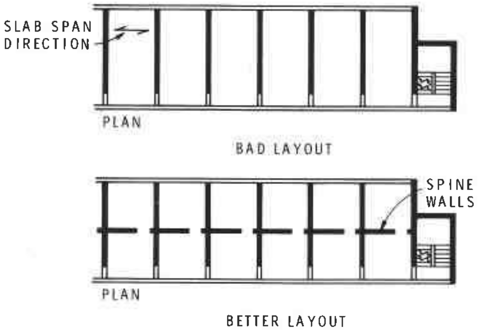

FIG. 11. Use of spine walls (only load-bearing walls (heavy lines) and exterior cladding are shown).

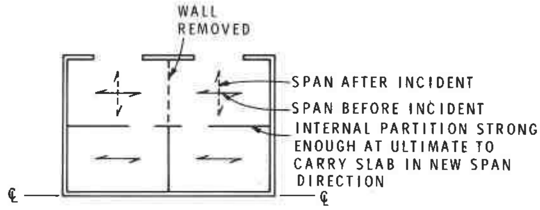

FIG. 12. Load-bearing internal partitions and change of slab span direction.

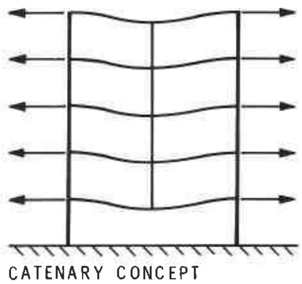

FIG. 13. Catenary action of floor slabs.

is a short length of wall usually at right angles to the first. The increase in transverse flexural and shear stiffness as a result of adding returns to internal or external walls enhances the stability. Clearly a long return is better than a short one, at least until its length is equal to about half the height of the wall.

(c) Strong points—In some designs certain elements may be strengthened to carry the abnormal load in order to complete an 'alternate path' and sometimes a wall return is used to advantage as a strong point when it is adequately reinforced.

(d) Change in span direction of floor slab—If a load-bearing wall supporting the edge of a slab collapses, the slab will probably fail unless there is enough reinforcement, perhaps temperature or distribution steel, to allow it to span in another direction and to carry the loads to the remaining supports. Very little reserve strength is required. Preventing the collapse of the slab is important because it reduces the danger of falling debris gathering enough momentum to carry away the floor below. The designer has only to consider the effect of debris such as the dead weight of a floor slab dropping through one storey height to appreciate the magnitude of the impact loads and the danger of progressive collapse.

In an *in situ* concrete slab little increase in reinforcement will be needed to achieve the required ultimate behavior, but with precast slabs, especially of the strip slab variety, a poured topping containing a mat of reinforcing bars will be necessary in many cases if significant two-way action is required. Unfortunately, this reinforcing mat is on the top side of the slab and therefore is best suited for cantilevering action, while often resistance to sagging movements is required. Indeed, it seems that precast strip slabs cannot ordinarily be counted on to change span direction.

(e) Internal partitions—To be able to use the capacity of a floor slab to change span direction, an internal wall must have at least enough ultimate strength, in some cases, to support the edge of the slab after the abnormal event occurs (Fig. 12).

(f) Catenary action of floor slab—Where the slab cannot change span direction because of the plan form of the building or the nature of the slab itself, the span will increase, often double, if an intermediate supporting wall is removed. If there is adequate reinforcing throughout the length of the slab and enough continuity and end restraint, the slab may be capable of carrying the loads by catenary action. Large deflections will likely result and will occur on the floors above if the load-bearing walls there coincide with the one that is removed. As the building will probably be extensively damaged, catenary action as shown in Fig. 13 may not be a favored method of providing integrity though it is important to remember that it is not necessary that the structure remains serviceable (with deflections or sway less than the design service limits of $L/360$ or whatever) after the abnormal event

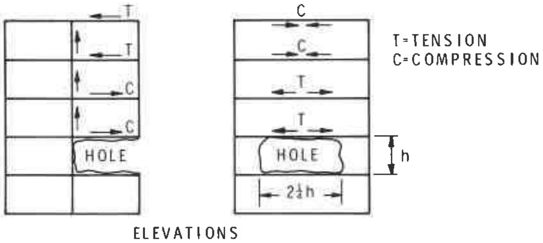

FIG. 14. Beam action on walls showing flange forces in floors.

occurs; only that it remains standing and allows relatively easy evacuation and that the locally ruptured zone remains confined.

A second case of catenary action, that of a single slab which has its bending stiffness sufficiently reduced by an abnormal event that it hangs in position, is very important. By this action, the fall of debris is prevented and thereby an important cause of progressive collapse eliminated.

(g) Beam action of walls—If a bearing wall is removed, beam action of the walls or panels and floor slabs above may allow the opening to be spanned without further distress to the structure (Fig. 14). To achieve this beam or cantilever action, there must be sufficient tying steel at the top and bottom of the wall, preferably in the floor slabs as shown in Figs. 8 and 9 or in bond beams, Fig. 10, in order that the wall may act as the web and the floor slabs above and below as flanges. Tying steel around the periphery of the building and at joints of internal load-bearing walls and slabs also contributes to a diaphragm action of the floor and horizontal beam action under lateral loads if some floor slabs are removed.

Design Examples

Designing for Prevention of Progressive Collapse

The following 'simple' calculations are given to indicate how to proceed with the design of a building in order that it will have adequate integrity to prevent progressive collapse. Some recommendations (Federal Housing Administration 1973; Comité Européen du Béton 1967; Nordic Concrete Association 1970; Lewicki 1971) give requirements for the location of reinforcing bars, and minimum reinforcing bars and tie forces to be carried, but the 1975 National Building Code of Canada does not. Hence in lieu of complex computer analyses which designers could undertake to determine optimum reinforcement required to prevent progressive collapse, very approximate procedures which are ordinary tools of the designer and which are adequate for the same purpose will be used here. When more sophisticated three-dimensional finite element and frame analyses have been completed by the author the results will be made available for comparison with those obtained by 'hand' calculations. For the results of other analyses reference can be made to a number of interesting references (Alexander and Hambly 1970; Haseltine and Thomas 1969; Morton *et al.* 1970; Burnett and Rajendra 1972).

To evaluate the resistance of a structure to abnormal loads, limit design procedures such as ultimate strength design, plastic design, and yield line methods should be used where possible. In Canada, building codes and standards with ultimate strength design provisions for structural steel and reinforced concrete are available. For these the ultimate criteria, with load factors slightly exceeding 1.0, applied to the abnormal load, should be used for designing to prevent progressive collapse. Unlike the steel and concrete standards, those for engineered masonry in this country use only working stress design. In the absence of any established practice, available data suggests that it would be reasonable to use a load factor just exceeding 1.0 and to multiply the allowable design working stresses by 1.75 or 2.0 for the purpose of checking against progressive collapse (1.75 for the lateral strength of walls) (Structural Clay Products Institute 1969). The same factor could also be applied to allowable loads in stability calculations.

Designers checking the integrity of structures made from other construction materials must use the best data available on their ultimate strength and behavior.

Example Calculations: Loss of a Cantilever Bearing Wall (Figs. 8 and 15).

The intent of this calculation is to compute the tying steel necessary in the floor/wall joint to allow the wall to act as a cantilever (Haseltine and Thomas 1969). Each storey is considered separately as it is not known at which level the wall will be 'removed' and also be-

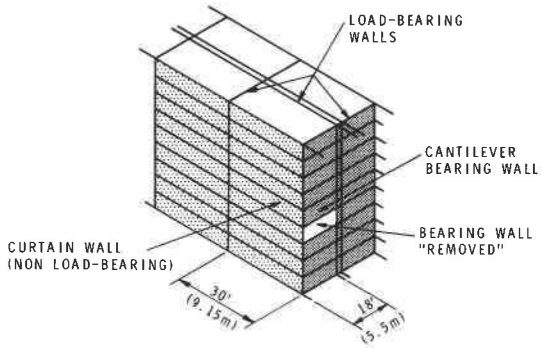

FIG. 15. Cantilever bearing wall 'removed'.

cause, for this example, extensive vertical tying steel will not initially be included in the design.

Design Parameters

Dead load (DL) = 81 p.s.f. (3.88 kPa)
Live load (LL) = 40 p.s.f. (1.92 kPa)
Steel yield stress F_y = 60 k.s.i. (413.7 MPa)
Storey height = 8 ft 10 in. (2.69 m)
Assume 8 in. (200 mm) precast prestressed slab 4 ft 0 in. (1.22 m) wide with 2 in. (50 mm) topping including a mesh
Slab spans 30 ft (9.15 m)
Wall length 18 ft (5.5 m) and wall thickness 9 in. (230 mm) masonry at 95 p.s.f. (4.55 kPa)
Floor load = DL + $\frac{1}{3}$ LL = 81 + $\frac{1}{3}$ × 40 ≃ 94 p.s.f. (4.50 kPa)
Load on cantilever wall = 30 × 94 + 8 × 95 = 3580 p.l.f. (52.3 kN/m)
Maximum moment on cantilever wall if curtain wall weighs 45 p.s.f. (2.15 kPa) is $18(30 \times 8 \times 45) + 3580 \times (18^2/2) = 774\,000$ ft lb (1.05 MN m)
Lever arm for T and C is 8 ft 10 in. (2.69 m)

$$\text{Hence } T = \frac{\text{load factor} \times \text{cantilever moment}}{\text{storey height}}$$

$$= \frac{1.05 \times 774\,000}{8.83}$$

$$= 92\,000 \text{ lb } (409 \text{ kN})$$

Hence tension steel

$$A_s = \frac{92\,000}{60} = 1.53 \text{ in.}^2 \ (990 \text{ mm}^2)$$

i.e. two No. 8 bars.

As this might be considered a rather large amount of steel, consideration could be given to a system of continuous vertical reinforcing bars in the walls capable of carrying at least the floor and wall loads at each storey level to enable the walls above to act together. If the wall is then considered as a cracked, transformed section, tying steel could be reduced somewhat; the reduction being greater in the lower floors and zero at the roof. However there is a minimum tying force required at each floor level to maintain diaphragm action of the slab to ensure that positive and negative pressures on the walls can be carried to the core of the building or distributed to the elements carrying lateral loads. Because of possible cracking, it might be prudent to limit to three or four the number of storeys considered acting together as a cantilever. It would also be wise to consider that the floor above the room in which an explosion occurs might be incapable of taking compressive forces (though still in position) for the purpose of computing stresses and steel areas in the cantilever wall. In any case the minimum longitudinal tying steel should probably be two No. 4 bars placed in the floor–wall joint or in a bond beam adjacent to the joint (Fig. 10).

General Tying Forces Between Walls and Floors

In order that only one storey height of wall will be affected when the abnormal event is confined to one storey, the tying force from wall to floor (in walls parallel to and at right angles to the span of the floor slab) must be at least as great as that lateral force on the wall which would cause the wall to collapse. However, until Canadian code committees make a recommendation it would seem reasonable that the design tie force need not be, in general, greater than that generated by a 5 p.s.i (34 kPa) explosion. Regardless of the design pressure finally chosen, the tie force must also be at least 3% of the vertical force on the wall to act as an effective lateral support to the wall at the floor level.

The tying steel required if the 5 p.s.i. (34 kPa) figure is used is about 0.1 in.^{2}/ft of wall length for a storey height of 10 ft (*i.e.* 210 mm^2/m length for a storey height of 3 m) and does not appear to be excessive.

Tying Forces in Direction of Floor Span: Catenary Forces

There are two cases of catenary action to be considered. The first is a floor slab that has lost its flexural stiffness due to damage but which is designed to hang in position in order to curtail the fall of debris.

Uniformly distributed loading W = dead load + $\frac{1}{3}$ live load, at least.

Midspan deflection Δ, span L

Δ set by designer $\geqslant L/10$

The required tensile force T per unit width at the ends of the slab, to be carried by reinforcing into the support is

$$T \simeq \frac{WL}{4\Delta}\sqrt{4\Delta^2 + L^2/4}$$

$$\simeq \frac{WL^2}{7.5\Delta} \simeq \frac{1}{\Delta}\left(\frac{WL^2}{8}\right)$$

It must be ensured that bars are properly lapped and have sufficient bond length or are properly fastened to peripheral ties or longitudinal tying steel in the joints to transfer this force.

The second case arises when an internal wall is removed and the span suddenly increases, often to $2L$. In this case T increases enormously unless the design allows for a larger sag, Δ. The continuity and proper bonding of reinforcing is very important in the two cases, but both will be ineffective if the undamaged structure does not provide enough horizontal restraint in the plane of the damaged floor slab.

From these examples it can be seen that though the calculations themselves are simple the attendant assumptions must be carefully weighed, a process that will initially take some time. Nevertheless, after completing the first few designs, it is anticipated that the engineer will find that designing for prevention of progressive collapse will involve relatively small extra design or structural costs.

Summary

It has been shown that by properly laying out structures at the concept stage, by using ductile joints, and supplying necessary peripheral, longitudinal, and transverse tying steel with quantities calculated by simple well-known methods, designers can achieve structures with adequate integrity to prevent the spread of local failures. This is surely the right direction for design to be progressing: towards the elimination of the 'house of cards' effect by direct, rational, and efficient designs based on sound structural engineering principles.

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