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CHAPTER I

INTRODUCTION

1.1 INTRODUCTION

The objective of this technical report is to identify all loading conditions assuring strength and serviceability in pre-cast concrete structures which must be measured. The relevance of each traditional loading condition, as contained in American, Canadian and European codes and standards and as used in the analysis and design of conventional structures is examined.

The term "pre-cast panel" concrete structure is used in order to describe a structural system composed of vertical pre-cast panels with pre-cast floors and roofs. These prefabricated component buildings can be considered as the industrialized form of conventional pre-cast-in-place shear wall construction. Pre-cast panel buildings are differentiated by the general arrangement of load-bearing pre-cast concrete walls.

1.1.1 Cross Pre-Cast Concrete Wall System

In this system, the load-bearing cross pre-cast concrete walls are perpendicular to the longitudinal axis of the building.

1.1.2 Spine Pre-Cast Concrete Wall System

In this system, the load-bearing pre-cast concrete walls are parallel to the longitudinal axis of the structure.

1.1.3 Mixed Systems

A combination of cross pre-cast concrete wall and spine pre-cast concrete wall systems.

Each of the systems, the transfer of their loads, takes place by the pre-cast concrete walls directly to the substructure without an intermediate frame. This type of construction restricts open plans at any level and is thus most typically suited to multi-storey housing where pre-cast concrete walls of substance are usually provided for the prevention of fire and noise. The following criteria have been developed to ensure the structural safety and serviceability of economical pre-cast concrete panels in residential buildings.

(1) The pre-cast concrete elements should not have low initial and life-time cost; the poor quality must not be the means of achieving low cost.

(2) The panels should have adequately structural capacity and durability.

(3) The elements should have efficient integration of structural, mechanical and electrical functions.

(4) The exterior pre-cast concrete elements and roof slabs should have good thermal insulation characteristics.

(5) The panels should have a good interior finish capable of taking paint or paper.

(6) The pre-cast concrete floor elements should have good finishes on both top and bottom surfaces so as to make on-site labour costs low when applying paint, paper, or floor surfacing.

(7) The elements should have satisfactory fire resistance and good acoustical properties.

(8) The exterior precast concrete panels and exterior joints should be weatherproof and durable.

(9) The elements should be light to facilitate handling, shipping and erection.

(10) The panels should be so designed as to provide for easy and efficient jointing, both in plant and on-site.

(11) The constituent materials of pre-cast concrete elements of composite construction must be compatible.

The above criteria must be achievable with simple manufacturing and erection procedures and the finished building must provide an architecturally pleasing and socially acceptable environment. While satisfying the above performance requirements the designers and developers will ultimately contribute to the expansion of knowledge required for the design and construction of pre-cast concrete structures, comparable to that existing in conventional cast-in-place concrete.

CHAPTER II

CONSTRUCTION PRACTICE - LOADING CONDITIONS

The following loading conditions should be considered in the design and construction of precast concrete structures:

(a) Dead load (D.L.)

(b) Live load (L.L.)

(c) Temperature effects and other volume changes (T)

(d) Soil and hydrostatic pressures (H and F)

(e) Snow load (S)

(f) Earthquake load (E)

(g) Wind load (W)

2.1 DEAD LOAD

Dead load is all permanent weight, including the walls, floors, roofs, partitions, stairways and fixed service equipment (such as electrical feeders, plumbing, stack, ventilating and air-conditioning systems.) In some cases, other loads may be included in dead weight, such as the frame, floor finishes and underfill, ceilings and their supports, permanent and moveable partitions, exterior walls and interior walls, forces due to prestressing and superimposed weight of earth-fill over an underground garage.

The dead load estimate in a thoroughly executed conventional design may be in error by as much as 1 to 20%. This degree of error may include or be caused by:

(a) Differences between the assumed and achieved on-site dimensions resulting in change of volume.

(b) Deviations in unit weights assumed in the design and those of the actual materials employed.

(c) Designers' alterations such as changes in floors, walls or ceiling surfaces.

2.2 LIVE LOAD

Live load is defined as the weight superimposed on the structure by the use and occupancy of the building. It is that portion of gravity load which is not climatologically dependent and not considered as a dead load. A transient type loading, live load typically includes loads due to movable partitions and vertical loads due to cranes; snow, ice,rain,wind and earthquake; earth and hydrostatic pressure; horizontal components of static or inertia forces; furniture, fixtures, vehicles and ordinary impact conditions. Provision shall be made in the structural design for uses and loads which involve unusual vibration and impact forces.

Live loads are the result of human actions which, except for those extreme cases subject to physical circumstances, have no predictable limits. Current codified live load requirements in the form of equivalent uniform loads and minimum concentrated loads represent those loads established by consensus through experience, judgement, practice, and more recently, statistical studies of loading surveys. These live-load requirements are defined in the National Building Code of Canada, and assure satisfactory results in that virtually no failures have occurred which could be directly attributed to inadequately specified live loads.

In the design of residential constructions, the live loads shall be the greatest by the intended use or occupancy, but no less than the minimum uniformly distributed unit loads prescribed. According to American National Standards Institute (ANSI) for live load of 100 psf or less on any member supporting 150 square feet or more may be reduced at the rate of 0.08% per sq. ft. of area supported by the member. The reduction shall exceed neither R as determined by the formula

$$R = 23(1 + D/L)$$

nor 60% where

R = reduction in percent

D = D.L. per sq.ft of area supported by the member

L = L.L. per sq.ft of area supported by the member

TABLE 2.1

RESIDENTIAL STANDARDS FOR UNIFORM DESIGN LOADS FOR FLOORS ACCORDING TO THE NATIONAL BUILDING CODE OF CANADA, 1970,1975

Use of Area of Floor	Design Live Load psf
Dwelling units	
- bedrooms	30
- all other rooms	40
Common space in buildings containing more than one dwelling unit	
- locker rooms	50
- entrance halls, ground floor corridors, exits and stairs	100
- corridors above the ground floor	40
Attics where there is no storage of equipment or material and not accessible by a stairway	10
Attics accessible by a stairway	30
Fire escapes, exterior balconies	100
Garages	
- for passenger cars	50
- for unloaded buses and light trucks	125
- for loaded trucks and buses and all trucking spaces	250
Sidewalks and driveways over basements or other open areas	250

TABLE 2.2

RESIDENTIAL STANDARDS FOR CONCENTRATED DESIGN LOADS FOR FLOORS ACCORDING TO THE NATIONAL BUILDING CODE OF CANADA, 1970, 1975

Use of Area of Floor	Concentrated Load, lb
Floors and areas used by passenger cars	2,500
Floors and areas used by vehicles not exceeding 8,000 lb gross weight	4,000
Floors and areas used by vehicles not exceeding 20,000 lb gross weight	8,000
Floors and areas used by vehicles exceeding 20,000 lb gross weight	12,000
Driveways and sidewalks over basements or other open areas	12,000

2.3 FORCES DUE TO TEMPERATURE AND VOLUME CHANGE EFFECTS (T)

Temperature loads (T) are defined as those loads or effects originating from dimensional changes induced by variations in ambient temperature, creep and shrinkage, along with those loads and effects induced by differential settlement of foundations. Temperature changes included in "T" are those resulting from weather cycles only. The various components of the "T" loading are time-dependent and, with the exception of the differential settlement component, are also climatologically dependent.

The temperature existing at any point in a wall under any given exterior and interior temperature conditions is of great significance in designing of problem-free building enclosures. An ability to calculate the thermal gradient permits the designer to forecast the magnitude of the movements caused by external temperature changes, to predict the location of condensation and freezing planes in the wall and to assess the suitability of the building. The first and most obvious effect of a difference of temperature between the inside and outside is that heat flows from the side of high temperature to the side of low temperature; adequate cooling or heating equipment must be provided to counterbalance the overall gain or loss, in order to maintain the desired internal temperature.

The effects of the various components may be additive in some cases, while cancelling each other in other cases. The residual displacement of an exposed exterior wall due to a temperature decrease along with the deformation associated with shrinkage and creep, and differential settlement of that wall, may be algebraically summed to obtain the total differential movement of the exterior wall at any particular storey level. The induced bending moments and shears may then be calculated from this movement. A designer making these calculations will soon realize that although adding insulation may alleviate some problems, it may also produce others. An increase of insulation in a wall will raise the interior surface temperature and minimize the risk of surface condensation. A decrease in the insulation value, will increase the heat loss, will lower the interior surface temperature, and will thereby reduce the risk of condensation or freezing within the wall construction.

The designer with an appreciation of the temperature variations in wall elements can estimate the magnitude of movements or stresses that may be induced in the components by expansion and contraction. He can then determine the required number and width of movement controlling joints or amounts of reinforcement necessary to prevent their failure. Therefore, the determination of the thermal gradient throughout a precast building element that separates two environments that have different properties,is the first factor to be recognized in designing walls.

2.4 SOIL AND HYDROSTATIC PRESSURES (H,F)

Loads denoted by "H" are those due to lateral pressures exerted by adjacent soils or other granular materials against basement walls, retaining walls, and similar below-grade structures. Loads denoted by "F" are those due to hydrostatic pressures exerted by water or other fluids applied normally to surfaces of a structure.

With the magnitudes defined, soil and hydrostatic pressures have to be considered in combination with other loadings, which may be critical for structures subjected to high soil and hydrostatic pressure or hydrostatic uplift. The critical loading combination can be associated with either the minimum or maximum vertical loads.

When a portion, or the whole, of the adjacent soil is below a free water surface, the design of basement walls shall be based on the weight of the soil diminished by buoyancy, plus full hydrostatic pressure.

The upward pressure of water, of basement floors, shall be taken as the full hydrostatic pressure applied over the entire area. The hydrostatic head shall be measured from the underside of the construction. The footing drain and the drainage layer adjacent to the wall will prevent the development of a hydrostatic head of water. The outer surface of the wall below grade is, however, in contact with a very moist environment, with a relative humidity in the soil air at or very close to saturation and, occasionally, water trickling down from the ground surface or as the result of condensation at a higher level. For most of the year the water vapour pressure in the soil adjacent to the wall will be higher than that in the air in the basement, with the result that water vapour will tend to flow into the basement. The amount of water vapour that will pass through good quality concrete is very low, but concrete made with a high water cement ratio is quite permeable. In addition, liquid water in contact with the basement wall can be drawn by capillarity through the wall. How serious this will be depends on the properties of the wall material and the severity of exposure (See Chapter 8.)

2.5 <u>SNOW LOAD (S)</u>

Snow load has to be recognized in design as "live" gravity load. Like other forms or live loads, it is a transient type load.

The basic snow load to be used in the design of a structure depends on the product of the climatological ground snow load and factors related to the type, size, proportions, site and orientation of the building. The climatological ground snow load is defined as the maximum expected snow load in pounds per square foot in a specified statistical basis for a period, typically 25 years, 50 or 100. These loads are usually obtained from maps containing isolines of ground snow load. The various factors associated with the building's layout can cause the total load on the roof to be less than, or greater than, that on the same area of ground. Factors which may reduce the snow load are: removal of snow by wind or evaporation, and melting due to diurnal rise in air temperature or to heat loss through the roof. The factor which may increase the total roof load includes: drifting, melting and consequent refreezing, and the accumulation of snow due to blowing or sliding from adjacent higher roofs. Since the variation of ground snow load with elevation and exposure, such as may exist in mountainous areas, is not yet fully understood, it is usually suggested that in determining the climatological ground snow load for these special areas, local snow records be consulted or the load to be used be obtained from the local building officials.

The design for a minimum snow load on a roof area to be considered is:

$$S = Cs \times g$$

where

S = design snow load in psf

g = ground snow load in psf

Cs = snow load coefficient

The basic snow load coefficient can be taken as 0.8, except that for roofs that have a clear exposure to winds of sufficient intensity to remove snow and that have no projections such as parapet walls, a basic snow load coefficient of 0.6 may be used.

2.6 EARTHQUAKE LOAD

Earthquakes usually introduce lateral forces either quasistatic or dynamic.

The behaviour of a structure depends on the rigidity, the stiffness and the period of vibration. Because the shorter period of vibration results in higher spectral accelerations, the stiffer structure may attract more horizontal force. Designing too large a force will not necessarily make the structure safer if the structure is less ductile. On the other hand, a less stiff structure will provide the structure with more flexibility, and in general, it will be lighter and generate somewhat of a less horizontal force. The greater flexibility of such a structure may invite a mere energy input, since the spectral velocity could be greater for a longer period.

Also, the greater flexibility may cause an undesirable vibration under earthquake or wind, and the structure may be unsatisfactory for reasons that differ from those assoeiated with its structural behaviour.

The design recommendations of the SEAOC Code (Seismology Committee Recommended Lateral Force Requirements, July 1969) require that the building be designed to withstand a minimum total lateral seismic force of magnitude given by

$$V = K \times C \times W$$

where

W = the total weight of the building

C = the seismic coefficient given by the following relation:

$$C = \frac{0.05}{\sqrt[3]{T}}$$

(<u>Exception</u>: C = 0.10 for all one- and two-storey buildings)

K = coefficient values from 0.67 to 1.33 for buildings having different framing systems.

The value K = 2.0 should be taken for buildings which exhibit little ductility and damping. This includes unreinforced masonry buildings and unreinforced masonry components. (The high value K = 3.0 is considered more appropriate because of the poor performance of such structures in past earthquakes.)

T = the period of vibration for the height H of the building and its dimension D in a direction parallel to the direction of motion can be computed from the formula

$$T = \frac{0.05H}{\sqrt{D}}$$

For buildings in which the lateral resisting system consists of a moment resisting space frame that resists 100 percent of the required lateral forces and which is not enclosed by or adjoined by more rigid elements, the value of the period T can be taken as 0.10 times the number of stories above the exterior grade.

$$T = 0.10N$$

where

N = the total number of storeys above the exterior grade

2.6.1 Seismic Coefficient (C)

The lateral seismic force on each storey or part of a building or structure, to be used in computing the total shear at the base, should be taken as C times the weight of that storey or part. The coefficient (C) was the same for every storey of a particular building and varied only with the site. If the seismic coefficient (C) were 0.07 (7 percent of gravity) and a building had several stories, 7 percent of the weight at each storey was applied as a lateral force at each floor level. Therefore, in most buildings, the lateral forces considered in computations of shears increased only slightly from the top floor levels downward because many of the massive elements, walls, slabs, stairs and partitions do not vary in weight from storey-to-storey.

The actual storey weights of a slender 15-storey building are based on the assumption that the weights are concentrated at the floor levels. If a hypothetical seismic coefficient of 0.07 had been used in the design of that building the shears would have been as shown in Figure 2.1. The seismic coefficient (C), even though constant for the building as a whole, would have different values depending upon:

(a) The soil conditions upon which the structure was founded, and

(b) The seismic rating of the geographical zone in which the structure was to be located.

However, its application to tall buildings is not only illogical from the dynamic standpoint, but would produce an inconsistent strength for the upper storeys, as compared with the lower storeys. If the upper storeys are

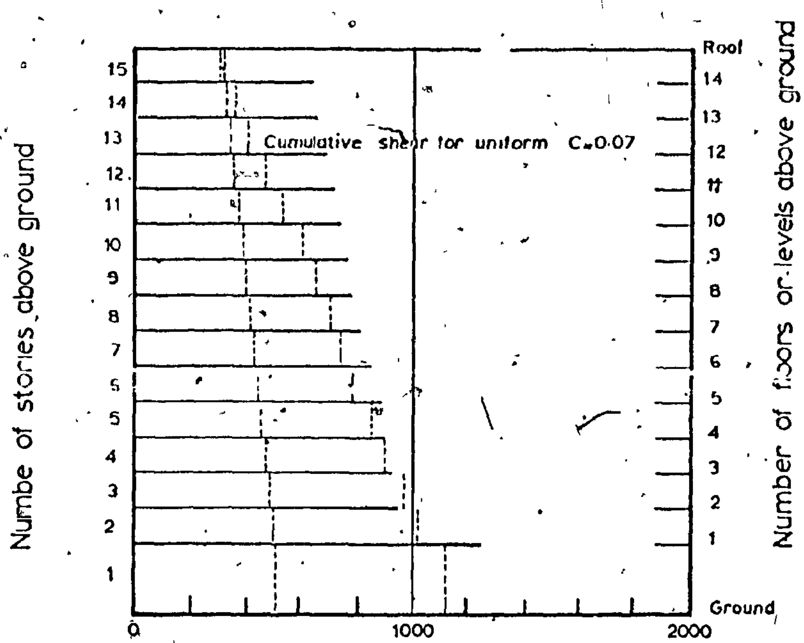


FIG. 2.1 Storey Weight and Cumulative Shear, kips. [9]

properly designed, the result would be much heavier and more costly construction in the lower storeys than in buildings that have withstood severe earthquakes satisfactorily.

In the Uniform Building Code of the International Conference of Building Officials they were revised in 1949, to provide a new seismic coefficient (C) that, instead of being constant over the entire height, would vary for each storey level in accordance with the following formula:

$$C = \frac{0.60}{N + 4.5}$$

where N is the number of stories above the storey under consideration.

For the upper storey of all buildings it became

$$C = 0.133$$

and for the lowest storey of various buildings it became as shown in Table 2.3.

In the National Building Code of Canada, 1960, earthquake loads are given by:

$$V = C \times W$$

where

V = the shear force at any level in a structure

W = the total load above this level and consists

TABLE 2.3

SEISMIC COEFFICIENT (C) [9]

N = number of storeys above bottom storey	C = the base
0	0.153
5	0.063
10	0.041
15	0.031
20	0.024
25	0.020

of the design stored load and service equipment loads

The seismic coefficient, C, is computed from

$$C = \frac{K(0.15)}{N + 4.5}$$

where

N = the number of stories above the level under consideration

and

K = the integer 1,2 or 4, representing the seismic risk associated with a region

Evidence of seismic activity demonstrates the need for earthquake protection in certain regions. In the light of present day knowledge, it appears that some of the codes do not adequately account for all the variables involved in the problem of earthquake design. There are strong arguments for considering the fundamental period of vibration of a structure as a basic design parameter and also for considering the relations between the type of structure and type of soil. Nevertheless, for many typical structures, the codes specify values of base shear that do not deviate far from the values calculated according to the regulations of other codes.

The National Building Code of Canada 1975, specifies that a structure should be designed for a minimum earthquake force given by the formula

$$V = A \times S \times K \times I \times F \times W$$

where

A = the value of the assigned horizontal design ground acceleration in units of gravity

S = the seismic coefficient which is given by the formula

$$S = \frac{0.5}{\sqrt[3]{T}}$$

In lieu of more accurate estimates, the following empirical formulas can be used for the determination of the fundamental period T for the height (h_n) of the building and its dimension D in a direction parallel to the direction of motion can be computed from the formulas

$$T = \frac{0.05\ h_n}{\sqrt{D}} \text{ and } T = 0.1N$$

for moment resistent space frames only (where N = the total number of stories above exterior grade.)

K = coefficient (k) assigned to different types of structural systems

The coefficient K reflects the design and construction experience, as well as the evaluation of the performance of structures in major and moderate earthquakes. The value

$$K = 2.0$$

should be taken for buildings which exhibit little ductility and damping. This includes the unreinforced masonry buildings and unreinforced masonry components. (The high value K = 3.0 is considered more appropriate because of the poor performance of such structures in post earthquakes.)

I = the factor (I) assigned for essential public services

It is imperative that these structures be operative after an earthquake. The value I = 1.3 is not intended to cover the design considerations associated with special purpose structures whose failure could endanger the lives of a large number of people or affect the environment well beyond the confines of the building. These would include facilities for the manufacture or storage of toxic materials, nuclear power stations, etc.

F = foundation factor (F)

The foundation factor (F) has been shown to exert a major influence on the amplitudes and nature of the earthquake motions at the ground surface. In cases where the motions propagate from bedrock to the surface, the soil can amplify the bedrock motions in select frequency ranges about the natural frequencies of the soil layers. For many soil layers the value of F is obtained by assuming an "average" soil type over the total depth. In all cases the applicable soil depth is measured from foundation level to the bottom of the layer under consideration. For the purpose of determining the foundation factor (F), small lenses of material having lateral dimensions of the order of 200 ft or less can be ignored. The advice of an experienced geotechnical engineer should be sought for the evaluation of the suitability of the site, and its possible behaviour under seismic forces and movements.

In comparison with wind, earthquake requirements govern the design of buildings lower than a certain critical height, which varies from below five to more than 30 storeys, depending on the building and its location. Wind requirements dominate for the lower part of the buildings which are over this critical height, although a substantial upper part may still be governed by earthquake.

2.6.2 Shear Wall-Type Buildings

Studies of recent earthquakes have indicated that cast-in-place shear wall buildings when properly designed and detailed, offer excellent resistance to earthquakes. A precast concrete panel building is basically a shear wall structure in that the in-plane stiffness of the wall panels constitutes nearly 100 percent of the building's resistance to lateral loads. It must be noted, that precast concrete panel buildings will act as a shear wall structure only if the wall panels and their connections have sufficient strength to match their rigidity.

The Codes generally classify precast concrete panel structures as "box" type systems and require an increased factor K in determining the design base shear. These design loads are not strictly theoretically correct. Therefore they can result in a safe design if applied judiciously with the following considerations.

(a) Vertical and horizontal joints between precast wall elements must be capable of transferring earthquake loads in bending, shear and thrust while remaining in the elastic range.

(b) The load factors should provide a sufficient margin of safety to resist increased shear forces should the structure be subjected to a strong earthquake.

(c) The precast wall elements should have sufficient vertical continuity throughout the height of the structure to assure their acting as vertical cantilevers.

(d) Adequate horizontal continuity inducing diaphragm action must be provided to ensure integral behaviour of all building elements.

(e) The design seismic forces specified by the codes are substantially less than those forces which will develop under seismic dynamic excitation.

2.7 WIND LOAD

Wind loads are defined as those forces originating from the natural movement of air. As a result of the mass and velocity of air, wind possesses kinetic energy which is transformed into the potential energy of pressure when the wind comes into contact with a structure. The natural phenomena associated with this loading are daily winds from normal atmospheric pressure fluctuations, thunderstorms and hurricanes.

The design external pressure or suction due to wind on a building as a whole, or on cladding, shall be calculated from the formula

$$P = q \times C_e \times C_g \times C_p$$

where

P = the design external pressure acting statically and in a direction normal to the surface either as a pressure (directed towards the surface) or a suction (directed away from the surface)

Q = the reference wind pressure which is determined by the following equation $q = C \times V^2$ (the factor C depends on the atmospheric pressure and the air temperature and V the reference wind speed)

C_e = the exposure factor as provided for the appropriate height of the surface or part of the surface

C_g = the gust effect factor is the ratio ot the expected peak loading effect to the mean loading effect. Therefore it makes allowances for the variable effectiveness of different sizes of gusts and the load magnification effect caused by gusts in resonance with the structure vibrating as a single degree of freedom cantilever

C_p = the pressure coefficients defining the pressure acting at local positions on the surface of a building or structure

The main objective of any design approach to wind loading is to ensure that the structural response to wind will not impair the strength, stability or serviceability of the structure. The main things to be avoided in proper design are:

(a) failure due to instability, fatigue or yielding with resultant excessive deformation, and

(b) loss of serviceability due to excessive sway causing occupant discomfort, or excessive deflection causing architectural damage to the interior finishes or exterior skin

The approach encourages the designer to develop an understanding of the dynamic nature of wind loads and the mechanics of wind interaction with structures.

The application of the standard requires:

(a) The choice of a basic reference wind speed with an appropriate mean recurrence interval;

(b) the determination of an exposure factor depending upon the roughness of the surrounding terrain;

(c) the determination of a gust response factor depending upon the roughness of the terrain and the dynamic response characteristics of the building; and

(d) determination of the pressure coefficients which relate the external and internal pressures on the various surfaces to the effective dynamic pressure of the wind. This last coefficient forces the designer to consider the extreme outward and upward acting forces in building components such as corners, eaves and ridges.

For buildings which have dynamic properties tending to make them wind sensitive, and for structures whose height exceeds five times the least horizontal dimension, the standard requires a detailed dynamic analysis to determine the response of the building to wind gusts. It also emphasizes that forces induced by vortex shedding and instability due to galloping or flutter are not provided for, and that these effects may govern in the design of slender structures with low natural damping. The fact that winds, as well as earthquakes, exert lateral forces on buildings invites a comparison between them.

The geographic location determines the wind gust pressure q and the seismic factor K. The considerable influence that location has on the design forces in the same structure has been analyzed in two different areas, one with high wind loads and the other with a high earthquake factor. The relations between shear and moment for wind and those for earthquake vary considerably from one region to the other.

Other factors being constant, the higher the building, the more probable it is that wind effects will exceed earthquake effects. Earthquake effects, on the other hand, exceed those due to wind for all buildings below a certain height, and, this critical height may well be as high as 30 storeys (360 ft) or even more.

Weight distribution is expressed by four parameters, P_1,d,w and r, where:

P_1 = average unit weight of top storey,psf

d = depth of building, ft

w = width of building, ft

r = ratio of increase in unit wt/storey:P_1

For relatively low buildings, an average storey weight, $w_{average}$, can be substituted as follows. The equation of shears which are considered in the following ratio act just above the foundation and the basement slab: [10]

$$\frac{V_w}{V_e} = \frac{0.294\ q'h^{9/7}w\ N^{9/7}}{0.15\ K\ P_1\ d\ w[\frac{4+rN^2+(2-r)N}{2N+9}]} = \frac{q'h^{9/7}}{K\ P_1\ d}\ x$$

$$x\ \frac{0.249N^{9/7}(2N+9)}{0.15[4+rN^2+(2-r)N]} = \frac{q'h^{9/7}}{K\ P_1\ d}\ x\ F_u(N)$$

where

q' = basic gust pressure q_{30} (30 ft.ht) multiplied by a total pressure coefficient of 1.5,psf

h = storey height, ft

N = total number of storeys (not including basement)

k = seismic zone factor (1,2 or 4)

However, the weight distribution is not critical for buildings up to 10 storeys in height. For taller buildings the ratio is more sensitive to the parameter "r", indicating the need for a more careful assessment of the weight distribution and a more liberal allowance for error in the result. It may be observed that when earthquake governs at the base, then it governs all the way to the roof; on the other hand, even though wind may govern at the base, earthquake invariably takes over at and above some upper levels of the building.

CHAPTER 3

PROGRESSIVE COLLAPSE

Progressive collapse may be defined as a chain reaction of failures following damage to only a portion of a structure. However, in buildings it typically occurs when abnormal loads cause loss of structural capacity of one or more critical parts. Progressive collapse of a significant portion of a structure, as a result of local failure, is deemed to be generally unacceptable. Recent studies indicate that progressive collapse should be dealt with in multistorey construction of all types and not be limited to precast concrete element constructions.

The progressive collapses that have occurred in the past and of abnormal loads, showed us the need for a new consideration in structural design resistance to progressive collapse initiated by local failure due to abnormal loads.

The progressive collapse is considered as the phenomenon in which local failure is followed by collapse of adjoining members which in turn, is followed by further collapse and so on, so that widespread collapse occurs as a result of local failure. This local failure may be initiated by explosion or impact or may simply be the result of local defective strength.

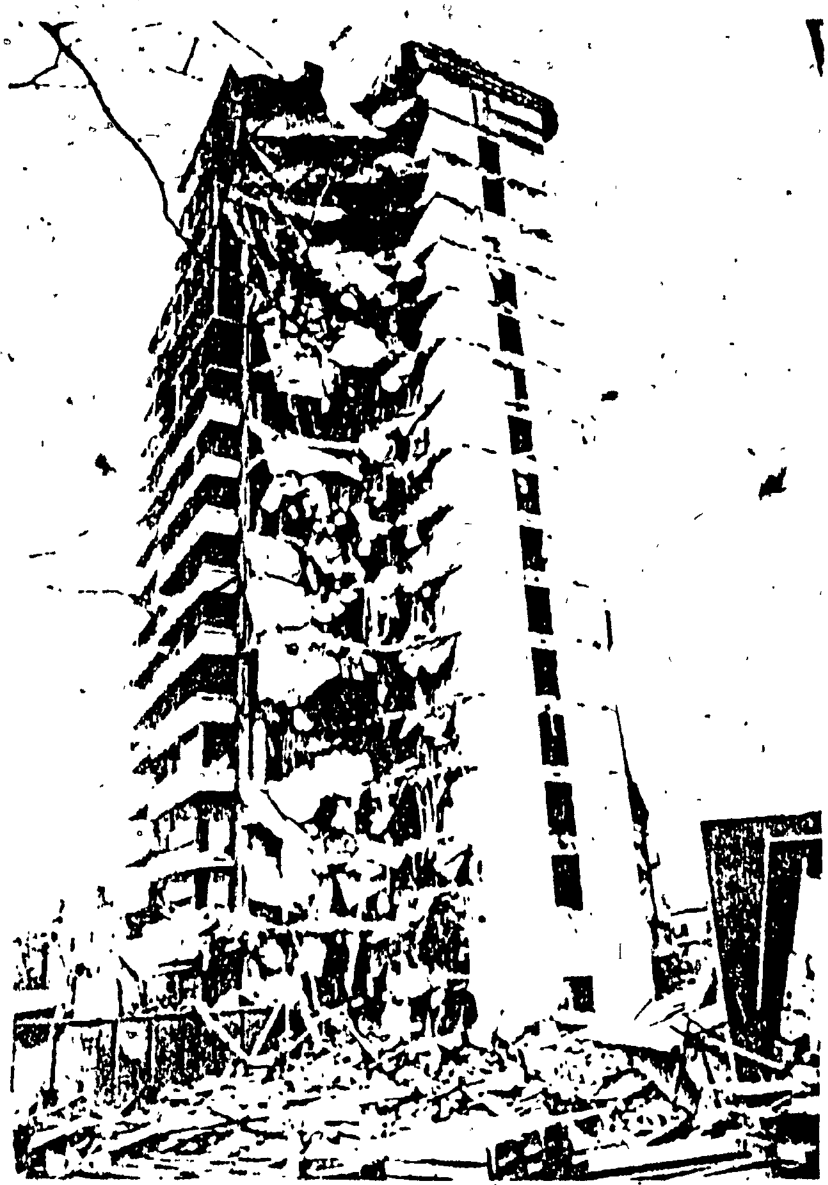


FIG. 3.1 Progressive collapse during construction of reinforced concrete building. Boston, January 25th, 1970. [28]

The progressive collapses have been categorized first according to whether collapse occurred during construction, during service life, during demolition of excavation nearby.

The number of progressive collapses is considerable, comprising approximately 15 to 20% of the total number of collapses which may happen during construction and due to errors in formwork, bracing or erection procedure.

Although there have been reinforced concrete buildings which have collapsed progressively after having been exposed to fire, evacuation is usually possible before collapse and therefore danger to life safety is usually not a problem. Exceptions are very tall buildings where evacuation is not possible and buildings where local fires can create large explosions (Fig.3.4).

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

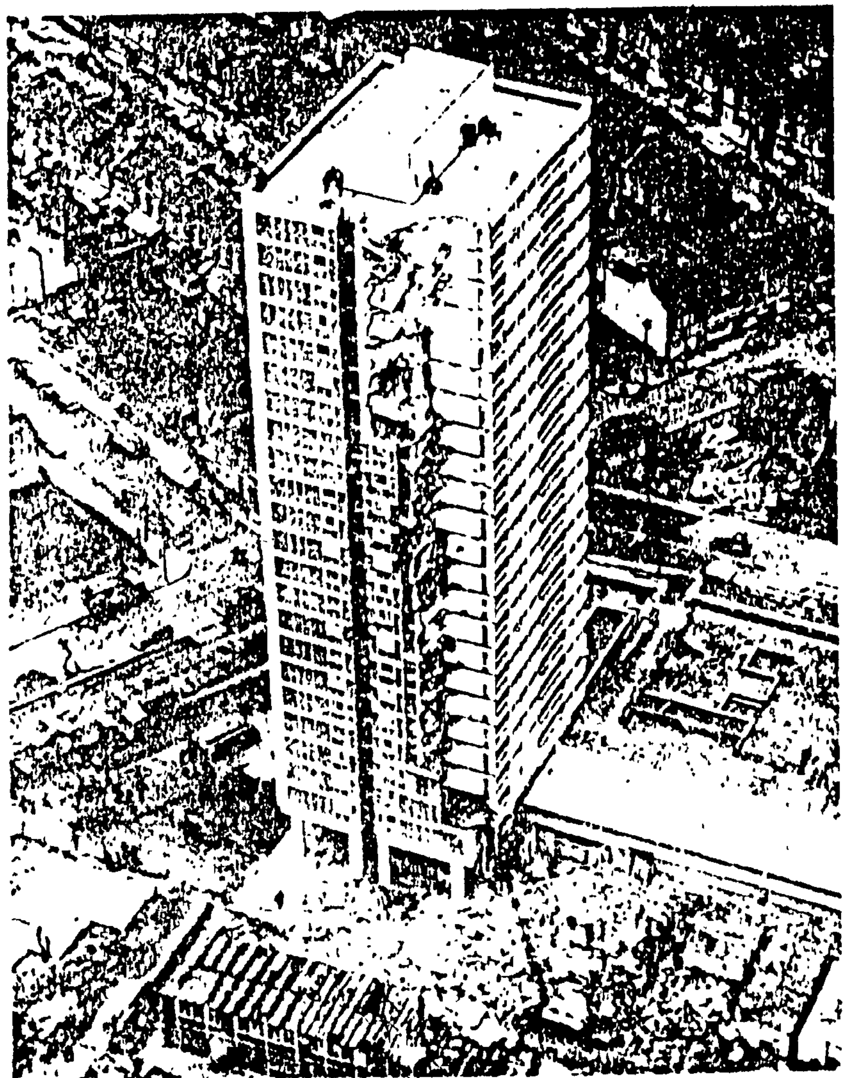


FIG. 3.2 Progressive collapse during the service life of the structure. England, May 16th, 1968. [28]

However, it is suggested that the risk of such collisions be treated as a site planning consideration, rather than as a structural design load, except for buildings in unusually exposed locations.

Certain values indicate an apparent necessity to consider abnormal loads. However, if design provisions are made to ensure a general structural integrity, the probabilistic approach which established the mortality risks becomes unnecessary; the assumption that a progressive collapse will occur does not apply. On the contrary, an adequate degree of structural safety established by assuring an appropriate level of general structural integrity obviates the necessity to develop deterministic or probabilistic specifications for particular abnormal loads.

The examination of progressive collapse causing heavy damage indicates the following facts:

(a) Although quite a number of collapses can be classified as progressive, most of these would not have occurred if normal good engineering practice, without special consideration of progressive collapse, had been carried out both in the design and during construction.

FIG. 3.3 Progressive collapse during the service life of the structure. New York, August 4th, 1970. [28]

(b) Many kinds of abnormal loads can initiate progressive collapse; eventually, various kinds of explosions, impact from vehicles, from falling objects and from collapsing structures, sudden changes in ground conditions and structural defects.

(c) Although vehicle impact has often led to progressive collapse, this danger can most easily be avoided by protecting certain exposed members. Many designers now recognize this in design.

(d) Only a few should definitely be considered structurally from the point of view of progressive collapse. The situation, however, could become more critical in the future due to a change in types of construction to high-rise precast concrete and due to possible increase in the likelihood of structurally damaging explosions of different kinds.

(e) The structural resistance to progressive collapse may have to be considered for (i) high-rise precast concrete construction, (ii) large span roofs where defects are possible in the material or in the manufacture and where there may not be sufficient warning of impending failure and (iii) precast or preassembled buildings during construction. Therefore, no single quanti-

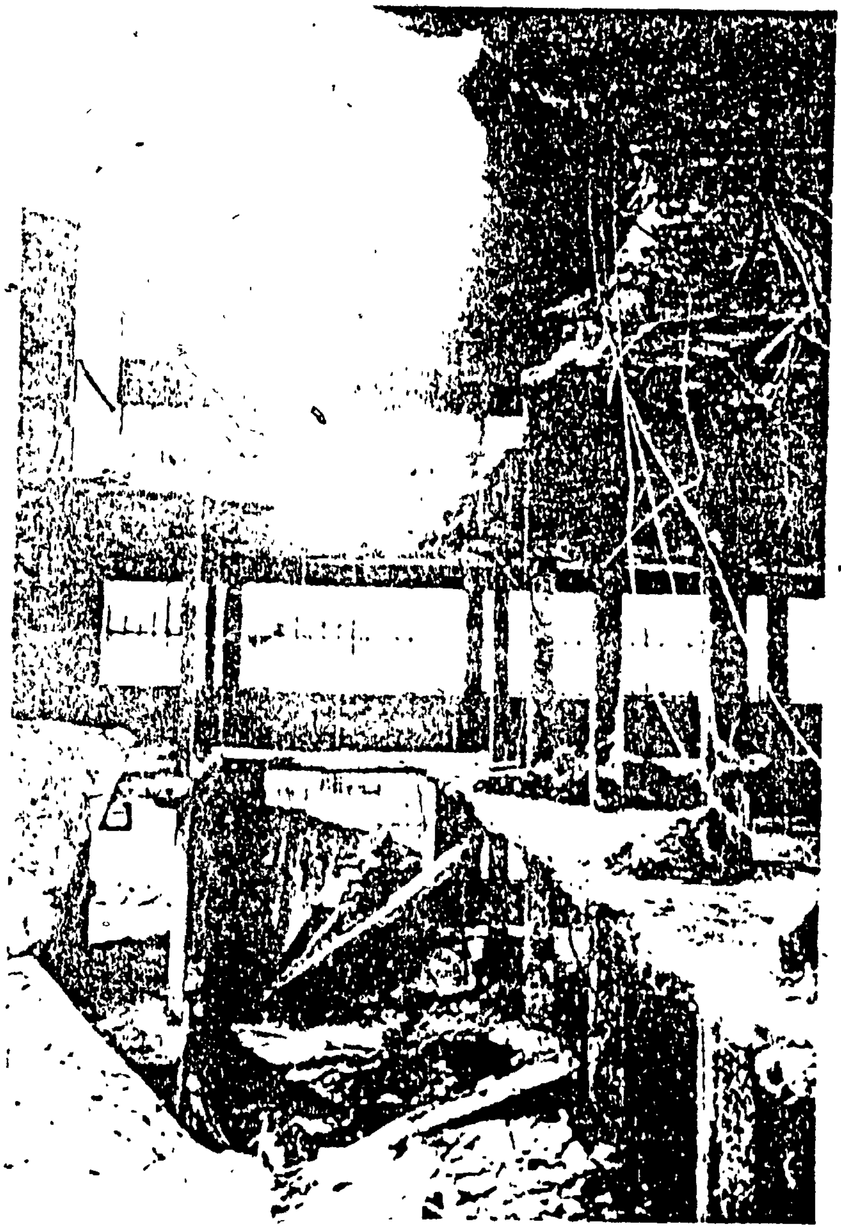


FIG. 3.4 Progressive collapse resulting from fire in the structure. Switzerland, May 9th, 1963. [28]

tative rule can be formulated for all buildings. Only a performance or warning type requirement can be given. Quantitative rules, however, will be required for specific types of construction. Until such rules are determined, the building designers may have to use an independent source of judgment, such as a review panel of experts or an independent consultant.

In the British Unified Code, [24] the modified progressive collapse provisions were incorporated for the structural use of concrete. This Code was published late in 1972.

In fact, the Unified Code does not refer to progressive collapse, rather, it refers to stability.

3.1 UNIFIED CODE REQUIREMENTS [24]

A summary of the latest Unified Code Requirements on progressive collapse are the following:

(a) Unless wind loads control, the building must be designed for a horizontal force equal to not less than 1.5 percent of the building dead load.

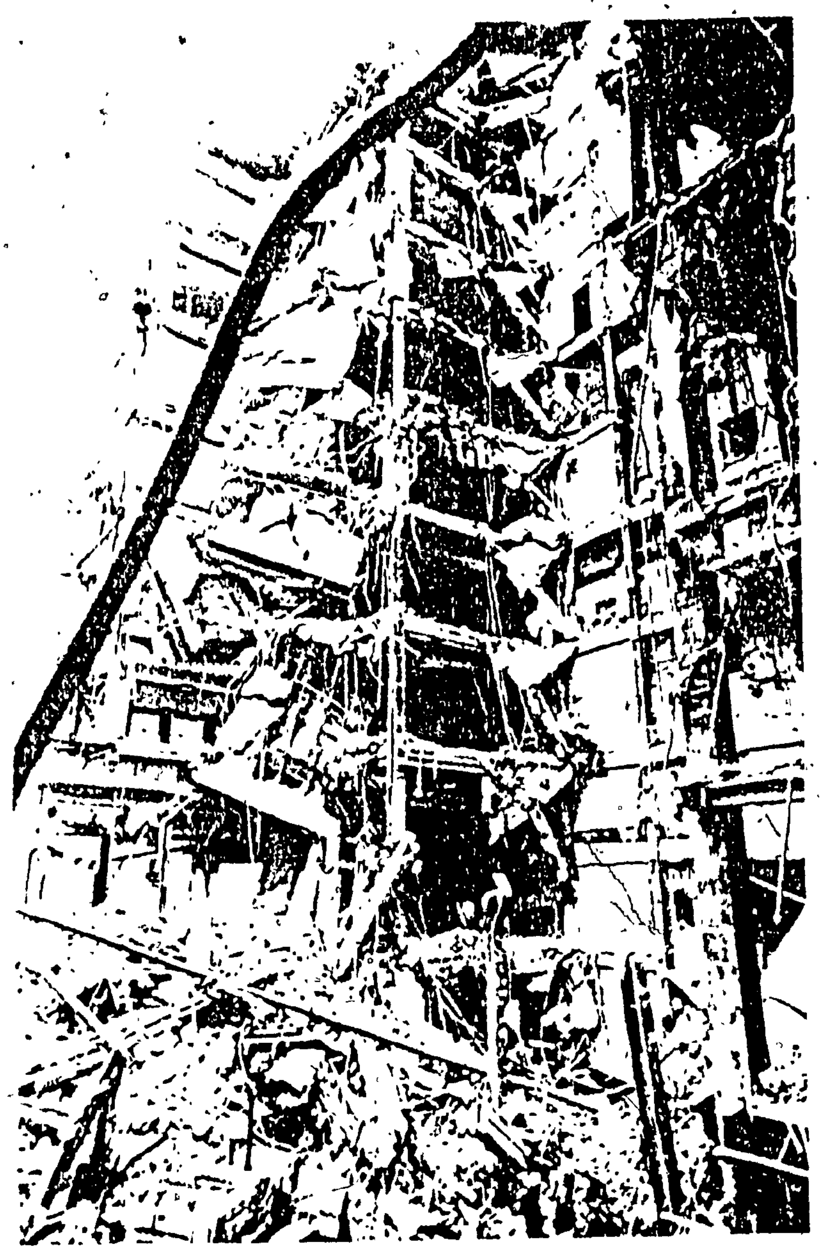


FIG. 3.5 Progressive collapse, during demolition, in the structure. Kansas City, February 1972. [28]

(b) All buildings must be provided with horizontal ties at each floor. These ties are peripheral and internal assuring diaphragm action and providing some ductility. The amount of tension to be resisted by ties is a function of load, span, storey height, and building height.

(c) External load-bearing walls and columns must be restrained horizontally at each floor with a force equivalent to not less than 3 percent of the ultimate.

(d) In buildings over five storeys in height, effective vertical ties must be provided in all columns and walls. The area of this tie is equal to the minimum main reinforcement.

(e) In buildings over five storeys where the vertical ties do not meet the requirement minimum: (i) the building must be designed so that any single vertical loadbearing element can become incapable of carrying its load without causing collapse of the structure or any significant portion of it except that (ii) any vertical load-bearing element, which cannot be allowed to be ineffective, must be designed, together with its connections and horizontal members providing lateral support, to withstand a load of 5 psi (720 psf) applied from any direction.

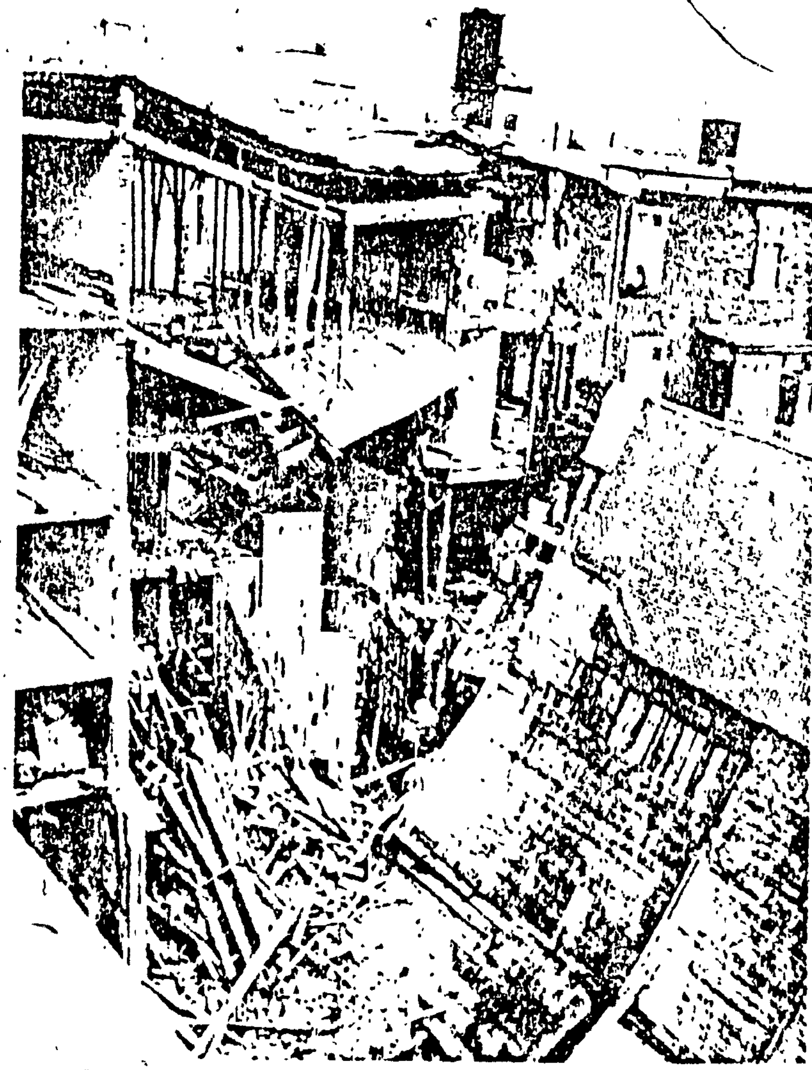


FIG. 3.6 Progressive collapse, due to adjacent excavation, in the structure. Montreal, November, 1964.[28]

For either alternative, the designer need only use a safety factor of 1.05 and include the dead load plus one-third the live load, plus one-third the wind load.

In Canada, the National Building Code introduced in its 1975 edition a performance or warning type clause which states that:

> "The structural integrity shall be sufficient to reduce to an acceptable level the hazards associated with progressive collapse due to local failure initiated by abnormal loads or severe overloads."

An explanatory commentary on this clause is given in the Code Supplement No. 4.

In the Unites States, to-date, none of the model building codes have any requirements aimed at preventing progressive collapse. However, the Department of Housing and Urban Development has developed preliminary criteria for high-rise precast concrete buildings based on the British requirements.

In the U.S.S.R., there is a requirement that large panel structures must withstand the notional removal of any wall panel of room size. This is a simple rule, less restrictive than the British one. In addition to this, the countries in Eastern Europe usually require more continuity steel than do those of Western Europe.

Therefore, the lack of definitive data in the Codes necessitates further experimental investigations.

CHAPTER 4

ABNORMAL LOADS

Abnormal loads impact and explosions, are not usually considered in the design of structures because the probability of their occurrence is considered extremely slight. If an explosion at any one location initiates progressive collapse of a large building, the risk becomes significant. To help assess the risk of progressive collapse due to different causes, it is useful to study the abnormal loads of the type that could initiate progressive collapse.

The abnormal loads have been categorized according to the type of explosion, vehicle impact, impact from large falling or flying objects, unexpected ground conditions, and miscellaneous conditions. Progressive collapse can also be initiated by abnormally low strengths due to faulty design or workmanship, or as a result of fire. In all, there are a large number and variety of abnormal loads or strengths that could initiate progressive collapse.

Generally, there are three causes for this new found significance, (a) the development of building types more susceptible to the effects of such loadings, (b) technological and sociological changes which cause an increase in such loads, and (c) public demand for greater protection from unusual events.

FIG. 4.1 Abnormal load, damaged by explosion, in the structure. North America: [28]

Some normal forms of construction have an ability to cope with conditions not accounted for in the design because of their inherent continuity or ductility. It was usually sufficient to assume that a structure designed elastically for normal conditions would react satisfactorily to the abnormal ones, or that abnormal loads large enough to cause collapse would be accepted as impractical for design. Developments in design and construction requirements, have allowed researchers to gradually reduce the margin of safety in design. New types of construction, much less capable of resisting abnormal loads, have been introduced.

Precast concrete element construction is especially susceptible to abnormal loads, due to the lack of adequate details, resulting in a limited continuity and ductility. The bearing walls in precast concrete construction also have a greater sensitivity to area loads than columns in normal construction because of the higher ratios of lateral-to-vertical-loads having depended heavily on bond or gravity and friction in the design of joints for such structures. As a result, unless measures are taken to assure an overall structural integrity in precast concrete element construction, it cannot be assumed that abnormal loads will be resisted satisfactorily. Effectively tying together the various components of the structure in order to obviate any tendency for precast concrete element buildings to behave like a house of cards under these abnormal loads.

FIG. 4.2 Abnormal load, damaged by gas explosion, in the structure. Copenhagen, January 1969.[28]

The code writers recognized the need for a systematic re-evaluation and determination of the various types of abnormal loads for all the types of construction, and undertook a study of such loads with particular reference to multiple unit residential buildings.

The fact that the examination on abnormal loads causing heavy damage indicates the following:

(a) Abnormal loads and service system explosions produce a risk high enough to warrant their consideration in the design process, as concluded from an actual versus acceptable risk analysis of abnormal loads. Such future specifications should be made applicable to all structural systems.

(b) Abnormal load-resistant designs can be achieved conceptually in the same way as for earthquake resistance by selecting relatively low equivalent forces coupled with a degree of ductility.

(c) Abnormal loads cannot be estimated at this time. The absence of required loads can be compensated for by adopting rigid site planning provisions.

(d) Lack of definitive data necessitates experimental investigations.

(e) Adequate structural safety with respect to abnormal

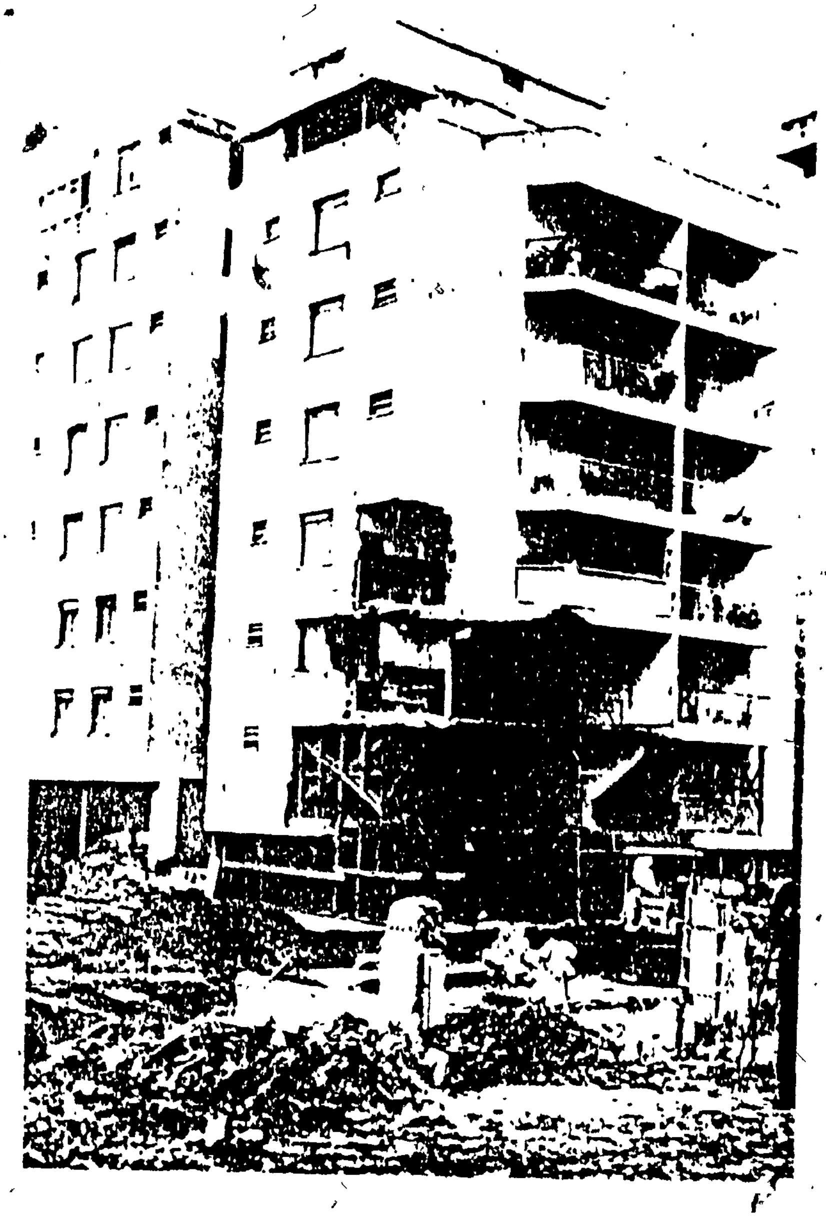


FIG. 4.3 Abnormal impact loads due to collapsing structural components. Caracas, July 1967. [28]

loads can be assured if general structural integrity is established within the structural system, thus obviating the necessity to develop deterministic or probabilistic specifications for particular abnormal loads.

CHAPTER 5

RECOMMENDATIONS FOR PRE-CAST CONCRETE PANELS

A typical precast concrete panel structure is made up of a substructure, precast concrete wall panels, and precast concrete floors and roof panels. The precast concrete wall panels are the principal vertical load-carrying elements since they transfer loads from the floor and roof precast concrete panels to the substructure without an intermediate frame.

The cost of a precast concrete panel structure is significantly affected by the number, size and shape of its precast concrete wall panels. Current production techniques for precast concrete wall panels primarily include battery mold, tilt table, and flat bed casting. These differ from assembly-line type casting beds used for hollow core precast concrete floor panels. This dissimilarity in the casting process invariably makes the precast concrete wall panel a relatively expensive structural element. As a result, to create an economical precast concrete panel structural system, the engineer should pay special attention, not only to optimizing the layout of the precast concrete wall panels, but also to the design of the elements, themselves.

To effectively use the structural precast concrete wall panel it is necessary to develop a rational method for its analysis and design. Such a procedure should be based on the general characteristics of precast concrete wall panels and should include an investigation of all critical analytical and design considerations.

5.1 ACI SPECIFICATIONS

A review of building codes indicates that the development of design procedures for loadbearing precast concrete walls has been at a much slower pace than for other reinforced concrete members.

The Joint Committee on Reinforced Concrete, formed in 1904, consisted of representatives from four national societies: The American Society for Testing and Materials, the American Society of Civil Engineers, and the American Railway Engineering and Maintenance of Way Association. The Joint Committee reported that incompatibility existed between theory and tests of reinforced concrete members. Several reports of the Joint Committee were produced during the period 1909 to 1925. Their final report in 1925 was the first code, a forerunner of the form and content of the Reinforced Concrete Building Code in use today. [13,14]

Under Section 1109 of the 1928 ACI Code, compressive stresses in walls with an ℓ_u/h ratio of 15 were limited to

$0.125\ f'_c$. A linear reduction was applied beyond this point to $0.0625\ f'_c$ when the height of the wall was 25 times its thickness. In the 1936 version of the Code, the allowable service load compressive stress was increased to $0.20\ f'_c$ for walls with an ℓ_u/h ratio of 10 or less and decreasing proportionately to $0.11\ f'_c$ for walls with an ℓ_u/h ratio of 25. Section 1112 of the 1941 Code increased the allowable working compressive stress to $0.25\ f'_c$ for walls with a height-to-thickness ratio of 10 or less, and reduced linearly to $0.15\ f'$ for walls having a height-to-thickness ratio of 25. The 1956 ACI Code retained the same allowable stresses and height-to-thickness requirements as the 1941 Code. The 1963 ACI Code presented the first major change in the design procedure since the original 1928 provisions by introducing the following equation for allowable compressive stress:

$$f_c = 0.225\ f'_c\ [1 - (\ell_u/40h)^3]$$

where

f_c = allowable stress

f'_c = specified compressive strength of concrete

ℓ_u/h = height-to-thickness ratio of the wall

The above equation resulted from the recommendation that the equation in the Uniform Building Code for allowable compressive stress in reinforced concrete bearing walls be used. That equation

$$0.2\ f'_c\ [1.0 - (\ell_u/30h)^3]$$

appeared in the 1943 edition of the Uniform Building Code by the Pacific Coast Building Conference. ACI Committee adjusted the Uniform Building Code equation to yield results fairly consistently with what has been used by the ACI since 1941. The coefficient 0.225 was chosen originally to agree with the coefficient being considered for the columns. When the column coefficient was later changed to 0.25, the coefficient for the wall equation was left unchanged.

The reduction of the allowable stress in the 1963 ACI Code for short walls from 0.25 f'_c, which had been used since 1941, to a value of 0.225 f'_c caused an all extensive discussion in the profession. Also, controversy existed over the term "reasonably concentric loads" in the 1963 Code equation for the position of the load.

The 1971 ACI Code adopted the "Strength Design Method" as the principal design procedure with the wall design equation given as: [6]

$$P_u = 0.55\ \phi f'_c\ A_g[1.0 - (\ell_u/40h)^2]$$

where

P_u = factored vertical load on the wall

f'_c = specified compressive strength of concrete

A_g = cross-sectional area

ℓ_u/h = height-to-thickness ratio of the wall

ϕ = capacity reduction factor

The ACI 1971 Code defined reasonably concentric loads as those applied within the middle third of the cross-section. In lieu of equation [6,7]

$$P_u = 0.55\phi f'_c A_g[1.0 - (\ell_u/40h)^2]$$

the code allowed for the design of wall elements as columns.

Figure 5.1 shows the basic design relationships for concrete bearing walls in accordance with ACI codes from 1928 to 1971.[6,7,13,14]

5.2 CHARACTERISTICS OF PRECAST CONCRETE WALLS

To better understand the behaviour of precast concrete wall elements under a system of applied vertical and horizontal forces the following general factors are discussed and evaluated.

(a) Types

(b) Production techniques

(c) Geometry

(d) Tolerances as in Chapter 6.

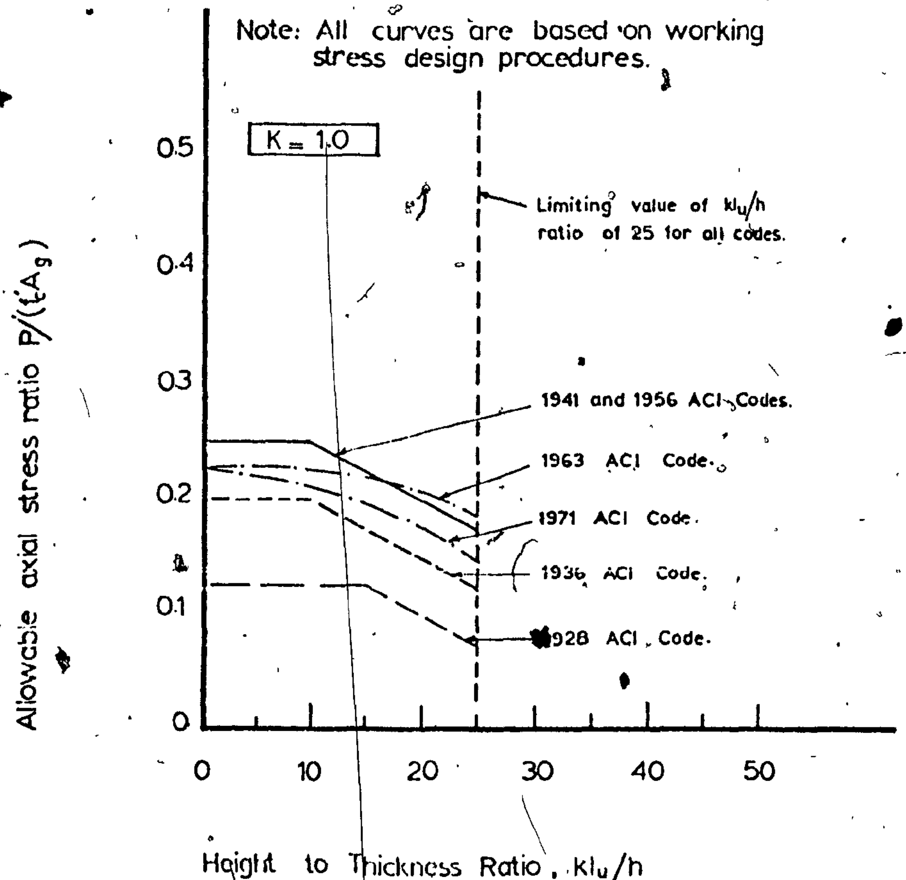


FIG. 5.1 Allowable Axial Stress in Bearing Walls as Recommended by Various ACI Codes [6,7,13,14]

(e) Handling stresses

(f) Loadings

(g) Eccentricities, and

(i) Reinforcing

5.3 TYPES

Structural precast concrete wall elements can be categorized on the basis of three characteristics:

(1) Cross-section of the precast concrete wall elements

(2) Function characteristics describe the principal structural purpose of the wall.

(3) Location characteristics describe the position and direction of the precast concrete wall element

The following Figure 5.2 shows the use of the cross-sectional function and location characteristics in categorization of precast concrete wall elements.

5.4 PRODUCTION TECHNIQUES

Current methods of precast concrete wall element production in North America includes:

(1) The battery mold method, the precast concrete wall elements are cast in arrays of vertical molds.

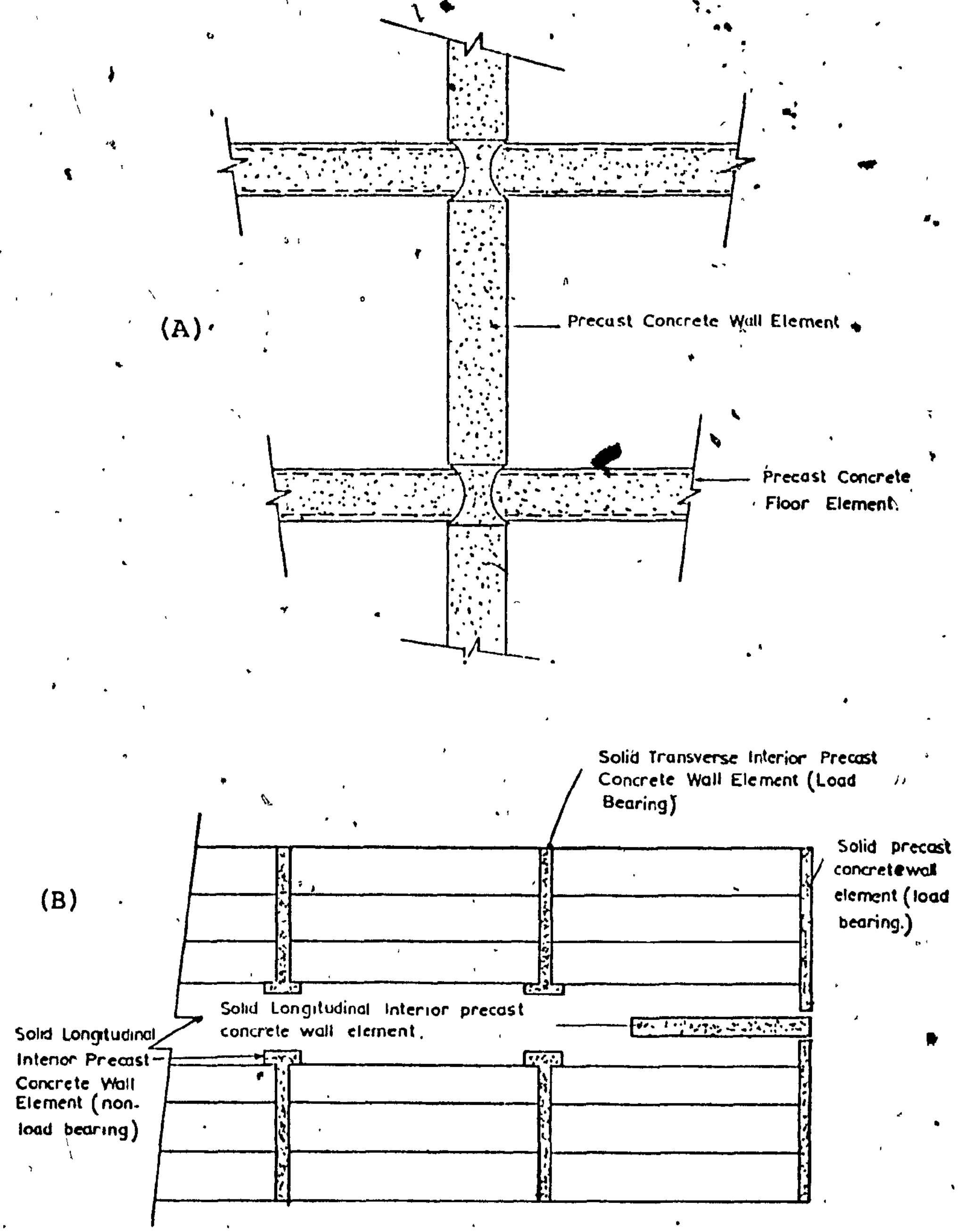


FIG. 5.2 Precast Concrete Wall Elements Showing Building in Plan and Section

(a) Precast Concrete Wall Element Section
(b) Precast Concrete Wall Elements Showing Building in Plan and Section

(2) The flat bed method, the precast concrete panels are cast in horizontal stationary tables.

(3) The tilt table, the precast concrete wall panels are cast in horizontal stationary tables. This method is a variation of the flat bed method and it is used primarily to optimize handling operations.

5.5 GEOMETRY CHARACTERISTICS

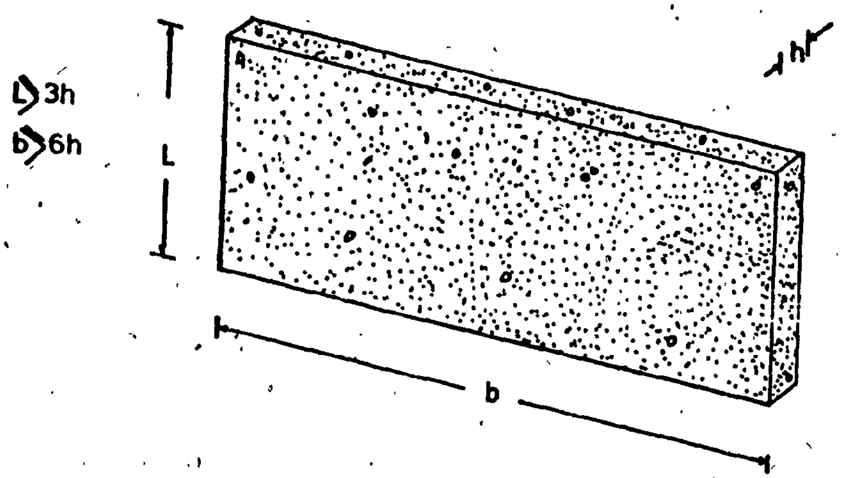
A survey of precast concrete element buildings in North America indicates that a typical precast concrete wall element will range in various geometrical characteristics as may be seen in Figure 5.3.

5.6 TOLERANCES

The acceptable tolerances in precast concrete element construction are presented in Chapter 6.

5.7 HANDLING STRESSES

Precast concrete wall elements should perform satisfactorily during three distinct parts, which will be discussed in the following sub-sections.



(1) Length (b) ranging from 10′ to 45′

(2) Thickness (h) ranging from 6″ to 12″

(3) Height (L) ranging from 8′ to 10′

FIG. 5.3 Geometry Characteristics

5.7.1 Demolding, Storage and Transmission of Components

The adequacy of inserts and the stresses in the precast concrete wall element should be checked, taking into account the adhesion of the concrete to the form work, the weight of the partially hardened concrete, the dynamic effects of lifting, and the production techniques. Stresses during the storage and transportation should also be considered.

5.7.2 Erection

The stresses in the precast concrete wall element should be checked against all conditions of the erection process. The stresses induced in the precast concrete elements during erection will be effected by such factors as temporary support, bracing conditions.

If a "crack-free" precast concrete wall element is required to resist to the normal loadings, tensile stresses in the reinforcement computed under either sub-sections 5.7.1 or 5.7.2, loading conditions should be kept relatively low.

5.7.3 "In-Place" Condition

Calculations for this part form the usual basis for design. The normal loading conditions corresponding to this part were identified in Section 5.1.

5.8 LOADINGS

Under normal loadings precast concrete wall elements are subject to act as follows.

5.8.1 Vertical Forces

Vertical forces, are caused primarily by gravity loads with some contribution as a consequence of lateral loads. Such forces occur in a direction parallel to the middle plane of the precast concrete wall element and are typically applied at a specific eccentricity. Therefore, precast concrete wall elements under the influence of vertical forces should be designed for the effects of combined flexural and axial loads as may be seen in Fig. 5.4.

5.8.2 Horizontal Forces

Horizontal forces, are caused typically by wind or earthquake loads. Such forces occur in a direction either parallel or perpendicular to the middle plane of the precast concrete wall element. Horizontal forces parallel to the plane of the precast concrete wall element, while creating nominal peripheral shear stress, can cause significant increase in the axial stress within the precast concrete wall element which should be considered in design. Horizontal forces applied perpendicular to the plane of the precast concrete wall element increase the slenderness effects and

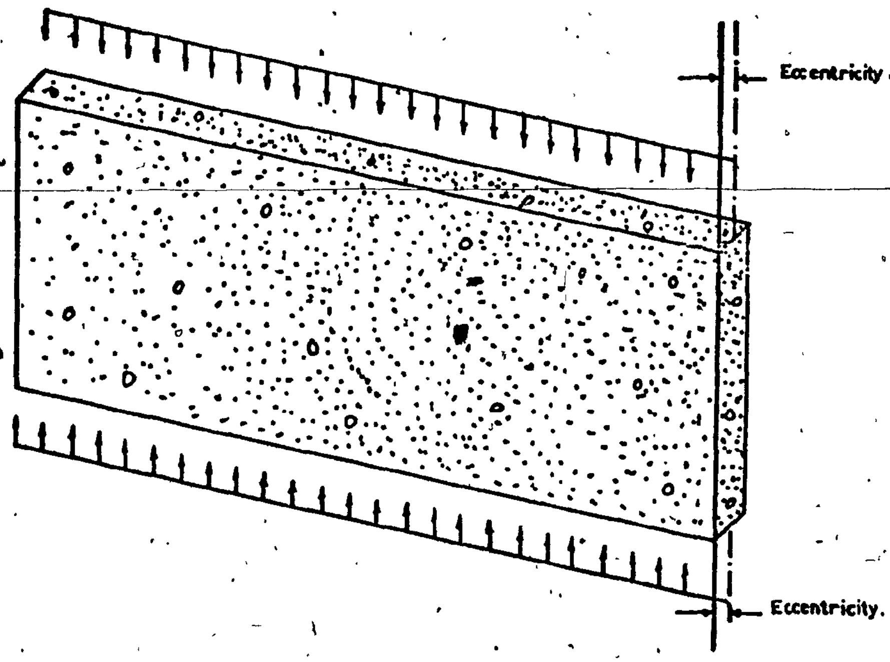


FIG. 5.4 Eccentric Vertical Forces

influence the precast concrete wall element capacity with respect to vertical forces. Such effects occur in flank precast concrete walls and should be considered in design, as may be seen in Figs. 5.5 and 5.6.

5.9 ECCENTRICITIES

In design, eccentricity accounts for the effect of end moments in relation to applied vertical forces. Although the design of most precast concrete wall elements is usually governed by code specified values for minimum eccentricity, the engineer should be aware of the sources of the various eccentricities.

Structural eccentricities occur primarily due to the relative positions of the precast concrete floor and wall elements existing at a connection. Eccentricities may also be generated at the connection either by moment transfer or by clamping action of the upper precast concrete wall elements, as may be seen in Fig. 5.7.

5.10 REINFORCING

Reinforcement requirements for precast concrete wall elements vary substantially in the different standards in use today. The British Code of Practice states that "a precast concrete wall element cannot be considered as a reinforced concrete wall unless the percentage of vertical reinforcement provided is at least 0.4% of the gross concrete cross-section." The PCI considerations for load-bearing precast concrete wall element structures suggests a minimum of 0.1% of the gross

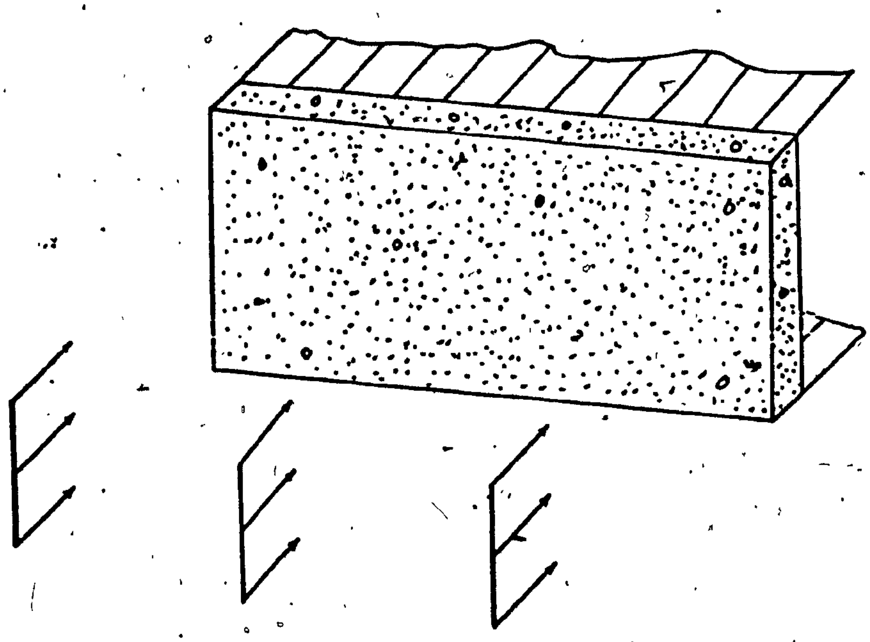


FIG. 5.5 Horizontal Forces Perpendicular to the Plane of the Precast Concrete Wall Element

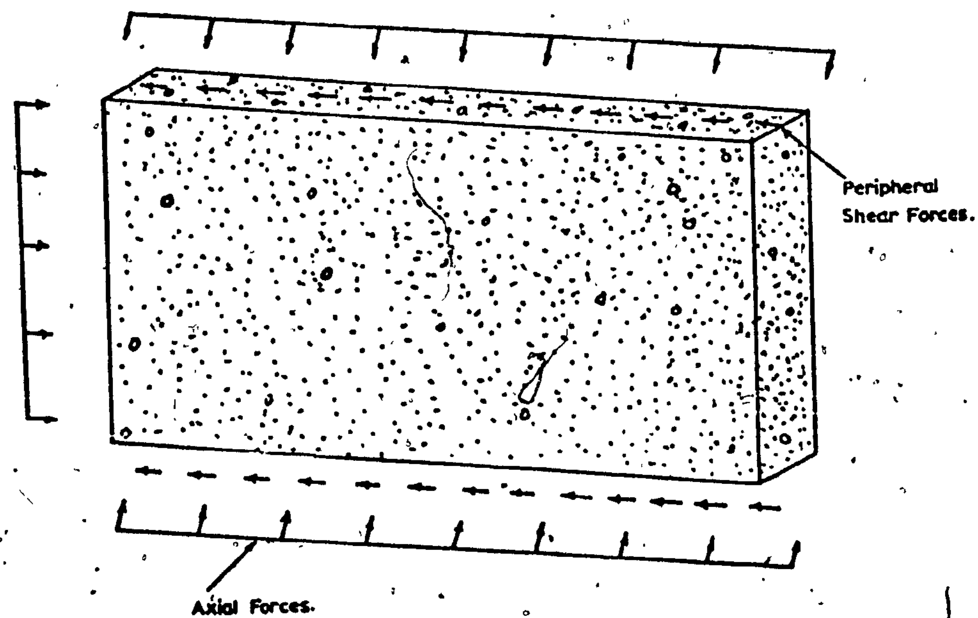
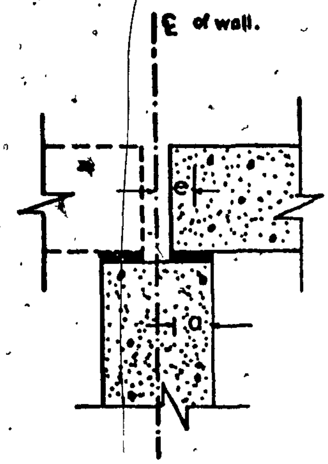
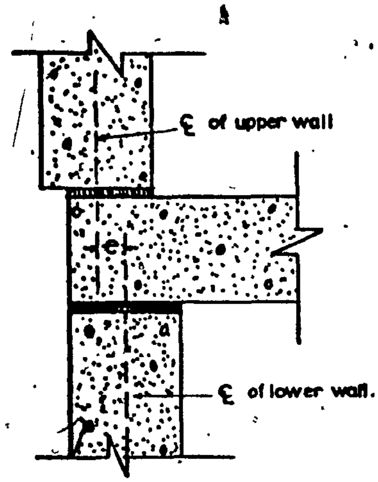


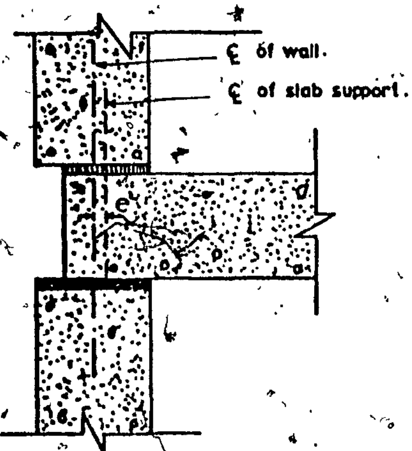
FIG. 5.6 Horizontal Forces Parallel to the Plane of the Precast Concrete Wall Element and Along With Vertical Axial Forces



Structural Eccentricity. of the precast concrete floor and wall bearing locations.



Structural Eccentricity nonalignment of joints and precast concrcte clcmcnts.



Structural Eccentricity nonalignment of Micdle planes of successive lifts of precast concrete elements.

FIG. 5.7 Structural Eccentricities

concrete cross-sectional area for both vertical and horizontal reinforcement.

The ACI 318-71 requires a minimum of 0.15% and 0.25% of the gross cross-sectional area as vertical and horizontal reinforcement, respectively. However, Section 14.1.2 of the ACI 318-71 Code allows a waiver of these requirements if reinforcement is not needed to satisfy either strength or serviceability. It is important to note that the ACI 318-71 reinforcement requirement does not make a distinction between cast-in-place walls and precast concrete wall elements. A precast concrete wall element is considered as uniformly reinforced if:

(1) Vertical reinforcement is required to satisfy strength (load-carrying capacity) and

(2) Such reinforcement is uniformly distributed and is at least 0.10% of the gross cross-sectional area of the precast concrete wall element.

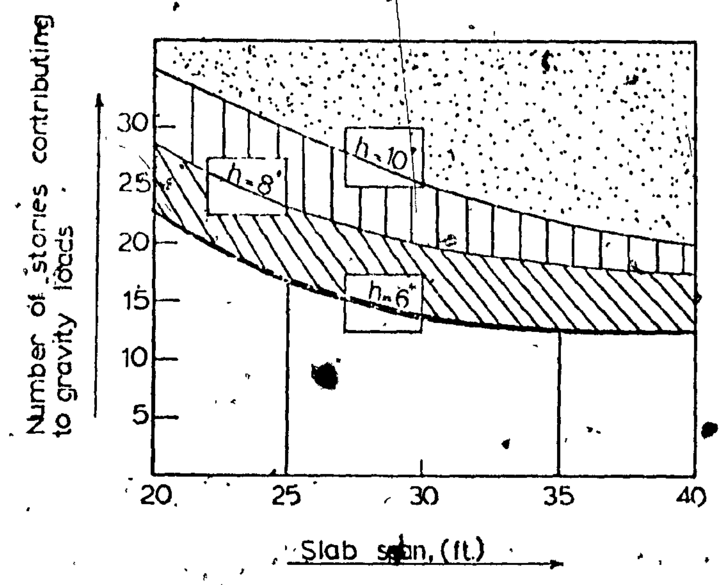
A typical uniformly reinforced precast concrete wall element has uniformly distributed vertical and horizontal reinforcement in addition to vertical tensile ties. The area of such reinforcement should be at least 0.10% of the gross cross-sectional area in both directions.

Precast concrete wall elements not requiring uniformly distributed vertical reinforcement for strength can be designed as peripherally reinforced.

A peripherally reinforced precast concrete wall element can be used generally for the majority of interior precast concrete wall elements in residential construction. The axial stress in precast concrete elements is of a relatively low magnitude. The relationship between the span, wall thickness, and number of storeys, together with the ranges for uniformly reinforced and peripherally reinforced solid wall elements, is illustrated in Figure 5.8. This illustration shows the schematic relationship between the slab span and number of storeys for uniformly reinforced or peripherally reinforced interior precast concrete wall elements used in a typical residential construction.

5.11 UNIFORMLY REINFORCED PRECAST CONCRETE WALL ELEMENTS

The design of precast concrete wall elements for flexural and axial loads should be based on the forces and moments determined from an analysis of the structure. An analysis should take into account the effect of deflections on the moments and forces, and the effects of the duration of the loads. To account for the effects in precast concrete wall elements, the moment magnification method of the ACI 318-71 Code should be used.



Uniformly distributed vertical reinforcement required in 6", 8" and 10" wall elements.

Uniformly distributed vertical reinforcement required in 6" and 8" wall elements

Uniformly distributed vertical reinforcement required in 6" wall element.

Uniformly distributed vertical reinforcement not required for vertical loads for all wall elements 6" or greater.

FIG. 5.8 Schematic Relationship Between Slab Span and Number of Stories [6,7,15,19].

Strength design of precast concrete wall elements for combined flexural and axial loads is based on satisfaction of the applicable conditions of the equilibrium and the compatibility of the strains. The relationship between the concrete compressive stress distribution and the concrete strain in reinforced concrete elements may be assumed to be rectangular, trapezoidal, parabolic, or any other shape which results in the prediction of strength in substantial agreement with the results of the comprehensive tests.

Uniformly reinforced precast concrete wall elements should preferably be braced sideways. Braced precast concrete wall elements are usually assumed to be simply supported along the horizontal connections, unless detailed otherwise. The effective length factor, k, is determined on the basis of the restraint conditions presently existing along the vertical connections.

Most precast concrete wall elements in precast concrete panel constructions are braced by orthogonal structural walls. Precast concrete walls in two directions are used to provide resistance to lateral loads such as wind and earthquakes. Therefore, the effective length factor should be estimated as follows.

(1) For precast concrete panels braced freely and sideways along both the vertical edges, for all values of $\frac{\ell_u}{b}$, [16,17,18,19]

$$k = 1.0$$

(2) For precast concrete panels braced sideways and restrained along one vertical edge, for [16,17,18,19]

$$\frac{\ell_u}{b} < 1 \qquad k = 1.0$$

$$1 \leq \frac{\ell_u}{b} \leq 2 \qquad k = 1.0 - 0.423 \left[\left(\frac{\ell_u}{b}\right) - 1 \right]$$

$$\frac{\ell_u}{b} > 2 \qquad k = \frac{1.0}{\sqrt{\left[1 + \frac{1}{2\left(\frac{\ell_u}{b}\right)^2}\right]}}$$

(3) For precast concrete panels braced against sideways and restrained along both vertical edges, for [16,17,18,19]

$$\frac{\ell_u}{b} > 2 \qquad k = \frac{1.0}{1 + \left(\frac{\ell_u}{b}\right)^2}$$

For

$$\frac{\ell_u}{b} < \frac{1}{2} \qquad k = 1.0$$

$$\frac{1}{2} \leq \frac{\ell_u}{b} \leq 1 \qquad k = 1.5 - \frac{\ell_u}{b}$$

For precast concrete panels not braced sideways, the effective length factor, k, is determined with due consideration to the effect of cracking and reinforcement on relative stiffness, and is typically greater than 1.2.

For precast concrete wall elements with a single layer of reinforcement the value of EI for use in Equation (10.6) of the ACI 318.71 Code ($P_c = \frac{\pi^2 EI}{(k\ell_u)^2}$) may be taken as:

$$EI = \frac{E_c I_g}{B_d}(0.5 - \frac{e}{h}) \geq \frac{0.10\ E_c I_g}{B_d}$$

For precast concrete wall elements with a double layer of reinforcement the value of EI for use in Equation (10.6) of the ACI 318.71 Code may be taken either as:

$$EI = \frac{\frac{E_c I_g}{5} + E_s I_s}{1 + B_d}$$

or conservatively:

$$EI = \frac{\frac{E_c I_g}{2.5}}{1 + B_d}$$

For precast concrete wall elements subject to transverse loading, the maximum moment can occur at a section away from the end of the member. In this case the value of the higher calculated moment occurring anywhere along the member is used for the value of

$$M_2 = \frac{M_c}{\delta}$$

in Equation (10.4) of the ACI 318.71 Code. C_m is taken as 1.0 for this case.

Precast concrete wall elements with openings for doors, windows or mechanical openings should have reinforcement around the openings to alleviate the effects of high local stress concentrations during handling and under in-place service conditions. It is customary to use the recommendations of ACI 318.71 for supplementary reinforcement.

5.12 PERIPHERALLY REINFORCED PRECAST CONCRETE WALL ELEMENTS

A typical peripherally reinforced precast concrete wall element has vertical tensile ties and horizontal reinforcement at the top and bottom of the precast concrete element. The top and bottom horizontal reinforcement in combination with the vertical tensile ties helps to prevent crack propagation from the precast concrete wall element edges while creating an ability to act as a simply supported beam. The horizontal reinforcement may be held in place by means of a ladder or truss-type of transverse tie, which can function to reduce end splitting tendencies in precast concrete wall elements, as may be seen in Fig. 5.9.

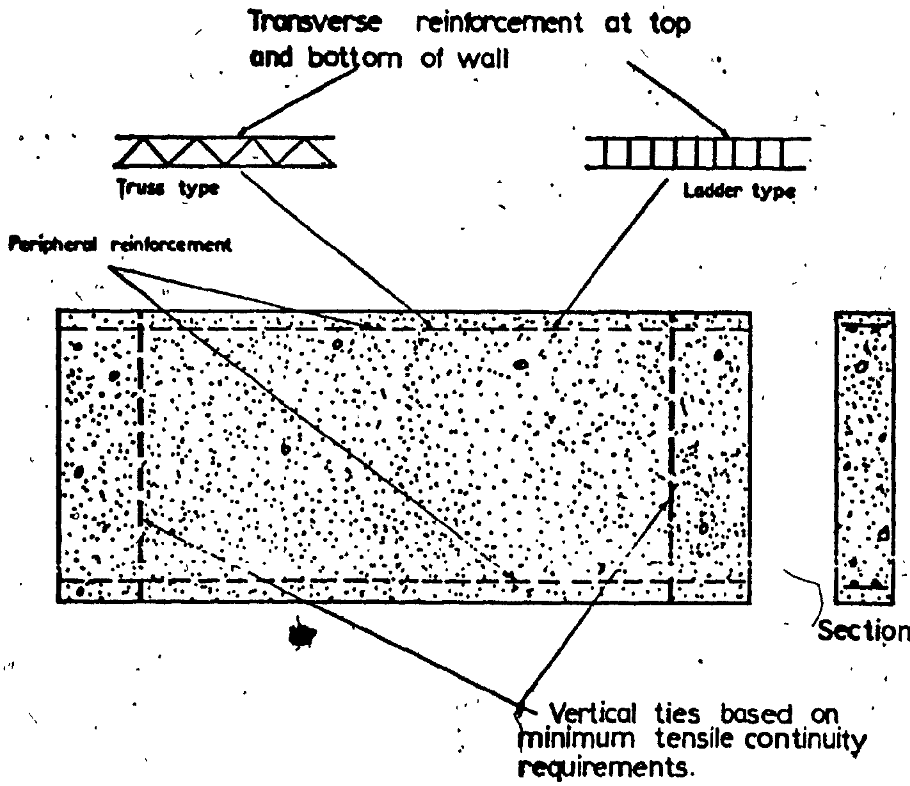


FIG. 5.9 Typical Peripherally Reinforced Precast Concrete Wall Element

The phenomenon of end splitting in precast concrete wall elements is dependent on the details of horizontal connections.

Peripherally reinforced precast concrete wall elements can be used only if:

(1) The precast concrete wall elements are concentrically loaded.

(2) There are no transverse loadings between horizontal supports.

The design of the peripherally reinforced precast concrete wall elements for flexural and axial loads should be based on the forces and moments determined from an analysis of the structure.

The axial load-carrying capacity of a peripherally reinforced precast concrete wall element is estimated using the equation (14.1) [6,7,15,19]

$$Pu = 0.55\phi f'_c A_g \left[1 - \left(\frac{\ell c}{40h}\right)^2\right]$$

taken from the ACI 318.71 Code, which is nearly identical to the recommendation of ACI 322-72 [15]. That is

$$Pu = 0.50\phi f'_c A_g \left[1 - \left(\frac{\ell c}{40h}\right)^2\right]$$

In the transverse direction peripherally reinforced precast concrete wall elements should preferably be braced sideways. Braced precast concrete wall elements are usually assumed as simply supported along the horizontal connection unless detailed to assure a restraint moment. The effective length factor, k, is determined on the basis of the restraint conditions existing along the vertical connections.

Precast concrete wall elements designed under the provisions of this section should be peripherally reinforced and may also require supplementary reinforcement around openings for doors and windows.

5.13 COMPARISON OF UNIFORMLY AND PERIPHERALLY REINFORCED PRECAST CONCRETE WALL ELEMENTS

The two methods give comparable capacities of design and experimental results. It is noted also for the range of precast concrete wall elements typically used in construction and for reasonably concentric loads, as may be seen in Fig.5.10.

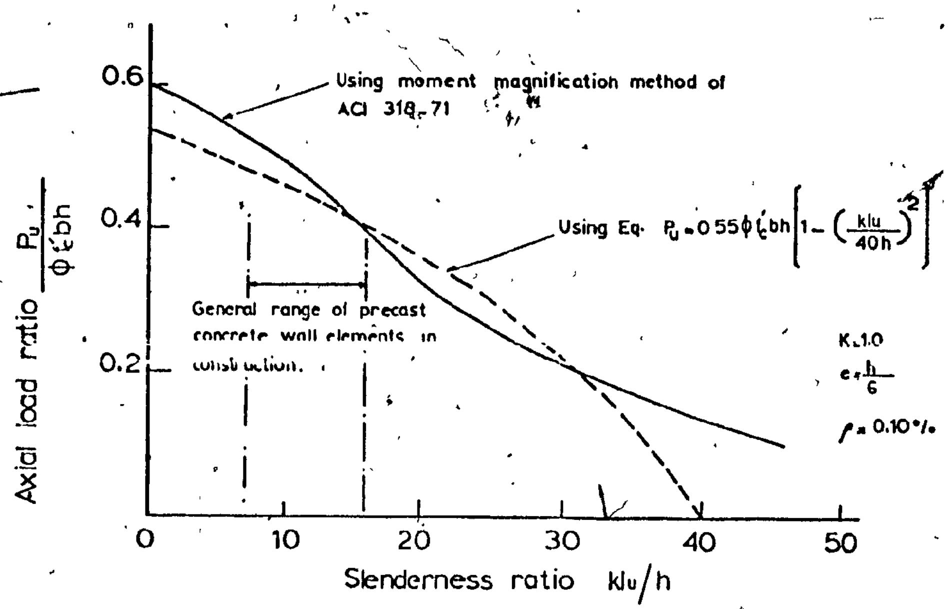


FIG. 5.10 Comparison of Computed Load Capacities of Precast Concrete Wall Elements for Vertical Loads [6,7,19,20,21,22]

CHAPTER 6

TOLERANCES AND FIT

In precast concrete panel structures, as in any precast structure, allowance must be made in design for dimensional deviations. Lack of fit may result in costly on-site modification and adjustment, and impairment of the load-bearing capacity of the structure. Therefore, the designer is expected to stipulate the allowable deviation or tolerance from a specified dimension. A reasonable system of tolerances is an economic and technical necessity for the success of any precast system: however, despite their vital importance tolerances have frequently been neglected.

Tolerances are determined by the requirements of the entire construction process considering technology and economics of currently used methods of precasting, setting out and assembly. The most influential factor affecting tolerances appears to be the connection type. More accuracy is required for bolted connections than for grouted connections. Tolerances should guarantee correct assembly and efficient functioning of individual precast concrete panels. Each panel is positioned within the basic space allotted it and should not encroach on the space allotted to another panel. Therefore, it is only necessary to fabricate and erect pre-cast concrete panel structures accurately enough to assure that the deviations do not fall outside known and account all possible loading conditions. The effect of camber and rotation, as well as erection stresses should be considered. Camber may result in displacement of the assumed locations of the reactions, and restrained rotation will cause moments in the connections. Good engineering decrees that the concrete members fail before the connections, normally achieved by providing a safety factor in the connections 10 percent higher than in the adjacent members.

(b) It must be compatible with the architecture of the structure, preferably not visible in the finished structure. If it must be exposed to view, it should be neat and unobtrusive, non-rusting and non-staining, and watertight.

(c) It must accommodate both manufacturing tolerances and erection tolerances. Both of these tolerances must be considered when determining sizes of holes, sleeves, dowels, corbels, and bearings, as well as erection clearances.

(d) It should be designed so that temporary bracing or connections can be made to hold the precast unit in place so the crane can be released as soon as possible. Tying up an expensive crane and crew for an extended time while the connection is welded, bolted, or otherwise completed is a needless expense.

(e) It should be the most economical connection possible that fulfills the requirements of a,b,c and d, considering all factors of precasting, handling, and erecting. This implies the use of standard manufacturing items readily available in the market rather than special items.

7.2 TOLERANCES

The tolerances that are required for precast concrete connections are a function of the size and type of member being connected. Tolerances should not be confused with clearances.

The "Manual for Quality Control for Plants and Production of Precast, Prestressed Concrete Products", MNL 116-70, gives recommended tolerances for structural precast members. The "Manual for Quality Control for Plants and Production of Architectural Precast Concrete Products", MNL 117-68T, gives recommended tolerances for architectural precast concrete members. Both manuals are published by the Prestressed Concrete Institute. The tolerances that apply to connections are given in the following Table 7.1. An important consideration is the compatibility of precast

TABLE 7.1

TOLERANCES FOR PRECAST CONCRETE CONNECTIONS [23]

Item	Recommended Tolerances (in)
Field placed anchor bolts (transit or template)	± 1/4
Elevation of field cast footings and piers	± 5/8
STRUCTURAL PRECAST CONCRETE	
Position of plates	± 1
Location of inserts	± 1/2
Location of bearing plates	± 1/2
Location of blockouts	± 1/2
Length	± 3/4
Overall depth	± 1/4
Width of stem	± 1/8
Overall width	± 1/4
Horizontal deviation of ends from square	± 1/4
Vertical deviation of ends from square	± 1/8 [per (ft) of height]
Bearing deviation from plane	± 1/8
Position of post-tensioning ducts in precast members	± 1/4

(continued)

Item	Recommended Tolerances (in)
ARCHITECTURAL PRECAST CONCRETE	
Length or width	± 1/16 per 10 ft but not less than ± 1/8
Thickness	± 1/4, - 1/8
Location of blockouts	± 1/2
Location of anchors and inserts	± 3/8
Warpage or squareness	± 1/8 in 6 ft
Joint widths	
- specified	3/8 in to 5/8 in
- min. and max. dimensions	1/4 and 3/4 in

tolerances with tolerances required for other construction materials.

7.3 LOAD FACTORS

In selecting appropriate load factors (factors of safety) for connections, it is recommended that they exceed those required for the individual members being connected. This recommendation is made because connections generally are subject to high stress concentrations whereas significant warning deformations and rotations of connected members occur under ultimate load conditions. Moreover, slight variations of the final connection as built from the designed connection may cause possible changes in the magnitude, direction and position of loads on the connection. It is not practical to determine fully all possible effects of minor variations other than by increasing the load factors.

In view of the importance of connections, and the current state of the art, the committee feels it prudent to provide an additional load factor of 4/3 for the ultimate design of connections. It is recognized that some connections may not require an additional 4/3 factor while others may require an even greater additional factor. Regardless, the selection of the ultimate design load factor is properly that of the engineer.

The load factors suggested apply only to permanent or final design loads.

Load factors of 1.4D + 1.7L are given in Section 9.31, ACI Building Code (ACI 318-71). When the volume change effects are considered, Section 9.3.7, ACI Building Code, they are to be included with dead load in 0.75 (1.4D + 1.7L). However, when considering volume change effects in brackets and corbels, the resulting tensile force should be included with live load with a load factor of 1.7 and no overall reduction Section 11.14.2, ACI Building Code. In the examples and other discussions in this manual, an approximate factor of 1.6 (D + L) is taken to simplify the explanations; it is not recommended as a substitute for the ACI Building Code load factors.

7.4 SHEAR-FRICTION

The shear-friction concept provides a lower bound ultimate strehgth approach that can be used to evaluate many different connection types.

A basic assumption in applying the shear-friction concept is that the concrete within the connection area will crack in the most undesirable manner. Ductility is achieved by placing reinforcement across the ultimate failure plane where the force $A_s f_y$ developed by the reinforcement is normal to the plane. This normal force in combination with a friction analogy results in shear resistance at the crack interface. The reinforcement for ultimate shear across any potential crack plane can be calculated by [23]

$$A_{vf} = \frac{Vu}{\phi(f_{yv})(\mu)}$$

where $\phi = 0.85$

The following Table 7.2 gives the recommended values for the coefficient μ, which is analogous to the coefficient of friction. Values of up to 1.7 may be used for this condition if the circumstances, in the judgment of the engineer, warrant a higher value.

If the unit shearing stresses Vu exceed the maximum values given but in no case shall Vu exceed 0.25 f'_c nor 1,200 psi, a reduced μ may be used as determined from the equation

$$\mu' = \mu[\frac{300\mu}{Vu} + 0.5]$$

TABLE 7.2

RECOMMENDED VALUES FOR THE COEFFICIENT (μ) [23]

Crack Interface Condition	Recommended μ	Max.Vu psi
Concrete to concrete cast monolithically	1.4	840
Concrete to hardened concrete, 1/4 in roughness	1.0	600
Concrete to steel with welded studs	1.0	600
Concrete to concrete smooth interface	0.7	420

7.5 DESIGN

7.5.1 Transfer of Shear

The transfer of shear may be accomplished using reinforcing steel extended as dowels coupled with cast-in-place concrete placed between roughened concrete interfaces, mechanical devices such as embedded plates or shapes, brackets, prestressing forces applied across the connecting surfaces, or any other ways which meet all accepted unit stress requirements for the materials involved and meet the ultimate strength requirements. The entire shear should be considered as transferred through one type of device mentioned above, even though a combination of devices may be available at the joints or supports being considered. The device should be designed to resist the maximum shear in the section at the connecting surfaces.

7.5.2 Reinforcing Dowels

The extension of reinforcing bars in a flexural member or the placement of dowels anchored in each connecting member either by mechanical anchorage devices or with minimum embedment required to develop the full yield strength of the bar through bond may be used to transfer shear, as may be seen in Fig. 7.1.

The allowable shear, based on extended bars or dowels

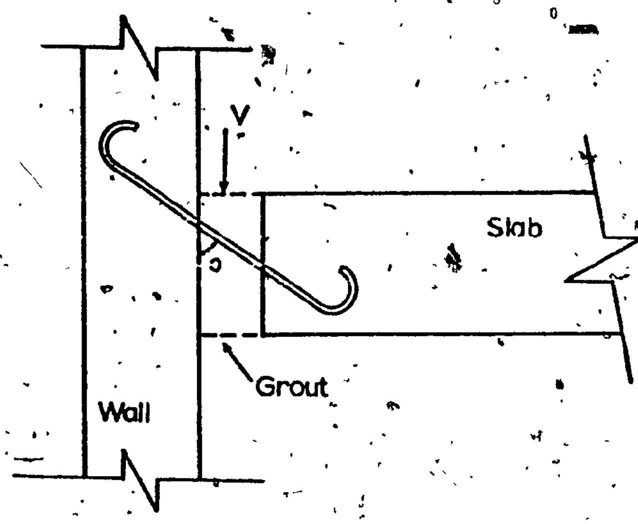


FIG. 7.1 Reinforcing Dowel

should not exceed. [34]

$$V = \sqrt{(A_s f_s \cos\theta)^2 + (1.5D^2 f_c' \sin\theta)^2}$$

where

- V total vertical shear in connections
- A_s total cross-sectional area of bars or dowels
- f_s allowable stress in bars or dowels
- D sum of the diameter of bars or dowels
- f_c' concrete design strength
- θ included angle between direction of shear force and extended bar or dowel

The second part of the above equation should not be considered if the concrete cover on the dowel is less than 3 in. Furthermore, cast-in-place concrete or grout or other positive means should be employed to fill gaps between precast members and the element to which they are connected with dowels. The use of stirrups or ties about the dowels is recommended.

7.5.3 Brackets

A bracket may be a corbel, that is, a protrusion cast onto the side of a wall to serve as a beam seat, the top of a column or a ledge on a column.

Positive means should be taken to prevent the reaction from bearing on the outermost edge of the bracket.

However, the vertical reaction should be assumed to act through the outer edge of the bearing pad as shown in Figure 7.2.

From the standpoint of the structural behaviour of the precast elements, a bracket serves as a shear transfer device, but the bracket itself, must be designed for flexure, shear, bearing, and the splitting forces accompanying the bearing.

The flexural design of a bracket with a/do ratio greater than 1.0 should follow the procedures of flexural design for ordinary reinforced concrete beams (ACI 318). The ultimate load of a corbel with a ratio $a/do < 1$ should not exceed [34]

$$V = \phi[6.5bdo\sqrt{f_c'}(1-0.5^{do/a})(1000P)^{1/3}]$$

where

V = total vertical load at ultimate

b = width of bracket

do = distance from extreme compression fiber to centroid of tension reinforcement at the column face

f_c' = compressive strength of concrete

a = lever arm of vertical reaction

p = As/bdo reinforcement ratio at the column face

ϕ = capacity reduction factor (recommended value ϕ = 0.85 for shear transfer device)

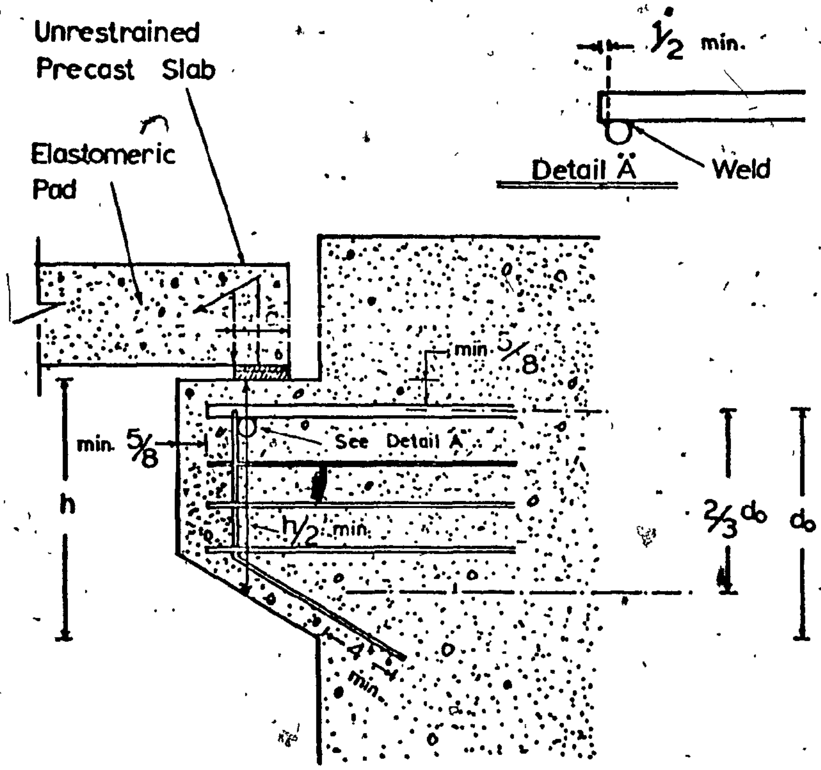


FIG. 7.2 Corbel Subject to Vertical Load Only

The reinforcement index

$$q = \rho f_y / f'_c$$

for all corbels should have a maximum of 0.15 and a minimum of 0.04. The depth of the bracket outer face should be at least from 0.4 to 0.5 the total bracket depth.

There is frequently insufficient room to develop the tensile reinforcement through bond; therefore adequate anchorage provisions should be made.

Additional reinforcement to resist horizontal forces which may develop from volume changes due to shrinkage, creep and temperature change should be considered in the design. The horizontal friction force may be estimated or taken at 0.5 times the vertical force.

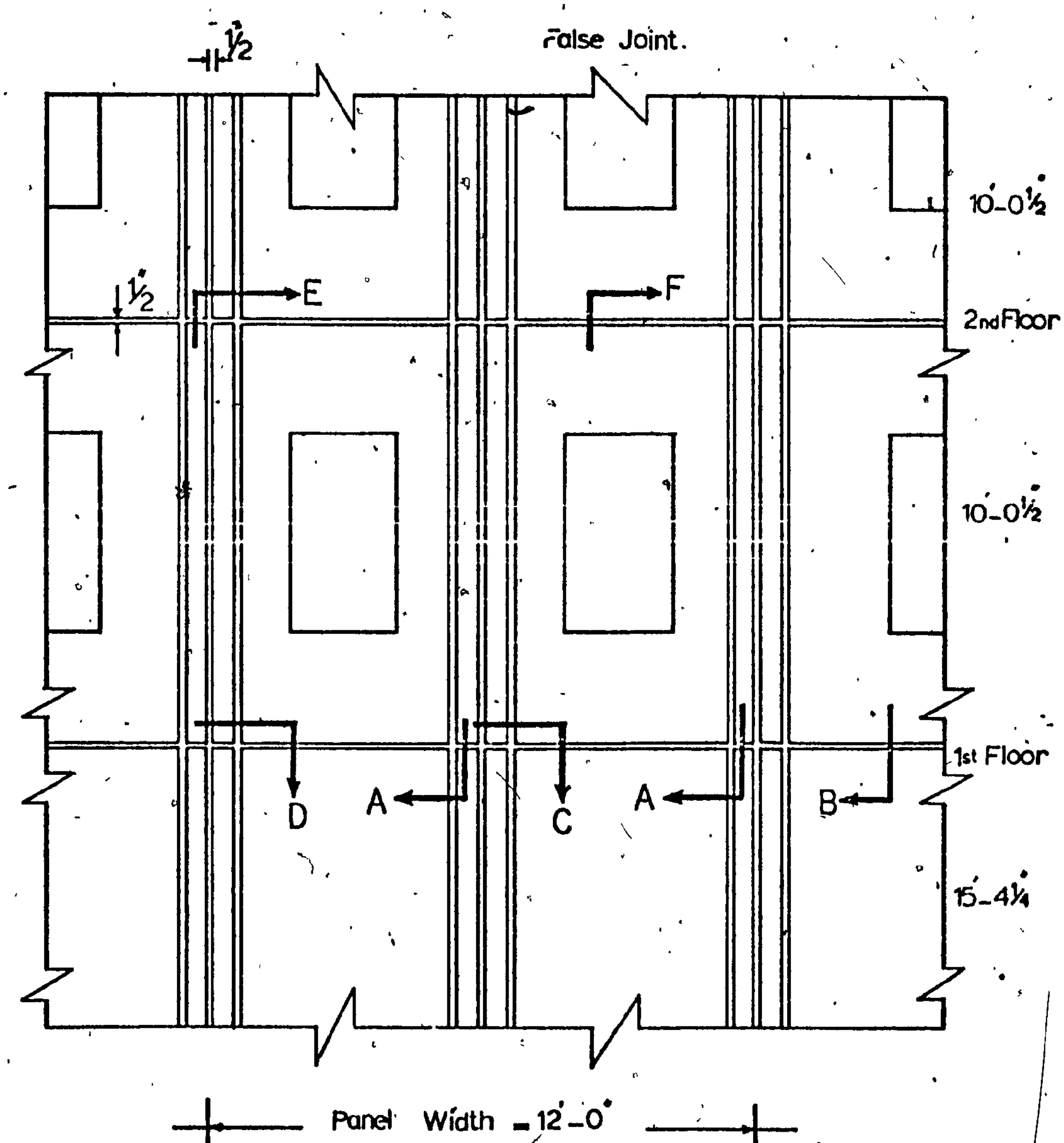
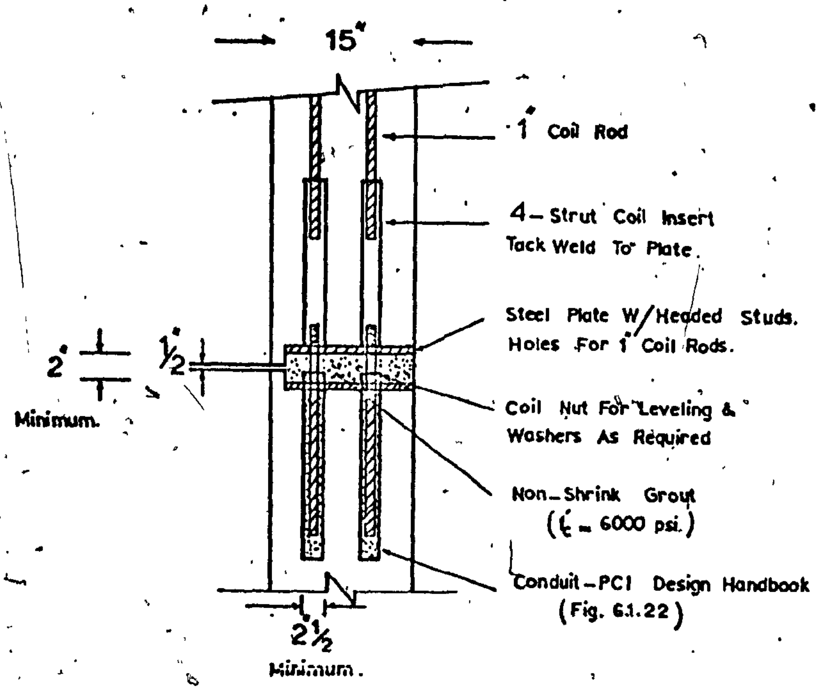
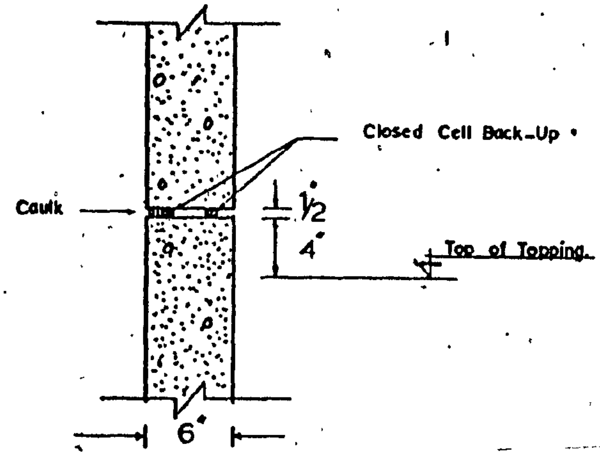


FIG. 7.3 Typical Connection Details. Typical Panel Connections

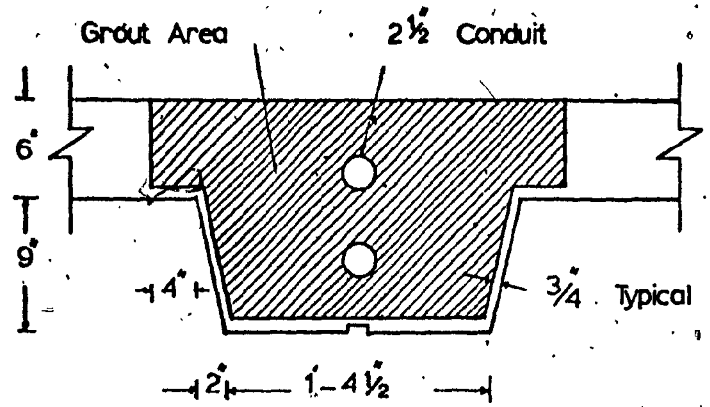


Section A

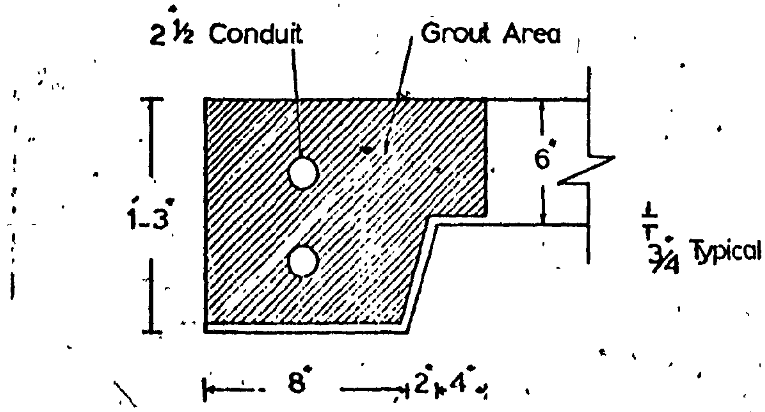


Section B

FIG. 7.4 Typical Mullion Connection

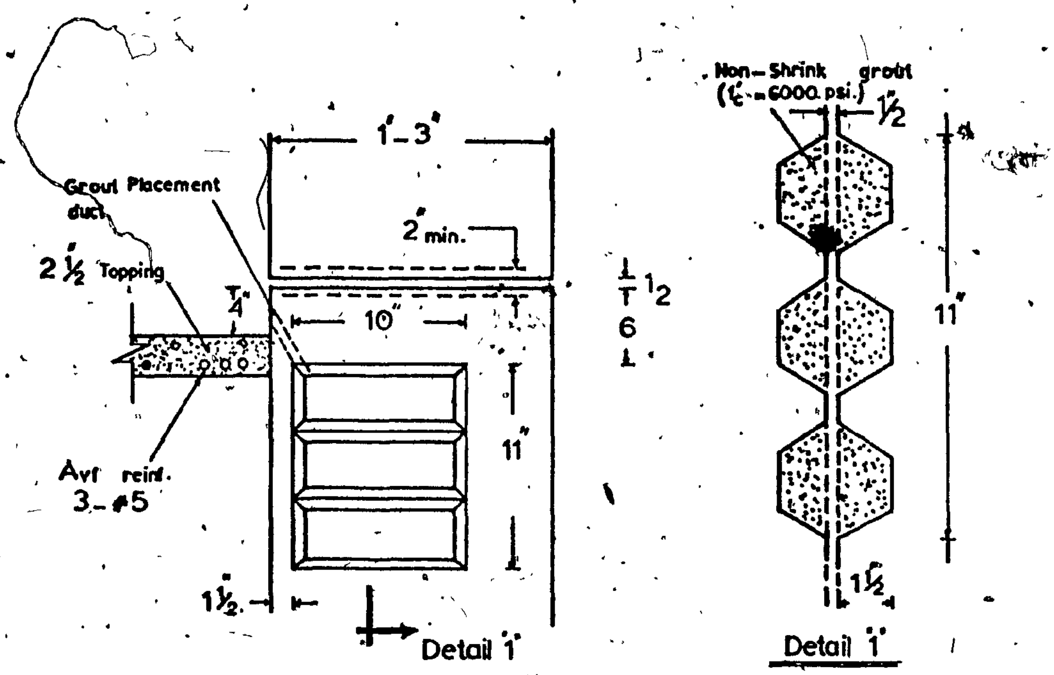


Section C

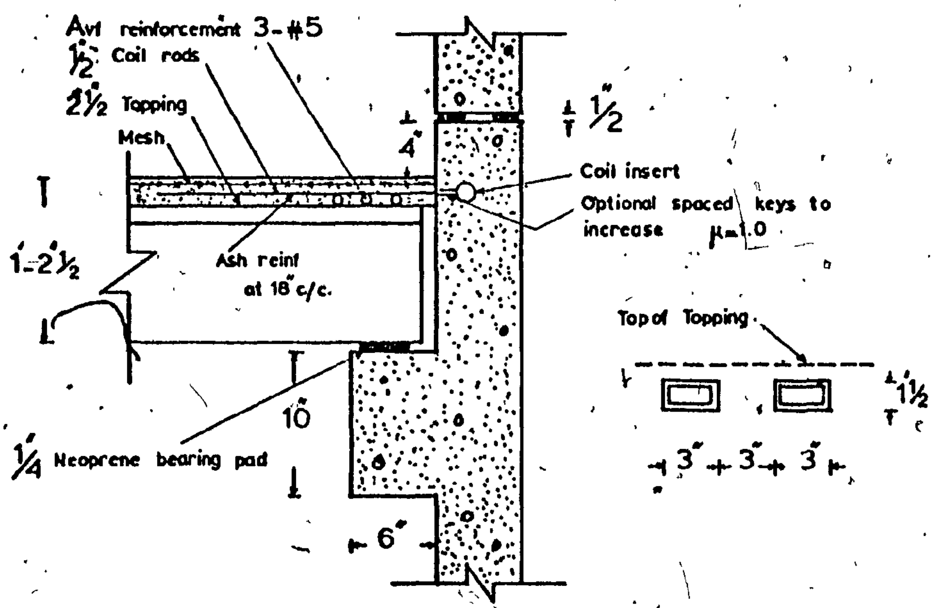


Section D

FIG. 7.5 Center and Side Mullion Connection



(A)



(B)

FIG. 7.6 (a) Section E - Typical Vertical Joint Shear Connection
(b) Typical Panel Bearing Connection

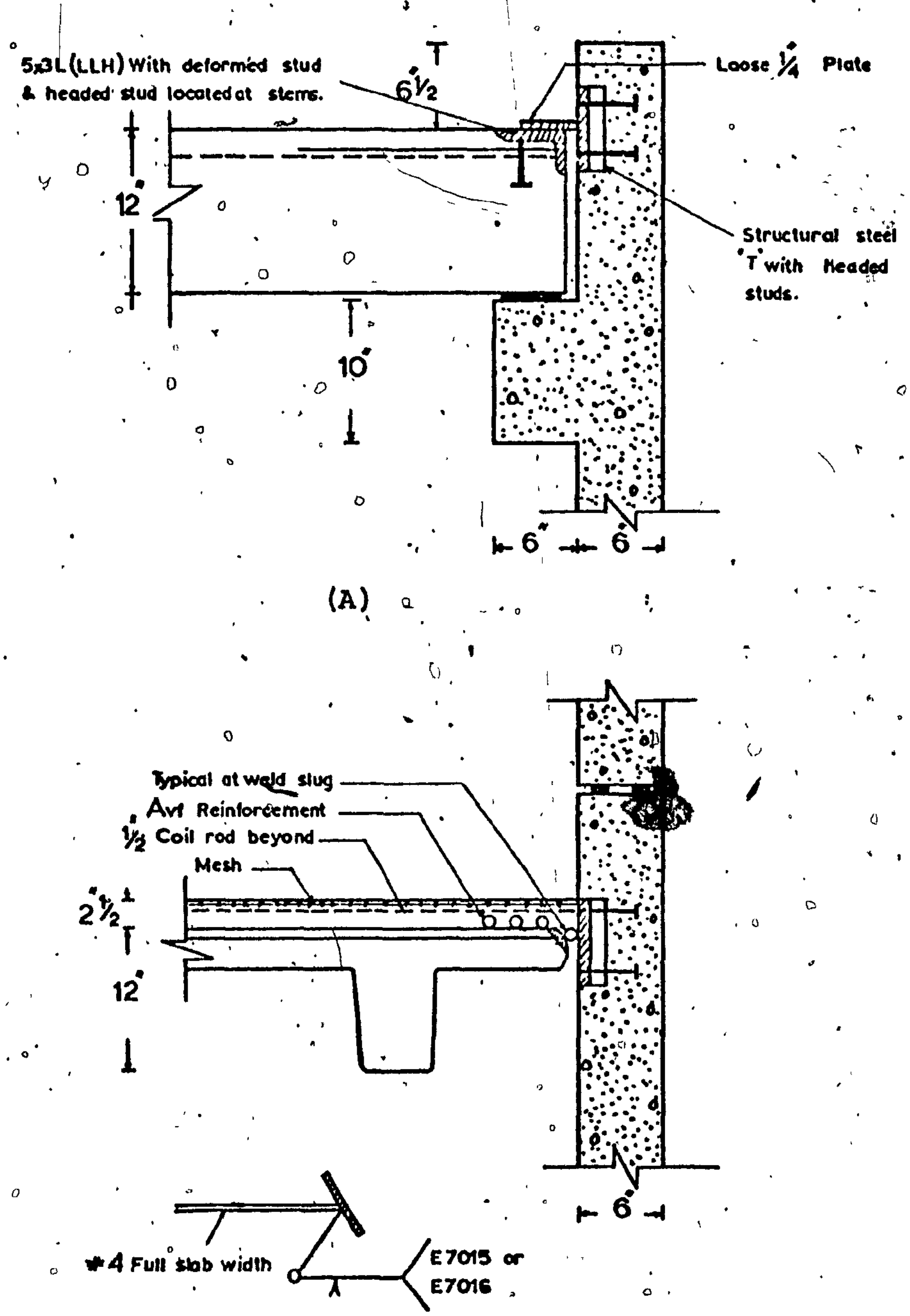


FIG. 7.7 Erection Connection (a) at Panel Bearing Connection
(b) at Non-Bearing Panel

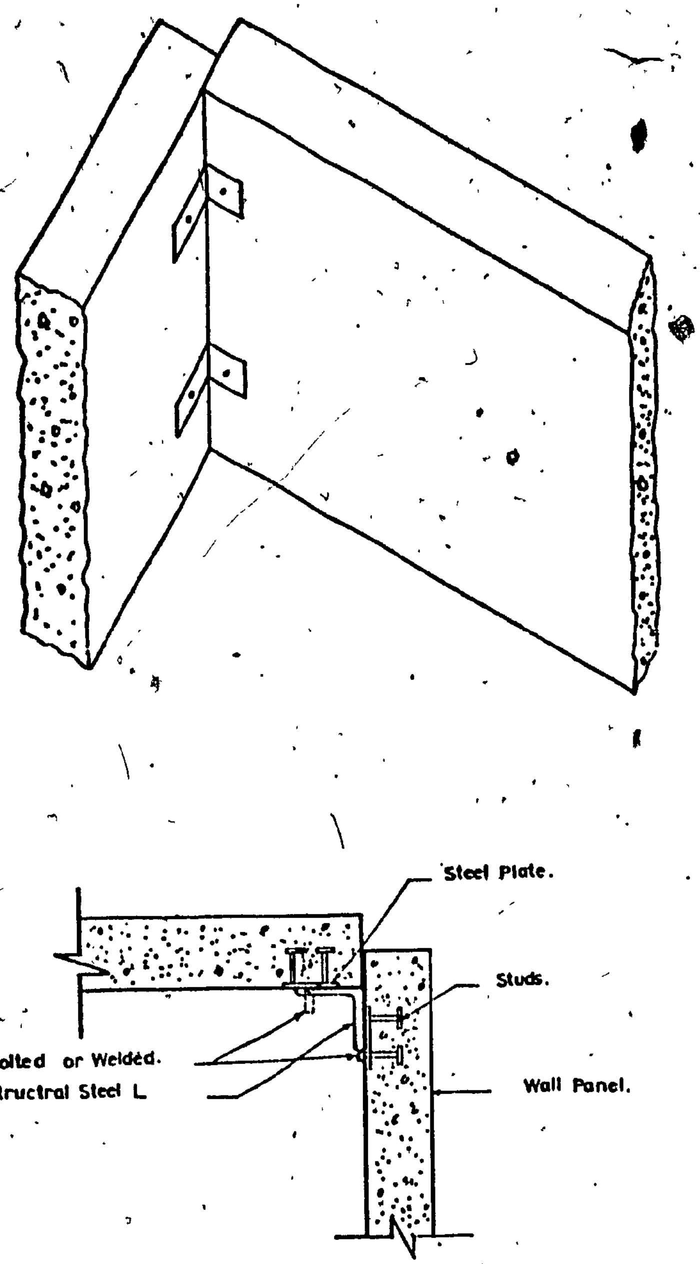


FIG. 7.8 Typical "DRY" Vertical Wall-to-Wall Connection

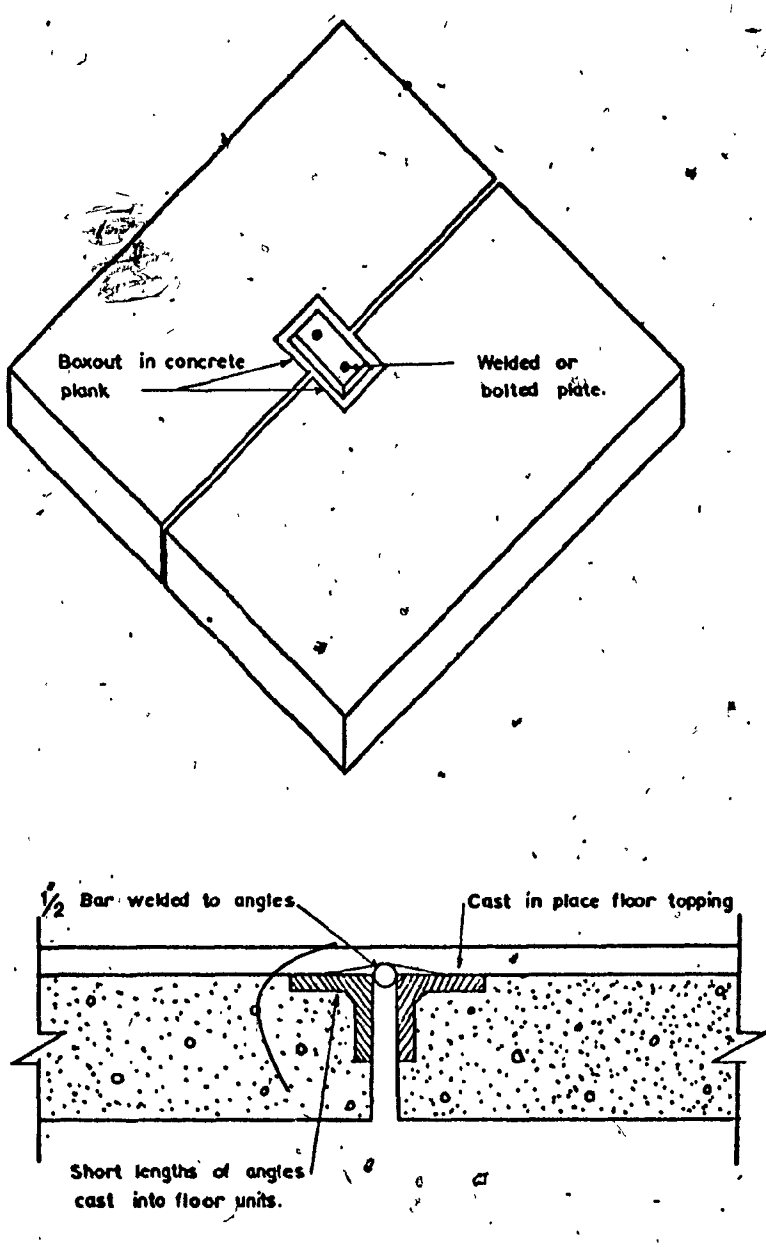
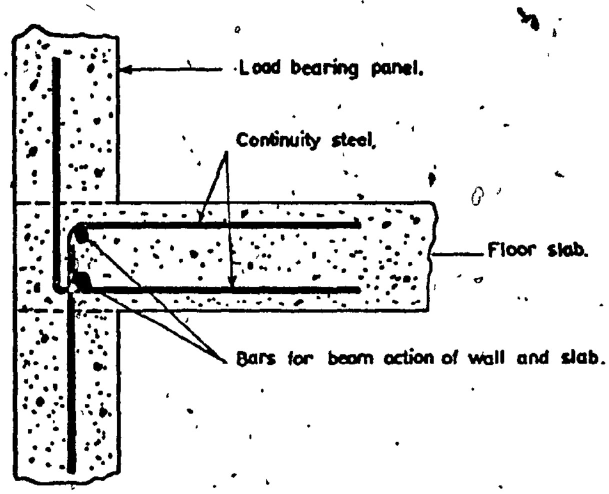


FIG. 7.9 Grouted Connections Between Floor Elements

(A)



(B)

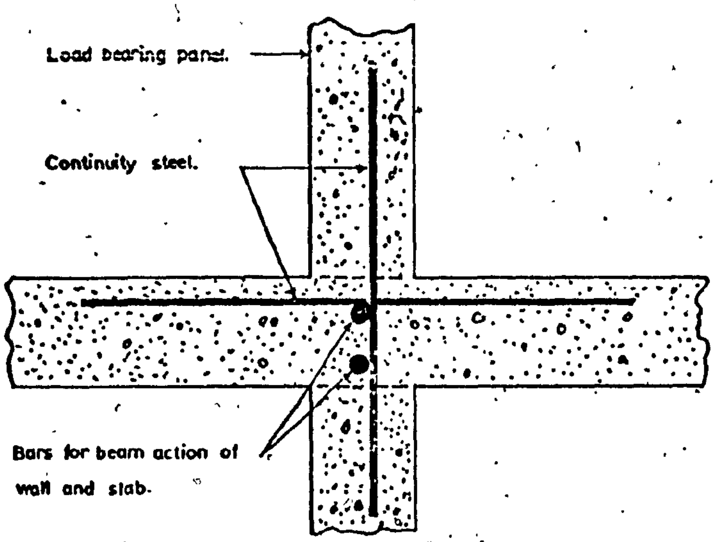


FIG. 7.10 Ductile Connections Showing Continuity Steel. Longitudinal Bars Providing Cantilever and Beam Action in Walls

(a) End Wall
(b) Internal Wall

CHAPTER 8

THERMAL PROPERTIES OF PRECAST CONCRETE STRUCTURES

Minimum insulation values for buildings are necessary for comfort, to avoid condensation and to limit operating costs.

Regulatory agencies specify maximum permissible heat loss values for housing. The thermal performance of most other buildings is determined by the architect or engineer. The trend is to use more insulation but modern criteria are flexible. This permits the use of precast concrete wall sections with somewhat lower insulation values than would be required if the heat loss through all surfaces had to meet a single criterion.

8.1 BASIC HEAT TRANSFER DATA

The thermal conductivities and resistances of all materials are usually tested and reported in an oven dry condition. The presence of moisture increases the thermal transmittance and reduces the resistance. For concrete, of the unit weights used in precast members, 100 to 150 pcf, the moisture present in the normally dry condition of use produces only a small change, about 25 percent. It should also be noted that structural concrete seldom contributes much to the total resistance of an assembly or system. The concrete in the precast member accounts for less than 3 percent of the total resistance. If the resistance were reduced 25 percent by moisture, the total resistance would be reduced by only 0.7 percent and the coefficient of heat transmission U-value would not change significantly. The heat storage capacity of concrete delays the passage of heat through precast roofs and walls. The effect is substantial when precast concrete is combined with insulation. In summer, when a roof is subjected to diurnal heat cycles, heat entry is delayed, spread over a longer period, and reduced in total daily flow. The required air-conditioner capacity and the operating cost are both lower than for a building with lighter walls or roof but with the same thermal transmittance. The following Tables 8.1, 8.2 and 8.3, give U-values of commonly used precast concrete assemblies for both summer and winter conditions, and are shown on pages 114, 116 and 118. (PCI Journal May-June 1971). [36]

8.2 METHODS OF CALCULATION

Thermal transmittance	(U)
Thermal resistance	(R)
Thermal conductance	(C)
Surface conductance	(f)
Thermal conductance of an air space	(a)

Conductances (C, f and a) of the components of a building assembly cannot be added to obtain the U-value, but

TABLE 8.1 U-VALUES OF ROOF ASSEMBLIES WINTER CONDITIONS HEAT FLOW UPWARD

[R(b.u.r) = 0.33, Rf_o = 0.17, Rf_i = 0.61, Ra = 0.85] [36]

PRECAST CONCRETE MEMBER	Thickness (in)	Concrete Resistance R		Roof Insulation							Acoustical Ceilings			
											Applied Direct		Ceiling Suspended	
				None	½"	1"	1½"	2"	2½"	3"	1/2"	3/4"	1/2"	3/4"
			C=	0	0.72	0.36	0.24	0.19	0.15	0.12	0.84	0.53	0.84	0.53
			R=	0	1.39	2.78	4.17	5.26	6.67	8.33	1.19	1.78	1.19	1.78
Tees and solid slabs-normal weight concrete(145 pcf)	2	0.18		0.55	0.31	0.22	0.17	0.14	0.12	0.10	0.33	0.28	0.26	0.22
	3	0.27		0.52	0.30	0.21	0.16	0.14	0.12	0.10	0.32	0.27	0.25	0.22
	4	0.36		0.50	0.29	0.21	0.16	0.14	0.12	0.10	0.31	0.26	0.25	0.22
	5	0.46		0.48	0.29	0.21	0.16	0.14	0.11	0.10	0.30	0.26	0.24	0.21
	6	0.55		0.46	0.28	0.20	0.16	0.13	0.11	0.10	0.30	0.25	0.24	0.21
	8	0.73		0.42	0.27	0.19	0.15	0.13	0.11	0.09	0.28	0.24	0.23	0.20
Tees and solid Slabs-structural lightweight concrete(110 pcf)	2	0.47		0.47	0.29	0.20	0.16	0.14	0.11	0.09	0.30	0.26	0.24	0.21
	3	0.70		0.43	0.27	0.20	0.15	0.13	0.11	0.09	0.28	0.24	0.23	0.20
	4	0.93		0.39	0.25	0.19	0.15	0.13	0.11	0.09	0.27	0.23	0.22	0.19
	5	1.16		0.36	0.24	0.18	0.14	0.12	0.11	0.09	0.25	0.22	0.21	0.18
	6	1.40		0.33	0.93	0.17	0.14	0.12	0.10	0.09	0.94	0.21	0.90	0.18
	8	1.86		0.29	0.20	0.16	0.13	0.11	0.10	0.09	0.21	0.19	0.18	0.16

(continued)

PRECAST CONCRETE MEMBER	Thickness (in)	Concrete Resistance R		Roof Insulation							Acoustical Ceilings Applied Direct		Acoustical Ceilings Ceiling Suspended	
				None	½"	1"	1½"	2"	2½"	3"	1/2"	3/4"	1/2"	3/4"
			C =	0	0.72	0.36	0.24	0.19	0.15	0.12	0.84	0.53	0.84	0.53
			R =	0	1.39	2.78	4.17	5.26	6.67	8.33	1.19	1.78	1.19	1.78
Hollow core slabs.	8	1.69		0.31	0.22	0.17	0.14	0.12	0.10	0.09	0.23	0.20	0.19	0.17
Normal weight	12	1.91		0.29	0.21	0.16	0.13	0.12	0.10	0.09	0.22	0.19	0.18	0.16
concrete (145 pcf)	6	1.21		0.37	0.24	0.18	0.15	0.13	0.11	0.09	0.26	0.22	0.20	0.18
Extruded rect-	8	1.44		0.34	0.23	0.17	0.14	0.12	0.10	0.09	0.24	[illegible]	0.20	0.18
angular voids.	10	1.73		0.31	0.22	0.17	0.14	0.12	0.10	0.09	0.23	0.20	0.19	[illegible]
Extruded oval voids	8	0.88		0.42	0.26	0.19	0.15	0.13	0.11	0.09	0.28	0.24	0.22	0.19
Wet cast, circular voids.														
Hollow core	6	3.05		0.21	0.16	0.13	0.11	0.10	0.09	0.08	0.17	0.15	0.15	0.14
slabs	8	3.63		0.19	0.15	0.12	0.11	0.10	0.08	0.07	0.15	0.14	0.14	0.13
Structural lightweight	10	4.30		0.17	0.14	0.11	0.10	0.09	0.08	0.07	0.14	0.13	0.13	0.12
Concrete (110 pcf)	12	5.10		0.15	0.12	0.11	0.09	0.08	0.07	0.07	0.13	0.12	0.11	0.11
Extruded, rectangular voids														

TABLE 8.2 U-VALUES OF ROOF ASSEMBLIES, SUMMER CONDITIONS, HEAT FLOW DOWNWARD

[R(b.u.r) = 0.33, Rf_o = 0.25, Rf_i = 0.92, Ra = 0.99] [36]

PRECAST CONCRETE	Thick-ness (in)	Concrete Resistance R		Roof Insulation							Acoustical Ceilings Applied Direct		Acoustical Ceilings Ceiling Suspended	
				None	½"	1"	1½"	2"	2½"	3"	1/2"	3/4"	1/2"	3/4"
			C =	0	0.72	0.36	0.24	0.19	0.15	0.12	0.84	0.53	0.84	0.53
			R =	0	1.39	2.78	4.17	5.26	6.67	8.33	1.19	1.78	1.19	1.78
Tees and solid slabs-normal weight concrete (145 pcf)	2	0.18		0.55	0.31	0.22	0.17	0.14	0.12	0.10	0.33	0.28	0.26	0.22
	3	0.27		0.52	0.30	0.21	0.16	0.14	0.12	0.10	0.32	0.27	0.25	0.22
	4	0.36		0.50	0.29	0.21	0.16	0.14	0.12	0.10	0.31	0.26	0.25	0.22
	5	0.46		0.48	0.29	0.21	0.16	0.14	0.11	0.10	0.30	0.26	0.24	0.21
	6	0.55		0.46	0.28	0.20	0.16	0.13	0.11	0.10	0.30	0.25	0.24	0.21
	8	0.73		0.42	0.27	0.19	0.15	0.13	0.11	0.09	0.28	0.24	0.23	0.20
Tees and solid Slabs-structural light-weight concrete (110 pcf)	2	0.47		0.47	0.29	0.20	0.16	0.14	0.11	0.09	0.30	0.26	0.24	0.21
	3	0.70		0.43	0.27	0.20	0.15	0.13	0.11	0.09	0.28	0.24	0.23	0.20
	4	0.93		0.39	0.25	0.19	0.15	0.13	0.11	0.09	0.27	0.23	0.22	0.19
	5	1.16		0.36	0.24	0.18	0.14	0.12	0.11	0.09	0.25	0.22	0.21	0.18
	6	1.40		0.33	0.23	0.17	0.14	0.12	0.10	0.09	0.24	0.21	0.20	0.18
	8	1.86		0.29	0.20	0.16	0.13	0.11	0.10	0.09	0.21	0.19	0.18	0.16

(continued)

PRECAST CONCRETE MEMBER	Thickness (in)	Concrete Resistance R		Roof Insulation							Acoustical Ceilings Applied Direct		Acoustical Ceilings Ceiling Suspended	
				None	½"	1"	1½"	2"	2½"	3"	1/2"	3/4"	1/2"	3/4"
			C=	0	0.72	0.36	0.24	0.19	0.15	0.12	0.84	0.53	0.84	0.53
			R=	0	1.39	2.78	4.17	5.26	6.67	8.33	1.19	1.78	1.19	1.78
Hollow core slabs. Normal weight concrete (145 pcf) Extruded rectangular voids. Extruded oval voids Wet cast, circular voids.	8	1.69		0.31	0.22	0.17	0.14	0.12	0.10	0.09	0.23	0.20	0.19	0.17
	12	1.91		0.29	0.21	0.16	0.13	0.12	0.10	0.09	0.22	0.19	0.18	0.16
	6	1.21		0.37	0.24	0.18	0.15	0.13	0.11	0.09	0.26	0.22	0.20	0.18
	8	1.44		0.34	0.23	0.17	0.14	0.12	0.10	0.09	0.24	0.21	0.20	0.18
	10	1.73		0.31	0.22	0.17	0.14	0.12	0.10	0.09	0.23	0.20	0.19	0.17
	8	0.88		0.42	0.26	0.19	0.15	0.13	0.11	0.09	0.28	0.24	0.22	0.19
Hollow core slabs Structural lightweight Concrete (110 pcf) Extruded, rectangular voids	6	3.05		0.21	0.16	0.13	0.11	0.10	0.09	0.08	0.17	0.15	0.15	0.14
	8	3.63		0.19	0.15	0.12	0.11	0.10	0.08	0.07	0.15	0.14	0.14	0.13
	10	4.30		0.17	0.14	0.11	0.10	0.09	0.08	0.07	0.14	0.13	0.13	0.12
	12	5.10		0.15	0.12	0.11	0.09	0.08	0.07	0.07	0.13	0.12	0.11	0.11

TABLE 8.3 U-VALUES OF WALL ASSEMBLIES-TEES OR SOLID SLABS [36]

TYPE OF CONCRETE	Concrete Thickness (in)	Concrete Resistance (R)	Summer Conditions $R_{f_0}=0.25$ $R_{f_1}=0.68$					Winter Conditions $R_{f_0}=0.17$ $R_{f_1}=0.68$				
			Insulation Resistance, R					Insulation Resistance, R				
			None	2.0	4.0	6.0	8.0	None	2.0	4.0	6.0	8.0
Normal Weight (145 pcf)	2	0.18	0.90	0.32	0.20	0.14	0.11	0.97	0.33	0.20	0.14	0.11
	3	0.27	0.84	0.31	0.19	0.14	0.11	0.89	0.32	0.20	0.14	0.11
	4	0.36	0.78	0.30	0.19	0.14	0.11	0.83	0.31	0.19	0.14	0.11
	5	0.46	0.72	0.29	0.19	0.14	0.11	0.77	0.30	0.19	0.14	0.11
	6	0.55	0.68	0.29	0.18	0.13	0.11	0.72	0.29	0.19	0.14	0.11
	8	0.73	0.60	0.27	0.18	0.13	0.10	0.63	0.28	0.18	0.13	0.10
Structural Lightweight (110 pcf)	2	0.47	0.72	0.29	0.19	0.14	0.11	0.76	0.30	0.19	0.14	0.11
	3	0.70	0.61	0.28	0.18	0.13	0.10	0.65	0.28	0.18	0.13	0.10
	4	0.93	0.54	0.26	0.17	0.13	0.10	0.56	0.26	0.17	0.13	0.10
	5	1.16	0.48	0.24	0.16	0.12	0.10	0.50	0.25	0.17	0.12	0.10
	6	1.40	0.43	0.23	0.16	0.12	0.10	0.44	0.24	0.16	0.12	0.10
	8	1.86	0.36	0.21	0.15	0.11	0.09	0.37	0.21	0.15	0.11	0.09

the sum of their reciprocals gives the total resistance, R. The reciprocal of the total R is the thermal transmittance U of the section, as may be seen in Table 8.4.

Assuming

$$U = 1/R = 1/7.94 = 0.13 \text{ Btu/hr/ft}^2\text{/deg F}$$

Heat loss or gain-heat flow = U-value × temperature difference

<u>Example:</u>

Heat loss through roof assembly shown.

Inside temperature = 75 F

Outside temperature = -15F

Heat loss = 0.13 × 90 = 11.7 Btu/hr/ft^2

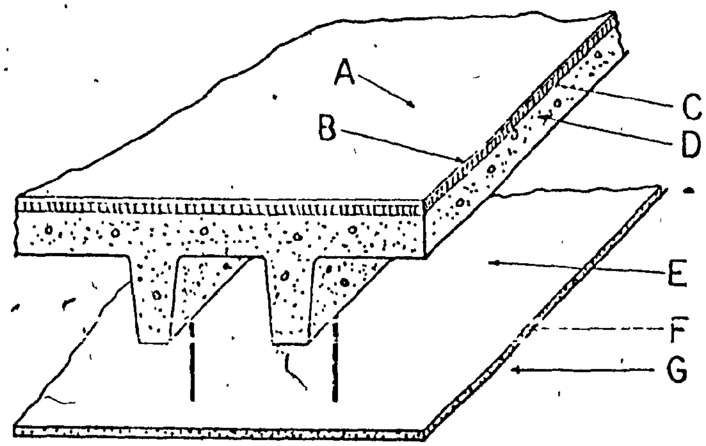
8.2.1 <u>Surface Temperature</u>

The temperature difference between any two points in a building assembly is proportional to the thermal resistance between those points.

<u>Example:</u>

Find the surface temperature at the bottom of the top slab of the tee in the example above.

TABLE 8.4 HEAT FLOW UP, WINTER, FOR THE ASSEMBLY SHOWN[36]



A - Surface coefficient outside	=	0.17
B - Built-up roofing	=	0.33
C - Insulation board 2 in. nominal	=	5.26
D - Concrete tee, 2 in. flange, 145 pcf	=	0.18
E - Air space	=	0.94
F - Gypsum board 1/2 in.	=	0.45
G - Surface coefficient inside	=	0.61
Total R	=	7.94

$$\frac{\text{temp. diff., room to slab}}{\text{temp. diff., room to outside}} = \frac{\text{R, room to slab}}{\text{R, room to outside}}$$

$$\frac{\text{temp. diff., room to slab}}{90\text{F}} = \frac{2.00}{7.94}$$

Temperature difference, room to slab = 22.7F

Surface temperature = 75.0 - 22.7 = 52.3F

8.3 CONDENSATION

Moisture which condenses on the interior of a building is unsightly and can cause damage to the building or its contents. Even more undesirable is the condensation of moisture within a building wall or ceiling assembly where it is not readily noticed until damage has occurred. All air in the buildings contain moisture in the form of water vapor. In many buildings moisture is added to the air by industrial processes, cooking, laundering or humidifiers. If the insulation in a wall, floor or ceiling is not sufficient, the building air contacting the inside surface will be cooled below its dew point temperature and leave its excess water on that surface. If an interior surface covering is very permeable, water vapour will move through the covering to a material or space behind it. The air in any space in an assembly, easily reached by moisture from within a building, will have a dew point close to that of the air in the building. All surfaces of such spaces should be above the dew point to prevent condensation within the assembly.

If building air can circulate through such a space, moisture can be carried into the space at an even greater rate than it is carried by permeance.

Condensation in the walls is frequently due to air circulation.

8.4 VAPOUR BARRIERS

Building materials have a range of very low to very high water vapour permeances. When properly used, low permeance materials keep moisture from entering a wall or roof assembly and materials with higher permeance allow construction moisture and moisture which enters inadvertently or by design to escape. When a material such as plaster, gypsum board or plywood has a permeance which is too high for the intended use, one or two coats of paint is frequently sufficient to lower the permeance to an acceptable level. Or a vapour barrier can be used immediately behind such products. Polyethylene sheet, aluminium foil and roofing materials (not just asphalt saturated felt) are commonly used. Proprietary vapour barriers, usually combinations of foil and polyethylene or asphalt, are frequently used in freezer and cold storage construction.

Concrete is a relatively good vapour barrier. Permeance is a function of the water cement ratio of the concrete. A low water cement ratio, such as is used in most precast concrete products, results in concrete with low permeance, as may be seen in Table 8.5.

The moisture flow through a surface is calculated using the formula: [36]

$$W = MA\theta \times \Delta p$$

where

W = weight of vapour transmitted in grains

M = permeance in grains per hour per sq. ft. per inch of mercury vapour pressure difference

A = area

θ = time in hours

Δp = vapour pressure difference in inches of mercury

TABLE 8.5 TYPICAL PERMEANCE VALUES [36]
(Dry Cup Method)

Material	Permeance
Concrete	0.5 to 3.0
Wood	0.5 to 5.0
Form Plastic	0.4 to 6.0
Plaster of Gypsum Lath	20.0
Gypsum Wallboard	50.0
Polyethylene, 2 mil	0.16
Aluminum Foil, 0.35 mil	0.05
Built-up Roofing	0.00
Water Base	4 to 12

CHAPTER 9

ACOUSTICAL PROPERTIES OF PRECAST CONCRETE AND SOUND CONTROL

A structure is acoustically correct when desired sounds are clearly heard by the intended listeners and unwanted noises are either absorbed or excluded. Because of its weight, concrete is a very good sound barrier. Precast concrete floors, roofs and walls without additional treatments provide adequate sound isolation for many buildings. Ceilings resiliently attached can further improve the acoustical performance. A carpet or a resilient floor system can be added to provide an excellent impact noise rating.

Except for specially fabricated units with porous concrete surfaces, precast concrete has a very low sound absorption. Properly used on a stage, near the front of a lecture room or in a band-shell, it reinforces and directs the sound to the audience where sound absorption is required. The smooth, hard surface of concrete is a good base for the adhesion of acoustical tiles or plasters. When the concrete is in the form of precast tees, the acoustical treatment can be in strips between the stems or it can be applied to the sides of the stems.

The weight and stiffness of precast concrete are utilized in the isolation of large unbalanced mechanical equipment, that might produce unwanted vibration, mounted in buildings. These properties in a floor are essential to the performance of the resilient mounts on which the equipment is supported.

9.1 CRITERIA

9.1.1 Airborne and impact noise ratings

The background sound level in a room is a factor in determining the level at which intruding sounds become objectionable. If the ambient sound level is low and the sounds outside the room are high then the sound transmission loss of enclosing building sections must be high. For example, the sound transmission loss between a bedroom and a bathroom must be greater than between two noisy spaces such as a bathroom and a corridor. Similarly, better impact noise performance is needed in floor ceiling assemblies above quiet rooms such as bedrooms and living-rooms. Apartment designs must avoid the placement of noisy areas adjacent to bedrooms and living-rooms.

The single figure ratings-sound transmission class (STC), impact insulation class (IIC) and impact noise rating (INR) were devised to fill the need for simplified ratings. The following Table 9.1 gives the ratings for precast concrete floor ceiling assemblies. The IIC is a newer rating system than the INR. The criteria were selected to avoid

TABLE 9.9 AIRBORNE AND IMPACT NOISE RATINGS BY PCI JOURNAL MARCH-APRIL 1971 [37]

Assembly Number	Description	STC	INR	IIC
1	14-in precast tees with 2-in concrete topping 75 psf	54	-27	24
2	Assembly 1 with: Std 44 oz. carpet and 40 oz hair felt pad, 75 psf	54	+21	72
3	Assembly 1 with: Resiliently suspended acoustical ceiling with $1\frac{1}{2}$-in mineral fiber blanket above 76 psf	59	+0	51
4	Assembly 3 with: Std. carpet and pad 76 psf	59	+31	82
5	6-in or 8-in hollow core precast units with $\frac{1}{2}$-in wood block flooring adhered directly 45 to 60 psf	49	-2	49
6	5-in flat slabs, 33 to 60 psf	46 to 52	-28 to 27	23 to 24
7	10-in flat slabs, 66 to 119 psf	53 to 58	-23 to-20	28 to 31
8	5-in flat slabs with std. carpet and pad 48 psf	50	+17	68
9	10-in flat slabs with std. carpet and pad 95 psf	56	+23	74

negative ratings. The IIC of a floor ceiling assembly is usually of the same magnitude as the STC.

Both INR and IIC are currently in use. At best the weightings are arbitrary. They are based on the sensitivity of the human ear and the subjective response of people to noises of different frequencies. As minimum standards for housing and for similar uses, single figure ratings are valuable. However, when the noise to be suppressed has concentrations of energy at specific frequencies, or any other unusual characteristics, then performance ratings at individual frequencies should be used instead of the STC, INR or IIC.

Impact and airborne sound transmission specifications for multifamily housing have been established in both the United States and Canada. The needs for commercial, institutional and industrial buildings vary greatly. Specifications have to be based on the use of the space and the various acoustical environments in the building and near it.

9.2 SOUND ABSORPTION

The location and amount of sound absorption in a small room usually is not critical. The amount of absorption needed increases with the size of the space and varies with the use of the space. For this reason, there cannot be a single sound absorption requirement for all ceilings or all walls. The architect or acoustical engineer should select sound absorption criteria for specific room designs.

Where a wall is used to reflect and reinforce sound, acoustical absorption is kept at a minimum. In other areas of ceiling, walls and floor, high sound absorption reduces reverberation time, quiets the room and stops undesired reflections of sound.

Sound absorption of a surface can be specified at individual frequencies or as a noise reduction coefficient (NRC). The NRC is an average of the absorptions at 250, 500, 1000 and 2000 Hz expressed to the nearest at 0.05. When there are concentrations of sound energy at specific frequencies, the sound absorptions at those frequencies, rather than the NRC, should be used to select materials for sound control.

Designers can use precast concrete effectively when minimum sound absorption is required. Where additional sound absorption is desired, precast concrete can be coated with spray applied acoustical materials; acoustical tile applied with adhesive, or an acoustical ceiling hung below the structural floor or roof. Most of the sprayed on, fire-retardant materials used to increase the fire ratings of precast concrete or other floor ceiling systems are also good sound absorbers. In addition, the acoustical ceiling would absorb a portion of the sound after entry and provide a few more decibels of quieting. This reduction is calculated as follows: [37]

$$NR = 10 \log_{10} \frac{A_o + A_a}{A_o}$$

where

NR = sound pressure level reduction in db

A_o = original absorption in sabins

A_a = added absorption in sabins

Values for A_o and A_a can be calculated from the values and in the following Table 9.2, and the areas of the surfaces. In a typical instance, the absorption might be doubled by the acoustical ceiling.

The added absorption in the space would also reduce the reverberation time which is calculated from the sabine formula: [37]

$$T = 0.05 \frac{V}{A}$$

where

T = time in seconds for a 60 db loss in sound level

V = volume of room in cubic feet

A = total absorption in sabins

9.3 CONTROLLING UNDESIRABLE SOUNDS IN PRECAST CONCRETE HOUSING STRUCTURES

Four rules apply to the control of undesirable sounds.

(a) Plan locations of noise sources so that they cannot be heard where maximum quiet is desired.

TABLE 9.2 SOUND ABSORPTION COEFFICIENTS IN SABINS PER SQ. FT.[1] [37]

	Materials	Frequencies					
		125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
1	Concrete block coarse	0.36	0.44	0.31	0.29	0.39	0.25
2	Concrete block painted	0.10	0.05	0.06	0.07	0.09	0.08
3	Floors concrete or terrazzo	0.01	0.01	0.015	0.02	0.02	0.02
4	Floors linoleum or cork tile on concrete	0.02	0.03	0.03	0.03	0.03	0.02
5	Wood parquet in asphalt on concrete	0.04	0.04	0.07	0.06	0.06	0.07
6	Marble or glazed tile	0.01	0.01	0.01	0.01	0.02	0.02
7	Carpet heavy on concrete	0.02	0.06	0.14	0.37	0.60	0.65

[1] From "Performance DATA, Architectural Acoustical Materials", published by Acoustical and Insulating Materials Association

(b) Avoid mechanical vibration and eliminate it wherever possible.

(c) Do not allow sounds to travel through openings in wall panels, through ventilating ducts, or around wall panels that are intended to separate a noisy area from a quiet area.

(d) Provide for absorption of excess sounds with porous units or units with high absorption coefficients.

The sound transmission loss is usually measured in decibels (which is a unit for measuring the loudness of sound). Average traffic noise in the precast concrete housing structures have decibel ratings from 30 to 40, depending on the activity. The amount of sound reduction afforded by a wall, floor or roof panel is called sound reduction coefficient or transmission loss coefficient. The sound transmission reduction coefficient is closely correlated with the weight of the precast concrete element. A heavier wall tends to reduce the transmission of sound more than a light element.

The precast concrete elements follow what is known as the mass law relationship. This relationship between transmission loss and mass can be expressed by the equation

$$\text{Transmission Loss (T.L)} = 23 + 14.5 \log_{10} m$$

where

m = the mass of the panel stated in pounds per square foot

The following curve, shown in Figure 9.1, is a graphic illustration of this equation.

As the mass increases to larger and larger values, it has a relatively smaller effect on the increase of the transmission loss.

9.4 VIBRATION AND NOISE ISOLATION

Large unbalanced mechanical equipment, which might cause vibrations, can be isolated from buildings by mounting on a heavy resiliently-supported base. The degree of isolation which occurs depends on the natural frequency of the resiliently supported mass and the frequency of the unbalanced machine. To be effective, such a system must, in turn, be mounted on a stiff heavy floor. If the static deflection of the floor is more than a small fraction of the static deflection of the resilient mounts, there is danger that the floor will act as a part of the vibrating system. Precast concrete floors supporting such equipment can be built stiff to avoid this element. A deflection much less than the otherwise satisfactory 1/360 of span is often

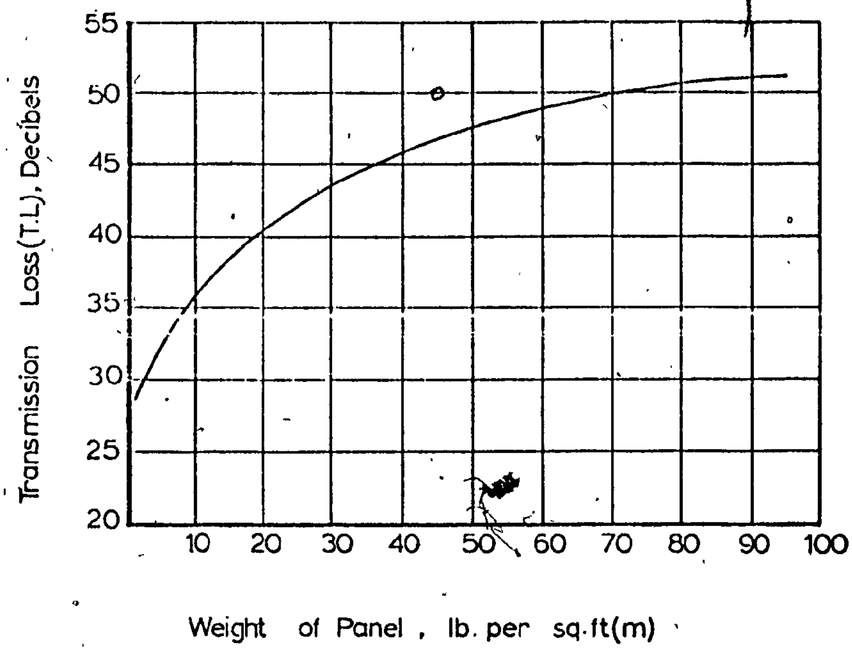


FIG. 9.1 Relationship Between Transmission Loss and Mass [39]

desirable. Locating the equipment near the end of a span, away from its center, also reduces static deflection. Impacts and vibrations in one part of a building are carried to other parts by the structural components. Flexible connections and separations between components in a building constructed of precast sections attenuate noise and vibration as it travels from section to section.

CHAPTER 10

FIRE-RESISTANCE OF CONCRETE

10.1 HISTORICAL BACKGROUND

During the past seventy years, much information has been developed on the fire-resistance of concrete structures. The rapidly advancing technology of structural designs and of fire safety has created a need for more precise information on the behaviour of structures during fires.

Prior to 1958, the PCA sponsored several broad research programs on the fire-resistance of concrete walls and floors. Much of the data developed as long ago as 1920, is still applicable today, even though the testing techniques used then were rather crude compared with today's standards. Early tests on walls, floors, roofs and columns demonstrated that concrete is a highly fire-resistive building material. Most of these early tests were conducted to determine the fire endurance of certain types of construction. Although much of the more recent testing has been aimed toward the same goal, a great deal of today's experimental work is directed toward the development of rational methods for calculating the fire endurance periods and for predicting the behaviour of concrete structures exposed to fire.

10.2 STANDARD FIRE TEST METHODS

Before discussing the factors that affect concrete's fire-resistance, it might be useful to review briefly certain features of standard fire test methods. The standard method for fire testing structural assemblies states that the end point of a fire test occurs when the first of three conditions is reached:

(a) When the specimen fails to support its design load at the structural end point.

(b) When cracks or fissures occur in the specimen flame passage end point.

(c) When the temperature of the unexposed surface rises above the ambient temperature an average of 250°F or 325°F at any one point occurs when the heat transmission reaches the end point.

The latter two criteria apply only to walls, floors and roofs, while the first applies to all load-carrying members.

10.3 PROPERTIES OF STEEL AND CONCRETE AT HIGH TEMPERATURES

In some fire tests, the structural behaviour of the specimen at elevated temperatures is of prime importance. In such cases, the high temperature properties of the materials used in the construction govern the behaviour during a fire test. In reinforced and prestressed concrete, steel resists tensile stresses and concrete resists compressive stresses; therefore, the behaviour of slabs and beams during fire tests depends upon the tensile strength of steel and the compressive strength of concrete at elevated temperatures. The following Figure 10.1 shows a typical relationship between temperature and tensile strength for two types of reinforcing steel.

Most ordinary reinforcing steel behaves about the same as the alloy steel bars shown in Figure 10.1. That reinforcing steel retains about half of its room temperature tensile strength at about 1,050^{O}F; however, for prestressing steel, the corresponding temperature is about 800^{O}F.

Preliminary results of tests in progress indicate that the strength of concrete at high temperatures depends somewhat on the type of aggregate used. Concretes made with siliceous aggregates retain about half their strength at 1,000^{O}F and about one-third their strength at 1,200^{O}F. At temperatures over 1,400^{O}F, most concretes lose much of their original strength.

10.4 FIRE-RESISTANCE OF CONCRETE

We consider a reinforced concrete floor slab supported on bearings at each end, which are free to rotate and to

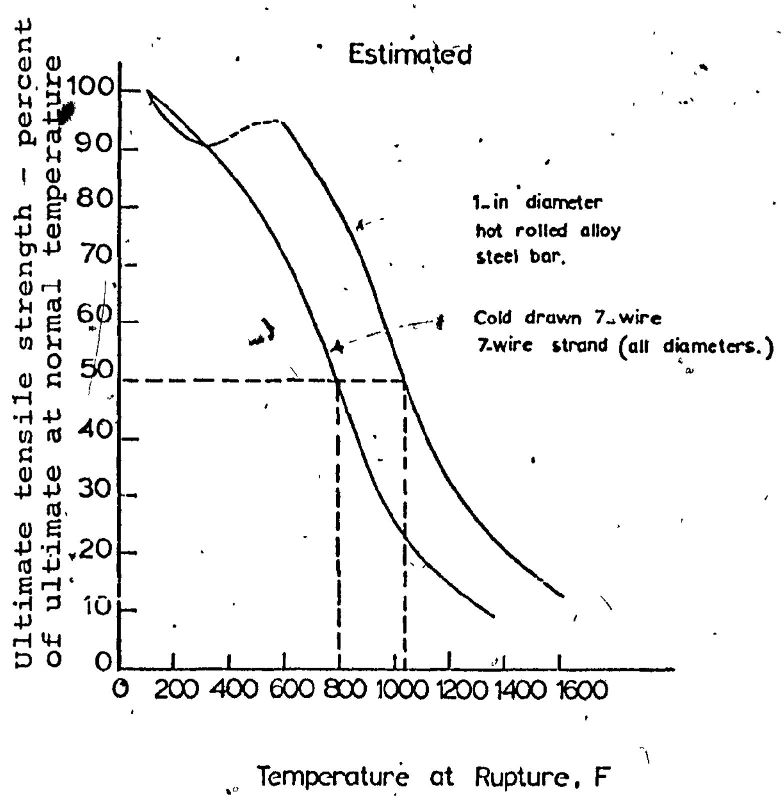


FIG. 10.1 Relationship Between Temperature and Tensile Strength for Hot-Rolled Reinforcing Steel Bar and Cold-drawn Prestressing Steel Wire [38]

accommodate longitudinal expansion, as shown in Figure 10.2.

Let us assume that sufficient longitudinal reinforcement is provided near the bottom of the slab to support a particular service load. Let us further assume that the ultimate capacity of the slab is such that it could support just twice the service load at room temperature. This means that the factor of safety against overload is two. Assume also that the underside of the slab is subjected to a standard fire as prescribed by standard fire test methods. During the first few minutes of fire exposure, the bottom of the slab will expand and cause the slab to deflect downward. After a short period, the temperature gradient through the slab will stabilize, and the deflection will then remain practically constant for a period of time. As the reinforcing steel becomes heated, it will lose strength, and as this occurs, the ultimate capacity of the slab will be lowered. When the strength of the steel has been reduced to a value such that the ultimate capacity of the slab is equal to the service load, the factor of safety will be one. As soon as the steel temperature is raised above that point, a structural end point will occur. The temperature at which such an end point occurs is sometimes called the critical temperature. With an initial factor of safety of two, the critical temperature would have been about 1,100°F, if ordinary reinforcing steel had been used. If the initial factor of safety had been higher than two, the so-called critical temperature would have been higher. Thus, the

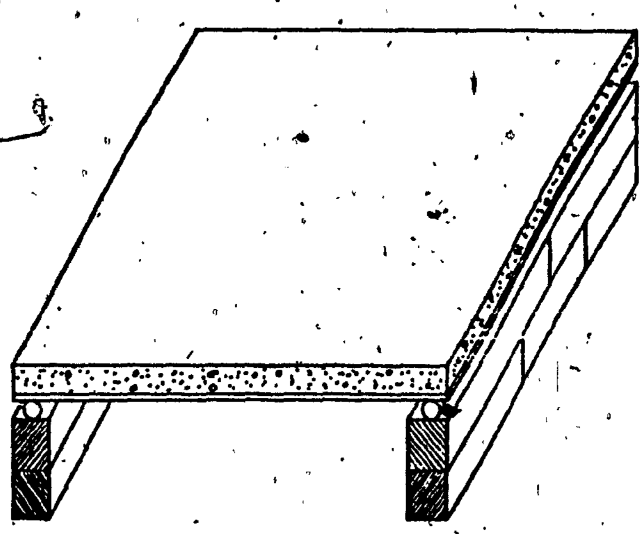


FIG. 10.2 Simply Supported Concrete Slab

factors that affect the steel temperature, such as cover thickness and concrete type, have an important influence on fire endurance of simply supported slabs. The cover thickness is the thickness of concrete between the reinforcing steel and the fire exposed surface. In general, the thicker the cover, the greater the fire endurance, and as a rule, concretes that transmit heat more slowly require less cover for the same endurance period.

10.5 FIRE RESISTANCE RATINGS (SUPPLEMENT NO. 2 OF THE NATIONAL BUILDING CODE OF CANADA) 1975

10.5.1 Minimum Thicknesses

The minimum thickness of unit masonry and monolithic concrete walls for fire-resistance ratings from 1/2 hr to 4 hrs are shown in Table 10.1.

Types of concrete are described as S, N, L, L_1, L_2, L40S, L20S or L220S. (Section 1, Subsection 1.4, Supplement No. 2, to the National Building Code of Canada). Where all the core spaces in a wall of hollow concrete masonry units are filled with loose fill materials, such as expanded slag, burned clay or shale, vermiculite or perlite, the fire resistance of the wall is the same as that of a wall of solid units of the same concrete type and of the same overall thickness.

TABLE 10.1 MINIMUM EQUIVALENT THICKNESSES OF UNIT MASONRY AND OF MONOLITHIC CONCRETE WALLS LOADBEARING AND NON-LOADBEARING, IN. [2]

Type of Wall	Fire-Resistance Rating						
	½hr	¾hr	1hr	1½hr	2hr	3hr	4hr
Solid brick units (80 percent solid and over) actual overall thickness	2.5	3.0	3.5	4.3	5.0	6.0	7.0
Cored brick units and hollow tile units (less than 80 per cent solid) equivalent thickness	2.0	2.4	2.8	3.4	4.0	4.8	5.6
Solid and hollow concrete masonry units equivalent thickness.							
Type S or N concrete	1.7	2.3	2.9	3.7	4.4	5.6	6.6
Type L_1 20S concrete	1.6	2.1	2.6	3.4	4.0	5.1	6.0
Type L_1 concrete	1.6	2.1	2.5	[illegible].2	3.8	4.8	5.6
Type L_2 20S concrete	1.6	2.1	2.5	3.2	3.7	4.6	5.3
Type L_2 concrete	1.6	2.1	2.5	3.1	3.6	4.4	5.1
Monolithic concrete and concrete panels							
Type S concrete	2.3	3.0	3.5	4.4	5.1	6.2	7.1
Type N concrete	2.3	2.9	3.4	4.2	4.9	5.9	6.7
Type L40S or Type L concrete	1.9	2.4	2.8	3.5	4.0	4.9	5.6

(continued)

Type of Wall	Fire-Resistance Rating						
	½hr	¾hr	1hr	1½hr	2hr	3hr	4hr
Gypsum partition tile or block non-load-bearing solid or hollow units, equivalent thickness	1.3	1.7	2.0	2.5	3.2	4.1	5.0
Column 1	2	3	4	5	6	7	8

Short reinforced concrete walls or portions of walls that may be exposed to fire on both sides simultaneously and that are required to carry a load during fire exposure shall have minimum dimensions and have minimum cover-to-steel reinforcement. [2]

Floors and roofs in a fire test are assigned a fire resistance rating which relates to the time that an average temperature rise of 250°F or a maximum temperature rise of 325°F at any location is recorded on the unexposed side, or the time required for collapse to occur, whichever is the lesser. The thickness of concrete which is required to resist the transfer of heat during the fire resistance period is shown in Table 10.2, Supplement No. 2 to the National Building Code of Canada, 1975.

The concrete cover over the reinforcement and steel tendons is required to maintain the integrity of the structure and prevent its collapse during the same period shown in Table 10.3, Supplement No. 2 to the National Building Code of Canada.

10.6 STRUCTURAL BEHAVIOUR

The structural behaviour of concrete floors, roofs and beams during fire exposure is mainly dependent upon the method of support. If slabs or beams are simply supported, the most important factors are the load intensity, the thickness of cover, the type of concrete, and the type of reinforcement. Concrete slabs and beams are not simply supported in most cases,

TABLE 10.2 MINIMUM THICKNESS OF REINFORCED CONCRETE FLOOR OR ROOF SLABS, IN. [2]

Type of Concrete	Fire-Resistance Rating						
	½hr	¾hr	1hr	1½hr	2hr	3hr	4hr
Type S concrete	2.3	3.0	3.5	4.4	5.1	6.2	7.1
Type N concrete	2.3	2.9	3.4	4.2	4.9	5.9	6.7
Type L40S and Type L concrete	1.9	2.4	2.8	3.5	4.0	4.9	5.6
Column 1	2	3	4	5	6	7	8

TABLE 10.3 MINIMUM CONCRETE COVER OVER REINFORCEMENT IN CONCRETE SLABS, IN. [2]

Type of Concrete	Fire-Resistance Rating						
	½hr	¾hr	1hr	1½hr	2hr	3hr	4hr
Types S, N, L40S of L-concrete	5/8	5/8	3/4	3/4	1	1¼	1½
Prestressed concrete slabs Types S, N L40S or L concrete	3/4	1	1	1¼	1½	2	2½
Column 1	2	3	4	5	6	7	8

rather they are supported in such a manner that longitudinal expansion and rotation at the supports are restrained. A redistribution of stresses occurs, which greatly enhances the fire endurance. In tests of restrained floors and roofs, the heat transmission end point generally governs. Transmission of heat through concrete is principally affected by the thickness of concrete, the type of aggregate, and the moisture condition of the concrete. It is hoped that research will lead to concrete structures that are even safer and more fire-resistant than those which have served mankind so well in the past.

CHAPTER 11

BUILT-UP ROOFING. (ASTM STANDARDS IN BUILDING CODES. 1964)

The use of built-up roofing, consisting of alternate layers of bitumen and felt, extends back over little more than a century. Roofing of this type is now often used on roofs with little or no slope, and depends for its effectiveness on its waterproofing properties. Organic felts, most commonly used in this system, can absorb moisture with consequent dimensional changes and deterioration. Bitumens oxidize from exposure to air, moisture, heat and ultra-violet light, and the oxidation products are water soluble and volatile. They also become very brittle when exposed to low temperatures. The quality of the finished built-up membrane depends on workmanship and weather conditions during application. Such roofing, therefore, has not always given good service, and a great deal of attention is being given at present to the problems of roof systems by the researchers.

During the past two decades, research groups in industry have devoted considerable effort to the search for new roofing products. This has been partially as the result of problems with conventional bituminous roofing, but it is also due to changes in architectural design. Designers have developed many new roofs of unusual contour-curved shells, domes, hyperbolic paraboloids, folded plates that are not easily roofed with conventional materials.

The search for new materials for these special applications has resulted in the development of many new roofing systems. Some of these still use bitumens as an adhesive and waterproofing element but with few exceptions thin films replace the old multiple systems. Although a few systems are applied in a manner somewhat similar to that used for conventional roofing, others are sprayed, brushed or rolled on.

11.1 BITUMINOUS ROOF SYSTEMS

Cold applied asphalt emulsions and cold applied coal tar pitch are bituminous-type systems that appear to offer some advantages over conventional hot applied bituminous roofs. Emulsions are very small droplets of bitumen in water and are stabilized by the addition of a material with small plate-like particles that orient at the interfaces between the bitumen droplets and the water. Curing of the emulsion takes place by the evaporation of water, and the particles form a honeycomb network throughout the remaining film. This gives the material excellent stability and permits the use of soft asphalts with characteristics most desirable for roofing. Bentonite clay is the principle stabilizing additive used for asphalt roofing emulsions. The weathering characteristics are good because oxidation is limited by the filler material.

Most roofing systems that use these emulsions start with a base membrane of coated asphalt felt applied over the roof deck or over the roofing insulation, with either a hot or cold asphalt adhesive. In some instances, where application is directly to a concrete deck, it is claimed that the base membrane is not necessary. This is only possible if adequate reinforcing is provided within the emulsion coating. In one system that utilizes a spray technique, asphalt emulsion and chopped glass fibres are applied simultaneously by means of a special spray gun. About 1 pound of glass fibre is used to 3 gallons of emulsion, and is intended to reinforce the resulting layer of asphalt. The effectiveness of the glass fibres as reinforcing depends on the distribution within the mass, with fibre lying across fibre to form a continuous mat of microreinforcing. When fibres fall in bundles leaving areas, the strength and moisture resistance of the material will be reduced. In other systems asphalt emulsion is brushed on over the base membrane and an open glass fibre mat is embedded by brushing to bring the applied emulsion up through the mat. Additional emulsion is then brushed on as the final coating.

Asphalt emulsions do not require the protective covering that is essential to ensure reasonable life of hot applied bitumens. Weathering characteristics, however, are improved by a light reflective coating to reduce surface temperatures and thermal movements. White and pastel shades are preferable, and these are often required for decorative considerations on roof exposed to view. Such decorative coatings will have a limited life, generally much shorter than the life of the material, and may have to be renewed about every five years. Damage to and deterioration of such roofs are readily discernible and relatively easy to repair with cold materials. There are, however, application limitations related to the nature of asphalt emulsions: because of the water carrier, the emulsions can only be used when temperatures are above freezing; and since the material cures to a stable condition slowly, there is danger of wash-off by rain during the first day after application.

11.2 PROCEDURE AND ANALYSIS OF BUILT-UP ROOFS - CALCULATIONS

The calculations of the procedure and analysis of built-up roofs is as follows:

The description of the built-up roof, including type and class of bituminous material (i.e., type of surfacing, type of insulation, type of roof decking, and type of felts or roofing sheets); the number of plies, in built-up roofing, to the nearest 0.1 in. ply; the weight of felts, interply mopping, top loading, total applied bituminous material and surfacing; to the nearest 1 lb/100 ft^2; and the measurements of felts, to the nearest 0.1 in (2.5 mm).

Table 11.1 shows a sample computation for a nominal 4-ply, aggregate-surfaced built-up roof, mopped to insulation 1 lb/100 ft^2 = 4.54 g/ft^2.

TABLE 11.1 COMPUTATION FORM AND SAMPLE COMPUTATION FOR A NOMINAL 4-PLY AGGREGATE SURFACE BUILT-UP ROOF MOPPED TO INSULATION (SAMPLE SIZE: 12 IN BY 12 IN.)

L_i No.	Identification	Factor Component	Computation	Example	Units
1	original specimen	measured weight		2890	g
2	original specimen size	measured area	12.0 x 12.0/144	1	ft^2
3	insulation with absorbed bituminous material			130	2
4	surfacing and top coating scrapings	measured weight		2080	g
5	surfacing cleaned of top coating	measured weight		1790	g
6	approximate top coating material	[line(4)-line(5)]/ [line(2)x4.54]	2080-1790/1 x 4.54	64	lb/100 ft^2
7	total felt area	measured areas of individual felts	0.6+1.0+1.0+1.0+ +0.7	4.3	ft^2
8	number of plies	line(f)/original specimen size	43/1.0	4.3	plies
9	felts and interply bitumen	specimen less top and bottom coating	130+170+200+150+ +110=	760	g

(continued)

L_i No.	Identification	Factor Component	Computation	Example	Units
10	interply bitumen	[line(9)/[(line(2)x x4.54)]-[(line 7xfelt weight]	[760/(1x4.54)]-(4.3x13)	112	lb/100 ft^2
11	approximate interply bitumen	line(10)/[line(8)-1]	112/(4.3-1)	34	lb/100 ft^2
12	weight of insulation after extraction	measured weight		104	g
13	bitumen absorbed into insulation	[line(3)-line(12)/[line(2)x4.54]	(130-104)/(1x x4.54)	6	lb/100 ft^2
14	total applied bitumen	[line(1)-line(5)/ line(2)x4.54 [line(f)xfelt unit weight+line(13)	(2890-1790)/1x 4.54-(4.3x13)+ +6=	192	lb/100 ft^2

11.3 PERIMETER FLASHING DETAILS [29]

Many engineers and architects have eliminated parapet walls from their planning, and all walls of their buildings are covered by the main roof. This can simplify the perimeter flashing details, since in the simplest form the membrane can be turned down over the eaves to shed water and no counter flashing is required. In practice, the eaves detail is complicated by other requirements such as the need to prevent water, bitumen or roof surfacing materials from spilling over the eaves. The designer will usually also want a better architectural finish than that provided by the edge of the roof flashing. This can be provided with reasonable certainty of freedom from trouble if proper detailing is carried out according to the principles of the following figures. Using this sort of detail the drip edge, gravel stop, architectural finish, or combination as required, is kept free of the roof membrane and eaves flashing. This allows completion of the roof membrane before it is necessary to apply the finish flashing, and allows as well, for movement of the flashing due to temperature changes separate from that of the roof membrane.

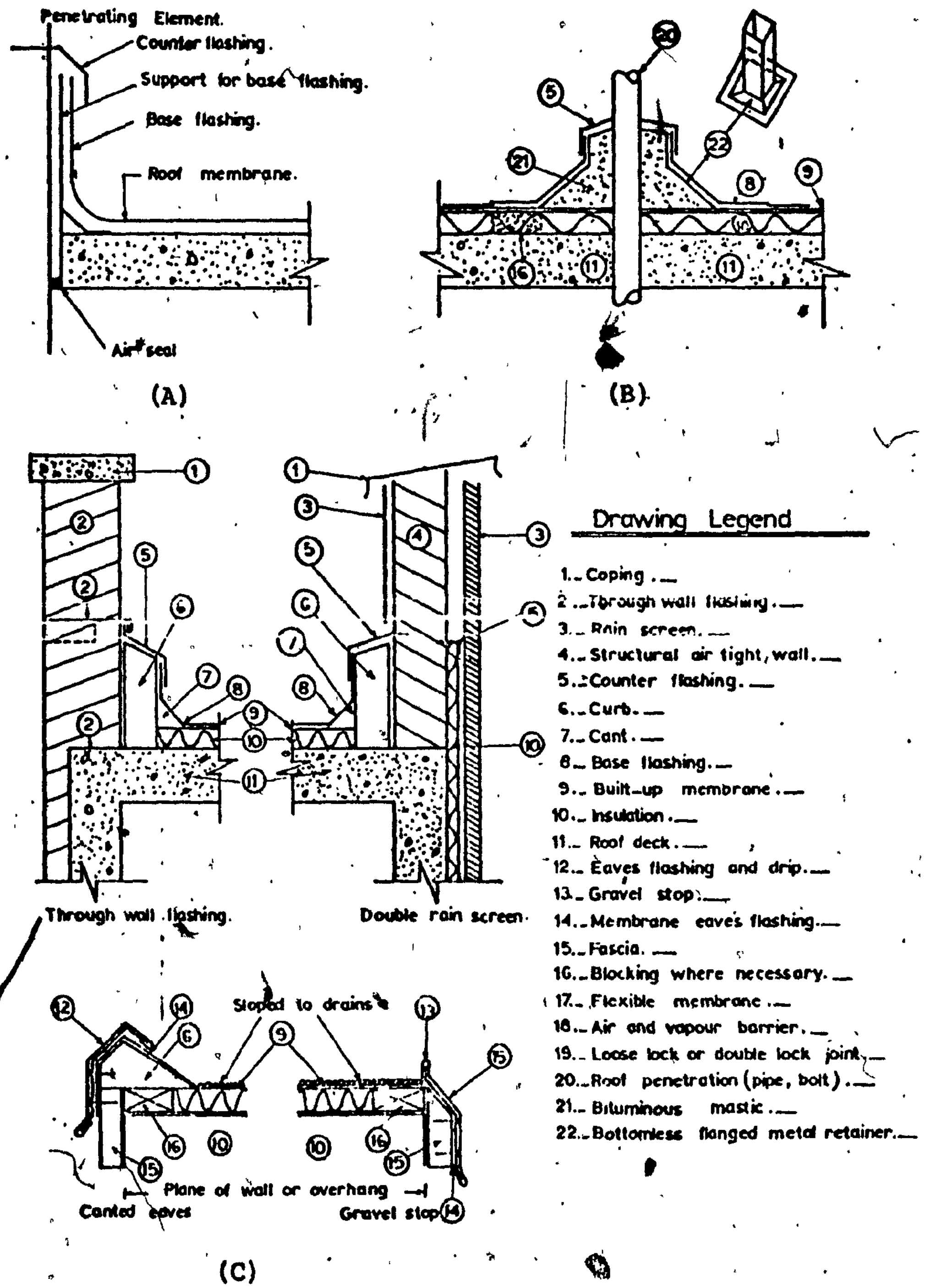


FIG. 11.1 Perimeter Flashing Details - (a) Basic Flashings
(b) Projections Through the Roof
(c) Finish at Roof Eaves

CHAPTER 12

CONCLUSION

During the last thirty years tremendous progress has been achieved in the field of industrial production and utilization of precast concrete structures. The enormous building construction, during this period, has greatly increased the technical knowledge, as well as the project planning and management, of precast concrete residential and industrial buildings.

With the gradual introduction of industrialized precast concrete technology into the construction of buildings in Europe and North America, prefabricated buildings have now become a reality.

New constructional concepts of precast concrete element connections and joints have been introduced. The thermophysical properties of the external precast concrete elements have been much improved. New composite roofs have been developed. The noise insulation properties of the inter-storey floor structure and the inter-apartment walls have been improved.

Further experience has been gained on the field under different systems, which have been created in Europe and North America and in which better constructional designs have been incorporated, concerning the economy of materials.

The erection of fully precast concrete structures has raised the techniques of building construction, particularly in North America, to a new and higher level.

Success in the creation and in the adoption of new types of precast concrete structures is, in a great measure, due to the results of long research and efforts in the development of experimental high-rise projects.

Further advancement in precast concrete technology means not only improving the finish of the surfaces of the precast concrete elements, but also enlarging their size and increasing their readiness for erection. It also accelerates the process during erection and reduces the cost and the volume of posterection building work which has to be carried out.

It is also very important to have good management at all levels. It is not enough to have top level people in production and construction only.

A key to any successful system is the design requirements which must be met. The reason why precast concrete systems can be so competitive is because the problems have been thought out beforehand. This requires a complete collaboration on the part of the architect, the engineer, and all the various mechanical and electrical consultants and subcontractors. The information that they can supply is needed at the very outset, in order that molds can be set up, equipment ordered and production schedules arranged. This must mean the abandonment of the concept of the isolated professional working in his own sphere with little concern for the problems of others engaged in the building process. The leadership of this new design team will largely be governed by the organization sponsoring the system, but can be either the engineer, or the architect, or the contractor. The omission of any participants, at the start, can only lead to an out-of-balance system.

The concept of precast concrete industrialized buildings serves thus, to get all those concerned to operate, in their respective tasks, in a more efficient way.

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