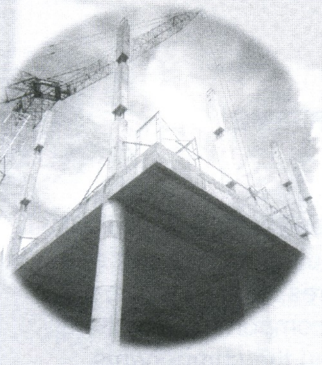


CHAPTER 1

STRUCTURAL CONCEPT FOR PRECAST CONCRETE SYSTEMS

CHAPTER 1 STRUCTURAL CONCEPT FOR PRECAST CONCRETE SYSTEMS

1.1 General

The main structural difference between cast-in-situ buildings and precast buildings lies in their structural continuity. The structural continuity of conventional cast-in-situ buildings is inherent and will automatically follow as the construction proceeds. For precast structures, there must be a conscious effort to ensure that structural continuity is created when precast components, such as slabs and walls, are connected. The connections act as bridging links between the components.

As the structural elements in precast building will only form a stable structural system after the joints are connected, structural considerations for stability and safety are necessary at all stages.

A load-bearing structure with stabilising elements which can sustain both vertical and horizontal loads and transmit these to the foundation and the soil is required. The structure must be robust and adequately designed against structural failure, cracking and deleterious deformations.

The overall behaviour of a precast structure is dependent on the behaviour of the connections which must respond to:

- resistance to all design forces
- ductility to deformations
- volume changes
- durability
- fire resistance
- production considerations
- construction considerations

1.2 Loadings And Load Tables

1.2.1 Design procedures in general

A logical design procedure for the structural engineer must include these four phases (Figure 1.1):

1. Load Assessment: Setting up load estimates and load tables.
2. Calculation Model: Defining the structural system, describing a possible load path, evaluation of stiffness of components and joints, the execution methods and load combinations.
3. Structural Analysis: Starting with the determination of loads on components and joints, calculation of the strength or carrying capacity of materials, cross-sections and joints; thereafter, the loads and the resistance are to be compared.
4. Documentation: Preparation of specifications, sketches, shop drawings and assembly drawings.

PROCEDURES	ACTIONS
1. LOAD ASSESSMENTS	Load Tables Load Estimates
2. CALCULATION MODEL	Structural System Load Path Stiffness of Components and Joints Execution Methods Load Combinations **Calculations:** Internal Forces Reactions
3. STRUCTURAL ANALYSIS (CODES OF PRACTICE)	**Design:** Stresses Deformations Deflections
4. DOCUMENTATION	Specifications Calculations Sketches Drawings

Figure 1.1 Procedure For Structural Design

1.2.2 Vertical and horizontal loads

The building design loads can usually be extracted from the load specifications in codes of practice or building bye-laws. If a load type, such as the wind load, cannot be established because it is a specially shaped building, tests must be conducted.

The vertical loads comprise the dead weight of the structure, superimposed dead loads and live loads. Load from lightweight partition walls is normally treated as a line load. It can be equated as a uniform load if the floor slabs distribute the load evenly.

For precast buildings with simply supported main structural members, it is easy to organise and accumulate the loads acting from top to bottom of the building in load tables. An example of how it deals with accumulating vertical loads in general is shown in Figure 1.2.

The horizontal loads are derived from either the wind forces or from the so-called notional force which is determined as a certain percentage of the total vertical load on the building. In CP65, the notional horizontal load is taken to be 1.5% of the characteristic dead weight of the structure and is treated as an ultimate load for which the stability of the building has to be checked against. It occurs due to eccentricities of the structure, tremors or subsoil settlements. In countries where earthquakes are frequent, a considerably higher value is used, or dynamic calculations have to be made.

The wind forces act on the facades and gables of the building. The notional force, on the other hand, is located at the points of application of the vertical forces, normally accumulated at the centres of gravity of the structural walls and of the floor slabs.

The wind forces and the notional force are assumed to act in an arbitrary direction. The designer only needs to consider the greater of the two forces.

Accidental action is action applied to the structure as the result of accidents and not due to specified imposed loads. Accidental action could occur from collisions, explosions or from vertical loads on air raid shelters.

For structures which must be assumed to be exposed to the risk of accidental action, specifications in the Code* normally give the designer a choice between:

- designing the structure in such a way that the parts of the structure subjected to the accidental action can withstand this, or
- designing the structure in such a way that failure of a given magnitude will not result in the progressive collapse or toppling of the entire structure, as detailed in Figure 1.3

Precautions Against Structural Failure
The figure demonstrates one of the principles of structural integrity. Special joint reinforcement bars are placed to avoid the progressive collapse of the building.

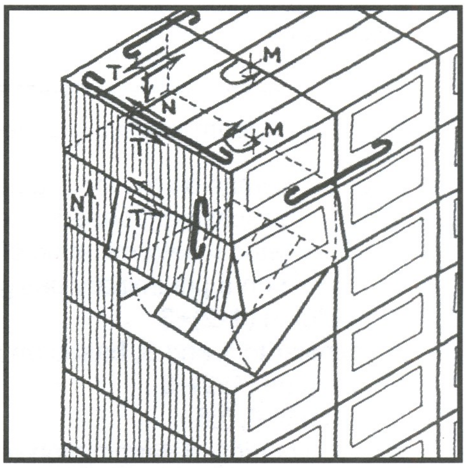
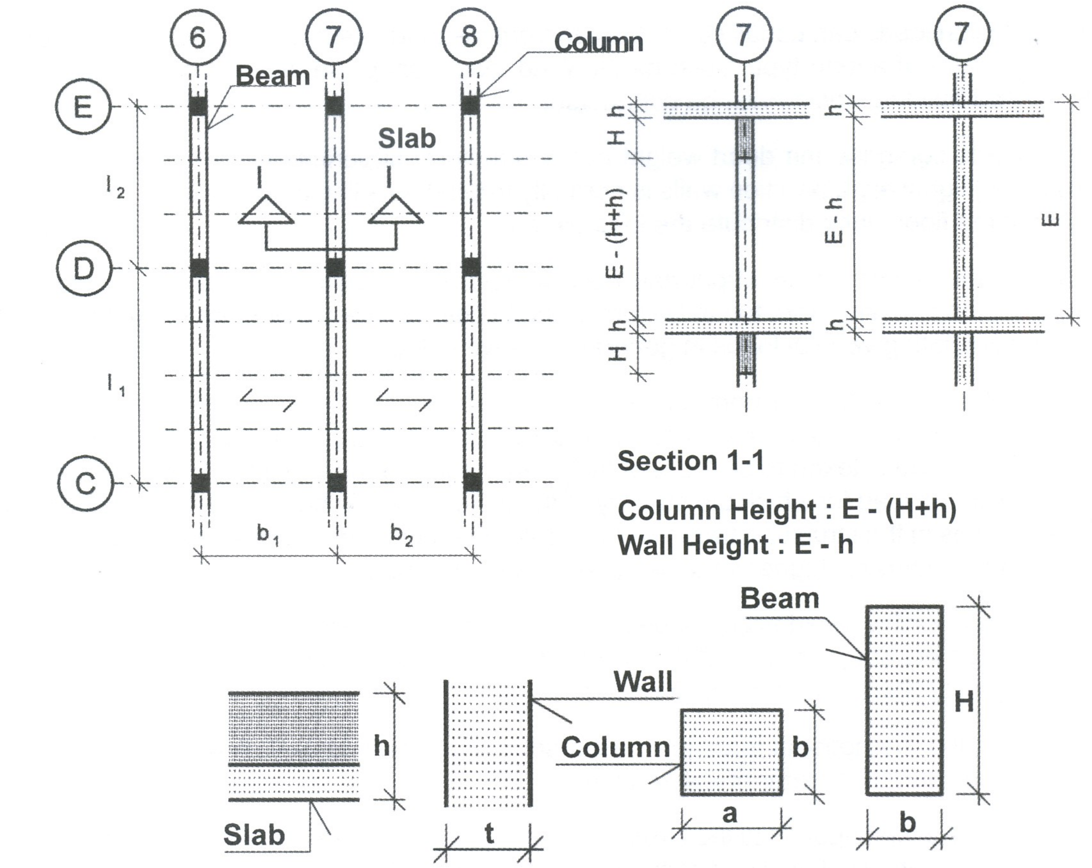


Figure 1.3 Structural Integrity Ties Against Failure

* : CP65 (1999) shall hereafter be referred to as the “Code”.



ACCUMULATION OF VERTICAL LOADS	1		2	3	4 (1x2)+3		5	6 (1x2)+5		7		8	9	10 (4x8)+9		11	
CHARACTERISTIC VALUES	SLAB LOADS		SLAB SPAN	BEAM SW	BEAM LOADINGS		WALL SW	WALL LOADINGS		$\Sigma 6$		BEAM SPAN	COLUMN SW	COLUMN LOADINGS		$\Sigma 10$	
	kN/m^2		m	kN/m	kN/m		kN/m	kN/m		kN/m		m	kN	kN		kN	
	DL	IL	$\frac{1}{2}(b_1+b_2)$	DL	DL	IL	DL	DL	IL	DL	IL	$\frac{1}{2}(l_1+l_2)$	DL	DL	IL	DL	IL
TOP FLOOR (ROOF)																	
NORMAL FLOOR																	
↓ x N																	
GROUND FLOOR																	
BASEMENT																	
FOUNDATION																	

DL is for superimposed dead load, IL for live load, and SW for self weight of structures.

The 11 columns on the load table are:

1. Load assessment based on a preliminary design
2. Geometrical determination of slab spans
3. Based on a preliminary design
4. Column (1x2)+3
5. Similar to 1 and 3, based on a preliminary design
6. Column (1x2)+5
7. Sum of 6, $\Sigma 6$
8. Geometrical determination of beam spans
9. Based on a preliminary design
10. Column (4x8)+9
11. Sum of 10, $\Sigma 10$

Figure 1.2 Accumulation Of Vertical Loads

1.2.3 Load path description

A building constructed from precast components becomes a so-called “house of cards”. It requires rather simple structural calculations as most of the load-bearing structural members are considered simply supported.

Using precast floor slabs, walls, beams and columns, it is seldom possible to achieve restraint in the joints, mainly due to small dimensions of the components. This calls for special attention when evaluating the stability of the entire structure.

After having made the load assessment and the choice of calculation model, the next and very important step is the load path description. It should explain in detail a possible load path for a specific load from the point of application to where the load is transmitted to the foundation and the soil.

By using a detailed load path description for vertical as well as horizontal loads, the designer is able to calculate all internal forces acting on components and joints necessary for a proper design.

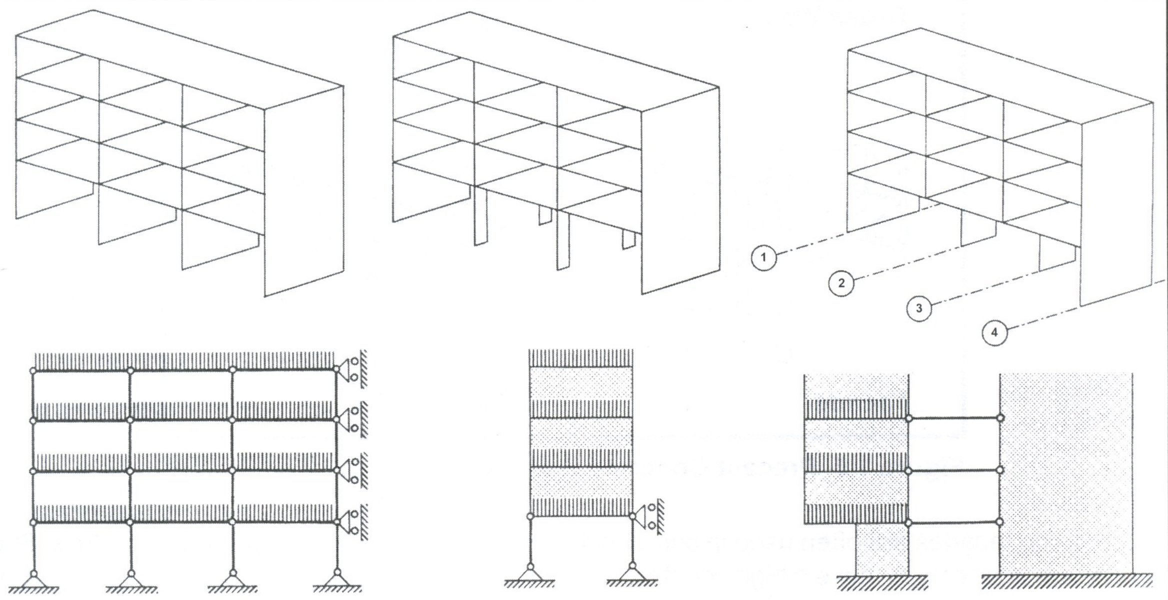
The description could also act very conveniently as a guide for the accumulation of loads and as a list of contents for the structural calculations.

1.2.4 Load distribution

The last preparatory step before the structural analyses can begin is the distribution of all loadings from their application points to the load-bearing and bracing systems. This process is very much linked to the load path description. Based on the determination of structural models and the evaluation of structural stiffness or rigidity, the distribution of loads can be easily accomplished.

With simply supported precast components, it is easy to allocate the vertical load to the load bearing elements, normally directly proportional to the span. Horizontal loads, on the other hand, are more difficult to handle. It is essential that precise and detailed structural description and evaluation are made.

The illustrations in Figure 1.4 give examples of statical models for vertical as well as horizontal loads for a panel system building with different supporting structures on the first floor.



(a) Proportionate Shear Wall (b) Wall Frame (c) Non-Proportionate Shear Walls

Figure 1.4 Statical Models For Vertical And Horizontal Load Distribution Of Large Panel Structures

1.3 Precast Concrete Systems

1.3.1 Building systems

Some examples of the more commonly used precast concrete building systems are illustrated in Figure 1.5. Skeletal frame systems are suitable for commercial buildings, schools, hospitals, parking structures and sporting facilities, where a high degree of flexibility in planning and disposition of floor areas can be achieved by using large spanning column-beam layout. Such buildings are easily adaptable to changes in use, thus giving architects a broader choice in facade claddings.

Systems with load-bearing cross walls, spine walls or facades are more suitable for domestic housing, apartments and hotels. Advantages are high construction speed, ready-to-paint surfaces, facades as architectural precast, good acoustic insulation and fire resistance.

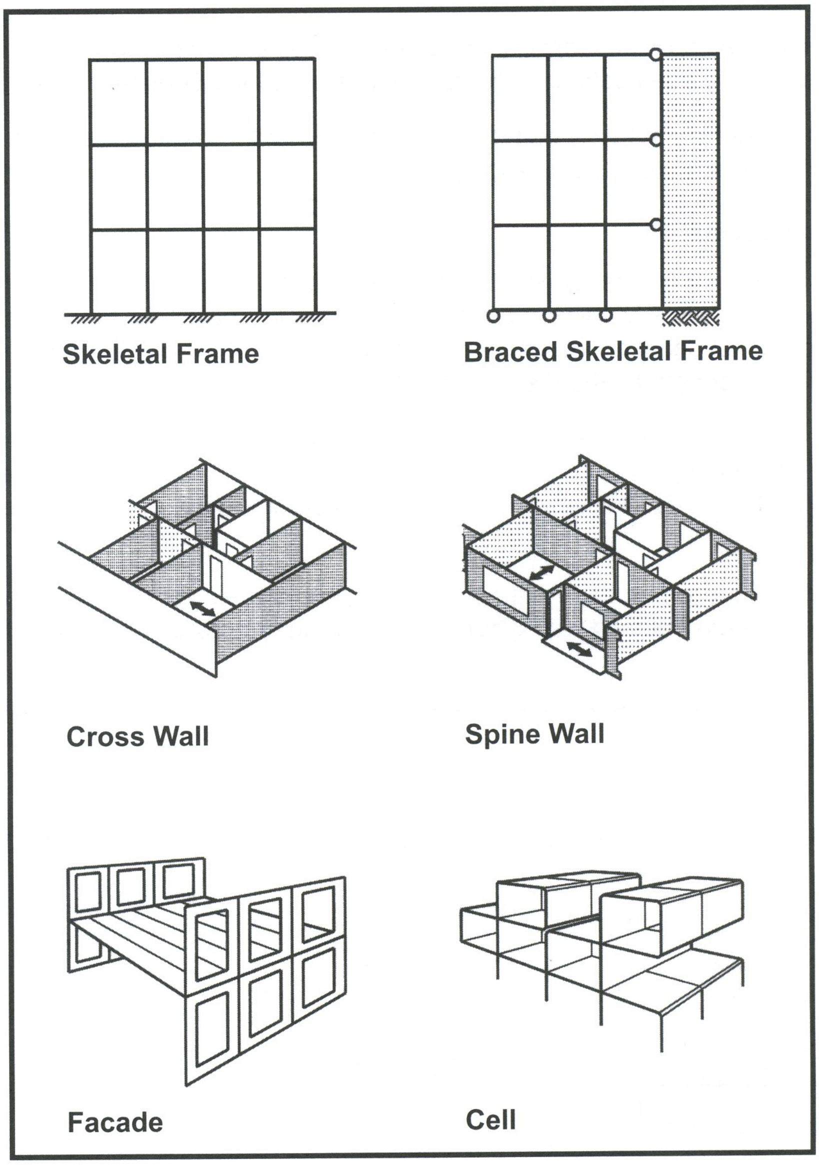


Figure 1.5 Precast Concrete Building Systems

Load-bearing facades are often used in combination with skeletal frames as internal structure. Such systems are economical, have a high construction speed and can incorporate architectural finishing.

Cell systems are mainly used for parts of a building, for instance, bathrooms or kitchens. Important elements in the design are manufacturing, assembly, transportation and erection considerations. Heavy cranage with special lifting devices may be needed.

Some assembly examples of frame and cell systems are shown in Figure 1.6.

The illustration (d) with the transverse crane shows a slab-column system in which stability is achieved by means of rigid cast-in-situ staircase towers.

Sketch (e) shows a cell system in which the individual rooms are prefabricated boxes assembled in a skeleton of precast frames.

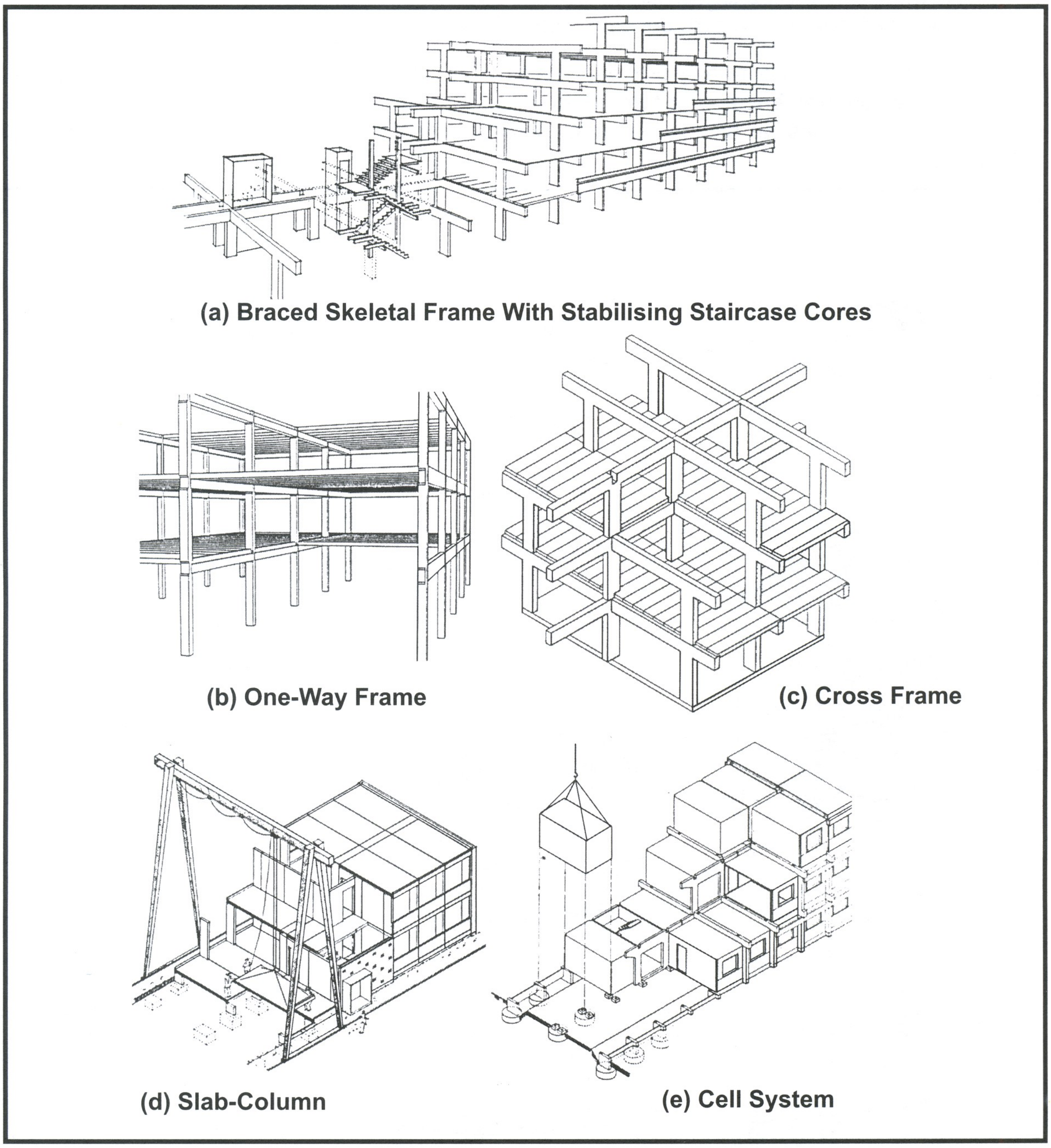


Figure 1.6 Assembly Of Precast Building System

Precast buildings with skeletal frames system may come in different forms and some examples are shown in Figure 1.7.

One- and two-storey column-beam systems together with four-storey high columns are shown. All of these are normally divided into components with joints at crossing points between the structural members.

Structures with other component divisions are also shown, for instance, with structural joints placed at the beam-span near to the zero bending moment point as in structural systems 5 and 9.

The structural joints can be broadly grouped into either hinged or pinned and rigid or fixed joints and they can either be prefabricated or formed at site. By manipulating the joints and their positions, various structural frames can be achieved from the assembly of the precast components as illustrated in Figure 1.8.

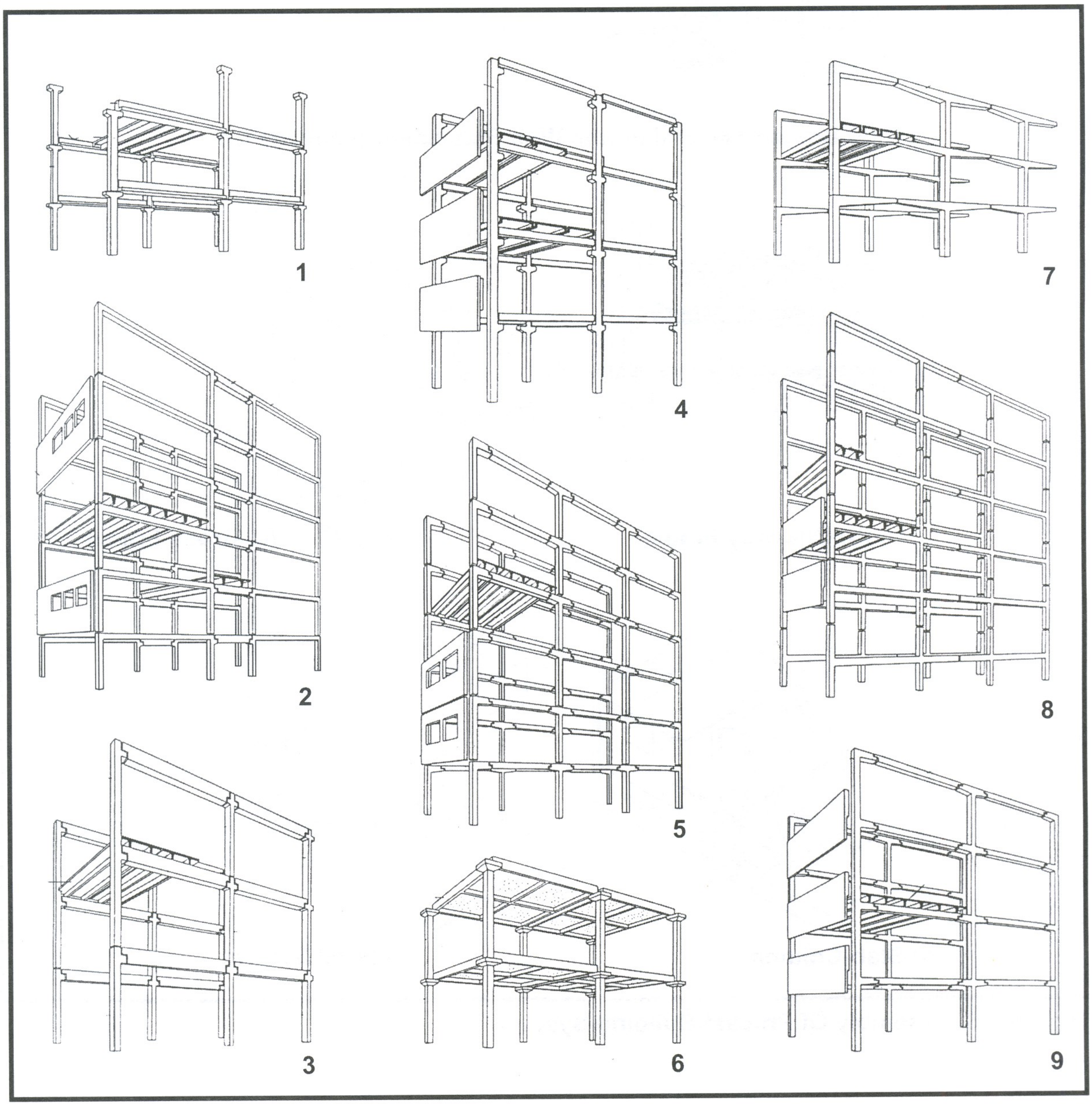


Figure 1.7 Variations In Skeletal Frame Systems (reference 2)

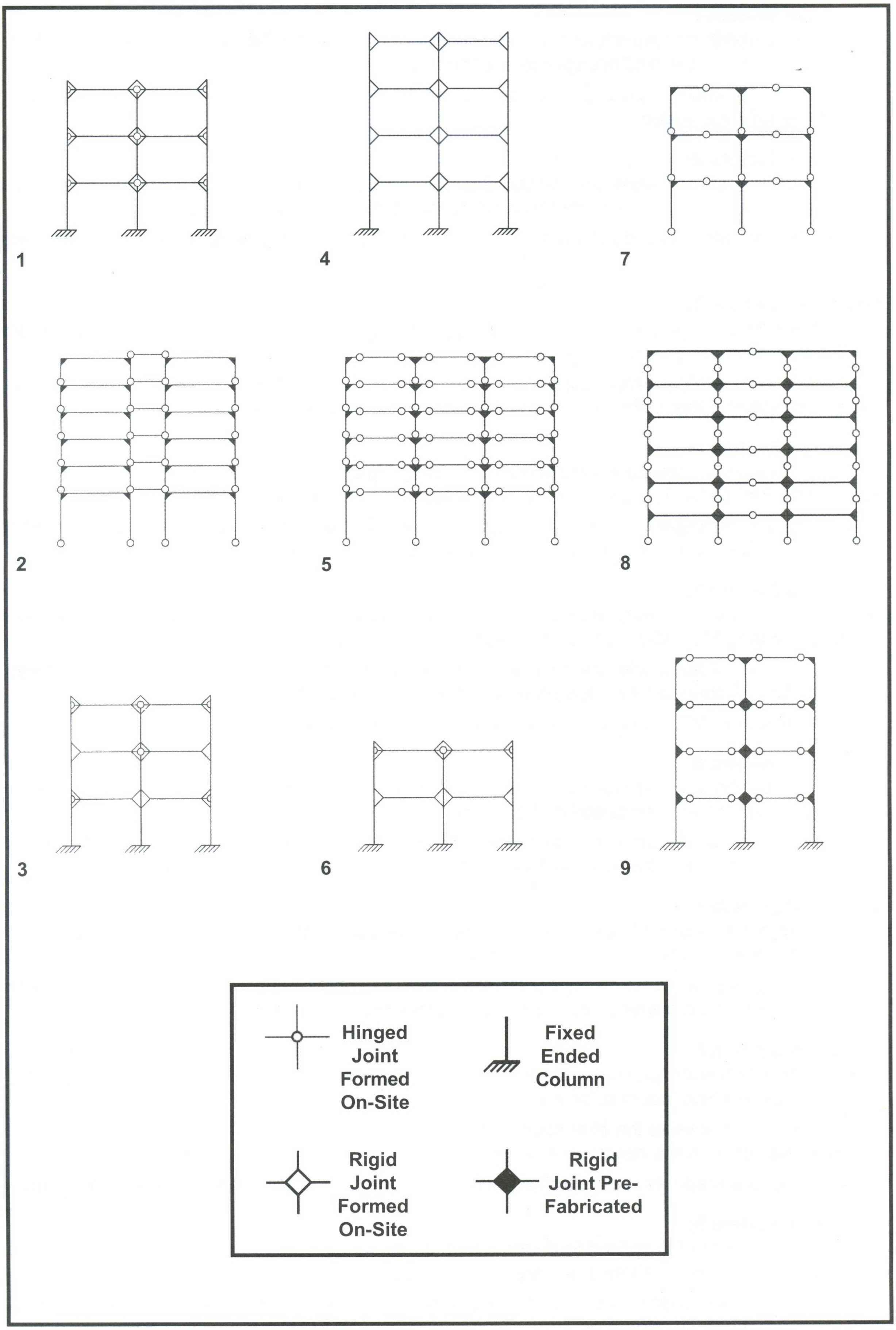


Figure 1.8 Skeletal Frames And Structural Idealisation

Structural System 1:
The system consists of columns with corbels, inverted T-beams and slab components. Columns are restrained at their bases and are spliced at each floor.

Beams are temporarily simply supported on the corbels, but rigid joints are formed afterwards. Floor slabs are simply supported.

Structural System 2:
The system consists of frames, beams and slabs. Frames are one-storey high, simply supported on top of each other, and are connected by a simply supported beam on each floor.

Floor slabs are also simply supported. (Structural System 2 is explained in detail on the following Figure 1.12.)

Structural System 3:
The system consists of one- and two-storey high columns, beams and floor slabs. Columns are restrained at their bases and erected with staggered joints.

Beams are temporarily in simply supported state. They are designed to have permanent rigid joints after construction. Floor slabs are simply supported from beam to beam.

Structural System 4:
Structural System 4 comprises unspliced four- storey high continuous columns with corbels, plus beams and floor slabs. The columns are restrained at their bases.

Beams behave as simply supported at temporary stage, but are designed to have rigid joints after construction. Floor slabs span from beam to beam, and are simply supported.

Structural System 5:
The system consists of T-shaped and L-shaped columns with beams suspended at the point of zero bending moment. Floor slabs span from beam to beam and are simply supported.

Frames of L- or T-shaped columns are placed simply supported on top of each other. At each storey, these frames are connected by beams spanning from frame to frame.

All rigid joints are prefabricated as an integrated part of the columns.

Structural System 6:
The system consists of unspliced two-storey high continuous columns and freely supported large floor units. Columns are restrained at their bases.

Floor units span in two directions and are supported only at the columns. Hidden beams are incorporated. Rigid joints between slabs and column drops are formed after erection.

Structural System 7:
The system consists of L-shaped and T-shaped frame units placed on top of each other. Simply supported floor slabs span between the frames.

The frame units are placed simply supported on the base and on top of each other. Hinged connections are made between the frame units as pin-joints at the mid-span of the beams.

Structural System 8:
The system shown is formed by H-shaped frames with a cantilever beam. Slabs are simply supported by spanning them from beam to beam.

The connections between the H-shaped frames are formed as pin-joints at the mid-height of the columns and between the cantilever beams.

All rigid joints are made as an integrated part of the frame units. Slabs span from frame to frame.

Structural System 9:
The system consists of unspliced four-storey high continuous columns with cantilever attachments for supporting the beams. Floor slabs are simply supported on the beams.

The columns are restrained at their bases. Beams are simply supported on column-attachments. All rigid joints are made as integrated parts of the columns.

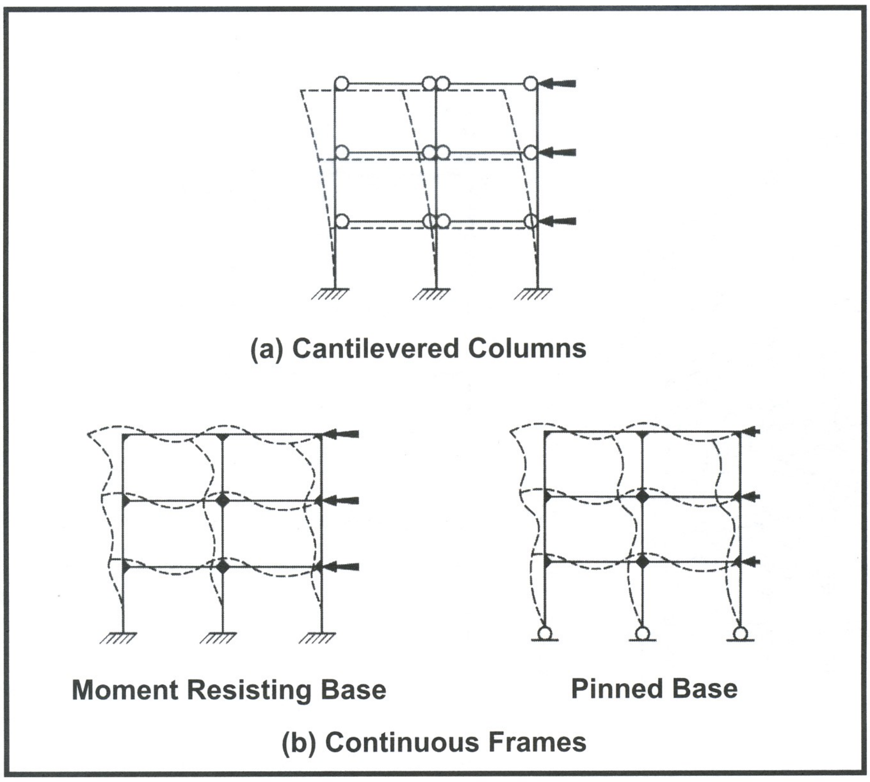


Figure 1.9 Unbraced Frame

1.3.2 Structural systems for horizontal loads

1. Unbraced frame system

a. Cantilever columns (Figure 1.9a)

- Beam-column connections are pinned. Moment resisting connections can be found at column-foundation intersection.

b. Continuous frame system (Figure 1.9b)

- Beam-column connections are rigid with flexural and shear continuity. The system allows either moment resisting connections or pinned connections at column-foundation intersection.

2. Braced skeletal system (Figure 1.10)

- Stability is provided by shear walls, shear cores or other bracing systems.
- The base may be pinned or moment resisting connections.
- Beam-column connections may be rigid or pinned.

3. Bearing wall and facade (Figure 1.11)

- Bearing walls which include core walls, spine walls, cavity walls, shaft walls and load-bearing facades can be designed to transfer vertical and horizontal forces to the foundations.
- Non-load bearing panels are designed to withstand the appropriate stresses, but are not intended to carry any load in the building.

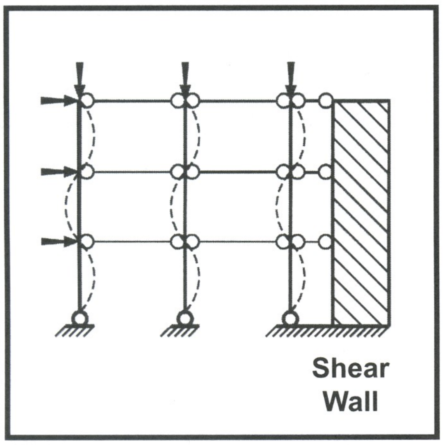


Figure 1.10 Braced Frame

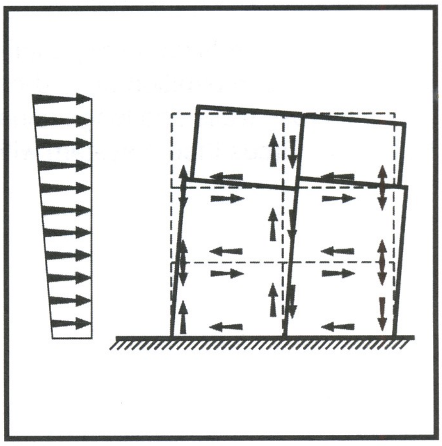


Figure 1.11 In-Plane Action Of Precast Concrete Walls

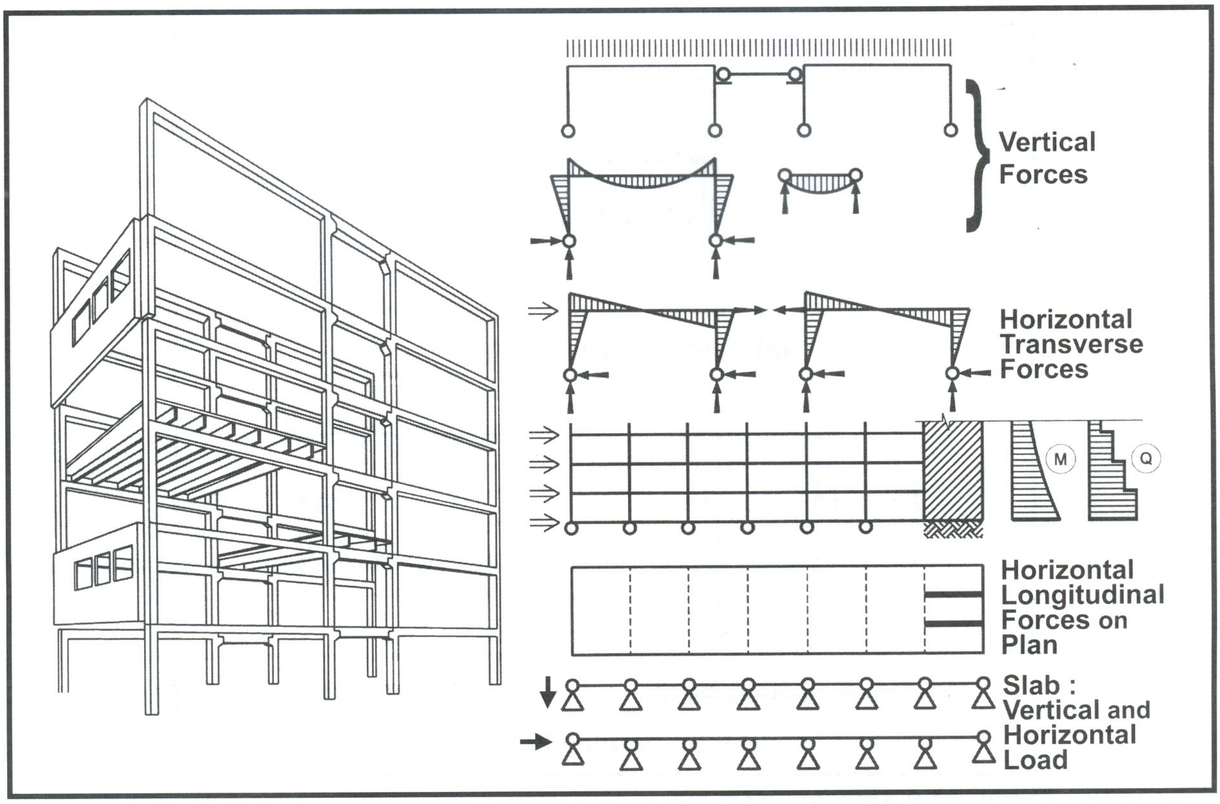


Figure 1.12 Statical Model Of Skeletal Frame For Horizontal And Vertical Forces

Figure 1.12 describes the structural model and behaviour of a skeletal frame system subjected to horizontal and vertical forces. It also covers a load path description. The vertical forces are transmitted from the double-T floor slabs to the beam and to the frames. The double-Ts are acting as simply supported slabs.

The horizontal transverse force is transmitted from the facade components to the floor slab structure, and by diaphragm action to the frame structure. The floor slab structure can act as a continuous beam spanning from frame to frame.

The horizontal longitudinal force from wind or notional force is transmitted from the gable or from gravity centres to the floor slab structure and further on to the bracing walls. These walls are considered as vertical beams restrained at the foundation or at the basement structure. They are able to sustain the longitudinal force as bending moment and shear at the wall-foundation intersection.

Figure 1.13 describes a panel system, possible load paths, and a structural model for vertical and horizontal forces. The system is a cross-wall structure with longitudinal bracing by three spine walls, two near the gables and one in connection with the stairwell structure. On the left, the isometric drawing shows how the floor slabs and the walls respond to the loadings.

To the right, alternative floor spanning directions are shown. The isometric drawing shows a structural system as a combination of load-bearing cross walls and of load-bearing facades and spine walls. As both cross walls and longitudinal walls are load bearing, this system is more suitable to absorb horizontal forces than a system with parallel load-bearing walls only.

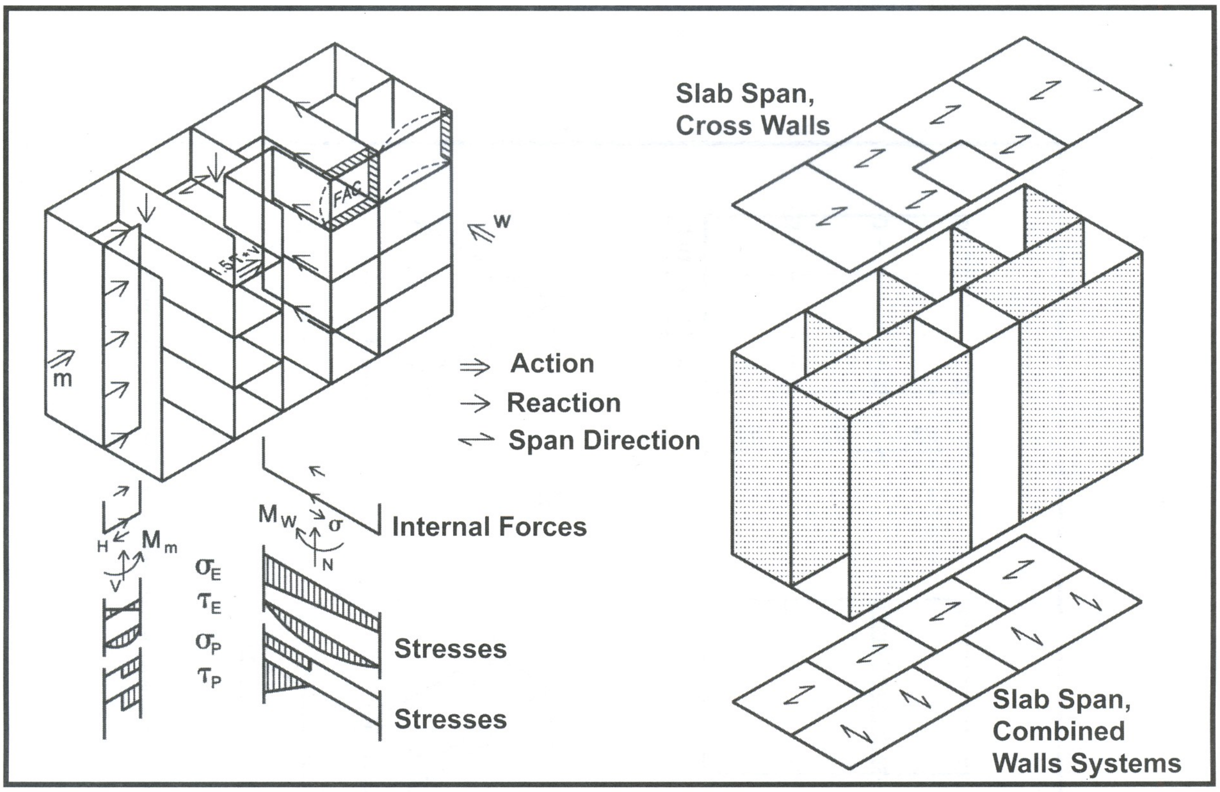


Figure 1.13 Statical Model Of Panel System For Vertical And Horizontal Forces

The bracing walls are all restrained at the foundations, so that internal forces as well as normal, bending and shear stresses can be calculated using either the theory of elasticity or the theory of plasticity.

1.4 Slab Wall Structures

1.4.1 Statically determinate slab-wall systems

Sketches 1, 2, 3, 4 and Sketches 1a, 2a, 3a, 4a in Figure 1.14 show respectively a floor slab supported by eight columns, one bracing wall and columns, two bracing walls and columns, and finally three bracing walls and columns. All columns are considered as pinned, and all walls are only able to sustain and transmit in-plane forces. A building is said to be stable when its individual structural members are in stable equilibrium and can resist the acting forces.

The structure in Sketch 1 (1a) is therefore unstable and unable to sustain any horizontal loads.

The structure in Sketch 2 (2a) is able to sustain only the horizontal force acting in-plane to wall 0. All other horizontal forces will cause movement in the structure.

The structure shown in Sketch 3 (3a) has two walls to carry horizontal loads. This system is sufficient if the horizontal loads can be resolved into components which are in-plane with walls 0 and 1. In other words, the result of the acting forces must go through the crossing between walls 0 and 1. All forces in other directions will cause movement in the structure.

The structure shown in Sketch 4 (4a) comprises the floor slab plus three bracing walls in lines 0, 1 and 2. The walls are placed in such a way that they are not crossing each other at the same point. Such a system is able to sustain any horizontal force. This structure is therefore in stable equilibrium.

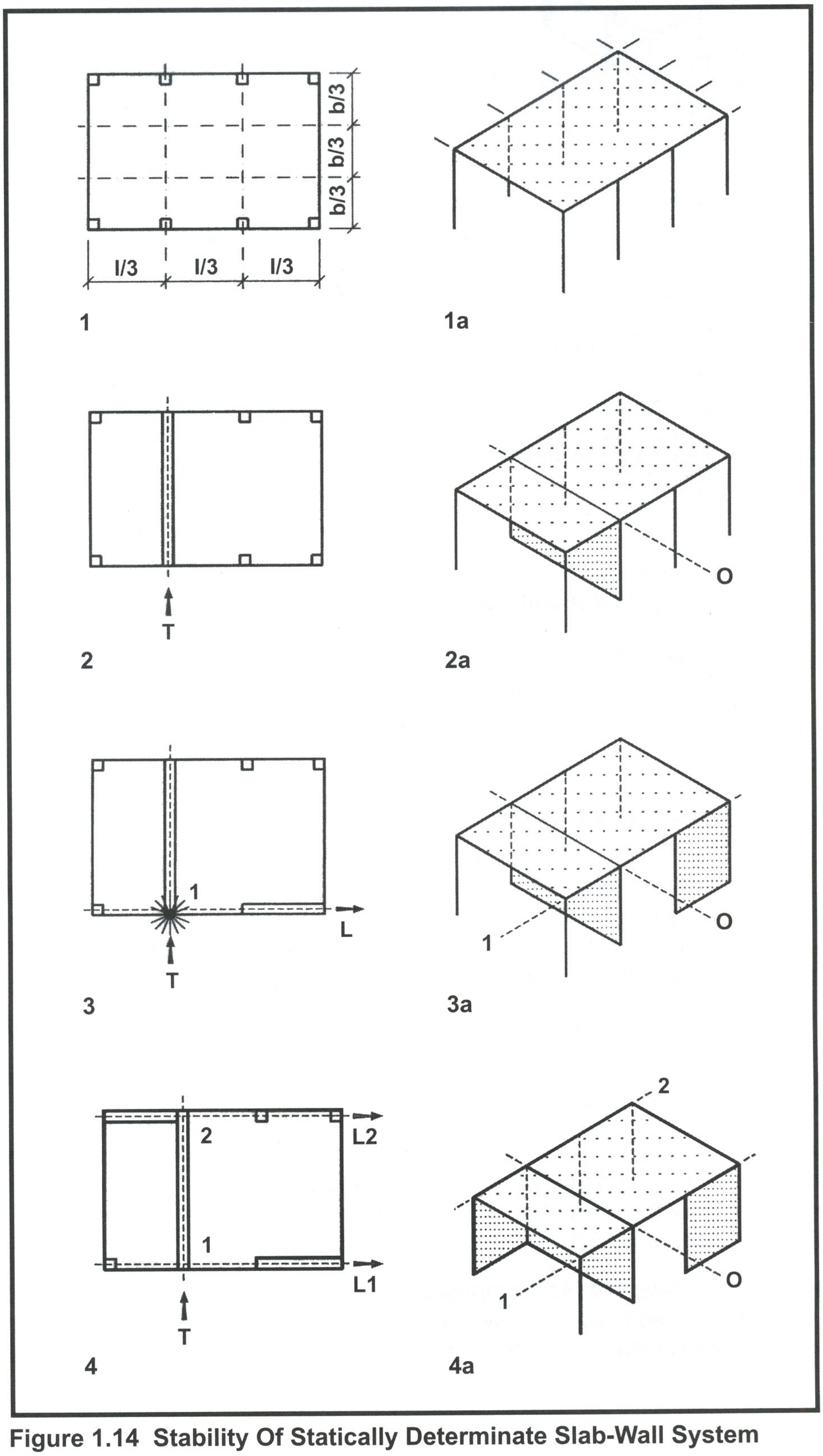


Figure 1.14 Stability Of Statically Determinate Slab-Wall System

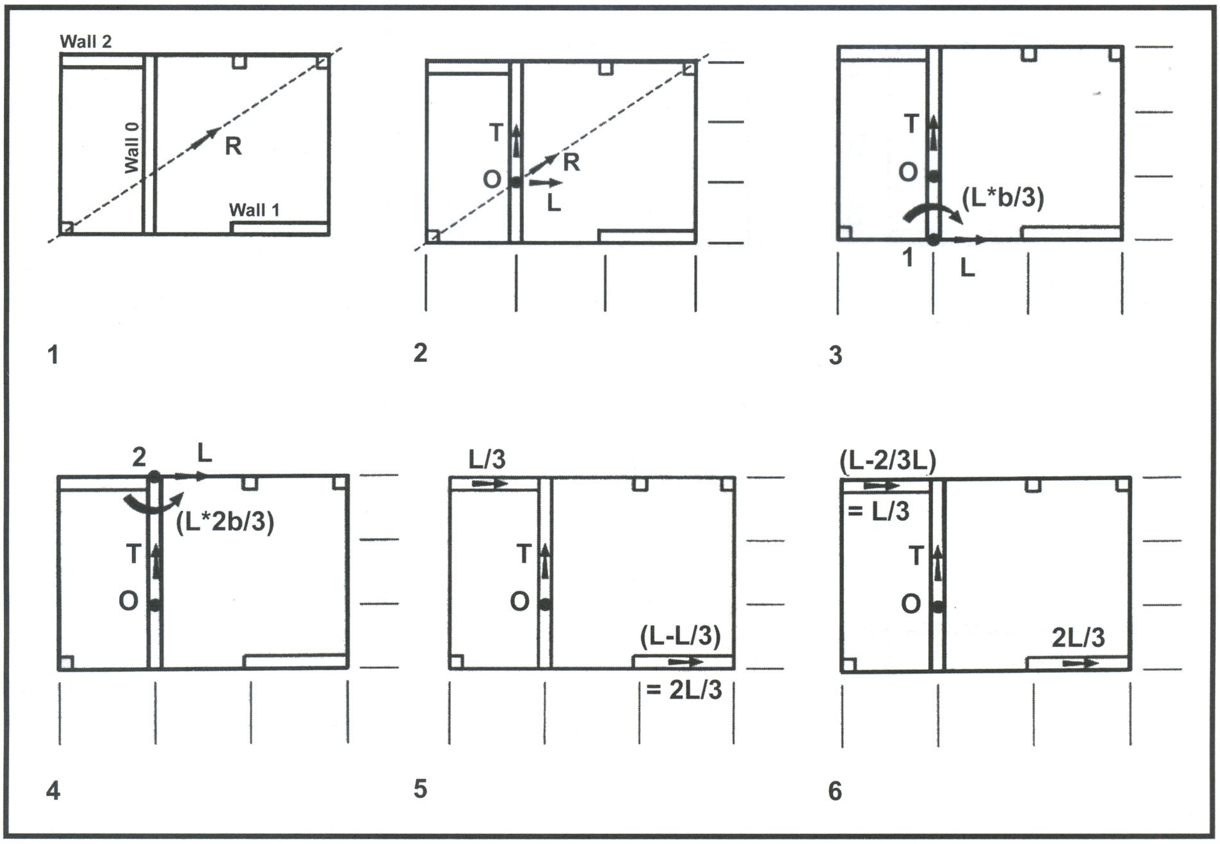


Figure 1.15 Resolution Of Horizontal Forces In Walls

1.4.2 Resolution of horizontal forces

Sketch 1 in Figure 1.15 shows a horizontal force R acting in-plane to the floor slab in the diagonal direction, from one building corner to the opposite corner.

The force R can be moved to point 0 and resolves into a transverse force T and a longitudinal force L as shown in Sketch 2.

Force T can now be sustained by wall 0. Force L has to be transferred to either the crossing point between walls 0 and 1, or to the crossing point between walls 0 and 2.

Sketches 3 and 5 show one of the possibilities where force L is displaced from 0 to the crossing point between walls 0 and 1. A moment L x b/3 has to be added.

Force L will go directly to wall 1, and the moment will be resolved into a pair of forces in walls 1 and 2 with a value of L/3. The final wall actions will be at:

wall 0 : T
wall 1 : L - L/3
wall 2 : L/3

Sketches 4 and 6 show the situation where force L is displaced to the crossing point between walls 0 and wall 2.

The moment to be added is now L x 2b/3 which has to be taken as a force pair still in walls 1 and 2. The value of these forces will now be : 2L/3, which again lead to the final actions at the three-wall system:

wall 0 : T
wall 1 : 2L/3
wall 2 : L - 2L/3 = L/3

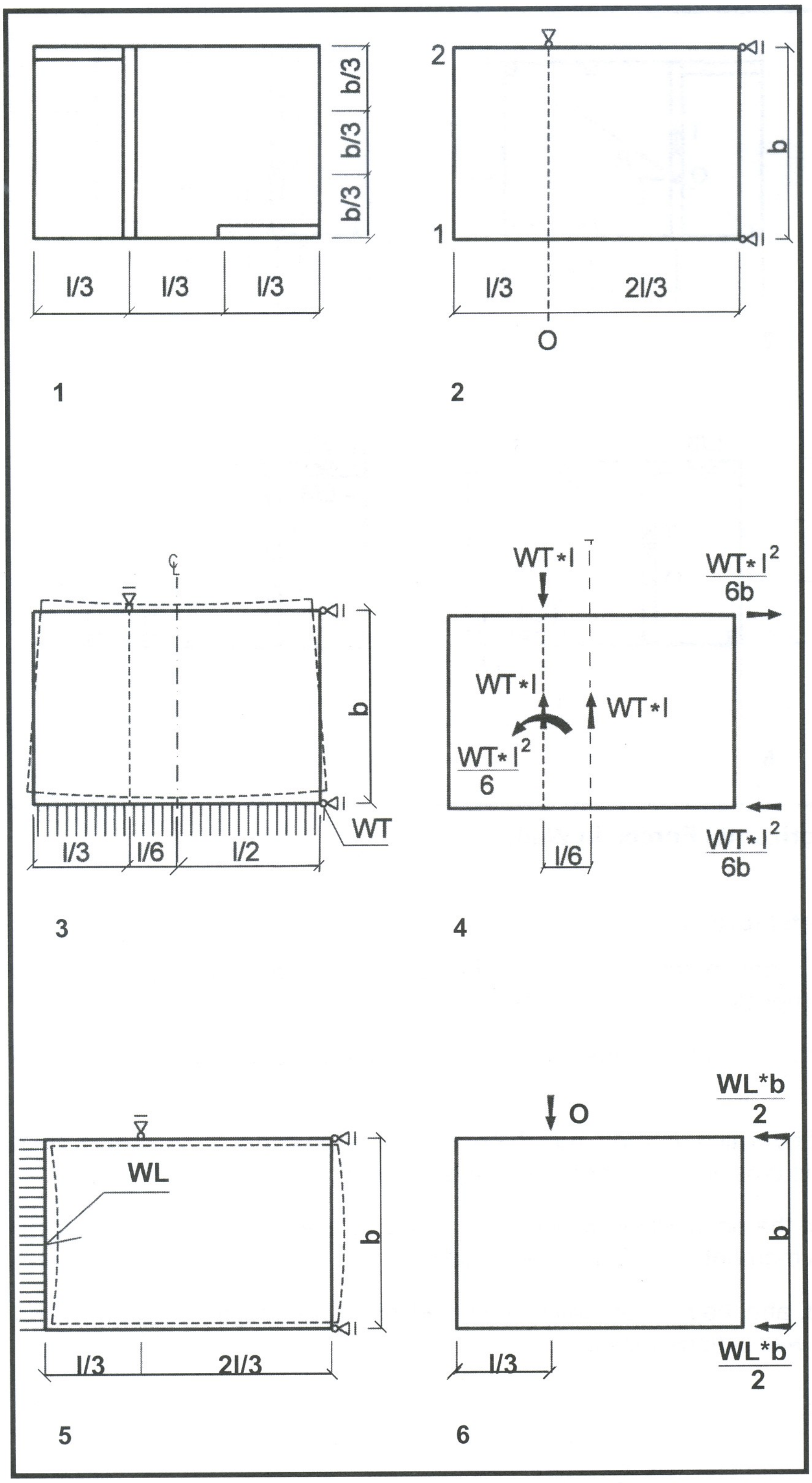


Figure 1.16 Statical Model Of Wall System For The Transfer Of Wind Load

Sketch 1 of Figure 1.16 shows the same very simple panel system as described in the previous pages. The system is statically determinate for any horizontal load acting in-plane in the floor slab.

The statical model of the floor slab for horizontal loadings is shown in Sketch 2 in which the supporting points for horizontal loads are depicted.

Transverse wind load WT is acting horizontally on the floor edge as shown in Sketch 3.

The wind WT x l, displaced from wall 0 by l/6, results in a moment WT x l x l/6 which is resolved into walls 1 and 2 with a value of WT x $l^2/6b$.

The final wall reactions from the horizontal line load WT are shown in Sketch 4 at the supporting points.

Longitudinal wind load WL is depicted in Sketch 5. This wind load acts along walls 1 and 2 with a value at WL x b/2 to each wall.

The final wall reactions from the horizontal line load WL are shown in Sketch 6 at the supports.

Design Example 1

Floor slab A in Figure 1.17 is supported by three walls which are subjected to vertical as well as horizontal forces.

The floor span is 6 m, walls 1 and 2 are 4x3 m, and wall 3 is 6x3 m. The panel system is loaded only with a longitudinal force L, acting in the gravity centre of the floor slab.

It is assumed that all walls act as shear walls and actions are displaced to supporting lines and moments are resolved into a pair of forces. The resultant forces as determined are shown in the figure.

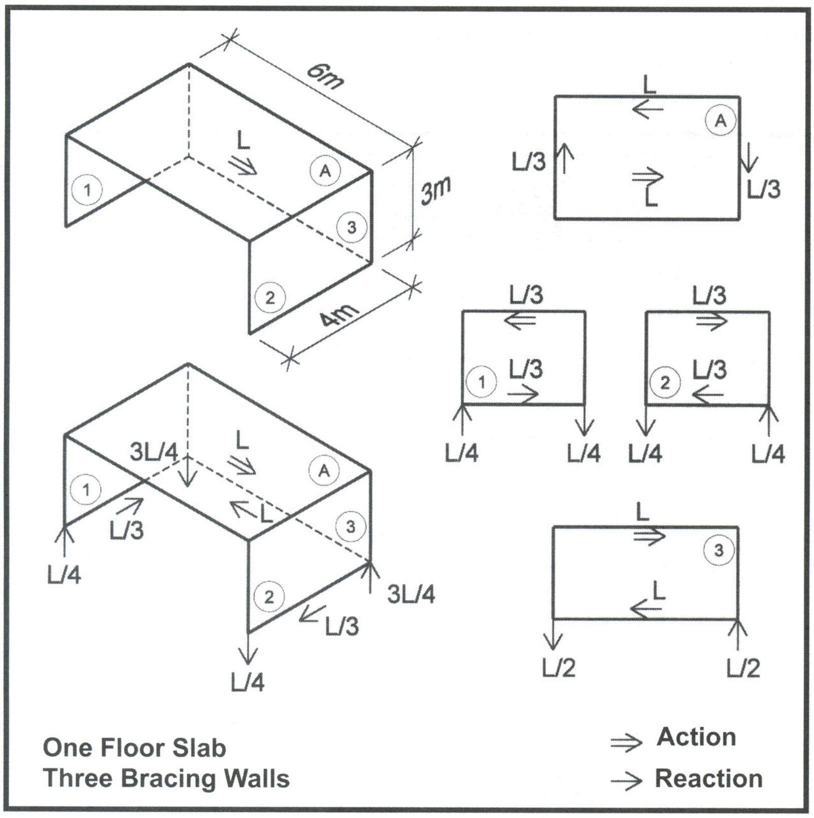


Figure 1.17 Example Of Action And Reaction Forces Due To Horizontal Load In A Determinate Slab-Wall System

Resolution Of Longitudinal Force
One floor slab interacts with three bracing shear walls which transmit the force L to the foundation by pure shear.

Design Example 2

Figure 1.18 shows one part of a building which is made up of a panel structure with load-bearing cross walls. The building is unusual, as the floor below is opened in the sense that cross walls corresponding to walls 6 and 7 are missing.

The normal floor spans at 4 m, but because of the missing walls 6 and 7, is now increased to 12 m. The structure could be considered as a three-dimensional panel structure. Floor slabs A and B together with the walls 1 to 7 act as diaphragm members.

The vertical actions on walls 6 and 7 are resolved into shear forces acting on floor slabs A and B and on the end-wall 5.

Floor slabs A and B as well as wall 5 are then considered as supported by the cross walls 3 and 4, which are, in turn, supported by cross walls 1 and 2.

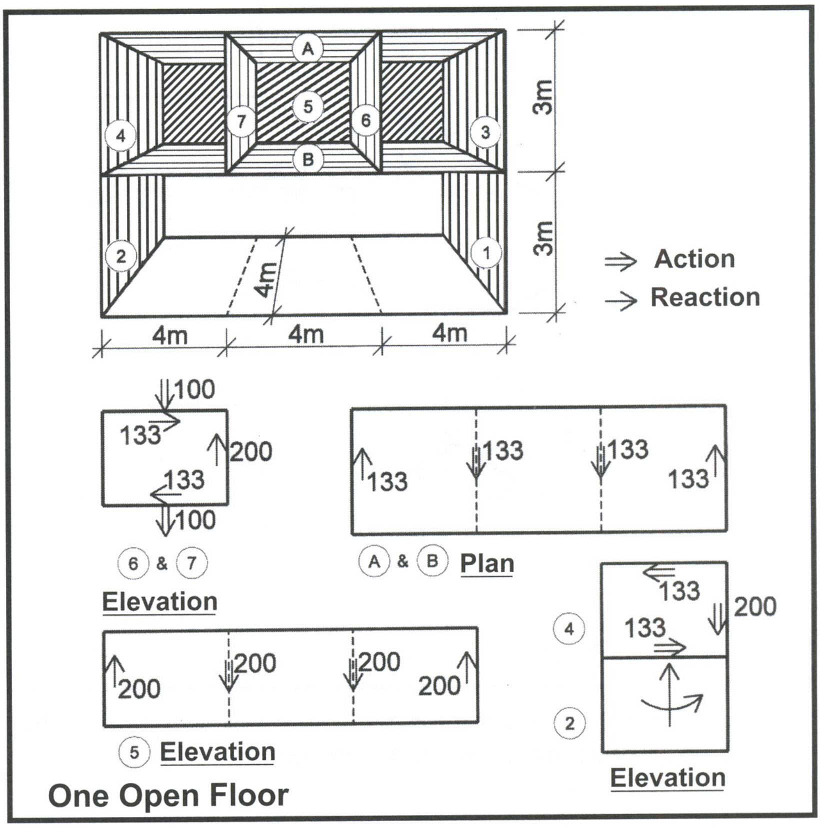


Figure 1.18 Horizontal And Vertical Load Transfer Of Determinate Discontinuous Slab-Wall System

One Open Floor
The floor slab A or B is supported by walls 6 and 7 which transmit the load to wall 5 via slabs A and B to walls 3 and 4.

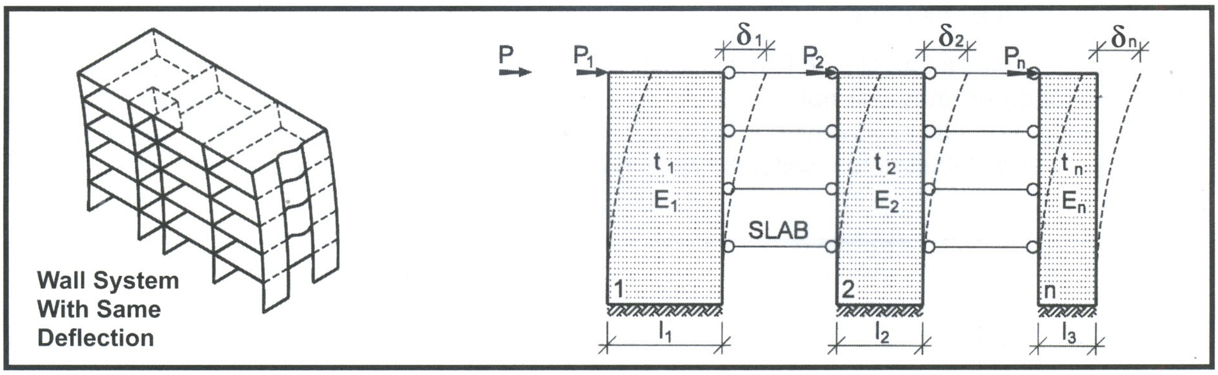


Figure 1.19 Statically Indeterminate Slab-Wall System

1.4.3 Statically indeterminate slab-wall systems

Panel systems in normal cases consist of more than three bracing walls. This means that such systems are statically indeterminate to horizontal loadings.

The wall systems may be regarded as a number of vertical beams restrained at the bottom and coupled together by the floor slabs or beams. They are assumed to be completely stiff due to horizontal deflection.

The illustrations in Figure 1.19 show symmetrical panel systems where all the bracing walls are having the same deflection at a given level, example $\delta_1 = \delta_2 = \delta_{n.}$

As a rough calculation, it is normally assumed that the deflection of the walls is due only to moment contributions. Contributions from shear can be neglected.

These assumptions lead to a distribution of the total horizontal load onto the individual wall profiles in proportion to their lateral stiffness.

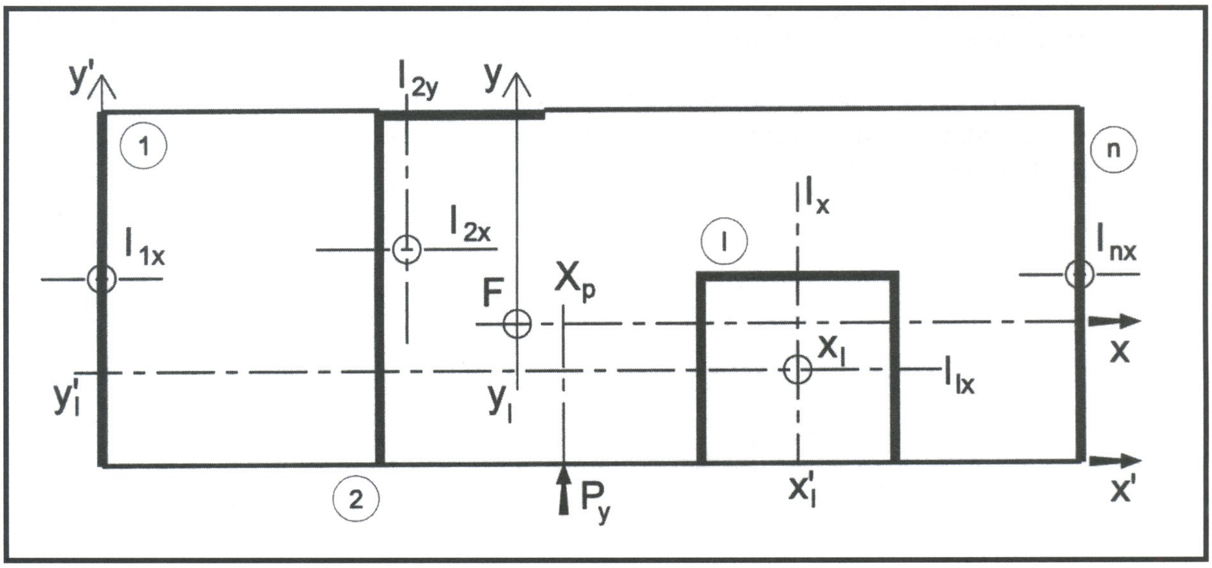


Figure 1.20 Wall System Subjected To Torsion

If a wall system is subjected to torsion as in the case of asymmetrical buildings, or if the acting horizontal loading is asymmetrical, the effect of torsion has to be taken into account. Figure 1.20 shows such a system. The distribution of horizontal load in this instance is proportional to the moments of inertia of the wall sections.

1.5 Shear Wall Behaviour

If the stiffness of wall joints or slab joints is nominal, the wall or slab structure will behave as a series of beams. Together, they will not be as stiff and will have a smaller load carrying capacity than homogeneous structures. However, if stiffness of such joints is significant as illustrated in Figure 1.21, the wall and slab structure will behave as homogeneous plates in respect of horizontal loads.

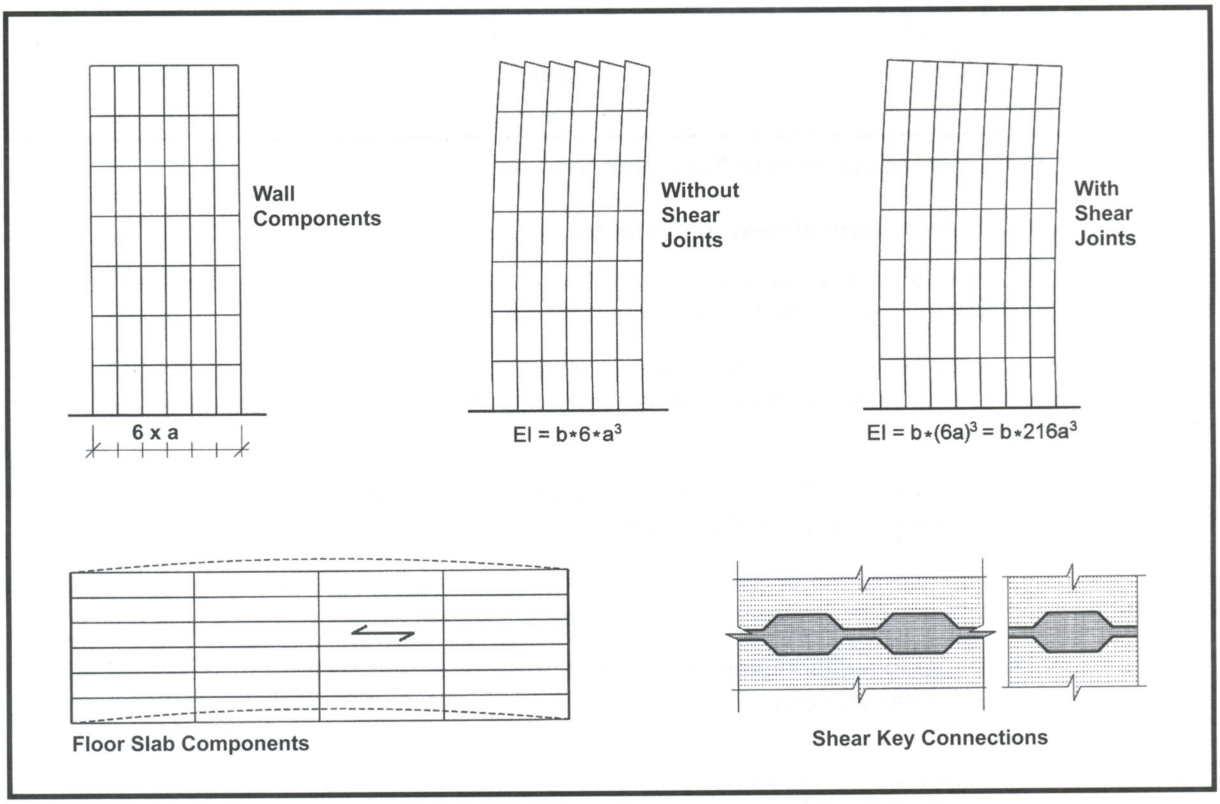
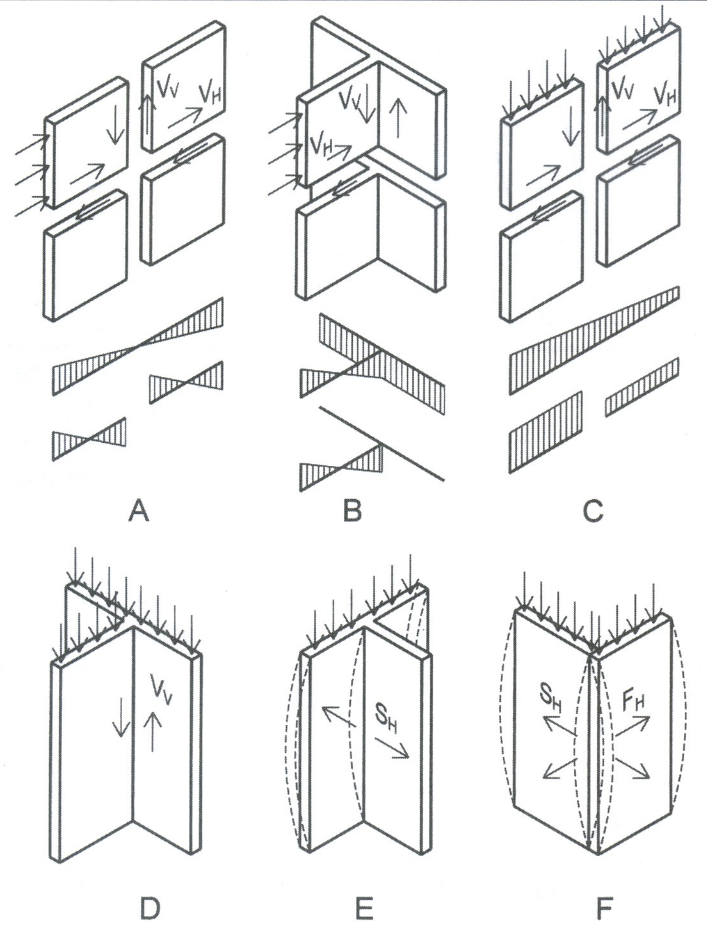


Figure 1.21 Connections And Shear Wall Behaviour

A horizontal load results in internal moment and shear force in the walls, which in turn, gives rise to normal and shear stresses in the vertical and horizontal sections as depicted in Figure 1.22. These stresses can be calculated using Navier and Grashof formulae.



A: A horizontal force calls for a horizontal and vertical shear joint if the components are assumed to interact.

B: If the two walls are not locked together with, for instance, a castellated joint, the profile will not act as a T-section.

C: The vertical load can differ from one wall to another due to differences in slab spans. This will result in shear forces at adjoining connections.

D: The vertical joint between the two walls has to resist shear forces resulting from differences in the acting vertical loads.

E: Stirrups and locking bars have to be installed to avoid possible horizontal wall deflections.

D: At wall corners, the vertical joint must be locked to withstand the risk of horizontal deflections.

Figure 1.22 Normal And Shear Stresses In Walls

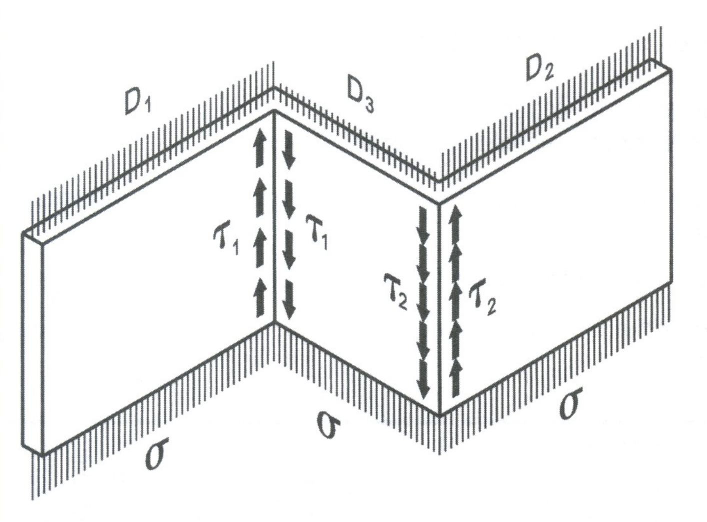


Figure 1.23 Normal And Shear Stresses In Wall Subjected To Non-uniform Vertical Load

Due to high in-plane stiffness of the wall, a non-uniformly distributed vertical load will normally result in uniformly distributed vertical stresses in the next horizontal joint. This will induce shear force in the wall to wall connections as shown in Figure 1.23. Simple equilbrium considerations can be used to determine these forces.

There are numerous connection techniques in the wall to wall joints. A popular method is the castellated joint with or without interlacing steel.

In the evaluation of castellated shear joints in bracing wall structures, the normal practice is to assume that the horizontal load on the structure can be increased until the shear stress at the most heavily loaded point of the joint reaches a permissible value. Up to this load, the wall can be assumed to be homogeneous. The horizontal bearing capacity of the structure at this loading point is also assumed to be fully utilised based on the theory of plasticity.

The ultimate load carrying capacity of castellated shear joints has to be determined by tests. To evaluate and design the joint, it is especially important to study the crack pattern and the distribution of cracks.

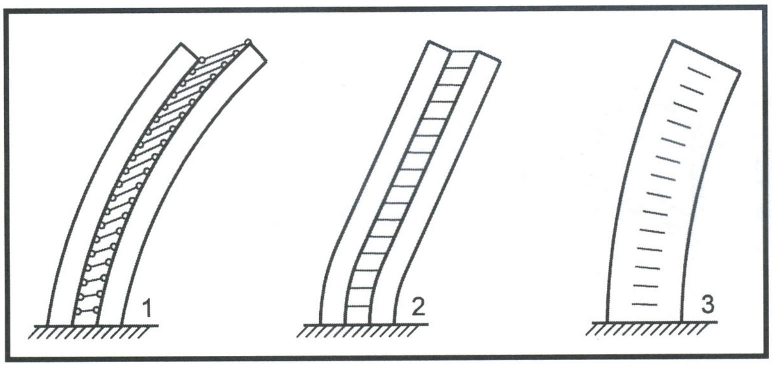


Figure 1.24 Shear Wall With Lintels

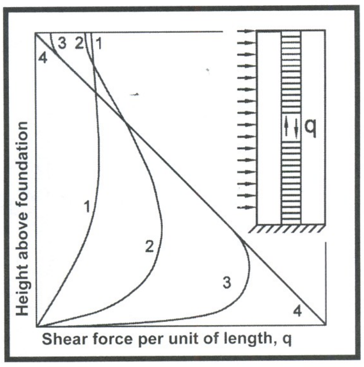


Figure 1.26 Variations Of Shear Forces Using Continuous Layer Method

1.5.1 Continuous layer method

There are various theoretical analyses of shear walls subjected to horizontal forces and one of the more common techniques is the continuous layer method. As the name suggests, the structure is simplified by making the assumption that all horizontal connecting elements are effectively smeared over the height of the building to produce an equivalent, continuous connecting layer between the vertical elements. The two dimensional planar structure is transformed into an essentially one-dimensional one.

The illustrations in Figure 1.24 show three different models of a shear wall subjected to pure bending. The two wall elements are coupled by door lintels.

1. System with completely flexible door lintels.
2. System with an elastic continuous layer in accordance with the continuous layer method.
3. System with completely stiff door lintels.

In Figure 1.25, the continuous layer method is applied to a wall with one row of doors.

1. The structural model.
2. The geometrical behaviour of the chosen structural model.
3. The main system with redundant shear forces.
4. The geometrical behaviour of the continuous layer model.
5. The main system using the continuous layer model with redundant shear.

Curves 1, 2 and 3 in Figure 1.26 show the results of calculations derived from the continuous layer method for increasing stiffness of the layer.

Curve 4 shows the result of a calculation based on the beam theory with a completely stiff layer.

For a stiff layer, there seems to be good agreement between the two theories, except at the boundary area near to the wall-foundation intersection.

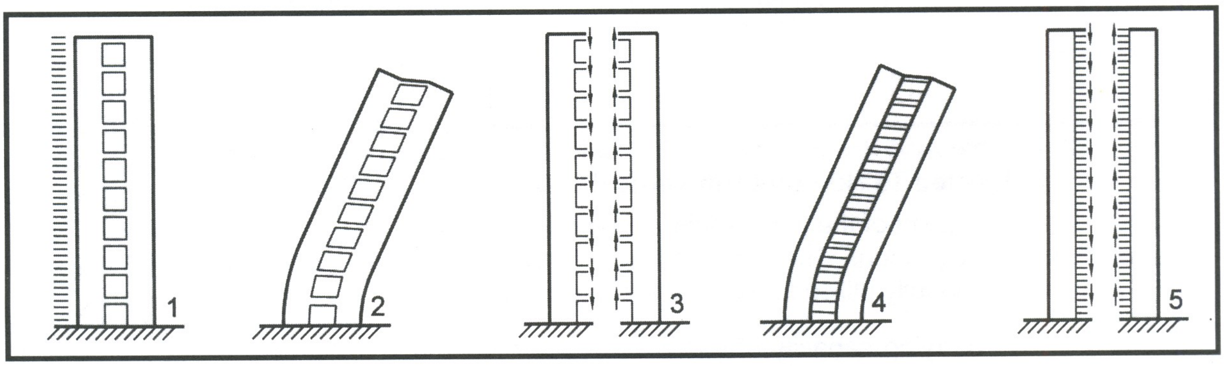


Figure 1.25 The Continuous Layer Method

The continuous layer method is reasonably accurate for uniform system of connecting beams or floor slabs. In many practical situations, the building layout will involve walls that are not uniform over their height but have changes in height, width or thickness, or in the location of openings. Such discontinuity does not lend itself to the uniform smearing of continuous layer representation and other analytical techniques such as the finite element, analogous frame etc, will need to be employed. The designer should refer to relevant literature in this matter.

1.6 Structural Integrity And Design For Progressive Collapse

1.6.1 General

The overall behaviour of a precast structure depends to a large extent on the behaviour of connections. Apart from the design of force transfer between individuals units, there should also be continuity across and ductility within the connections, structural members and of the structure as a whole. This is to ensure structural integrity which is the ability of the structure to bridge local failure.

A badly designed and/or badly detailed precast building is susceptible to progressive collapse which is a chain-reaction failure causing extensive damage or total collapse as a result of localised failure to a small portion of a structure. The failure is initiated by the so-called accidental loads which are not generally considered in the design. The accidental loadings which can be structurally significant include:

1. errors in design or construction
2. local overloading
3. service system (gas) explosion
4. bomb explosion
5. vehicular and falling material impacts
6. intense localised fire
7. foundation settlement
8. seismic effects

1.6.2 Design for progressive collapse

The most direct way to prevent progressive collapse is to reduce or eliminate the risk of accidental loadings by measures such as prohibiting gas installations or erecting barriers to prevent vehicular impact. This, however, may not be practical because every conceivable hazards must be eliminated and all accidental loading conditions fully dealt with.

CP65 adopts three alternative methods in the design for accident damages:

1. Design and protection of structural members

The design, construction and protection of structural members are covered in the clauses 2.2 and 2.6 in Parts 1 and 2 of the Code respectively. The protected members or in the Code's terminology, "key elements", are elements which include connections to adjacent members on which the stability of the structure is to depend. All other structural components that are vital to the stability of the key elements should also be considered as key elements. To prevent accidental removal, the key elements and their connections are designed to withstand an ultimate pressure of 34 kN/m^2, to which no partial safety factors should be applied. The Code recommends that key elements should be avoided as much as possible by revising the building layout within the architectural constraints.

2. Alternative load paths

In this method, which is covered in Part 2, clause 2.6.3, the beams, walls, columns or parts thereof, are considered to have failed and the loads supported by the failed members are transferred by bridging elements to other load bearing members. In designing the bridging elements, the following materials and design load safety factors may be used:

a.	Materials (Part 1, clause. 2.4.4.2)	concrete	= 1.3
		steel	= 1.0

b. Design loads (Part 1, clause. 2.4.3.2)

dead load	=	1.05
live load	=	1.0 for warehouses and industrial buildings and 0.33 for others.
wind load	=	0.33

3. Provision of structural ties

The design method is aimed at providing minimum levels of strength, continuity and ductility. It is the most commonly adopted solution to prevent progressive collapse in precast structures.

The structural ties are continuous and fully anchored tensile elements consisting of reinforcing bars or prestressing tendons (stressed or unstressed). They are placed in in-situ toppings, infill strips, pipe sleeves or joints between precast components and form a three dimensional network in the longitudinal, transverse and vertical directions as illustrated in Figure 1.27. The design of structural ties is covered in Part 1, clause 3.12.3 of the Code.

The above three methods may be employed separately or use in combination in different part of the structure. It is not permitted, however, to superimpose the effect of the three methods ie, a member must be either fully protected or fully tied and not partially protected and partially tied.

Apart from designing for progressive collapse, the Code also requires that all buildings must be robust and are capable to resist the greater of:

1. an ultimate notional horizontal load of not less than 1.5% of the total charactertisic dead weight of the structure acting at each floor or roof level simultaneously, or

2. the wind load.

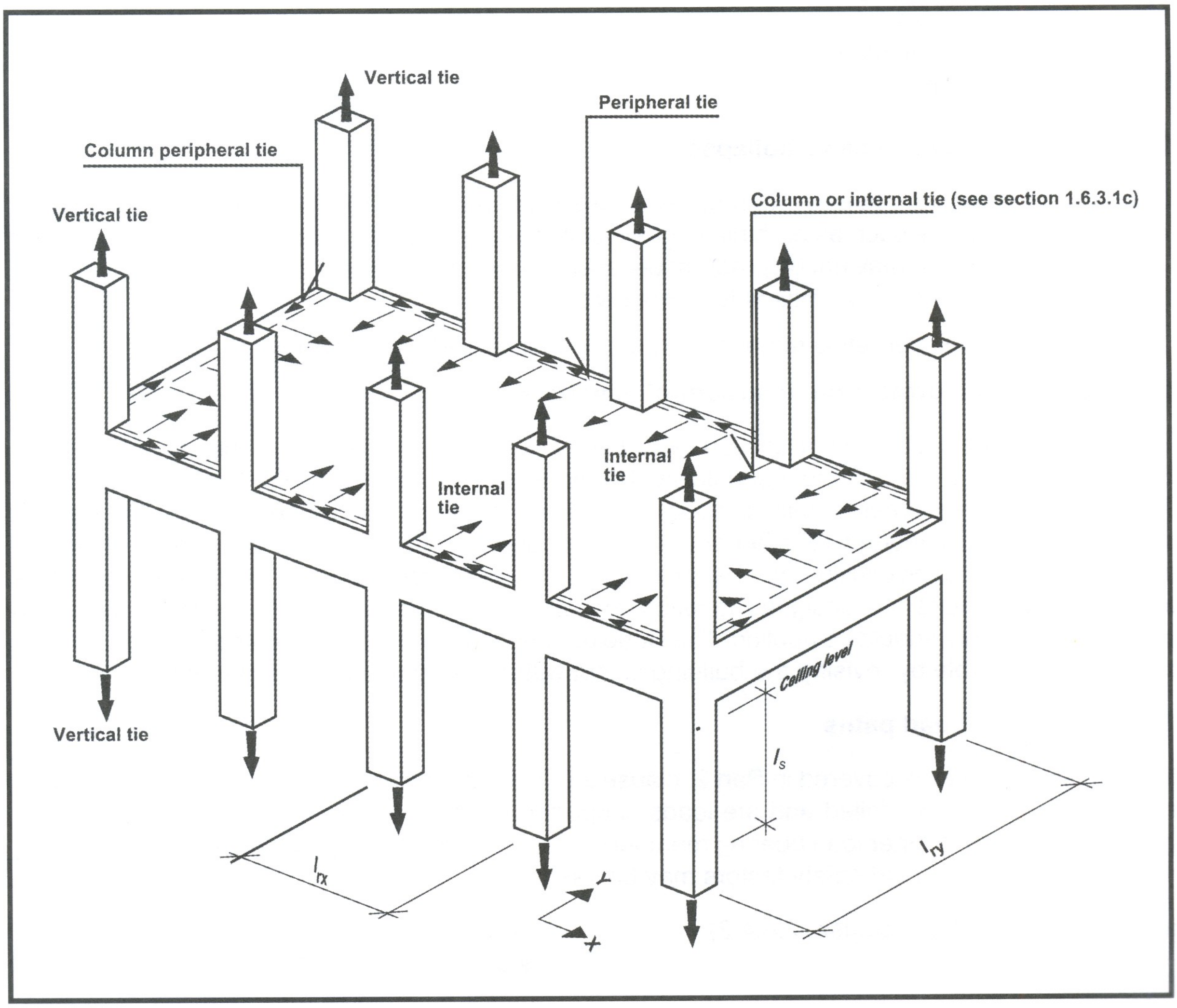


Figure 1.27 Structural Integrity Ties

1.6.3 Design of structural ties

The following ties should be provided and detailed in precast structures:

1. Horizontal floor ties

The basic tie force on each floor or roof should be the lesser of

F_t = 60 kN or
F_t = 20 + 4 x number of storey (in kN)

Horizontal floor ties are further divided into peripheral, internal and column/wall ties.

a. Peripheral Ties

Design tie force = $1.0F_t$. Peripheral ties should be located within 1.2m from the edge of a building or within the perimeter walls or beams. Perimeter reinforcement for floor diaphragm action may be considered as peripheral ties. From the maximum basic tie force of 60kN above, the steel area for peripheral ties is 130mm^2 (= 60 x 10^3 /460) which is equivalent to 1 number of T13 bar.

Structures with internal edges eg. atrium, courtyard, L- or U-shaped floor layout etc., should have peripheral ties detailed in Figure 1.28. At re-entrant corner of the perimeter, the tie reinforcement should be anchored straight inwards on both sides.

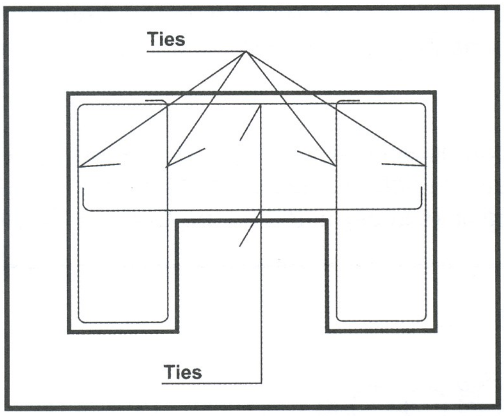


Figure 1.28 Peripheral Ties In Floor Layout With Internal Edges

b. Internal Ties

The ties are in two orthogonal directions and anchored to peripheral ties or to columns and walls. The spacing of these ties must not be greater than $1.5l_r$ where l_r is the greater distance between centres of vertical load bearing elements in the direction of the tie being considered.

The tie should be capable of resisting a tensile force equal to the greater of (in kN/m)

i.
$$\frac{g_k + q_k}{7.5} \times \frac{l_r}{5} F_t$$
$$= 0.0267(g_k + q_k)\, l_r \times F_t \qquad \text{or}$$

ii.
$$1.0 \times F_t$$

where $(g_k + q_k)$ is the sum average of the characteristic dead and imposed floor loads (kN/m^2).

The reinforcement acting as internal ties may be spaced evenly across the floor or grouped within the beams or walls as convenient.

c. Column and wall horizontal ties

Design tie force will be the greater of (in kN)

i. $2 \times F_t$ or $(l_s/2.5)F_t$ or

ii. 3% of the total vertical ultimate load carried by the column or wall at the floor or roof level being considered.

where l_s is the floor to ceiling height (m)

At corner columns, the ties are to be in each of the two directions. If peripheral ties are located within the columns and walls and the internal ties anchored to the peripheral ties, no other horizontal ties to columns and walls need be provided. Otherwise, the columns and every metre length of the walls should be tied back to the floor or roof.

2. Vertical ties

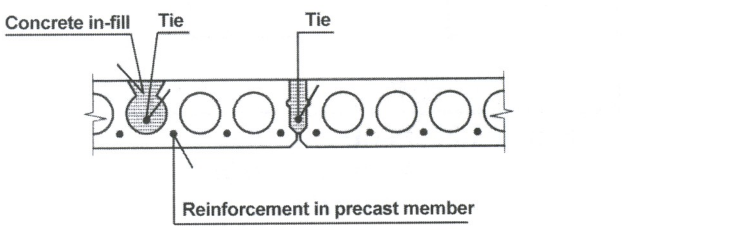
Each load bearing column and wall should be tied continuously from foundation to roof. The tie force in tension will be the maximum design ultimate dead and live load imposed on the column or wall from any one storey or the roof.

3. Proportioning of ties

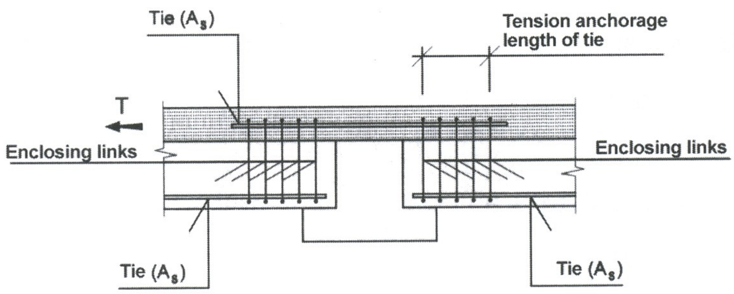
Reinforcement bars acting as ties are designed to its characteristics strength and the bars provided for other purposes may be used as part or whole of the tie requirement. Ties may be located partly or wholly within precast member as long as continuity of the tie is assured.

4. Continuity of ties

Continuity of tie reinforcement can be achieved by lapping or welding of reinforcement, or by using threaded couplers, cast-in sockets or anchors. Tie continuity created by lapping with precast member reinforcement or using enclosing links is permitted by the Code as illustrated in Figure.1.29.



(a) Bars In Precast Member Lapped With Bars In In-Situ Concrete



Minimum link area over tension anchorage of tie, $A_{sv} = \frac{T}{f_{yv}}$

$$= A_s \frac{f_y}{f_{yv}}$$

(b) Anchorage By Enclosing Links

Figure 1.29 Tie Continuity By Lapping And Enclosing Links

5. Anchorage of ties

The Code requires that the internal tie reinforcement is to be effectively anchored to that in the peripheral ties. Tie bars are considered fully anchored to the peripheral tie if they extend:

a. 12ϕ or equivalent anchorage length beyond all the bars forming the peripheral tie

b. an effective anchorage length (based on the actual force in the bar) beyond the centre-line of the bars forming the peripheral ties.

Figure 1.30 illustrates the above anchorage requirements of internal ties to peripheral ties.

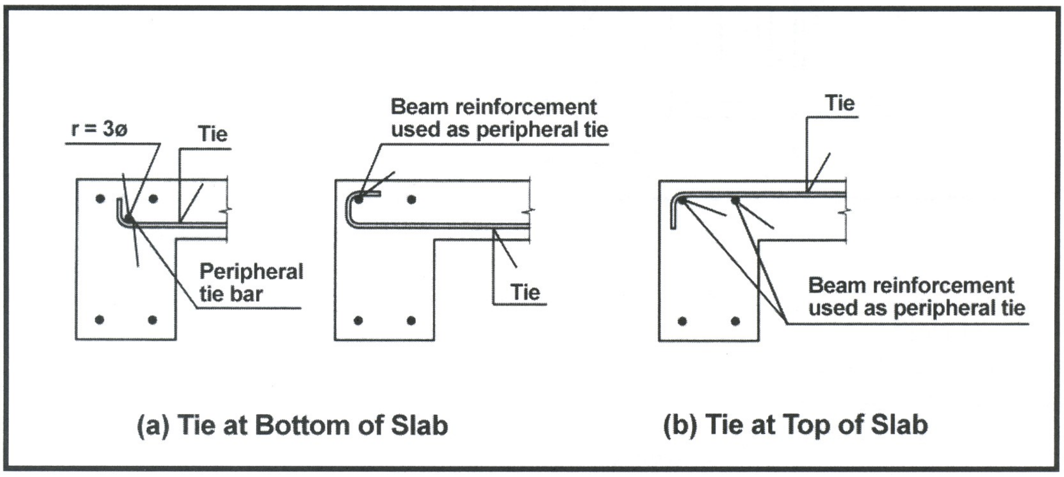


Figure 1.30 Anchorage Of Ties To Peripheral Ties

Figure 1.31 illustrates the tie backs from edge column in two orthogonal directions. It should be noted that the tie backs may also be part of the main reinforcement from the perimeter beams framing into the column.

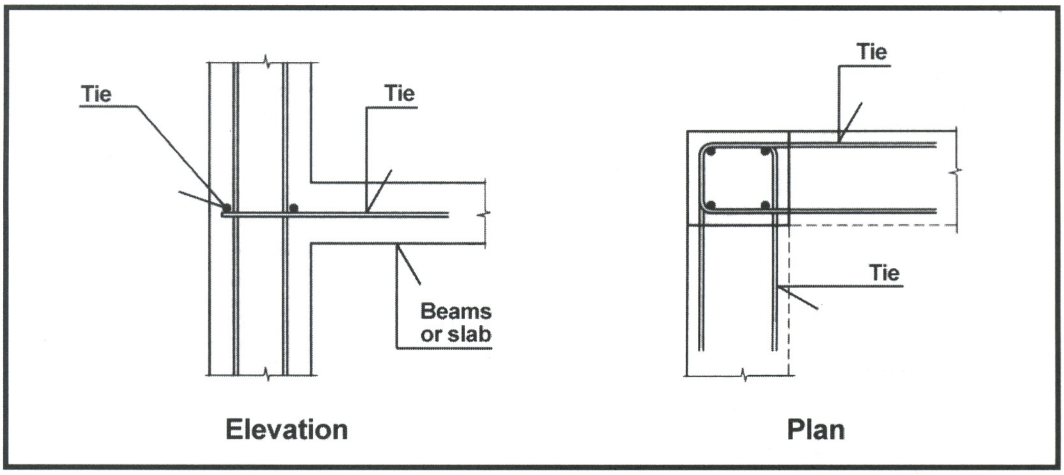
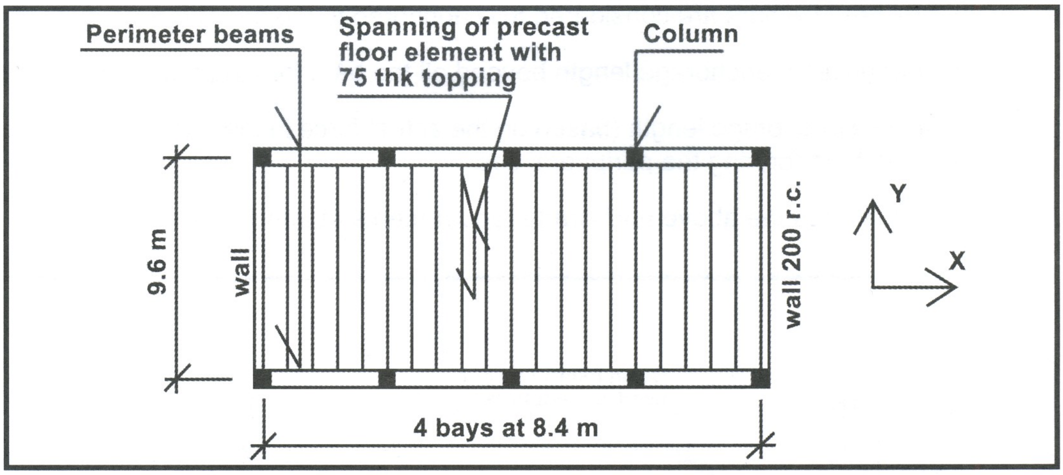


Figure 1.31 Tie Backs For Edge Columns

Design Example 3 : Structural Integrity Ties



Design Data

Total no. of storey	= 8 (including roof)
Floor to floor height	= 3.5m
Characteristics dead load, q_k	= 10 kN/m^2
Characteristics live load, g_k	= 3.5 kN/m^2
Characteristics steel strength, f_y	= 460 N/mm^2 for T-Bars
	= 485 N/mm^2 for steel mesh
Column size	= 400 x 600

1. Basic Tie Force

F_t = 60 kN or
F_t = 20 + 4 x 8
= 52 kN < 60kN
Use F_t = 52 kN

2. Horizontal Ties

a Peripheral tie

Design tie force = $1.0F_t$

Steel area, A_s = F_t/f_y
= $(1.0 \times 52 \times 10^3) / 460$
= 113 mm^2 (1T13, A_s = 132 mm^2)

b Internal Tie

Tie force the greater of
i. $0.0267(q_k + g_k)\, l_r\, F_t$
ii. $1.0F_t$

i. Ties in the X-direction :
l_r = 8.4m

Tie force = 0.0267 x (10 + 3.5) x 8.4 x 52
= 157.4 kN/m

or = 1.0 x F_t
= 1.0 x 52
= 52 kN/m

Use design tie force = 157.4 kN/m. Tie reinforcement will be provided by the steel mesh embedded within the 75mm thick concrete topping. The total steel area required:

$$A_s = 157.4 \times 10^3 / 485 = 325 \text{ mm}^2/\text{m}$$

ii. Ties in the Y-direction

$l_r = 9.6$ m

Tie force = 0.0267(10 + 3.5) x 9.6 x 52 = 180 kN/m > 52 kN/m

Use mesh within 75 mm thickness topping, steel area required:

$$A_s = 180 \times 10^3 / 485 = 371 \text{ mm}^2/\text{m}$$

Use D7 steel mesh (ϕ7 @ 100 c/c both ways) within topping (A_s = 385 mm²/m)

c Column Horizontal Ties

Total ultimate load carried by the column at 1st floor

$$N = (1.05 \times 10 + 0.33 \times 3.5) \times 8.4 \times 9.6 \times 7/2 = 3289.5 \text{ kN}$$

Design tie forces :

i. $2 \times F_t$ = 2 x 52 = 104 kN or

$l_s \times 52/2.5$ = 3.5 x 52/2.5 (assume l_s = 3.5m) = 72.8 kN

ii. 3% of ultimate vertical load = 0.03 x 3289.5 = 98.7 kN

From above, the design tie force for column F_t =98.7 kN (the greater of (i) & (ii))

$$A_s = 98.7 \times 10^3/485 = 203 \text{ mm}^2$$

According to the Code, [1]if the internal ties are properly anchored into the peripheral ties, no additional ties for column is needed. Hence D7 mesh is adequate as column tie.

The corner columns at both ends of the floor require ties in two orthogonal directions with A_s = 203 mm² in each direction. This will be provided by the main steel in perimeter beams framing into the columns.

3. Vertical Column Ties

Design tie force = ultimate load on column at the floor being considered = 3289.7/7 = 470.7 kN

$$A_s = 470.7 \times 10^3 / 460 = 1022\text{mm}^2 \ (=0.43\% \text{ of } A_c)$$

This area of tie will not be in addition to the column reinforcement if it is greater than 0.43%. If the columns are precast with floor to floor joint, the connections must be designed for the tension force of 470.7 kN and tie continuity from foundation to roof properly detailed.

1.7 Floor Diaphragm Actions

Horizontal loads on the structure are transmitted to the vertical stabilising cores, shear walls, structural frames or bracings, etc, by the floors and roofs which act as rigid horizontal diaphragms as shown in Figure 1.32.

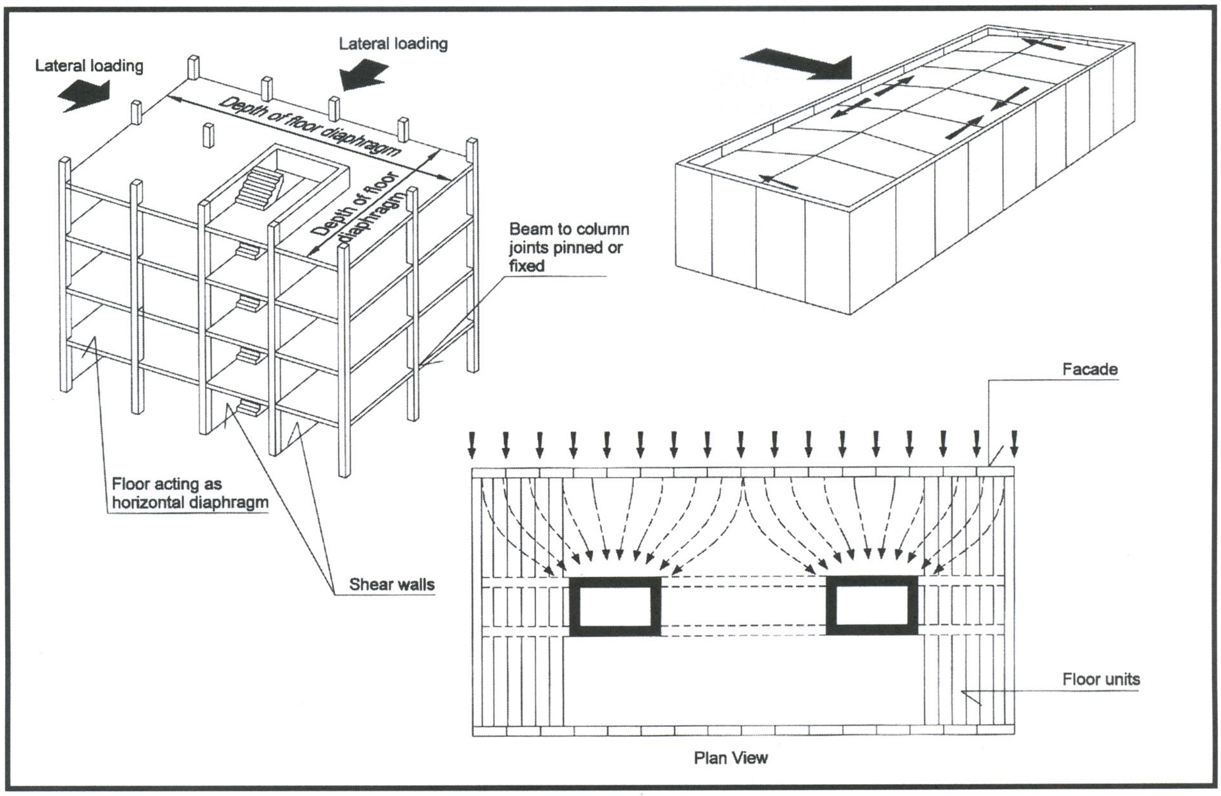


Figure 1.32 Diaphragm Action In Precast Floors And Roofs (reference 3)

1.7.1 Method of analysis

The precast concrete floor or roof is analysed by considering the slab as a deep horizontal beam, analogous to a plate girder or beam containing chord elements as shown in Figure 1.33.

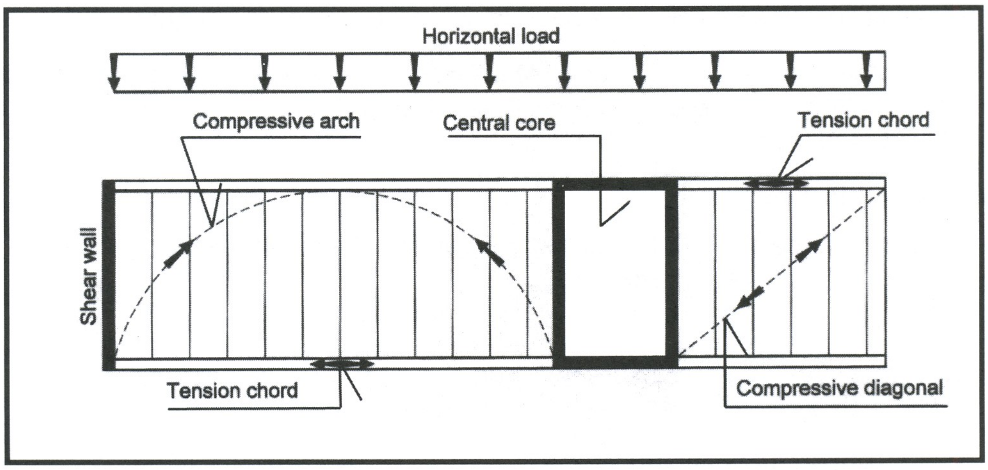


Figure 1.33 Analogous Deep Beam (reference 3)

The stabilising cores, shear walls, frames or other bracing components act as supports for this analogous deep beam and the lateral loads are transmitted as reactions.

The model for a deep beam is usually an arch and tie structure. The tensile, compressive and shear forces in the diaphragm can be calculated by normal statical method as shown in Figure 1.34.

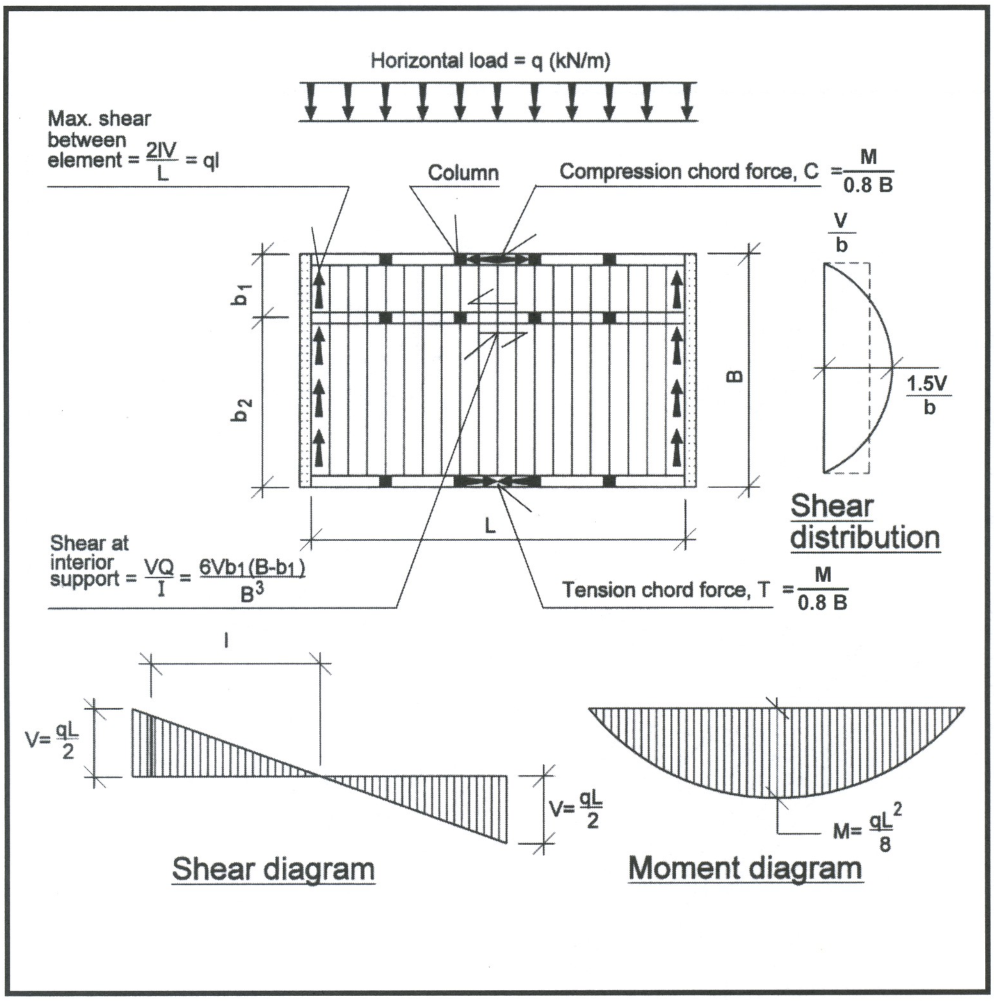


Figure 1.34 Analogous Beam Design Of A Floor Diaphragm

1.7.2 Transfer of horizontal forces

In general, the horizontal forces are transferred between precast units (Figure 1.35) by a combination of shear friction, aggregate interlocking, dowel action and mechanical welding. To resist these forces, it is necessary that the units are tied together so that shear forces can be transferred across the joints even when the units are cracked.

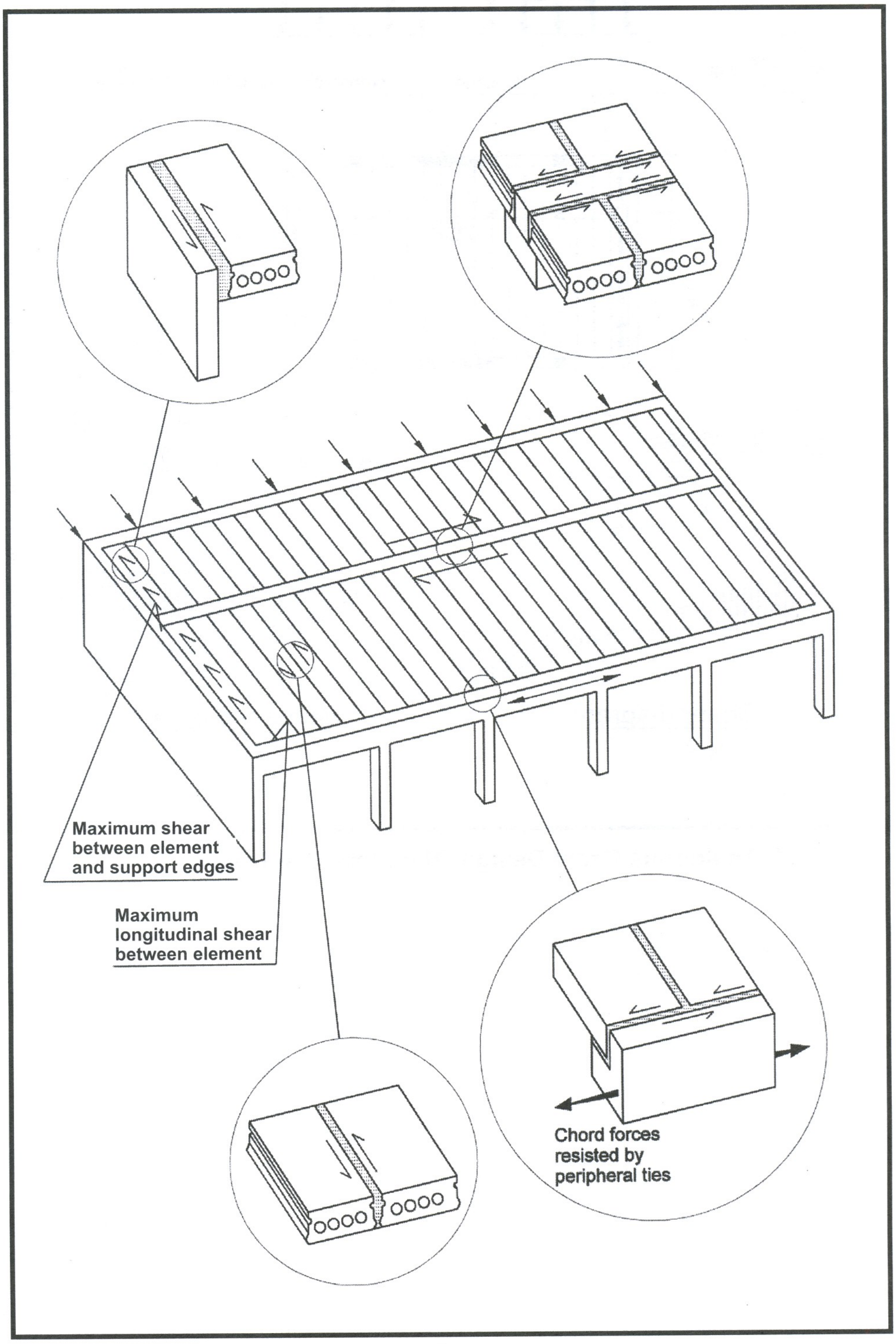


Figure 1.35 Horizontal Force Transfer

1. Chord forces

The chord forces are calculated as shown in Figure 1.34. At the floor perimeter, the chord tension is usually resisted by the peripheral ties or by the reinforcement in the perimeter beams.

2. Shear transfer between elements

The most critical sections are at the joints between the floor and the stabilising elements where the shear forces are at their maximum. Examples of joint details are shown in Figure 1.36.

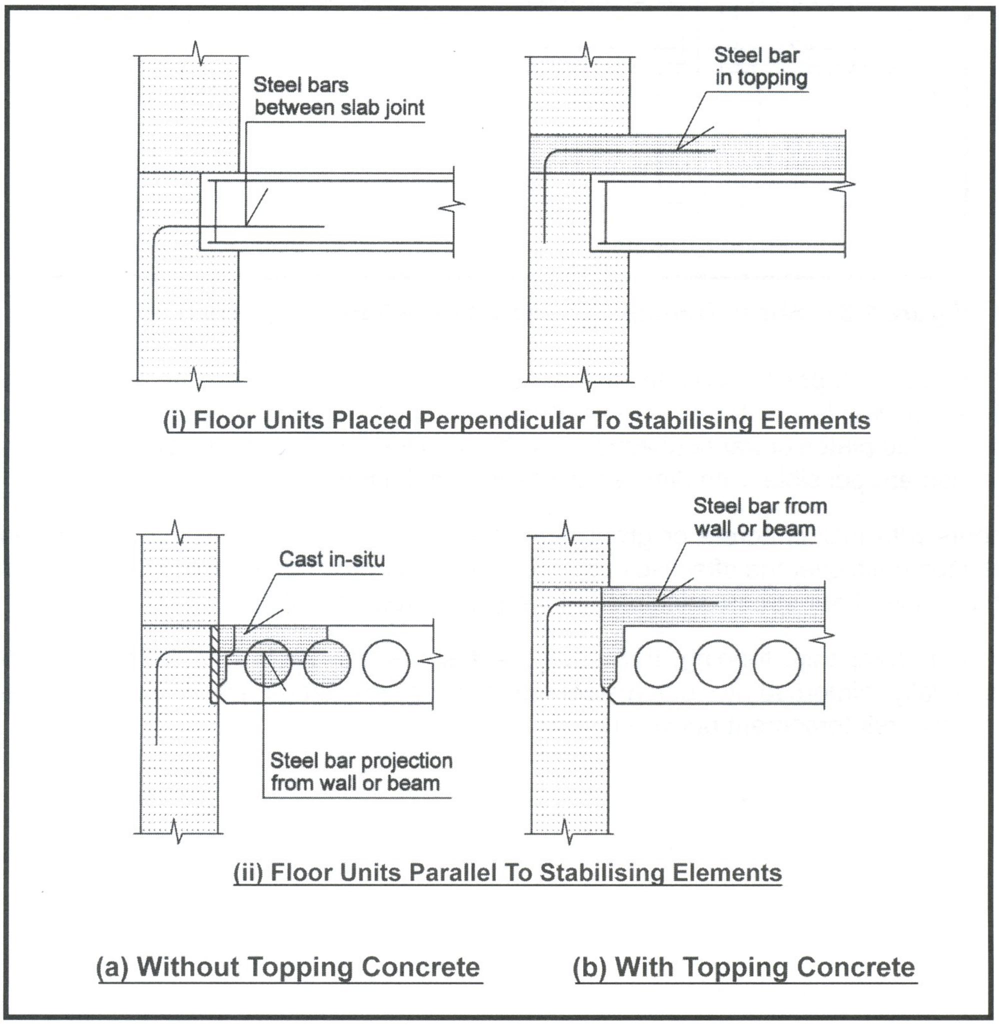


Figure 1.36 Shear Transfer Across Edge Supports

At intermediate supports, the shear force is carried across by reinforcing bars as shown in Figure 1.37. The reinforcing bars for shear transfer is usually determined by the shear friction design method. In general, the forces are quite low and only as many bars as required should be used.

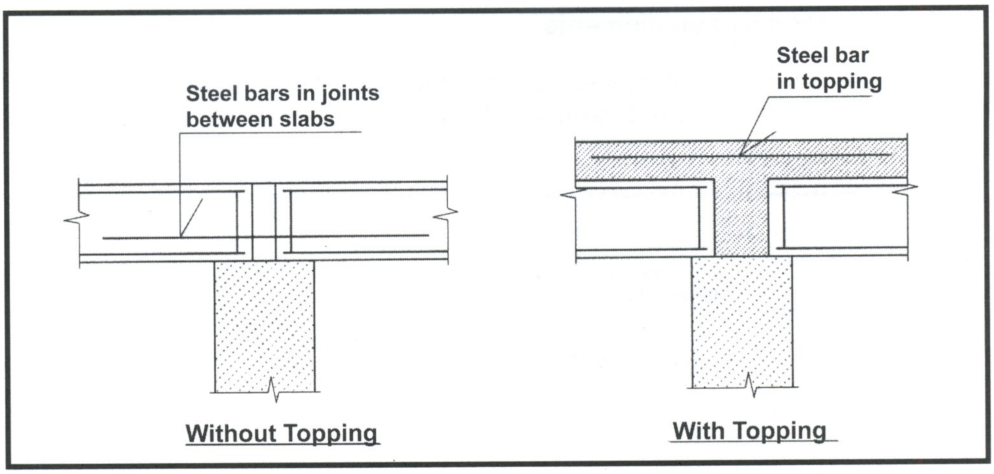


Figure 1.37 Shear Transfer Across Intermediate Support

In floors without composite topping, the longitudinal shear transfer between units is usually accomplished by welded plates or bars in flanged deck elements or grouted keys in hollow core slabs. The welded plates or bar connection may be analysed as shown in Figure 1.38. Variations of the connection are possible from different precast manufacturers.

For elements with infill concrete or grout along the joints, the design of average ultimate shear stress between units over the effective depth of the joints should not exceed 0.10 N/mm^2 (reference 4). In general, the shear stress calculated at the joint is seldom critical.

For floors with composite topping, the topping enhances the diaphragm action of the floor. The topping is usually reinforced by welded steel mesh which serves both as structural floor ties as well as shear friction reinforcement between units.

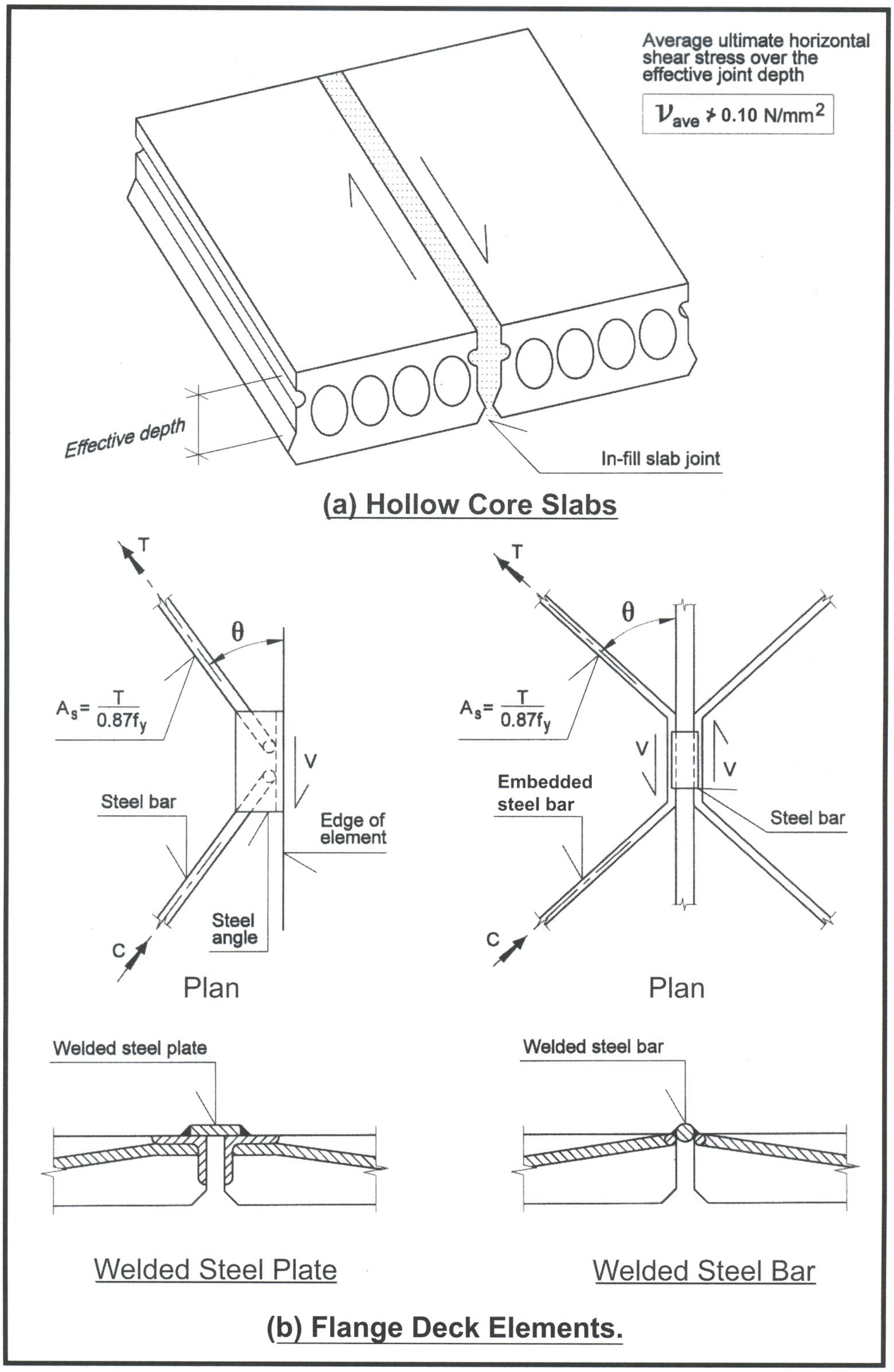
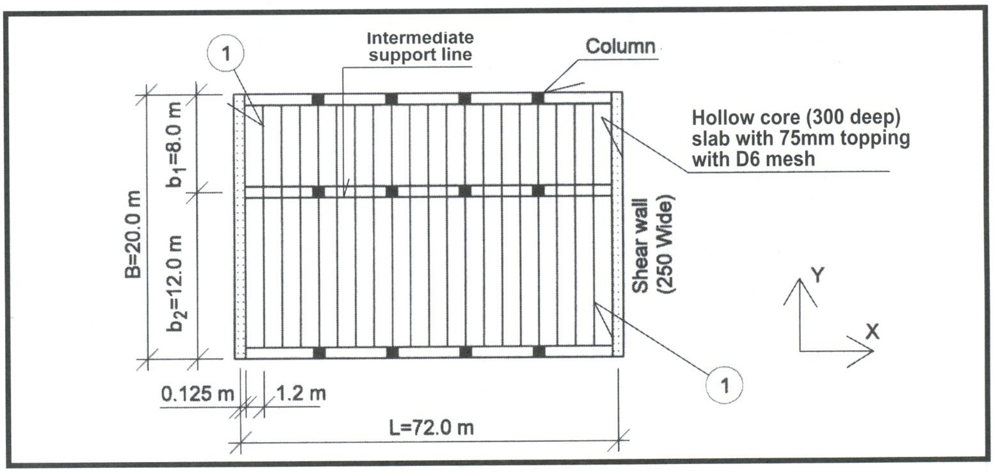


Figure 1.38 **Examples Of Shear Transfer Along Longitudinal Joint Between Elements In Floor Diaphragm Design**

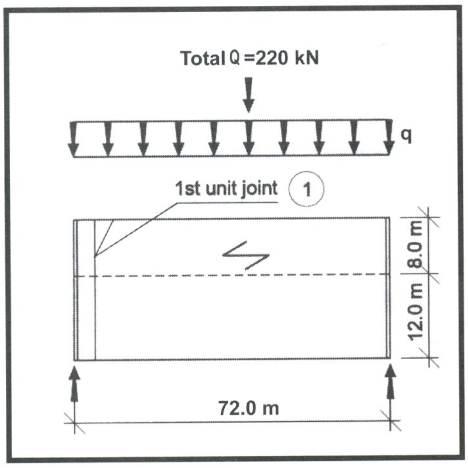
Design Example 4 : Floor Diaphragm Action

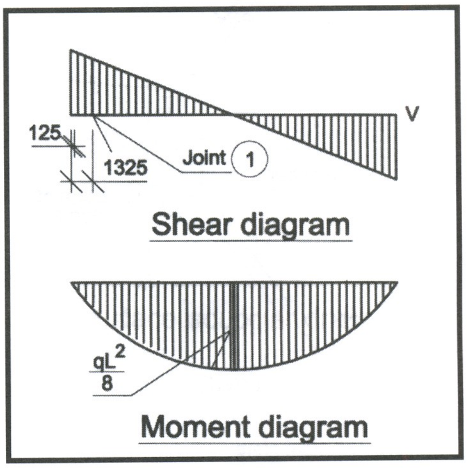
A precast floor is subjected to a total notional horizontal load of 220 kN in each orthogonal direction. The floor elements consist of 300 mm deep hollow core slabs with 75 mm thick concrete topping reinforced with D6 mesh (ϕ6 @ 100 both ways). Determine the critical forces and design the reinforcement for effective floor diaphragm action.



Typical Floor Plan

A. Notional load acting in Y-direction





1. **Shear in floor diaphragm**

a. Reaction at wall as supports :

$$V = 220/2 = 110 \text{ kN}$$

b. Shear at joint between hollow core slabs and wall :

$$V = 110 \times (36.0 - 0.125)/36.0 = 109.6 \text{ kN}$$

c. At first joint between hollow core slabs (Joint ①) :

$$V = 110 \times (36 - 1.325)/36 = 105.9 \text{ kN}$$

d. Shear at intermediate support

$$V_i = 6Vb_1(B - b_1)/B^3 = 6 \times 110 \times 8(20 - 8)/20^3 = 7.9 \text{ kN}$$

2. **Bending moment in floor diaphragm**

Maximum mid-span moment :

$$M = qL^2/8 = 220 \times 72/8 = 1980 \text{ kNm}$$

Chord forces at floor perimeter :

$$T = C = M/0.8B = 1980/(0.8 \times 20) = 123.8 \text{ kN}$$

3. **Design of reinforcement for shear transfer**

a. Across the joint between hollow core slab and wall:

$$V = 109.6 \text{ kN}$$

$$\text{Average shear} = 109.6/20 = 5.5 \text{ kN/m}$$

$$A_s = V/(0.87f_y) = 5.5 \times 10^3/(0.87 \times 460) = 14 \text{ mm}^2\text{/m}$$

Note : No additional reinforcement needed if D6 mesh in topping concrete continues and anchored into the walls.

b. Joint location 1

$$V = 105.9 \text{ kN}$$
$$\text{Average shear} = 105.9/20 = 5.3 \text{ kN/m}$$

Check horizontal shear stress at hollow core joint :

$$\text{Assume effective depth} \approx 0.8h = 0.8 \times 300 = 240 \text{ mm}$$

$$\text{Average shear stress} = (5.3 \times 10^3/240) \times 10^{-3} = 0.02 \text{ N/mm}^2 < 0.10 \text{ N/mm}^2 \quad \text{O K}$$

No additional steel needed.

c. At intermediate support

$$V_t = 7.9 \text{ kN/m}$$
$$A_s = 7.9 \times 10^3/(0.87 \times 460) = 20 \text{ mm}^2\text{/m}$$

No additional steel needed as D6 mesh (A_s = 283 mm²/m) is placed continuous throughout the floor.

d. Peripheral chord ties

$$\text{Tension chord forces} = 123.8 \text{ kN}$$

$$A_s = 123.8 \times 10^3/(0.87 \times 460) = 309 \text{ mm}^2$$

Note : The required A_s should be compared with the peripheral tie reinforcement as calculated from Part 1 clause.3.12.3.5, of the Code. In most instances, no additional steel is specifically needed for diaphragm action at the perimeters.

Separate calculations may need to be carried out for notional load acting in the X-direction. The computation is similar as above. It can easily be shown that the floor diaghragm action is adequately ensured by the D6 steel mesh placed within the concrete topping.