DESIGN GUIDE

Precast Concrete Building Concepts

Written by Barry Crisp

HOLLOWCORE
HOLLOW CORE CONCRETE PTY LTD

Ph: (03) 9369 4944
www.hollowcore.com.au
@hollowcoreconcrete

FOREWARD

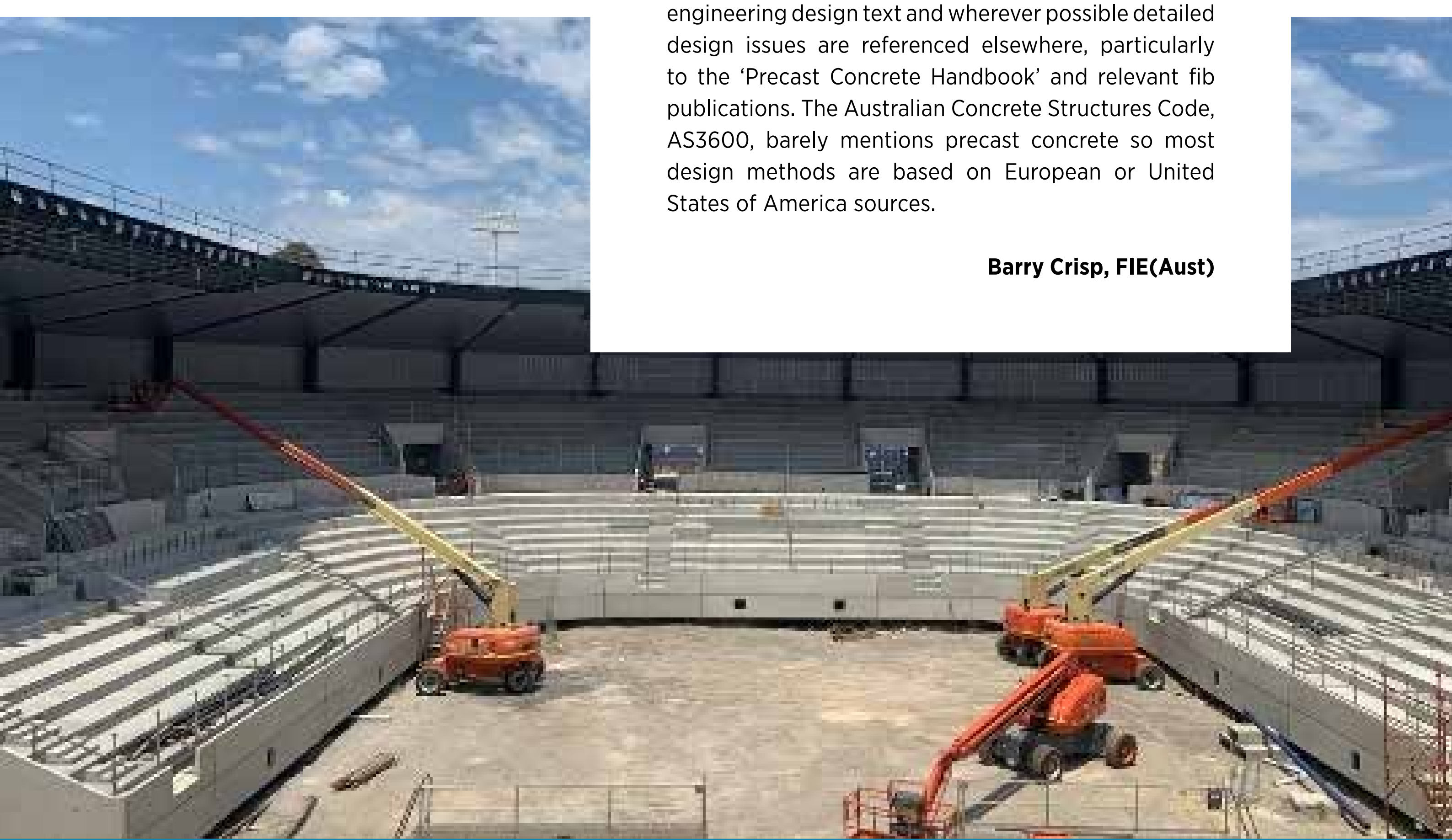
This document is intended to show how these precast concrete elements are incorporated into the various types of precast concrete structures that are built in Australia. In other words, how precast concrete elements are assembled to construct complete buildings.

This document is inspired by and is based on and the 'Planning and Design Handbook on Precast Building Structures', published by the fib [1] Commission on Prefabrication, Task Group 6.12, convened by Arnold Van Acker. I wish to acknowledge the input provided by Mr. Van Acker and Dr Kim Elliott and fellow members of fib Commission 6 and to the authors and publishers of other fib documents that have been used as source material. Without their input this document would not have eventuated.

This document has also been written as a companion document to the National Precast Concrete Association Australia 'Precast Concrete Handbook' [2].

This is a practical 'how to' document rather than an engineering design text and wherever possible detailed design issues are referenced elsewhere, particularly to the 'Precast Concrete Handbook' and relevant fib publications. The Australian Concrete Structures Code, AS3600, barely mentions precast concrete so most design methods are based on European or United States of America sources.

Barry Crisp, FIE(Aust)

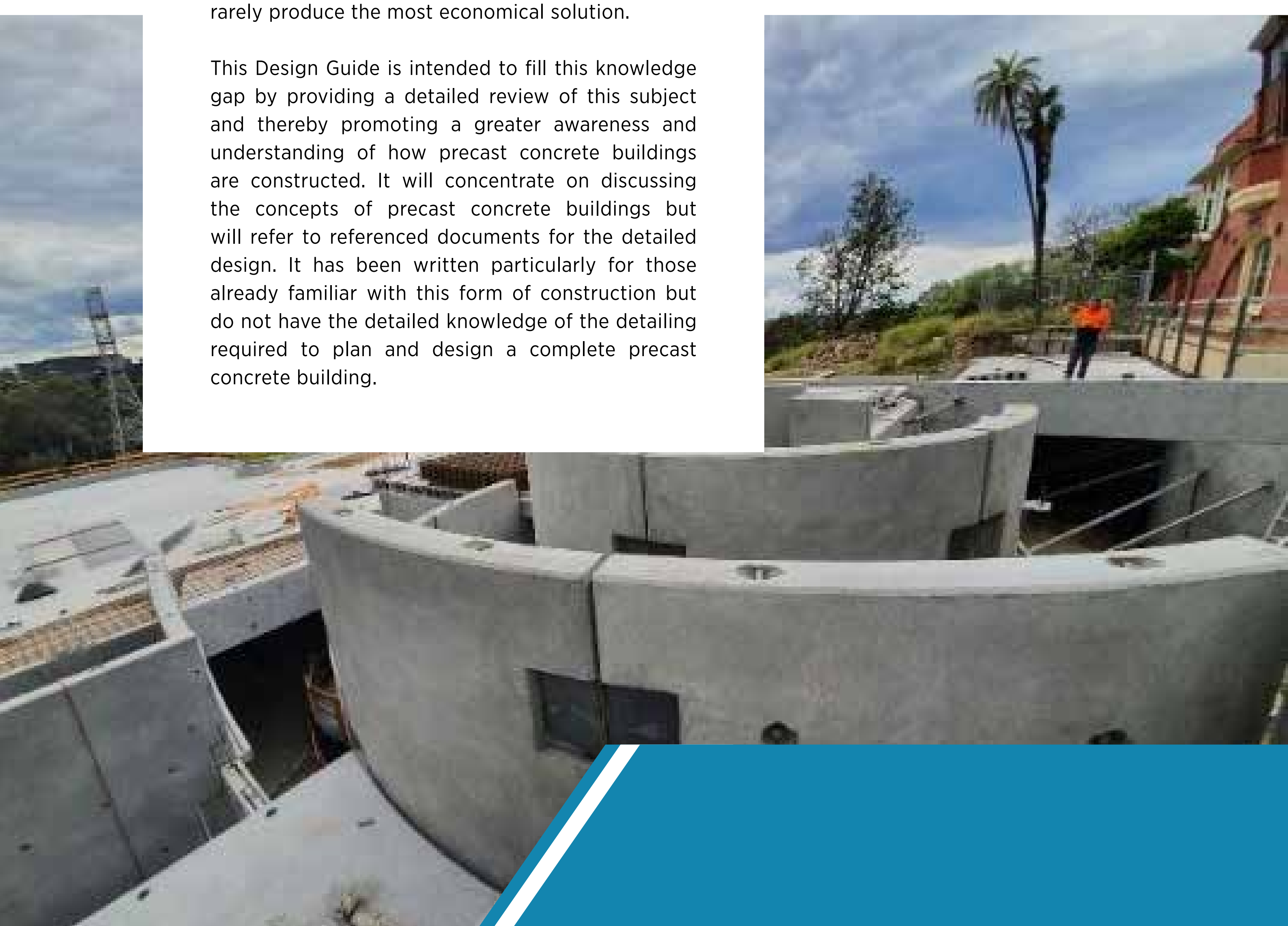


INTRODUCTION

The use of precast concrete in building construction is an economical, durable, environmentally friendly, structurally sound and architecturally versatile form of construction. The precast concrete industry is continuously making efforts to meet the demands of modern society such as environmental friendliness, off-site prefabrication, affordability, technical performance and safety.

Prefabrication of concrete structures is an industrialised process. Like every construction system, prefabrication has its own characteristics, which to a greater or lesser extent influence the structural lay-out, span width, stability, etc. For the best results a design should, from the very outset, respect the specific and particular demands of the intended structural system. Unfortunately, prefabrication of concrete is often considered by uninitiated designers as a variant of cast in-situ construction whereby only parts of the structure are precast in specialised plants. Assembled afterwards on site in such a way that the initial concept of a cast in-situ structure is emulated. This approach will rarely produce the most economical solution.

This Design Guide is intended to fill this knowledge gap by providing a detailed review of this subject and thereby promoting a greater awareness and understanding of how precast concrete buildings are constructed. It will concentrate on discussing the concepts of precast concrete buildings but will refer to referenced documents for the detailed design. It has been written particularly for those already familiar with this form of construction but do not have the detailed knowledge of the detailing required to plan and design a complete precast concrete building.



CONTENTS

1 CHARACTERISTICS OF PRECAST CONCRETE

1.1 General

In this Chapter

Although precast and insitu concrete buildings are constructed from the same basic materials, namely concrete and steel, the design methods, construction process and the in-service performance of the finished structures are completely different.

The advantages of building in precast concrete include, manufacture off site in a stable working environment, economy of the project, speed of erection and low labour demand on site.

The main characteristic of precast concrete buildings is they comprise a series of discrete elements assembled and connected on site to form a complete structure. Connection between elements is critical and continuity between elements can be difficult to achieve. Realisation of a three-dimensional framework is seldom applied unless specifically required. Stability can be provided by in-plane stiffness of shear walls, braced frames, restraint of columns in foundations, diagonal bracing, floor and roof diaphragms, or combinations of the above systems.

Conversely cast in-situ concrete structures are monolithic and behave as three-dimensional frameworks. Continuity of reinforcement through the joints results in bending and torsional moments, along with shear and axial forces being distributed throughout the structure by frame action.

A common misconception is that precast concrete has little or no flexibility in design. Modern precast concrete buildings can be designed safely and economically, with a variety of plans (figure 1.1) and with considerable variation in the treatment of the elevations (figure 1.2). Technically there is no limit to the building height that can be constructed in precast concrete; the limitation is generally one of logistics and material handling on site.

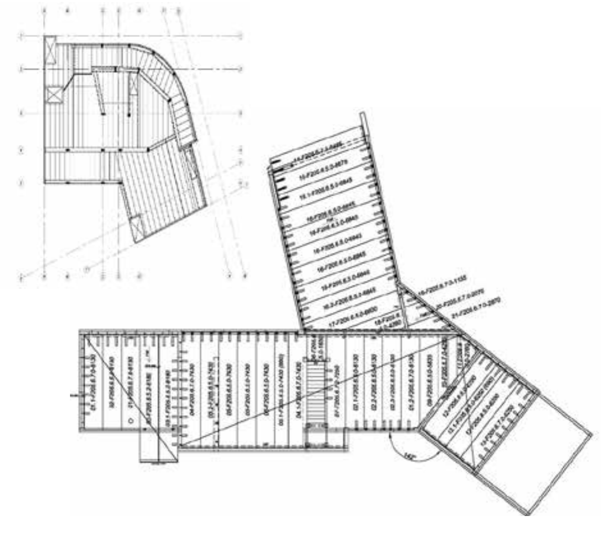


Figure 1.1 Irregular Layouts. complex plans with simple framing

Figure 1.2 Complex elevations in precast concrete

1.2 Advantages & Limitations

Most buildings are suitable for construction in precast concrete either in part or as a whole. Buildings with an orthogonal plan are of course ideal for precasting, because they exhibit a degree of regularity and repetition in their structural grid and spans and member size. However irregular layouts are on many occasions equally suitable for precasting, if not totally, then certainly partially. Irrespective, when designing any building, the design team should always strive for standardisation and repetition in the context of economical construction, not only in precast concrete, but in any design. Precasting should not be ignored at the concept design stage of buildings, either for the total structure or for parts of the structure when mixed with other materials.

A major advantage of precasting is that the manufacturing process is performed under controlled working conditions in a factory environment. Material handling is optimised by the industrialised process and quality control is centralised. The effects of weather on production and quality are eliminated and the workforce can be arranged into specialised teams to perform specific tasks. Down time can be minimised leading to greater productivity and more economical production.

A perceived limitation concerns the lack of structural flexibility, the multiplicity of precast elements offered by different manufacturers and that long lead-in times are necessary to allow for the design and manufacture. Such misconceptions are not justified in Australia where most manufacturers either have in-house designers and/or work closely with their consulting engineer to rapidly produce concept and final design solutions to the requirements of the project consultants. Thanks also to computer aided design, building information modelling (BIM) and modern production and manufacturing techniques, flexibility combined with short delivery times has become a major commercial advantage of prefabrication.

One limitation of a precast skeletal frame compared with insitu concrete is beam depth. To erect a precast frame without falsework requires beam depths typically greater than those of an equivalent insitu beam poured on falsework. Although this has the potential to add to floor to floor height this can be minimised or eliminated by co-ordination of the structure with services requirements to produce an appropriate overall solution.

Another major difference between precast and cast in-situ concrete lies in the fact that everything to be cast into the precast elements needs to be planned at an earlier stage. The current method of defining a broad outline and leaving the detailed design to later does not work with prefabrication. Both the architect and services engineer must be ready to define their requirements in time for the precaster to prepare drawings. Therefore, the final study of the building services has to be made earlier than in conventional construction, but this could be seen equally as an advantage.

For wall structures precast concrete offers considerable advantages over masonry or lightweight construction by allowing for rapid construction of the structure combined with improved thermal and acoustic performance.

Precast buildings can also be erected without a fully enclosed perimeter protection system because perimeter handrails can be pre-installed and erected with the precast elements.

Figure 1.3 Flexibility in design allows irregular Buildings

1.3 Opportunities with prefabrication

Compared with traditional construction methods and other building materials, prefabrication as a construction method, and concrete as a material, have a number of positive features. It is an industrialized way of construction, with inherent advantages:

- **Factory made products**
  The only way to industrialize the construction process is to shift the work from the site to permanent factories. Factory production means rational and efficient manufacturing processes under controlled conditions, skilled workers, repetition of actions, quality surveillance, etc. Automation of production is an area offering significant advantages. Examples already exist in Europe where preparation of the reinforcement, assembly of moulds, concrete casting, surface finishing and de-moulding is fully automated.

- **Optimum use of materials**
  Prefabrication has much greater potential for economy, structural performance and durability than cast in-situ construction because of the higher potential to optimise materials in individual elements. Additives and admixtures are used in the mix design to obtain the specific mechanical performances needed for each product. Casting and compaction of the concrete is performed under controlled working conditions with optimum equipment. The water content can be reduced to a minimum, and compaction and curing are done in controlled environments.

- **High performance concrete**
  High performance concretes, including high flow or self-compacting concrete (SCC) have been developed over the last decade and have a significant beneficial impact on the production process.

  SCC needs no or minimal vibration and thus minimises labour requirements and creates advantages such as low noise level during casting, less mould pressure, rapid casting, easy casting when using dense reinforcement or with thin or complicated cross-sections, less air pores at the surface and easy pumping.

  High strength concrete with cylinder strength greater than 65MPa is common in prefabrication and most factories are using it daily. The major benefit for building structures concerns the improved structural efficiency enabling more slender elements and optimum use of materials.

- **Prestressing**
  Pre-tensioning of steel tendons, either small diameter wires or compound helical strands, are often applied in precasting due to the ability to use long-line prestressing beds and tendons anchored by bond. This technique gives not only all the construction advantages of prestressed concrete, but also improves economy at manufacture because of the low labour input and the absence of expensive anchorage devices and duct grouting required in post-tensioning.

- **Quality**
  The term quality has a broad meaning, the final aim being to supply products and services that meet the expectations of the customer. Prefabrication in a controlled environment offers the customer marked advantages with respect to assuring that the products and services provided meet their expectations. Factory quality control consists of procedures, instructions, regular inspections, tests and the utilization of the results to control equipment, raw materials, other incoming materials, the production process and the final elements. This process is usually based on a system of self-control, with or without supervision by a third party. The results of the process are recorded and available to customers.

- **Good, clean architecture**
  The design of a precast concrete building need not be fixed by rigid concrete elements and almost every building can be adapted to the requirements of the architect or the builder. Prefabrication no longer means industrialised production of large numbers of identical units; on the contrary, an efficient production process can be combined with skilled design and workmanship to permit a modern architectural design without extra cost.

- **Structural efficiency**
  Precast concrete offers considerable scope for improving structural efficiency. In office buildings the trend is to construct large open spaces. Long spans and shallow construction depths can be obtained by using precast prestressed concrete for beams and floors. For car parks, precast concrete can provide occupiers with greater parking densities, because of the large span possibilities and fewer columns. In industrial buildings, perimeter walls can be loadbearing to support the roof structure and so eliminate internal wall framing. In structures such as medium density apartment buildings, wall panels can be loadbearing to support precast floors, eliminating column and beam framing systems.

- **Flexibility and adaptability**
  Certain types of buildings are frequently required to be adaptable to the user's changing needs. The solution to this problem lies in designing a building that can accommodate future renovations or refurbishment without demolition of the structure. The most suitable solution to this is to create a large free internal space. This is possible with precast floors that can span 12-16 metres without a significant cost penalty compared with shorter spans. On apartment buildings, precast floors can span the clear distance across the apartment, allowing all walls within the apartment to be non-loadbearing and therefore be easily relocated or moved at any time.

  Compliance with the Code of Practice, Tilt-up and Precast Concrete in Building Construction, [3] requires buildings to be designed so that they can be dis-assembled and will lead to the ability to re-cycle precast concrete building elements. This is already occurring on industrial buildings where wall panels are being re-used when buildings are extended.

- **Fire resistant construction**
  Precast structures in reinforced and prestressed concrete normally have a fire resistance rating of 60 to 120 minutes or more. This is easily obtained by increasing the concrete cover to the reinforcement to comply with code requirements.

- **Shorter construction time**
  Projects are becoming more and more complex, leading to longer construction cycles for cast insitu construction. Prefabrication shortens activity onsite allowing work to proceed both on and off site

- **Environmentally friendly way of building** Preserving the environment is becoming globally increasingly important. In the context of environmental friendly construction, the precast concrete industry is showing the way by reduced use of materials, reduced use of energy, increased use of recycled material and reduced waste at demolition.

  In future, all waste materials from building sites will be required to be recycled. In Europe some plants are already working as a closed production system, in which all waste material is processed and reused. This is being driven by the requirement to account for life-cycle costing in establishing green energy credentials.

- **Appearance and surface finishing**
  Precast concrete components can be produced with a wide variety of finishes. These range from carefully moulded surfaces to high quality visual concrete. Considerable architectural freedom and range of expression can be obtained. This technique is called architectural precast concrete, to indicate that the material and the way of production and application contribute to the architectural and aesthetic function of the project.

- **Transport and site erection**

  Transportation of precast elements to site is normally by truck with the majority of precast elements being used within a few 10's of kilometres of the manufacturing location. The maximum economical distances for transport will vary depending on the type and weight of elements, traffic infrastructure, commercial competition etc but can range up to several hundred or more kilometres. There are limitations on the size of the elements that can be transported and these must be established before commencing the final design.

  The erection procedure is closely related to the maximum weight of the elements and depends on accessibility to the site and the capacity of the lifting crane. Establishing the maximum possible weight and size of the elements requires close coordination with the proposed erection process and should be determined before commencing the final design.

- **Building services**

  Building service requirements can be integrated into the precast building system. Elements can be provided with a variety of holes, fixings can be cast in the units, and in some instances non-structural finishes can be incorporated and erected with the precast element. Pre-glazed façade elements are a good example of this.

Precasting also offers certain advantages with respect to building performance. For example, thermal mass of concrete is being used to store thermal energy in hollow core floors, leading to substantial savings on heating and cooling costs.

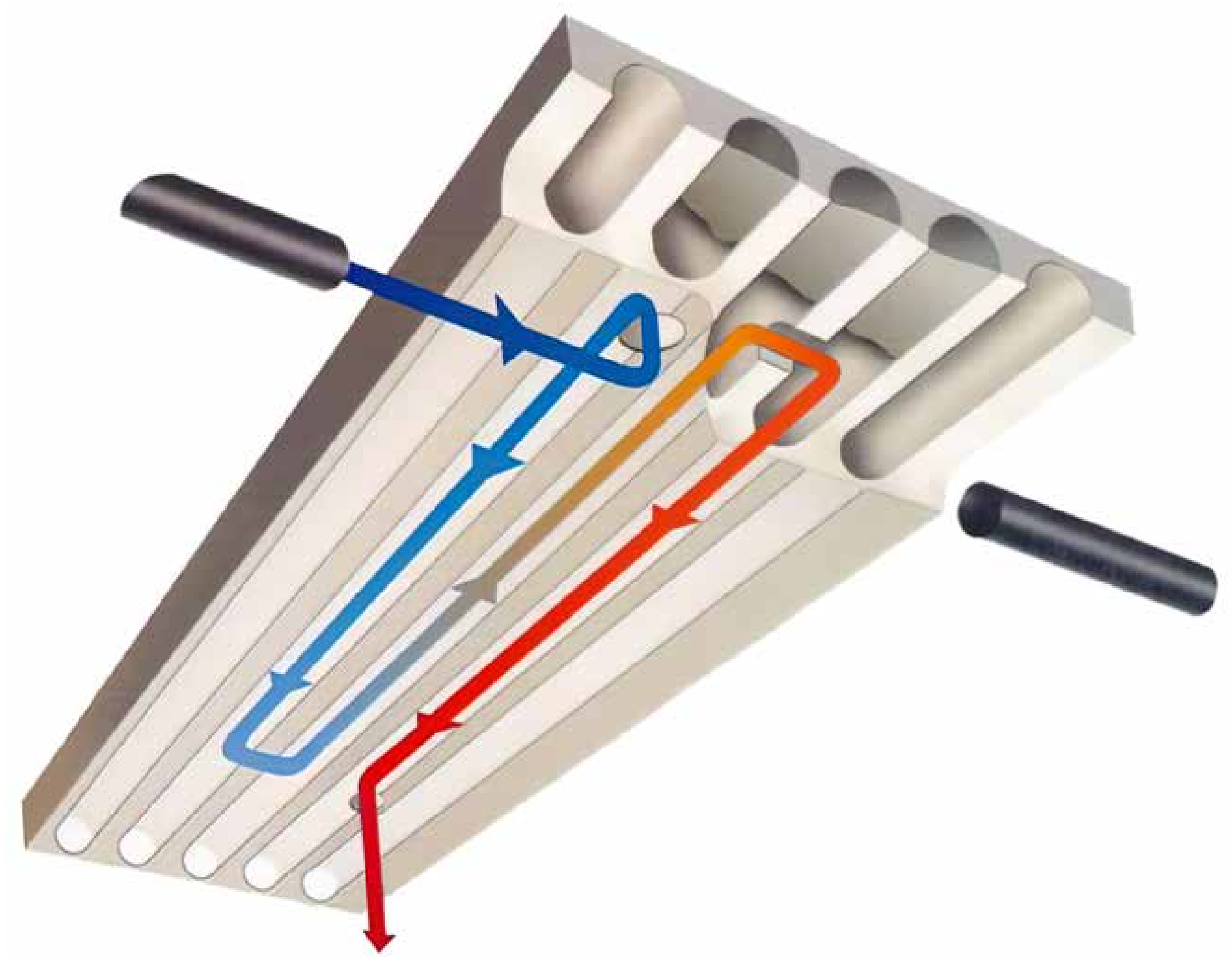


Figure 1.4 Hollowcore slabs used for heating and cooling

1.4 Quality assurance and product certification

Quality assurance and plant certification are important items in prefabrication. They are a response to an ever-increasing demand from the market place for quality of products and services.

Quality assurance and quality control of precast concrete elements are based on two levels: an in-house quality assurance programme with continuous in-house control, and plant certification with quality control supervised by an independent body.

Quality control requires much more than achieving concrete strength. Many other factors are involved in the control of quality of precast concrete products. Some of the most important include:

- Completeness of work orders.
- Clarity of the drawings and specification for preparation of the shop drawings.
- Testing and inspection of the materials selected for use.
- Accurate manufacturing equipment.
- Proportioning and adequate mixing of concrete.
- Handling, placing and consolidation of concrete.
- Curing of concrete.
- Control of dimensions and tolerances.
- Handling, storing, transportation and erection of members.

In Australia there are currently no specific quality assurance and plant certification programmes available. The procedures to be followed for the quality control are normally based on ISO 9001 standards. The Precast Prestressed Concrete Institute, USA, [4] has published quality control manuals for plants and production of precast and prestressed concrete elements. The fib Commission on prefabrication has published a Guide to good practice on 'Quality Assurance of Hollow Core Slab Floors' [5] and the fib Bulletin 41 'Treatment of Imperfections in Precast Structural Elements' [6].

2 DESIGN CONSIDERATIONS AND PROCESS

2.1 Design considerations

Before commencing a design in precast concrete the possibilities, restrictions, and advantages of precast concrete including the design process, manufacture, transport and erection and serviceability stages should have been considered by the project team.

The project design team should make themselves aware of the product information that is available from the precasting industry. This will ensure that all parties are aware of the preferred methods to be adopted in all phases of the project, leading to maximum efficiency and benefit. This is particularly important with the manufacturing and erection stages. It should be born in mind that in the case of prefabrication it is difficult to make additions or changes once construction has started.

It is also very important to realize that the best design for a precast concrete structure is arrived at if the structure is conceived as precast from the very outset and is not merely an adaption of a traditional cast in-situ building.

The major benefits of a precast concrete solution will be met when at the conceptual design stage the following points are considered:

In this Chapter

2.1.1 Safe design

The safe design of the structure requires consideration of a broad range of objectives including practicability, aesthetics, cost and functionality. Each of these need to be balanced against the requirements to produce a safe design.

Safe design means that early in the design process of a building, control measures need to be implemented to identify workplace hazards during the construction and life of the building and to eliminate or minimise, so far as is reasonably practicable, the risks associated with those hazards. Managing this risk is a systematic process that is usually easier and cheaper to achieve at the design stage than trying to make changes later when the hazards become real risks in the workplace.

Precast concrete construction can be a high risk process that requires serious consideration of these issues by all relevant people involved in the work, including the principal contractor, designers, engineers, prefabricators, and erectors.

Safe design starts at the concept design stage when consideration needs to be given to materials, methods of construction, maintenance and eventual demolition or dismantling. This topic is discussed in detail in the Code of Practice, Safe Design of Structures [38] and in AS3850. [35]

2.1.2 Respect the specific design philosophy

One of the most important objectives of this Design Guide is to explain the specific design philosophy of precast structures, since it is the key to efficient and economical construction.

Generally in all cases the objective of the design team is to develop a structural system characterized by structural simplicity and geometric clarity, providing short and direct paths for the transmission of the vertical and horizontal actions, thus minimizing uncertainties concerning the modelling, analysis, dimensioning and detailing for the construction. This can be obtained by;

- Optimising the spans for each element.
- Minimising the number of building elements.
- Using simple lateral restraining systems.
- Providing for structural integrity.

Structural simplicity in turn, is best characterized by uniformity and regularity in the geometrical configuration of the structural system in plan and elevation. Load transfer should be avoided or otherwise limited and if required specially treated at one level.

2.1.3 Modulation

Although modulation, designing the building on a rigid module, can be an economic factor in designing and constructing buildings, it should not be the over-riding factor. The precasting industry in Australia has developed in response to architectural requirements and few buildings are constructed to a rigid module. Grid layouts and dimensions are usually based on site limitations or car park dimensions.

2.1.4 Standardization

Standardization and modulation are often used to mean the same. In prefabrication they are significantly different. Section profiles and details can be standardised without being limited to a set dimensional module.

Standardization of products and processes is wide spread in prefabrication. Precast manufacturers have standardized their components by adopting a range of preferred cross-sections for each type of component and all Manufacturers have their own detailing manuals, for example the Hollow Core Concrete 'Detailing Manual'. [29] Standardization is generally limited to connection details, cross-sectional dimensions and geometry, but seldom to the length of the units. Typical standard elements for buildings are walls, columns, beams and floor or roof slabs.

Standard products are cast in existing moulds. Each precast manufacturer will have a preferred range of dimensions for each element cross-section that combined with variations in reinforcement content will provide the full range of required capacities.

The designer can establish the required length of the element and then select the dimensions and corresponding load bearing capacity. Information on preferred cross-section dimensions can be found in product catalogues from the precast manufacturer. The 'Precast Concrete Handbook' [2] also provides guidance for establishing section dimensions and capacities.

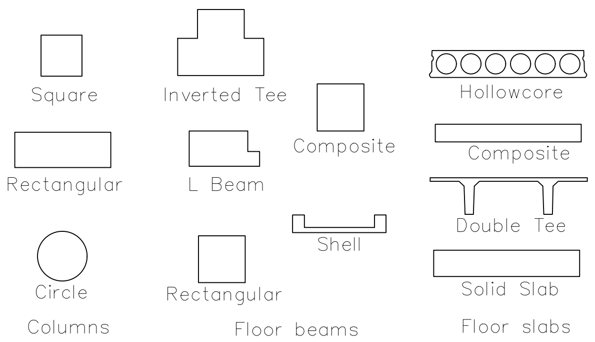


Figure 2.1 Common examples of standard cross-sections

Wall elements usually have a range of standard thicknesses but the height and width are flexible within certain limits that are dictated by transportation or erection. Facades are always designed individually for each project but wall panels for industrial buildings are sometimes available in standard dimensions. Within limits, wall thicknesses have little influence on the erected cost of the wall. It is usually more economical to provide extra thickness with less reinforcement than the opposite.

In addition to the already mentioned components, the precast industry also produces other purpose-designed elements, for example stairs and landings, balconies, special shaped sections, etc.

Standardization constitutes an important economic factor in prefabrication. Repetition resulting from standardisation has a beneficial effect when manufacturing any series of identical elements, resulting in a significant reduction of the cost of moulds and labour input per element.

2.1.5 Detailing

A good design in precast concrete should use details that are as simple as possible. Connection details that are too elaborate or difficult to implement should be avoided as they are expensive and time consuming on site.

In terms of this Design Guide the following definitions are used.

- Joint. The interface between two discrete precast elements, or between a concrete element and some other portion of the structure.
- Connection. Method by which one or more concrete or steel elements are joined together to transfer loads and/or provide stability.
- Fixings or fitments. The hardware components of all connections including bolts, washers, weld plates and anchors.

2.1.6 Dimensional deviations

There will be inevitable differences between the specified dimensions and the actual dimensions of the elements and the finished building. These deviations, often referred to as tolerances, must be recognized and allowed for. Precast concrete is generally manufactured with relatively small dimensional variations, but designers should take a realistic view of dimensional variability. It is essential to consider this from the very outset and to discuss tolerances as early as possible with the precast producer.

Dimensional variations occur at both the precasting plant and on site.

Production tolerances at the plant include dimensional deviations of the products, non-linearity, non-flatness, non- orthogonal cross-section, camber deviations of prestressed elements, position of cast-in fixings, etc.

Site tolerances include variations on the setting out of construction axis and levels. In addition, during erection, deviations will occur with respect to position and alignment of the elements.

Figure 2.2 shows the relationship between the various deviations and how they interact within the total building. Manufacturing, setting-out and erection tolerances all need to be taken into account in the total building.

Information concerning allowable tolerances can be found in the 'Precast Concrete Handbook' [2] and relevant Australian Standards [7].

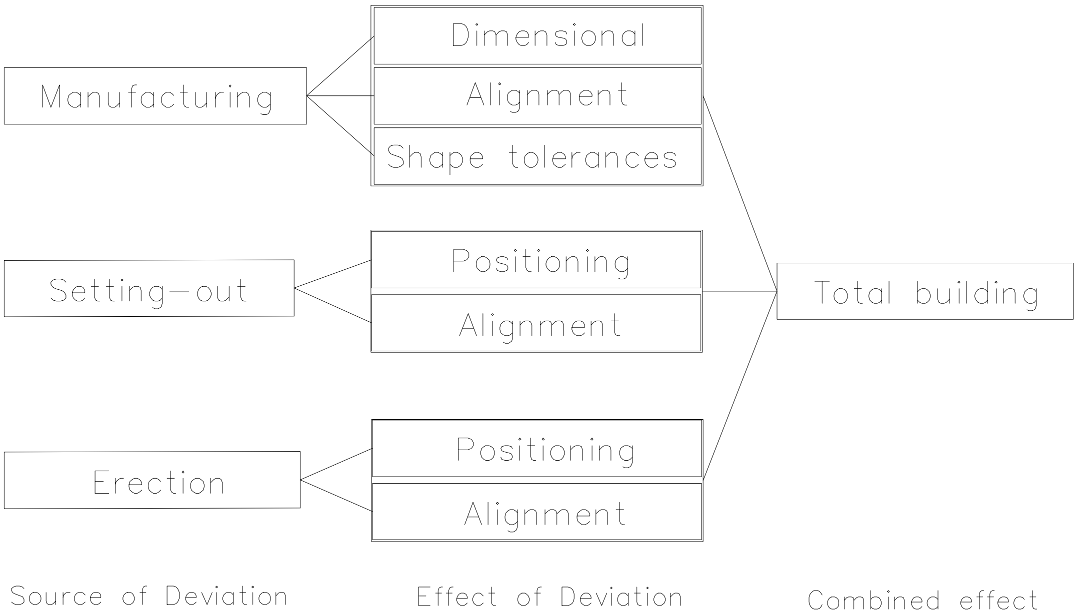


Figure 2.2B Combination of construction tolerances

2.1.7 Industrialization of the process

Precast concrete production should be based on industrialization with the objective to minimise labour requirements. This is partly influenced by the design. For example:

- Standardise components and details to enable standardization of the production process. This allows simultaneous manufacture of elements for a range of projects.
- Detail elements such that they can be set-up, poured and de-moulded within a 24 hour period. It is often more economical to simplify and use extra material if it results in more efficient production and mould utilisation.
- Use prestressing to enable long-line production of elements.
- Simplify reinforcement to minimise labour and set-up time.
- Avoid complex details that require high labour input and tight tolerances.
- Simplify documents to help avoid mistakes. Shop drawings should be standardised so that each type of element is presented in a similar way for every project.
- Avoid last minute modifications that affect the production planning and can lead to mistakes.

2.2 Design considerations in moderate seismic areas

Although this document does not cover specific earthquake design requirements, most areas of Australia are subject to mild to moderate earthquakes. This means that buildings are likely to be subjected to lateral displacement and induced forces and designers should take this into consideration when conceiving and detailing a building. This is important for precast structures as the seismic response of an assembly of discrete precast elements is quite different to that of a monolithic structure. It is therefore important in the conceptual design to incorporate some basic earthquake design principles. Doing so, will in most cases satisfy the fundamental requirements of non-collapse and damage limitation, within acceptable costs and to meet the requirements of the BCA [10]. A more detailed coverage of this topic is given in the fib technical report, 'Precast Buildings in Seismic areas – practical aspects' [8].

Where seismic design becomes the critical design case a more detailed analysis in terms of actions and behaviour of the structure must be carried out by the structural engineer. However, at the conceptual design stage it is essential that the following points are considered. These concepts apply equally to all structural systems and not just to precast concrete.

2.2.1 Structural simplicity

Structural simplicity and geometrical clarity of a structural system is essential for providing direct or alternative paths for the transmission of the seismic loads. The uncertainties in the modeling, analysis, dimensioning, detailing and prediction of the seismic response of the structure are minimised with a simple structural system.

Structural simplicity is characterized by the uniformity and regularity in configuration of the structural system in plan and/or elevation as shown in Figure 2.3.

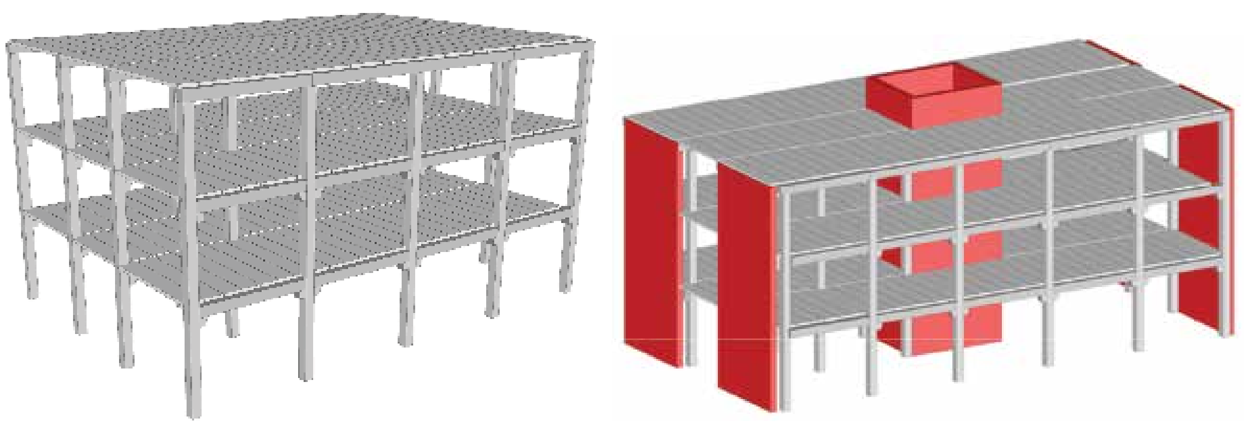


Figure 2.3 Two schematic examples of structural simplicity

2.2.2 In plan uniformity

Uniformity in plan is affected by the geometrical configuration of the building which in turn is imposed by architectural aspects. However re-entrant corners, edge recesses and non-uniform shapes should be avoided or otherwise limited and specially treated. To minimise torsional effects the centre of mass and centre of stiffness should be closely symmetrical in plan with respect to two orthogonal axis.

The maximum building's length to width ratio should not be greater than about 4:1. In this respect, it may be necessary to subdivide the entire structure into dynamically independent units by means of seismic joints.

In-plan uniformity and regularity leads to the uniform distribution of the structural elements in plan and reduces possible torsional effects.

2.2.3 In-height uniformity

For a building to be characterized as regular in elevation all lateral resisting systems such as cores, structural walls, or columns in frame systems, should run without interruption from their foundations to the top of the building.

Both the lateral stiffness and the mass of the individual levels should remain constant or reduce gradually with height. A reduction in stiffness at a single level, particularly ground level, should be avoided. This is commonly known as a 'soft storey' as shown in Figure 2.4.

A natural flow of forces should be ensured by avoiding staggered beams, or worse, staggered columns.

Figure 2.4 Building examples of soft storey ground floors and soft storey upper floors to be avoided

2.2.4 Torsional resistance and stiffness

Each structural system should be able to resist seismic actions in two orthogonal directions by providing similar resistance and stiffness in both directions.

Torsional irregularities should be avoided as much as possible. In all cases, special care should be given to the position of the lift-shafts and staircases. These parts of the building which are usually provided by structural walls around their area contribute significantly to the torsional behavior of the structural system. Figure 2.5

The analysis and observation of structures in past earthquakes has identified a group of common causes of damage as follows;

- Soft storey ground or upper floors due to discontinuity of stiffness. (Fig. 2.4)
- Asymmetrical positioning of lateral bracing elements. (Fig. 2.5)
- Short columns due to unintended restraint, for example by spandrels. (Fig. 2.6)
- Poor connections of floors to walls and lift shafts and the like. (Fig 2.7)
- Inability of the floor to act as a diaphragm to distribute horizontal loads to the lateral restraining elements.

2.2.5 Connections

The connections in precast buildings are often designed only for gravity loads without any consideration of lateral sway. Such connections can therefore be very vulnerable to lateral displacement due to seismic events. For this reason all connections should be designed for ductility so they have post-yielding capacity under load reversal. Cast-in fixings in particular are vulnerable and should never be the weak link. They should be designed in accordance with the capacity design method.

Lack of concrete confinement in connections, short support lengths, weak or missing dowels, and unsatisfactory overturning restraints of beams are frequently the cause of collapse.

In the case of precast floors an adequately reinforced concrete structural topping should cover and connect the floor elements in order to guarantee a diaphragm action. Diaphragms also need to be adequately tied into the shear walls or frames especially where openings such as a stair core may make these connections difficult.

In precast braced wall buildings, such as single storey industrial buildings, it is recommended that the wall-to-roof connections are designed to accommodate lateral displacement without failure. These connections are particularly sensitive to unintended loads from deflections of the steel roof diaphragm or thermal movement.

Experience from past earthquakes shows that inadequate connection details between cladding panels and the corresponding structural elements can lead to collapse of the panels during the seismic event. (Fig 2.7) where external wall panel connections failed due to their inability to accommodate in-plane displacement.

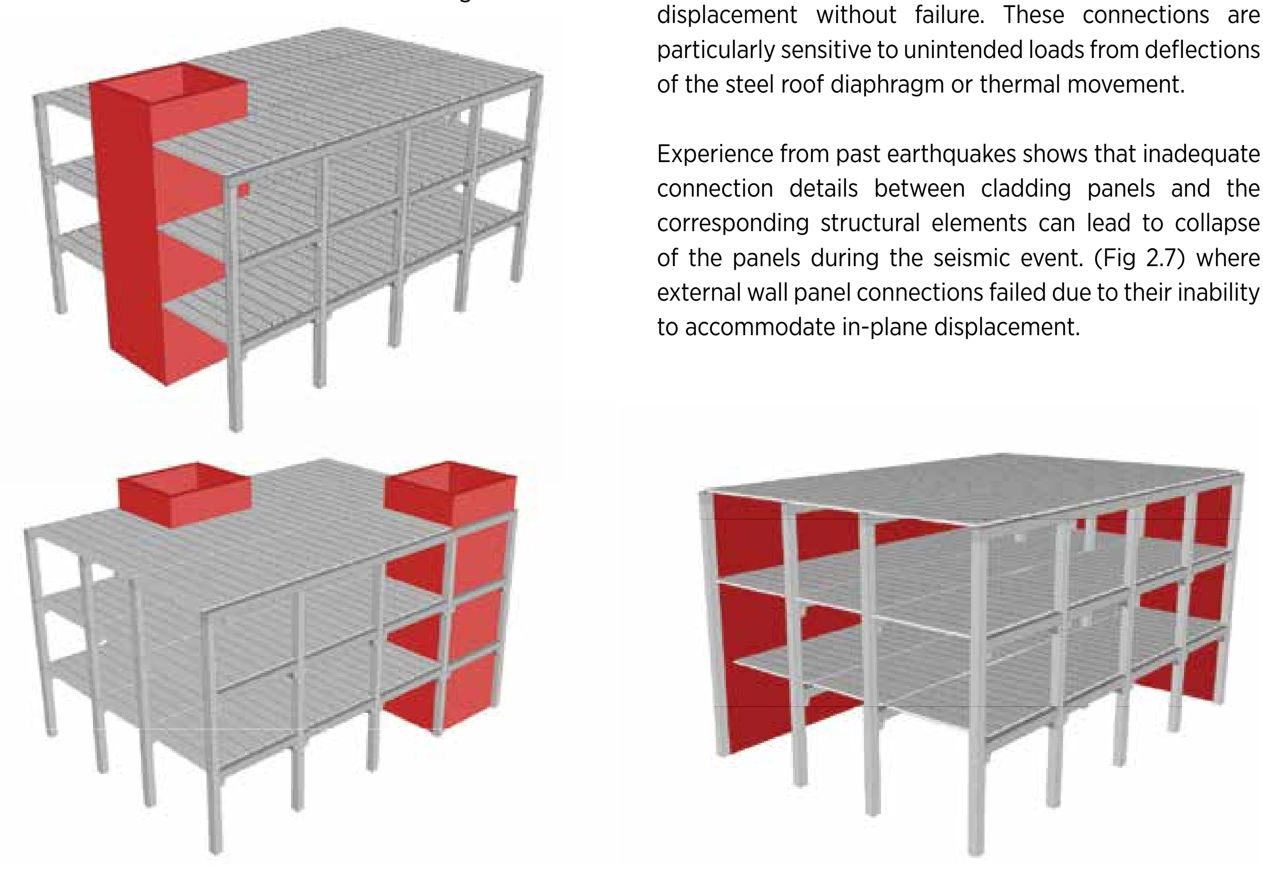


Figure 2.5 Shear wall configurations to be avoided due to the fact that they are not symmetrical in plan with respect to two orthogonal axes.

2.2.6 Infills partitions and claddings

When designing precast structures and particularly frame systems the interaction of non-structural secondary elements such as infills, partitions and claddings under seismic action should be given particular attention. The torsional response of the structure can be changed by these elements providing unintended stiffness.

Generally, non-structural secondary elements need to be connected with the structural elements in a way that they will not affect the anticipated response of the structure during the seismic event.

Appropriate measures should be taken to avoid brittle failure of secondary elements and special care should be given to the detailing of their connections.

In the case of rigid infills, the possible shear failure of columns under shear forces induced by the diagonal strut action of the infills should be taken into account. (Fig 2.6)

2.2.7 Foundations

In seismic situations, the interaction of the soil with the superstructure should be carefully studied. The response of a building to earthquakes is very dependent on the founding conditions. Generally, the design and construction of the footings should ensure that the whole building is subject to a uniform seismic excitation.

Figure 2.7 In-plane failure of cladding panel connections

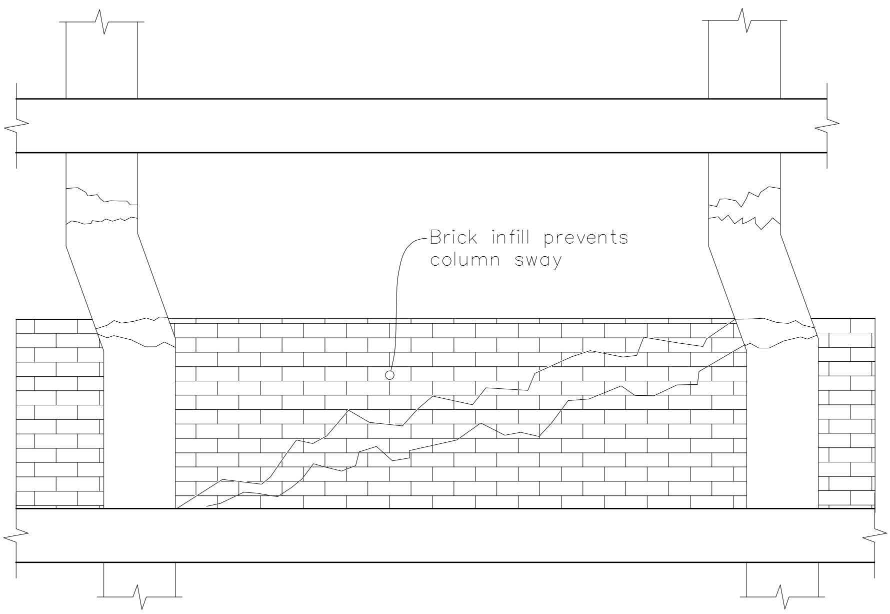


Figure 2.6 Building example with column failure due to spandrel infills

2.3 Design process

The design process for a building has a great influence on the success of the subsequent construction and final product. There are a number of ways of facilitating the design and the construction process but all start with a client who identifies the need for a building and prepares a brief based on quantifiable requirements such as space and budget. This process is the same for all type of building structures.

The brief, prepared by the client or his representative should set out the type of building required, the proposed method of procurement, budget constraints, program requirements and any other client issues related to the proposed building.

From here there are numerous ways of proceeding with the design and construction of the building ranging from the traditional full design and tender process via an architect as the principal consultant through to the appointment of a builder/project manager to 'design and construct' the building. Contractual arrangements under a 'design and construct' process can vary considerably

Irrespective of the delivery method, a project design team needs to be established to design and document the building.

2.3.1 Design team

In all cases the project design team should consist of at least the Client or building owner, Architect, Structural Engineer, Services Engineer, other specialist consultants and possibly a Cost Planner and Project Manager.

Precast concrete construction lends itself to some form of 'design and construct' process. From a precast manufacturers point of view this is the preferred option due to particularities from both architectural and structural point of view caused by the precast construction technique. This in turn leads to the recognition that from the very beginning of the project the design team should include input on precast concrete design and precasting techniques. Precast manufacturers with access to their own designers may become part of the design team and carry out the structural design of the precast elements under the direction of the Project Structural Engineer.

Irrespective of the procurement arrangement the design process of any building can be divided into three separate phases as follows.

- Concept design.
- Design development.
- Final design.

2.3.2 Concept design

Concept design, or schematic design, is usually based on vague and limited information and initial structural design should therefore be simple quick and conservative without being heavy handed. Detailed analysis of framing systems or elements is not warranted as the final form of the building is almost certainly still evolving.

Ideally, if precast concrete is being considered for the structural system a decision in principle should be made to this effect as part of this phase. This then allows future decisions to be based around the unique attributes of prefabricated construction. Sufficient structural design needs to be carried out to ensure that the concepts proposed are feasible.

2.3.3 Design development

The design development phase is where the concepts that evolved in first stage are developed in more detail. Architectural sketches of the building will have been produced showing a general layout of the floors with possible locations of the fixed vertical elements including shear walls, stair and lift shaft walls etc.

On the basis that the structure of the building is to be precast concrete the following stages in developing the design for the structure are recommended. They are exemplified on a simple office building given in figure 2.8.

First step: The various structural options for column and wall locations established as part of the concept design are reviewed and confirmed.

Second step: Once the preferred wall and column locations are finalised a structural system and appropriate structural grid can be determined.

Third step: Methods of providing horizontal stability are investigated. This will depend on the type of structural system. For the example given, a skeletal frame braced by shear walls is the logical solution. The façade cladding should be investigated and recommendations provided.

For other building layouts the structural system could range from a braced wall system for single storey structures to a loadbearing wall system for a wall structure or moment resistant frames for an open plan structure.

Fourth step: A preliminary layout is prepared that shows the preferred dimensions of the precast elements. Information about preliminary design of precast components with regard to load and span is available in the Precast Concrete Handbook [2], the Hollowcore flooring technical manual [33] and catalogues and technical brochures from precast manufacturers.[29]

Fifth step: The preliminary structural concept is presented to the design team for review and costing as required.

Note that the building in this example has a layout that is not ideal to resist seismic actions. If this were a requirement, serious consideration would need to be given to providing a seismic movement joint across the re-entrant corner to split the building into two separate structures.

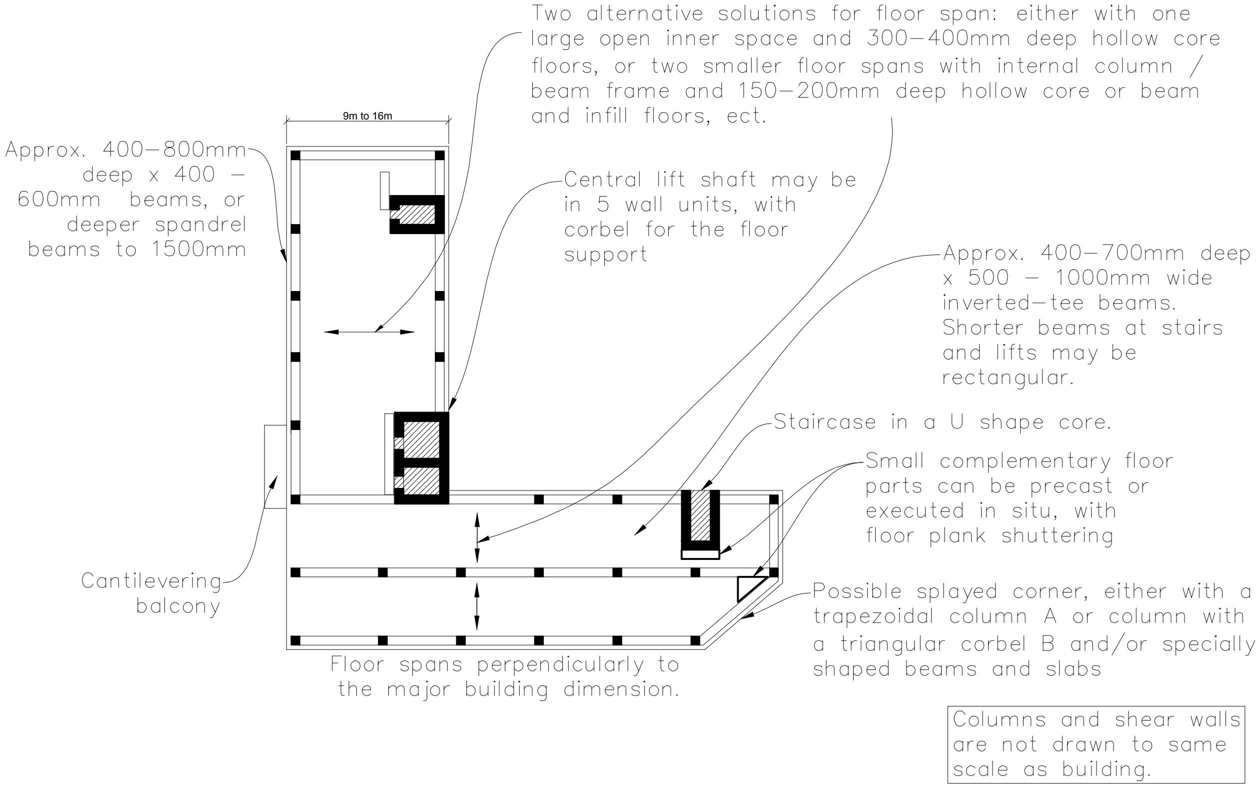


Figure 2.8 Example of possible floor layouts for an office building

2.3.4 Final design

The final design phase is where the chosen preliminary scheme is designed and detailed. Where precast manufacturers are part of the project team and have access to their own designers the final design may include two separate but integrated sets of documents. In this case the structural design of the precast elements would be carried out by the precast manufacturer under the direction of and in accordance with design information provided by the project consulting engineer. This requires discussion and agreement between the two parties to clearly establish the split up of work. If the design is split it is important that the project consulting engineer fully coordinates the overall design to ensure that it complies with the project design criteria.

Prior to issuing the final structural design the documents should be reviewed to ensure the following.

- That the final documents have been coordinated to comply with all architectural and other requirements.
- The project is fully designed and detailed. It must be adequately documented by way of drawings and specifications to allow the structure to be costed and built.
- Confirmation that the design computations and drawings have been checked in accordance with quality assurance procedures.

The conclusion of this stage is provision of a detailed design that meets the requirements of each member of the design team. The degree of documentation should be such that permit applications can be submitted, pricing finalised and construction started.

As part of the manufacturing and erection process for precast buildings, shop drawings and erection designs are prepared, usually by the precast manufacturer and erector.

Individual shop drawings of all precast concrete elements are provided by the precast manufacturer to allow the element to be checked by the designers before manufacture. Regulatory authorities [3] as well as Australian Standards [35] have specific requirements for shop drawings. Shop drawings are the responsibility of the precast manufacturer and are not part of the building design documentation.

Precast concrete buildings are assembled from a series of discrete elements so they usually require temporary support or bracing during erection. A separate detailed design is prepared that determines the erection sequence and calculates the bracing requirements for each element. There are OHS regulatory requirements [3] [35] that specify that this design is to be carried out by a temporary works or erection engineer. If the project has special requirements for construction sequence or other matters that may affect the erection then these requirements must be clearly detailed on the project consulting engineer's drawings.

Erection design of precast concrete buildings is a specialist field that requires a practical understanding of both the construction process and OHS issues and is not covered by this Design Guide. Further details can be obtained in the Code of Practice, Tilt-up and Precast Concrete in Building Construction, [3] and AS3850 [35].

3 PRECAST BUILDING CONCEPTS

3.1 General

Every construction material and system has its own characteristics which to a greater or lesser extent influence the layout, span length, construction depth, stability system, etc. This is also the case for precast concrete, not only in comparison to steel, timber and masonry structures, but also with respect to cast in-situ concrete. Theoretically, all joints between the precast units could be made in such a way that the completed precast structure has the same monolithic behaviour as a cast in-situ structure. However this concept of emulating insitu concrete, is the wrong approach and one which is generally very labour intensive and costly.

If the full advantages of precast buildings are to be realized, the structure should be conceived according to its specific design philosophy: long spans, appropriate stability concept, simple details, etc. Designers should from the very outset of the project consider the possibilities, restrictions and advantages of precast concrete, its detailing, manufacture, transport, erection and serviceability stages before completing a design.

In this Chapter

3.2 Structural systems

When looking at the precast concrete industry, there are apparently a large number of technical systems and solutions for precast buildings. This is often a matter of confusion and distrust for designers who are not familiar with precast concrete, and consider it an insurmountable handicap to precast design. However, they all belong to a limited number of basic structural systems, of which the design principles are more or less identical. In a similar manner with an insitu concrete building there are many solutions, the aim is to provide a system which is simple and sensible.

Consequently, the designer does not need know details of all commercial systems to design a project in precast concrete, but only the basic principles of precast structures.
Some details need not be considered at the initial design stage but can be resolved in conjunction with the precast manufacturer during finalisation of the design.

The most common basic types of precast concrete structural systems are:

- Braced wall structures.
- Skeletal frame structures.
- Loadbearing wall structures.
- Cellular structures.
- Hybrid or mixed structures.

In practice most buildings comprise a combination of the above basic structural types and are completed by using a range of precast elements for the construction of walls, floors, roofs and façades.

The selection of a structure for a given project is based on a number of parameters related to the type, occupancy, the needed spans and grids, the applied solution for the façades, the required fire resistance, available lifting capacity during erection, etc. Each of the above parameters will have an optimum structural solution and each need to be weighed against the others in making the final choice.

3.2.1 Braced wall structures

Braced wall structures are designed and constructed in a number of ways, the differences being in the manner in which the wall is stabilised, and secondly in the type of roof structure. The most common is where the walls are both loadbearing and shear walls and support a lightweight roof structure. Stability is provided by bracing the roof to transfer lateral actions to the shear walls. A variation of this is where the loadbearing walls cantilever from the foundations to provide lateral stability. Where stability is provided by a complete portal frame system the walls can be non-bearing cladding panels, spanning either vertically or horizontally. Wall panels are usually solid, but hollowcore or double Tee sections can be used.

These systems are particularly suited to use on single storied buildings but are equally applicable to the top floor of low to medium rise buildings with a light weight roof structure. Braced wall structures give a high degree of flexibility by providing large structure free areas and combine the advantages of loadbearing walls with the long span capabilities of the steel roof structure. The individual structural wall elements are well suited to rational and rapid production and erection and give the architect a wide choice of fenestration and finishes.

Detailed information on braced wall structures is given in Chapter 6.

Similar concepts can be used on residential buildings as shown in figure 3.3 where loadbearing wall panels support a metal framed roof structure which in turn is braced to provide lateral stability. The slab-on-ground is often poured and tied into the walls after erection of the structure.

Figure 3.1 Horizontal cladding structure

Figure 3.2 Braced wall structures

Figure 3.3 Braced wall structure for residential building

3.2.2 Skeletal frame structures

Precast skeletal frames consist of a series of columns, beams and floor slabs assembled and connected to form a robust skeletal structure able to support and transfer vertical and horizontal actions to the foundations. Skeletal structures are most commonly used for low to medium rise buildings of up to 15 storeys. The limiting factor for height is not so much technical restrictions but cranage, material handling and competition with sophisticated formwork systems that have become very cost effective on high-rise buildings.

Horizontal stiffness and lateral stability can be provided in a number of ways. For buildings up to 2 to 3 storeys with few walls, the structural system can be based on the cantilever action of the columns, which are fixed into the foundations. The most efficient skeletal frame solution irrespective of the number of storeys is in braced structures where the lateral stability is provided by stair or lift shafts or shear walls. In this way, connection details between beams on columns can be designed as pinned, greatly simplifying the design and construction.

Maintaining stability of the structure during erection of the precast is important and erection design requirements may determine the details used.

Skeletal structural systems are very suitable for multi-storied buildings requiring a high degree of flexibility. This is mainly due to the ability to use large spans and to achieve open spaces without interfering walls. This is very important in shopping centres, parking structures, sporting facilities and office buildings. The skeletal structural concept gives great freedom in planning of floor areas that are not hampered by load-bearing walls. The individual structural units are suited to rational production and erection processes and the skeletal concept gives the architect a wide choice for the facade cladding.

Since the load-bearing system of a skeletal structure is normally independent of the mechanical and electrical services and partition walls etc. the buildings are easy to adapt to changes in use, new functions and technical innovations.

Detailed information on skeletal structures is given in Chapter 7.

Figure 3.4 Cantilevered precast skeletal frame

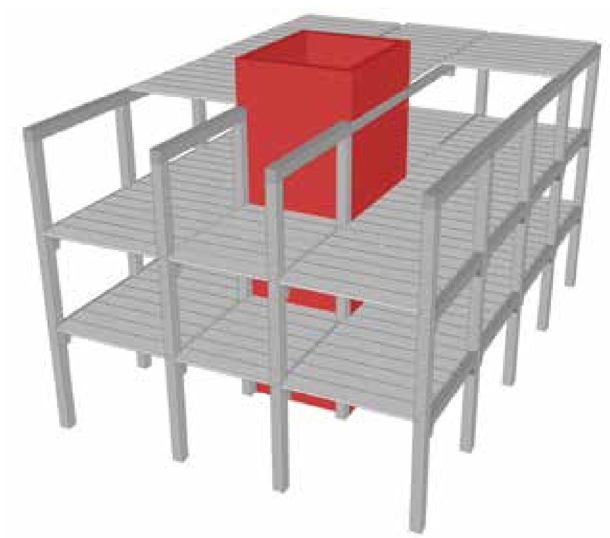


Figure 3.5 Braced precast skeletal frame

3.2.3 Loadbearing wall structures

Loadbearing wall structures are obviously ideal for buildings with many external and internal walls. Precast bearing walls are also used as cross-walls, walls in stair and lift shafts, cores and load-bearing facades. This is a common form of construction with various combinations of floor to floor or multi-height panels being used.

Precast walls offer the advantage of speed of construction, prefinished surfaces, acoustic insulation and fire resistance.

Precast bearing wall systems are mostly used in residential construction, both for low and high-rise apartments. Indeed, the ideal solution on apartments is to build free open spaces between the load-bearing perimeter walls of each apartment and to use light partition walls for the internal layout. (Fig 3.6) This offers the possibility to later change the interior layout without major costs.

Precast lift and stair shafts are common on commercial buildings and where necessary individual wall panels can be connected on site to form 'box shafts' or in-plane shear walls.

Loadbearing wall structures are also suitable for multi-storey commercial buildings. With building widths of up to 16 metres precast floors can clear span between loadbearing wall facades on each side of the building. Walls can be single or multi-level and are often architectural panels acting as and both loadbearing and shear walls.

Maintaining stability of the structure during erection of the precast is important and erection design usually requires all walls to be braced during construction.

Detailed information on loadbearing wall structures is given in Chapter 8

Figure 3.6 Low-rise apartment building with load-bearing cross-walls and corridor walls

Figure 3.7 High-rise apartment building with loadbearing walls and precast floors

Figure 3.8 Load-bearing facades and clear-span floors

3.2.4 Cellular structures

Cellular units are mainly used for parts of a building, for example lift or stair shafts, bathrooms and kitchens, hotels, prisons and occasionally as modules to make up complete buildings. For example an apartment building can be assembled from a series of prefabricated modules complete with internal finishes.

The advantage of the system lies in the speed of construction, and industrialisation of the manufacture. For cellular units containing non-structural finishes these can be completed at the precasting plant and delivered to site as a finished unit. The disadvantage is that the size and weight of the unit is often limited by transport and erection considerations.

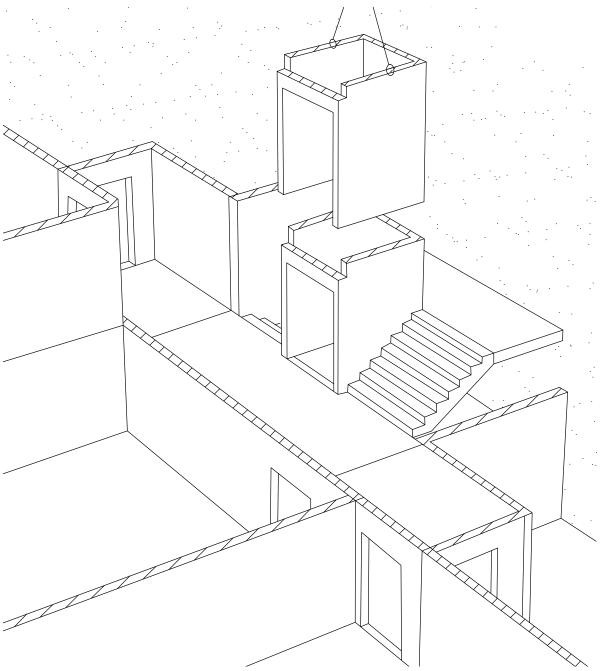


Figure 3.9 Precast concrete cellular systems

3.2.5 Hybrid or mixed structures

The term hybrid construction, or mixed construction, is used to describe a type of structure where precast concrete is used in combination with other building materials, such as cast in-situ concrete, steel, masonry or timber. The term must not be confused with "composite" construction, where structural performance relies on the interaction between two separate materials. Nor does it apply to the common situation where for example precast buildings may contain a mixture of loadbearing wall panels and skeletal frames.

The use of the term in this Design Guide applies to the use of mixed materials within the same structure. Common examples are; steel frames supporting precast floors, masonry walls supporting precast floors, precast walls or beams supporting metal deck floors, precast walls supporting insitu floors and insitu lift or stair shafts combined with precast skeletal frame systems. A further refinement is where a small part of a precast concrete project may be of some other type of structure.

Hybrid structures are becoming more and more common and in response to the more complex demands of modern architecture. The choice of structure for all or part of a building should be driven by the best solution for individual parts combined with an overall assessment of the buildability of the structure.

Most precast buildings will contain some form of hybrid structure and frequently the most economical structural solution involves mixing precast with other structural materials for at least some part of the building.

Detailed information on hybrid or mixed construction is given in chapter 8 and in the fib bulletin 19 'Precast concrete in mixed construction' [9].

Figure 3.10 Hybrid construction, steel frame roof, timber floors and precast walls

3.3 Applications of precast structural systems

The application of the above-described basic structural systems in buildings is closely related to the type of building, which in turn depends on the occupancy. For example, housing, offices, commerce, industry, etc. In the following, the criteria used to help choose the most appropriate precast system for each type of building are given. An overlying consideration is the possible need for future changes or extension of the building.

3.3.1 Residential buildings

Residential buildings contain many internal and external walls and can be designed as loadbearing wall structures where the walls also act as shear walls. This is a common form of construction and many such buildings have been constructed in Australia, ranging from single dwellings to apartments of 2 to 30 or more floor levels. With increasing emphasis being placed on energy consumption, the use of insulated concrete sandwich panels in houses and apartments is becoming common.

For apartment buildings the optimum solution is a loadbearing cross-wall system with party walls running across the width of the building with precast floors spanning between. Other walls can be non-loadbearing of preferably lightweight construction. The exterior cladding can be precast concrete or traditional brick masonry or any other façade material. This solution provides excellent acoustic and fire performance and offers the possibility to change the interior layout at a later stage without major costs.

Alternatively, all walls within the building can be loadbearing precast with floors normally spanning in the direction of the shortest span. To optimise the layout it is possible to the span the floors in different directions within the apartment.

In apartment buildings, floor spans are typically 8 to 12 metres and large openings and offset loadbearing walls from level to level are common. The slenderness of the floor construction, a flat soffit and the speed of construction are important. Insitu concrete is currently the most common floor solution for apartment buildings but prestressed hollowcore slabs, beam-and-infill and prestressed solid slabs are attractive options and there are numerous buildings being constructed as complete precast structures. Erection times for such precast buildings are considerably faster than for mixed construction of precast and insitu.

In individual housing, floor systems usually have small to moderate spans of 4 to 8 metres, light imposed actions (2kPa) and usually no fire rating requirement. Other criteria in the choice of the type of flooring are the presence of large openings in the floors, the requirement for a flat smooth soffit, the available lifting capacity on the project and building tradition, etc.

The use of insitu concrete floors combined with precast walls on individual houses does not seem logical and runs counter to the advantages of speed provided by a precast system. In these cases prestressed hollowcore floor slabs, with or without structural toppings are more suited to the longer spans and or heavier loading where moderate lifting capacity is available to handle the heavier units..

Figure 3.11 Apartment buildings with loadbearing and shear walls

3.3.2 Offices and commercial buildings

Modern offices and commercial buildings normally require a high degree of flexibility and adaptability. The interior space should therefore be as open as possible. Such buildings are usually conceived as skeletal frame systems with lift and stair shafts providing lateral stability. The facades can be constructed in any material but lightweight glazed systems are the current preference. If precast architectural concrete facades are used they can be either load bearing or cladding panels.

The current tendency for office buildings is to create large open spaces with floor spans of 12 metres or more. When the total width of the building lies within these dimensions, the most appropriate solution is to use load bearing façades and to span the floors from one façade to the other. (See figure 3.8) For larger buildings, the same system is complemented by a skeletal frame of one or more rows of internal columns and beams. The cores provide lateral stability with precast walls acting as both loadbearing and shear walls.

For the longer floor spans, hollow core slabs are the most suited floor type because of the large span capacity, slender floor thickness and ease of construction. There are a range of slab thicknesses available for different spans but it is common to have hollow core units of up to 400 mm thickness with a design capacity of 5kPa or more on spans up to about 16 metres.

Reduced construction depth is an important parameter for office buildings and beam configuration and depths should allow for services, particularly major air-conditioning ducts. The traditional office building has a false ceiling below the soffit of the floor where beams and services can be concealed. A recent innovation that is becoming popular is the use of the floor system as a thermal mass to conserve energy use and in this case services are often run in a false floor set above the structural floor. There are several variations of this concept but all involve using the floor as a heat sink.

Figure 3.12 Example of commercial building with skeletal frame

3.3.3 Hotels

Hotels are a mixture between an office building and apartment building and the optimum structure often reflects this mixture. Areas containing hotel rooms are ideally suited to a loadbearing wall structure, such as described previously for apartments, while front of house and service areas are more suited to a skeletal frame structure as, described for an office building. Building geometry will usually dictate the solutions. For example if large open areas are required on the lower levels it may be more economical to use a skeletal frame full height than to provide a transition level to a wall frame structure on the upper levels. Also a skeletal frame provides for future flexibility that is limited with a wall frame structure.

Figure 3.13 Hotel building with open lower levels and loadbearing upper levels

3.3.4 Hospitals

These types of buildings have similar requirements to office buildings but imposed loads tend to be larger, in the order of 5-8kPa. The projects are often large, containing a significant service component and lend themselves to industrialized systems. To accommodate future changes in technology and service requirements clear open structures are desirable. Provision is often made within the floor system for future changes to penetrations.

Such buildings are usually conceived as skeletal frame systems with long span floors and stabilizing cores.

Figure 3.14 Hospital building

3.3.5 Educational buildings

Educational buildings cover a range of different occupancies and uses, from classrooms, administrative areas, auditoria and laboratories. The façades are often characterized by large window openings. The buildings can also range from low-rise to multi-level structures. To cover these requirements and to allow for future changes, multi-level educational buildings are usually either skeletal frame or loadbearing wall systems. Single storey buildings are usually braced wall structures.

Floor spans on multi-level buildings are moderate to large and range from 8 to 16 metres for classrooms to more than 24 metres for auditoria. Imposed loads are in the range of 3-5kPa and dynamic performance is often critical. Selection of the floor system will depend on the spans and the required performance.

Figure 3.15 Educational building & auditorium

3.3.6 Sports halls and gymnasia

If single level, these building are similar to industrial buildings and typically are conceived as braced wall structures. Where multi-level, they are similar to educational buildings, characterized by moderate to large spans, from 8 to 16 metres or more with imposed loads in the range of 5kPa. Dynamic performance is critical and floor stiffness and structural damping are the determining design factors. Skeletal frames or wall frame structures are a typical solution. As with educational buildings selection of the floor system will depend on the spans and the required performance.

3.3.7 Industrial buildings, warehouses

Industrial buildings are typically single storied with large spans and simple roofs and facades. The buildings are normally designed as braced wall systems with loadbearing concrete walls that also act as shear walls. Stability is realized by bracing the steel roof structure to transfer lateral loads to cross walls and shear walls. Alternative load paths should be provided in any light roof bracing system to ensure redundancy and in many cases The Building Code Australia [10] requires that in the event of a fire, the external walls remain standing, or if they fail they must fall inwards.

For very large buildings where roof bracing can become difficult and expensive, stability can be enhanced by provision of internal columns cantilevering from the footings.

Figure 3.16 Example of braced wall system on industrial building

3.3.8 Shopping centres

Shopping centres generally require large column-free areas. If single storied, they are normally designed as braced wall systems with loadbearing concrete walls similar to industrial buildings.

Where multi-storied, these buildings may have to perform several functions, namely car park at the basement, shopping on the lower levels and offices or apartments above. The structural concept is usually a combination of a skeletal structure at the car park and the shopping levels and a loadbearing wall or skeletal structure at the above levels. Where appropriate the up-most level maybe a braced wall structure.

3.3.9 Car park buildings

The basic requirements for modern multi-level car parks are large open spaces with a minimum of internal columns, reduced constructional depth, aesthetic outlook, etc. Parking and aisle dimensions dictate the column grid with common spans being 8.4, 10.8, 12.6 and 16.8 metres. They are usually designed with skeleton frame systems, in combination with precast stair and lift shafts acting as shear walls. In all cases the skeletal frame beams span the shorter dimension. A further complexity is added by the internal geometry of the circulation and parking space and whether the parking facility is in the lower part of another type of building.

The overall stability of car park structures of up to about three levels is generally assured by cantilevering columns in combination with lift shafts and stairwells. With higher structures skeletal frames combined with loadbearing walls acting as shear walls are used. When parking structures are built partially or totally below ground level, it is usual to use the floor system as a diaphragm to brace the retaining walls.

Figure 3.17 Split level car park system

3.3.10 Grandstands

There are many different types of grandstand, each having their own specific requirements. A modern tendency is for raking seating areas to be combined with extensive corporate entertainment areas.

In their simplest form grandstands comprise radial precast concrete skeletal frames combined with loadbearing walls. Horizontal floors can be precast beams with precast floor slabs and raking areas comprise precast seating elements supported on precast beams. Floor loads are generally in the range of 5-10kPa and dynamic performance of the complete structure is frequently the critical design factor. The distance between the radial frames varies but should be dictated by the longest economical span floor unit on the outer perimeter.

The cantilevering roof above the grandstand and even the beams supporting the upper level of seating are often executed with steel beams.

Figure 3.18 Grandstand frame in precast concrete

4 PRECAST STRUCTURAL ELEMENTS

4.1 General

Precast structures are comprised of a small number of different elements selected and assembled in different ways to form the building structure. As shown in Chapter 1 the flexibility of use is such that buildings assembled from similar elements can appear to be quite different. The main structural elements are; walls, columns, beams and floor slabs.

In this Chapter

4.2 Walls

4.2.1 Basic principles

Wall panel manufacture in Australia is characterised by the wide range of dimensions and panel profiles that are used. The great success of the industry is a result of providing complete flexibility to produce the architectural effect required by the clients.

Besides the fast and industrialized construction, precast walls can offer good loadbearing, acoustic and thermal properties, high fire resistance and a wide variety of finishes ranging from smooth concrete to polished, coloured, textured or natural stone appearance.

Wall panels with polished, coloured, textured or natural stone appearance are usually referred to as "Architectural precast concrete". Architectural concrete is not covered in this Design Guide and is described in detail in the 'Precast Concrete Handbook'. [2]

Precast wall elements can be either single or multi-storey height. The thickness is a function of requirements related to structural capacity, stability, acoustic insulation, fire resistance of the wall, the equipment at the precasting plant and particularly the ability to transport to site. Length can vary up to about 17 metres but typically widths should be limited to about 3.6 metres. Wider panels can be transported on special low-loaders but transport costs increase significantly. Transport regulations vary from State to State and wall panel dimensions should be adjusted to optimise transportation so that each load is maximised.

Panel thickness can range from 125-350mm with 150-200mm being the most common. The weight of the panels should generally be in the range of 5-12 tonne to suit cranage and transport. Optimum panel weight is a trade-off between cranage costs and time taken for each lift. On low-rise projects where mobile cranes are used the optimum panel weight will be at the upper end of the range and on projects where fixed tower cranes are used the optimum panel weight will be at the lower end of the range.

Where high thermal performance is required insulated sandwich panels are becoming common. These have generally comprised an inner structural layer tied to an outer non-structural veneer with insulation between but

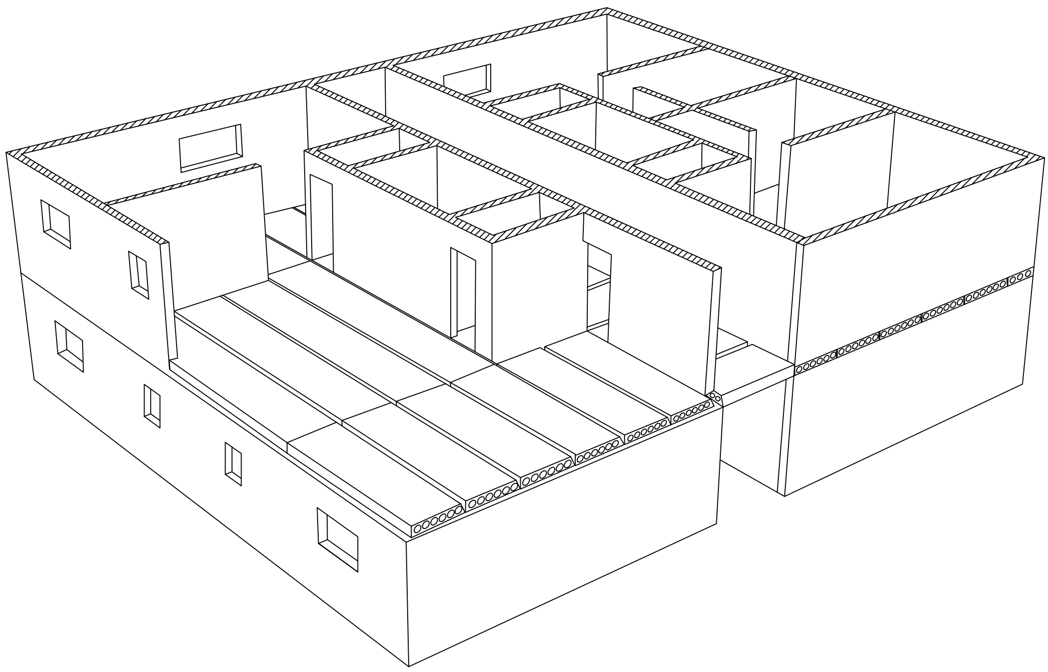


Figure 4.1 Typical use of walls on a loadbearing structure

recent technical advances have led to the introduction of fully composite sandwich panels.

Precast walls are mainly used in industrial buildings and housing and apartments, but also in hotels, hospitals, office buildings and other similar constructions. Precast walls are ideal for fire separating walls when joints are filled with a suitable fire rated sealant.
Load bearing wall panels can also be used to construct lift and stairwell shafts and can be connected after erection to form composite T, L, U or box-shaped sections. Where possible the design should be based on the walls being individual elements rather than trying to combine them into a box. Connections between elements on site are time consuming and the cost of the fixings can become very expensive. In some exceptional cases, cell units are totally precast or pre-assembled at the plant.

The advantage of precast cores and shafts over cast in-situ ones lies mainly in the speed of construction, quality of the surface finish and the improved opportunities in organizing the erection of the structure.

Figure 4.2 Example of precast core with multi-level panels

4.2.2 Design concepts

There are several different methods used to design wall panels. The most appropriate method is dependent on the panel height to thickness ratio and the ratio of axial to flexural actions.

AS3600, Section 11 [7] covers the design of two types of walls as follows;
(a) Braced walls that are subject to in-plane load effects.
(b) Braced walls that are subject to simultaneous in-plane and out-of-plane load effects and un-braced or cantilevered walls.

Neither of these categories specifically covers precast wall systems that comprise discrete elements. Loadbearing walls that have high height-to-thickness ratios (are slender) and low in-plane compared to out-of-plane effects (high flexure) can generally be designed as flexural slabs in accordance with Section 9 of AS3600 provided second order effects are considered. Loadbearing wall structures with low height-to-thickness ratios (are stiff) and high in-plane compared to out-of-plane effects (high axial loads) can generally be designed as columns in accordance with Section 10 of AS3600.

When designing walls as deep beams a strut and tie analysis may be required. This means walls need to be thick enough to accommodate the necessary reinforcement, particularly confinement reinforcement at the supports.

4.2.2.1 Braced wall structures.

On Braced wall structures, (see Section 3.2.1) where wall panels generally have large height to thickness ratios and low vertical loads, the panels act more like flexural members due to out-of-plane loads and eccentricity of axial load. Second order (P-delta) effects induced by horizontal actions need to be taken into account in the design. The design of such wall panels is outside the scope of AS3600 and the BCA [10]. The Concrete Institute of Australia [11] and Cement Concrete & Aggregates Australia [12] both have extensive publications on this topic that include design methods and design charts. There are now also a number of software programmes available that are specifically written for the design of ‘tilt-up wall panels’ that take the second order effects into account.

Axial load eccentricity should include a realistic value for construction tolerance as well as roof framing eccentricity. The bending moment from eccentricity of the roof framing can be exacerbated if the connection provides any degree of continuity, whether intentional or not. This highlights the care required in detailing roof framing-to-wall connections to ensure that unintentional load, particularly flexure is not introduced into the wall. Figure 4.3 shows a typical elevation of a braced wall with design cases and load model for wall design.

If vertical dowel connections into ducts in the wall are used the minimum thickness of the wall panel should be 125mm. For panels up to 180mm thick a single layer of reinforcement is generally used and above that two layers. Note that it is usually more economical to use a thicker panel with a single layer of reinforcement than to provide two layers in a thinner panel.

Overlying the above structural aspects is the Building Code of Australia [10] requirement to design and construct perimeter walls so they do not collapse outward if the supporting structure is removed by fire. This is a performance requirement and there have been a number of documents published on this topic with various solutions on how to comply. The CSIRO Division of Building, Technical Report 93/2 [13] discusses this topic and describes the likely fire performance of one and two storey buildings with precast concrete loadbearing walls or cladding. The detailed solution to meeting this BCA performance requirement is dependent on the wall height, roof framing system and building geometry.

4.2.2.2 Loadbearing wall structures.

On loadbearing wall structures (see Clause 3.2.3) axial loads in wall panels are generally high compared with in-plane and out-of-plane loads. Loads from floors and upper wall elements are also transmitted to the lower walls with a certain eccentricity and this introduces bending moments in the walls and tensile forces in the connections to the floor diaphragm.

The design of these wall panels is based on the principle of hinged connections between the elements. Wall eccentricities can be critical due to the relative slenderness of the wall element. The following eccentricities should be considered in the design of the walls and the connections to the floor:

a) Structural eccentricities.
- eccentricity of the floor support on the lower wall.
- eccentricity of the load from the upper wall unit.
- eccentricity of the self-weight of the panel.

b) Eccentricities due to geometric imperfections and construction tolerances.
- variations in panel flatness.
- variations of panel alignment at erection.
- variations due to non-uniform grouting of horizontal joints.

a) Structural eccentricities

For a precast floor simply supported on the lower wall as shown in figure 4.4, the total load is transferred to the wall with an eccentricity efl. When the floor element is placed without a supporting pad or mortar, (rigid bearing) the location of (G + Q)floor is at about 1/3 of the support length. In the case of mortar or bearing pads, the contact pressure is assumed to be uniformly distributed and the floor load is located at the centre of the support.

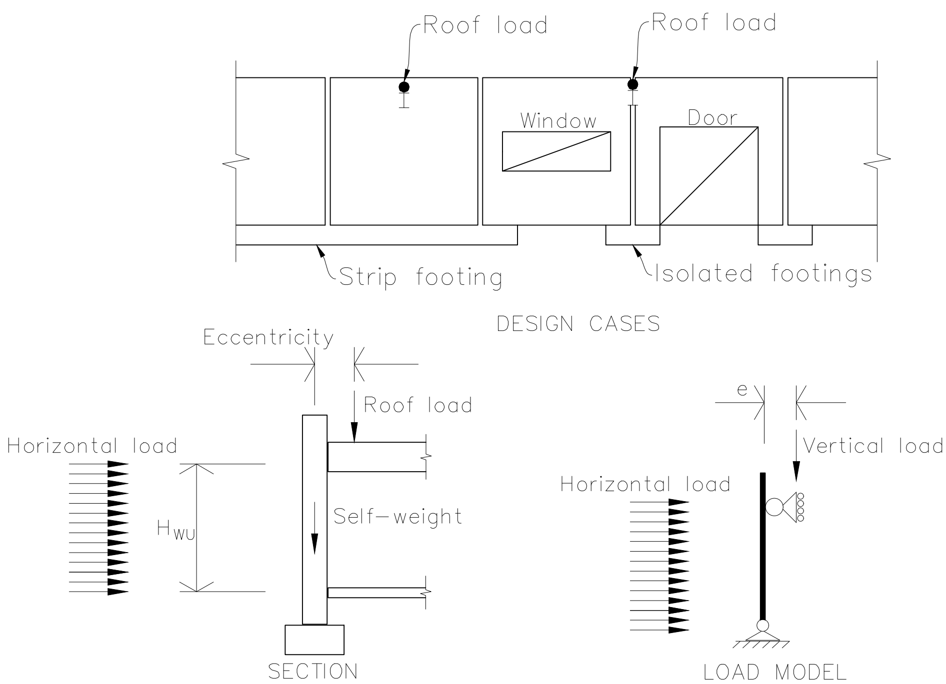


Figure 4.3 Design cases and load model for braced wall design

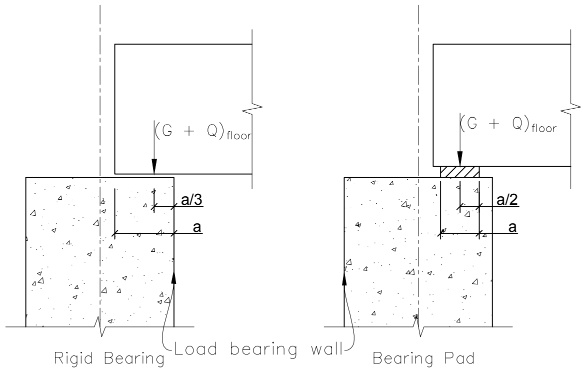


Figure 4.4 Eccentricity of floor load

For precast floors with insitu toppings that provide restrained at supports, the floor load is applied in two steps with two different eccentricities. Gfl is the part of the load which is transferred to the wall before hardening of the in-situ concrete. The eccentricity is the same as for simple supported floors. Qfl is the part of the loading transferred after hardening of the in-situ concrete and is applied at the centre of the wall. It should be noted that these eccentricities do not take any possible positioning tolerances into account.

The forces acting on the lower of a series of walls that also supports a simply supported precast floor are shown in Figure 4.5. The floor eccentricity in relation to its support on the corbel is as described above and the effective design eccentricity for the lower wall panel is calculated from the sum of the individual eccentricities. Note that the worst case of any possible positioning tolerances should be taken into account.

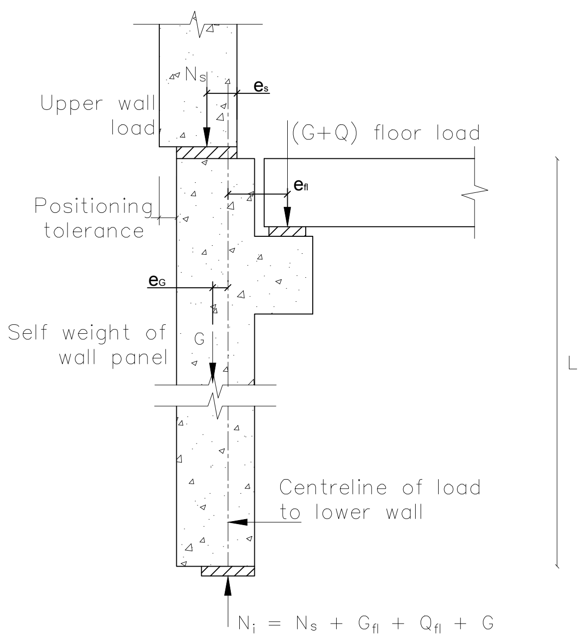


Figure 4.5 Eccentricities on floor wall system

b) Eccentricities due to geometric imperfections and construction tolerances

Allowance needs to be made for an eccentricity introduced by bowing or lack of flatness of wall panels. There are no specific code requirements but the allowable flatness tolerance can be used as guidance. The Precast Concrete Handbook [2] nominates a figure of 1mm/metre height of panel.

The unfavourable effects of possible erection deviations in the geometry of the structure and the position of the loads need to be taken into account in the analysis of members and structures. For members with axial compression the forces resulting from any deviations from floor to floor as shown on Figure 4.6 need to be taken into account.

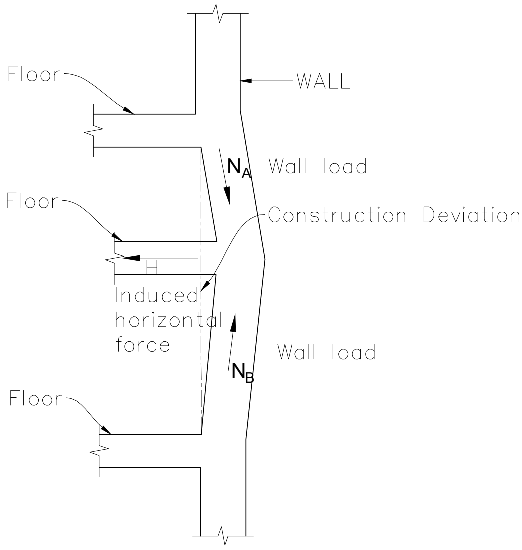


Figure 4.6 Effect of inclination of wall panels

This construction deviation induces a horizontal force H at floor level that needs to be resisted by the floor diaphragm. These deviations can also affect the design eccentricity for individual wall panels. The allowable tolerance deviation at each floor level can be used as a guide to calculate H. The Precast Concrete Handbook gives 20mm as the limit of deviation from the specified position in plan.

The eccentricities due to tolerances and geometric imperfections are not necessarily cumulative and careful assessment is needed in determining design values. Once axial load and eccentricity have been established the wall section can be designed in accordance with AS3600.

4.2.2.3 Precast façade systems.

CConcrete facades can be designed as load-bearing or non-load-bearing walls, depending on their function within the building,. Being the external face of the building most facades will have some type of finish, either applied after erection or preferably integral with the panel. Such finishes are described in detail in the 'Precast Concrete Handbook'. [2]

Load-bearing façades support the vertical loads from the floors and the structure above. They can also contribute to the horizontal stability of the building. Figure 3.7 shows a loadbearing façade structure. The design concepts for loadbearing facades are as described in the previous sections.

A loadbearing façade can also be composed of load bearing spandrel panels or beams as shown in Figure 4.7. Here the spandrel elements act like beams, transferring vertical loads to the columns.

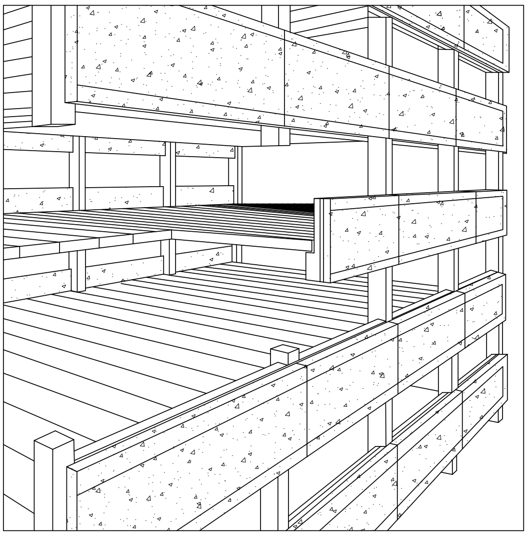


Figure 4.7 Load-bearing spandrel beams

Non load-bearing façades fulfil only an enclosing function. The elements are either individually fixed to the structure of the building or they can be supported from the ground or a load transfer element. In the first case the structure of the building supports the self-weight of the cladding elements at each level. In the second case the elements are stacked vertically and are only anchored horizontally to the structure. In principle, the shape of the elements can be designed without any restriction. Non load-bearing façade elements are described in detail in the Precast Concrete Handbook [2].

When combining non-loadbearing precast supported on the ground with precast supported on a floor by floor basis the relative movements that can occur between the two need to be taken into account.

4.2.3 Composite wall systems

The horizontal stability, or bracing of a loadbearing wall structure, is assured by means of shear wall action, usually involving a series of individual or connected walls. Walls resist loads only in their own plane and may need to be combined with other walls perpendicular to their plane or with assembled core structures to provide the required stiffness. (See Fig. 4.1). Composite action of adjacent walls forming L, H, or T shapes are possible on condition that the vertical joints between the panels can transfer the required forces.

Wall panels connected to form L, H, or T shapes or box-shaped sections for stair or lift shafts will generally not provide the same stiffness as an insitu section. This is because the connections between elements are discrete and result in stress concentrations and slight yielding. This reduced stiffness needs to be considered in the design.

Behaviour of composite wall systems is discussed in greater detail in Chapter 5, Structural Stability.

4.2.4 Manufacture

Most precast walls are manufactured on horizontal steel tables or casting beds. Some manufacturers use tilting tables to allow early de-moulding but with modern concrete mix designs, early strengths are usually sufficient to allow lifting on a one day casting cycle without needing to tilt the beds. Battery moulds are rare in Australia and are only used where considerable repetition in panel size and shape occur.

For reasons of stability at de-moulding and handling, minimum dimensions of lintels and mullions between windows and at panel edges are required. Figure 4.8 gives an example of recommended dimensions for typical 150mm thick panels. On irregular shaped panels or where minimum dimensions are structurally insufficient temporary strong-backs can be used to stiffen the panel to allow lifting, transport and erection.

Service ducts and electrical conduits can be incorporated into the panel prior to casting. The dimensions for door openings and windows are generally unlimited and door and window frames are often cast-in.

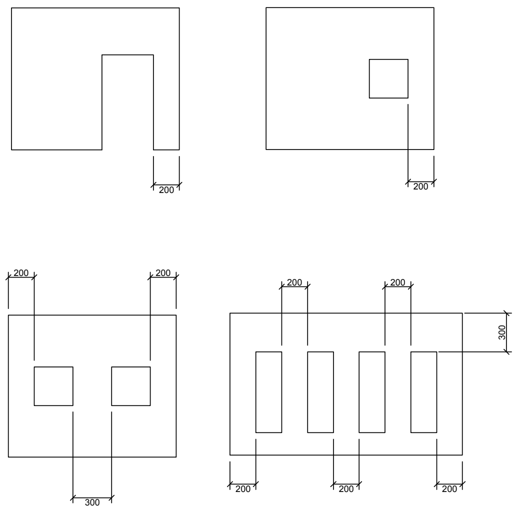


Figure 4.8 Recommended minimum dimension for lintels and mullions

4.2.5 Erection

This document is not intended to cover the transport and erection of elements. These aspects are covered in detail by State and Commonwealth authorities. Many aspects of erection are also covered by statutory requirements under OH&S regulations [3] [35]. Reference should be made to these documents to ensure compliance.

4.3 Columns

4.3.1 Basic principles

PPrecast columns can be square, rectangular or circular, with the size frequently dictated by column-to-beam connection details. Individual elements can be either floor-to-floor height or multi-level.

Besides offering fast and industrialized construction, precast columns can offer a high fire resistance and a smooth and ready to paint surface finish. Textured or profiled finishes can be provided where specified.

Figure 4.9 Precast columns in frame structure

Precast columns are mainly used in frame structures in conjunction with precast beams but can also be used with insitu concrete floors in hybrid structures or with wall cladding panels in braced wall structures.

The column size is function of requirements related to structural capacity, stability, and fire resistance but generally a minimum dimension of 400mm is adopted to accommodate column to beam connections. This minimum dimension can be obtained by providing corbels on smaller or rectangular columns.

Columns may be of constant dimension for the full height of the building or stepped back at intermediate levels to satisfy structural and architectural requirements. As with any form of construction it is desirable to keep the columns in vertical alignment to avoid introducing eccentricity of load or transfer structures.

The relative difficulty in providing moment capacity in column-to-beam connections means that most column connections are designed as pinned. The exception is in low rise structures where lateral stability can be provided by cantilevering columns from the footings. Column-to-column connections are typically grouted dowels projecting from the lower to the upper column. Where beams are supported on top of the column at floor levels the dowels project through ducts in the beam ends. An alternative is to provide a concrete or steel corbel to allow a continuous column. These concepts are discussed in greater detail in Chapter 6.

Figure 4.10 Multi-level columns with corbels

4.3.2 Design concepts

The design of a precast column is the same as that for an insitu column. They are typically designed in accordance with Section 10 of AS3600 for applied axial loads and bending moments that are derived from analysis.

Where precast columns are used as part of a skeletal frame with pinned connections, the structure is subject to staged loading and partial continuity. It is therefore difficult to establish the continuity and eccentrically induced bending moments without carrying out a complex non-linear analysis. A non-linear analysis should be carried out whenever the connection contains complex geometry or indirect load paths. For simple connections load eccentricities can be determined as upper and lower bound limits as set out in the following section. In all cases the connection between column and beam also needs to be designed to resist the induced bending moments.

Confinement reinforcement within the beam connection zone is often critical and can determine the column size. The most economical column is one that maximises the concrete strength and minimises the vertical reinforcement. If available, concrete strengths of 65-80MPa can be used and are usually specified as 56 day strengths.

4.3.2.1 Load eccentricity normal to the frame

A bending moment is induced into the column by eccentricity of load from the floor slab on the supported beam. There are two cases, an interior column and beam or an edge column and beam.

The above diagrams clearly show that the edge column and beam is the worst case for bending moment normal to the frame. The applied actions at each stage of assembly of the frame are calculated and the appropriate eccentricity determined.

For R* due to the precast floor, wet screed and construction load, the upper bound eccentricity is assumed to be from the column centreline to the bearing strip under the precast floor. Once the beam is composite with the slab, torsion is resisted by tie bars (see Section 7.3.6) and the eccentricity of R* due to the additional imposed actions is assumed to be from the column centreline to the outer edge of the column. Summing the forces and eccentricity for each case gives the upper bound design bending moment normal to the frame.

In some cases, particularly for interior columns, the greatest bending moment can be produced during erection where the floor is erected and screeded only on one side of the beam. Beams can be propped during erection to reduce the applied bending moment in the column but in most cases it is more economical to design the column and the beam-column connection so that no propping is required.

4.3.2.2 Load eccentricity in the plane of the frame

A bending moment is induced into the column by eccentricity of load from the supported beams. There are two cases, an interior column and beam or an end column and beam.

a) interior column

To establish the upper bound eccentricity it is generally assumed that one beam reaction is applied at the centreline of the column and the other at a distance of one third of the support length from the face of the

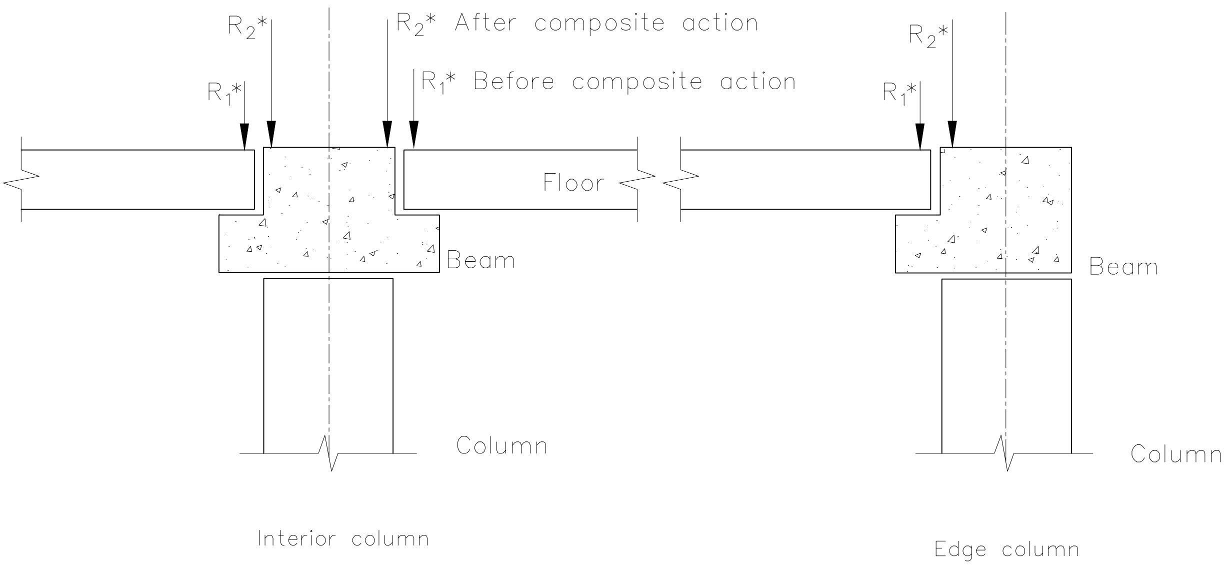


Figure 4.11 Eccentricity normal to frame

column. Although this may seem to be conservative it rarely penalises the column design. With minimum dimensions, columns on low-rise buildings are generously sized and on high-rise buildings axial load is the governing case.

The case for erection with beam and slab on one side should also be checked but this is not normally critical.

b) End column

To establish the upper bound eccentricity it is generally assumed that the beam reaction is applied at a distance of one third of the support length from the inner face of the column.

The final column design is based on biaxial bending using the critical bending moments about each axis.

4.3.3 Manufacture

Most precast columns are designed as reinforced concrete sections and can therefore be manufactured individually or in gang moulds. Pre-tensioning on a long line bed is usually not economical and can only be justified for crack control in long slender columns or on columns with significant lateral loads.

Because of their robustness and the use of high strength concrete, columns can be easily manufactured on a daily casting cycle.

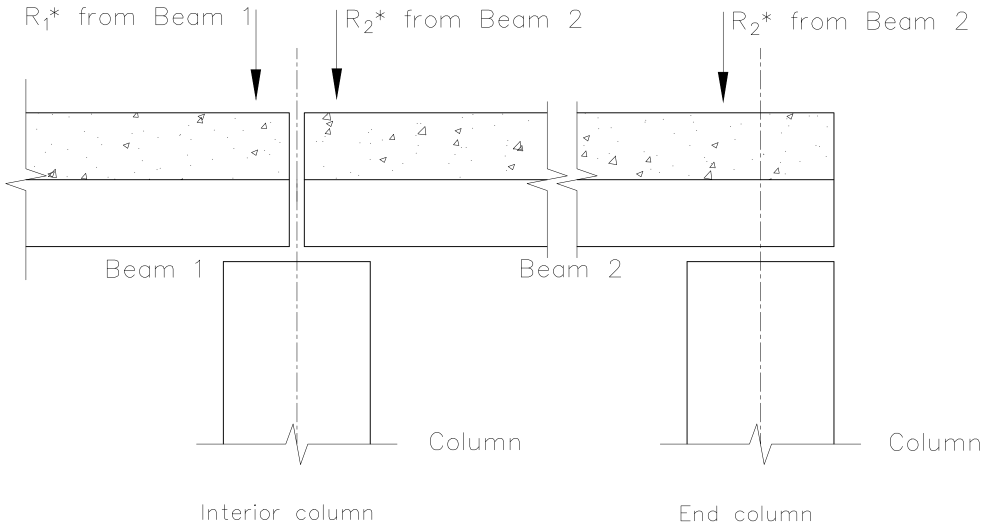


Figure 4.12 Eccentricity in the plane of the frame

4.4 Beams

4.4.1 Basic principles

There are two basic types of precast beams. Solid beams and shell beams.

Solid beams are usually designed to allow erection without requiring falsework and are available in various profiles and dimensions. They range from rectangular to inverted T or L shape cross sections and contain all of the main reinforcement within the solid section. They can be designed to act as either composite or non-composite with the floor system.

To optimise use of material and labour, beams are usually pre-stressed and manufactured on a long line pre-tensioning bed The use of concrete strengths of 50MPa allows early de-stressing and results in fast and industrialized production.

Shell beams are composite precast members that can contain part or all of the main positive reinforcement and stirrups within a minimum volume of concrete for weight and ease of handling. They are generally U shaped in cross section and act compositely with the infill concrete and the flooring. Unlike solid beams they almost always need to be supported on falsework during erection.

The width and depth of beams is a function of structural capacity, stability, and fire resistance but is often dictated by the requirements of supporting a precast floor system. Maximum weight for handling and transport is another limiting factor. Typical spans range from 4 to 16 metres with the range 6 to 9 metres being the most common.

Most precast beams are used with precast floors as part of a pin jointed skeletal frame structure but can also be used with insitu concrete floors in hybrid structures. Although beam elements usually span from column-to-column many variations are possible with cantilevers and infill beams used to optimise the bending moment capacities. To this end they are frequently designed as partially continuous for imposed loads. The most economical solution is generally obtained by maximising the positive bending moment capacity and minimising the required negative bending moments.

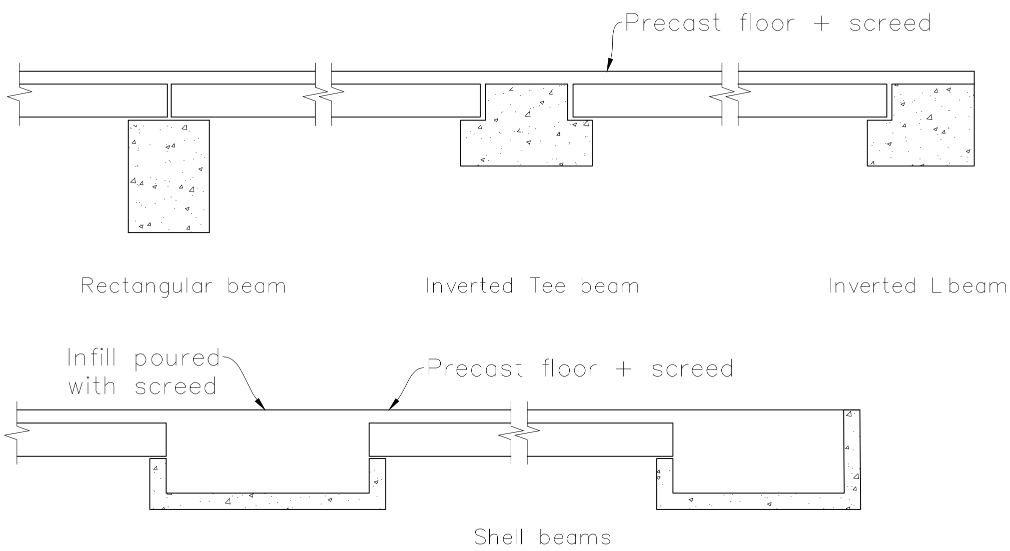


Figure 4.13 Typical precast beam profiles

Beam support on columns can be by direct bearing on top of each column or by half joints on the beam ends supported on concealed corbels on the columns. A variation is the use of proprietary concealed steel corbels and supports cast into the beam and column. Top bearing simplifies the provision of continuity of negative reinforcement in the beam but where columns run past the beams this can be obtained by passing bars through ducts in the column.

These concepts are discussed in greater detail in Chapter 6.

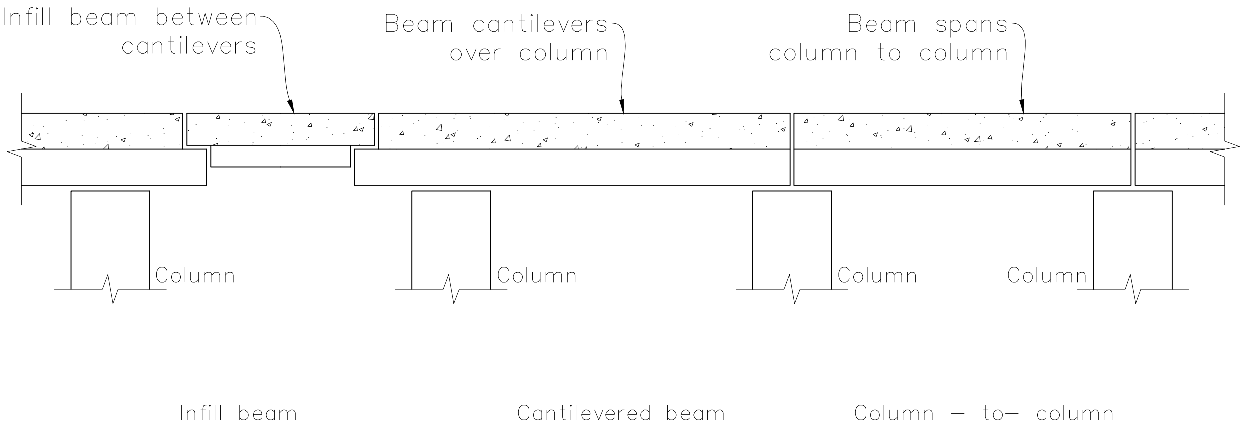


Figure 4.14 Beam variations

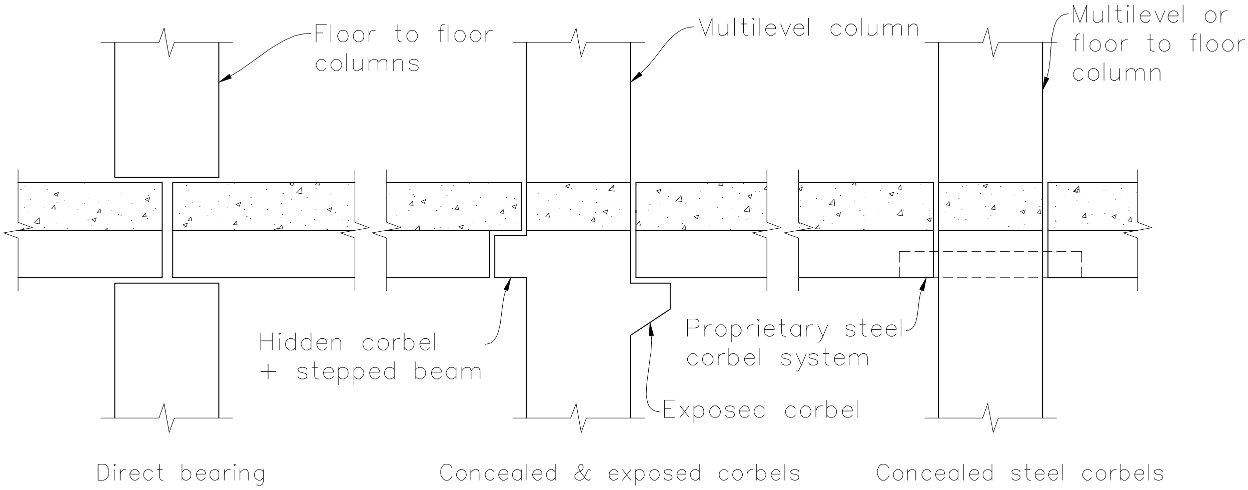


Figure 4.15 Beam support

4.4.2 Design concepts

The design of a precast beam is the same as for any other flexural section and should be in accordance with Section 8 of AS3600. Where precast beams are used as part of a complete floor system, the structure is subject to staged loading and the beams can be provided with partial continuity. The segmental construction and staged loading process of a precast structure also means that the method of analysis is different for different types of beams.

4.4.2.1 Non-composite beams

Non-composite beams are usually designed to span simply supported from column to column. They are typically rectangular and only the strength of the basic beam is considered with the supported floor slab or any other loads treated as imposed actions. The analysis and design of such beams is simple and need only consider direct and torsional loads. Support conditions and termination of the flexural reinforcement need more careful consideration. Where beam connections involve vertical ducts for dowels, hairpin reinforcement round the duct can be detailed to act as anchorage for the flexural reinforcement along with confinement to the vertical dowel.

4.4.2.2 Composite beams

Composite beams, whether solid or shell profiles, involve a multi-stage analysis and a design process which reflects the assembly process of the precast structure. They can be of any profile and the strength of the basic beam is enhanced by incorporating the floor slab into the beam as a composite profile. Being composite means that it is simple to provide continuity over the supports for loads imposed after composite action is formed. This is referred to as partial continuity because only the superimposed part of the total load produces continuity.

Analysis of composite action requires the effective width of the compression flange to be determined in the same way as for monolithic structures. Where the floor is a solid precast slab with topping the section is assumed

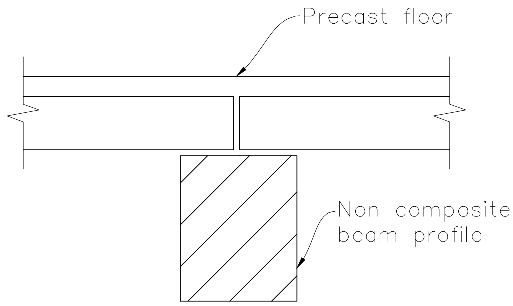


Figure 4.16 Non-composite beam & precast floor

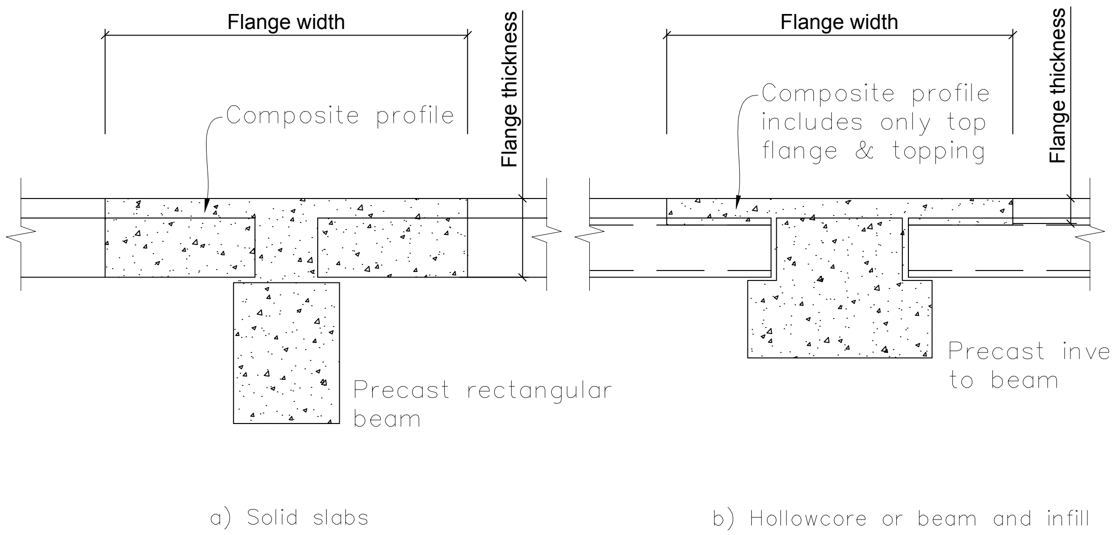


Figure 4.17 Composite beam profiles

to be monolithic and the flange width determined in accordance with AS3600, Section 8.8.2 and as shown in figure 4.17(a). For hollowcore slabs a reduced effective flange thickness is used which is based on the geometry of the cross section as shown in figure 4.17(b).
A more detailed discussion on this topic is given in fib Guide to good practice, Composite floor structures. [14]

For composite action to occur the interface between the precast units and the insitu topping must be able to transfer the longitudinal shear forces. A roughened interface on the precast and reinforcement across the interface is usually required. This can be determined in accordance with AS3600 Section 8.4.

The analytical and design process to include composite action and partial continuity involves the following staged process.

- Beam analysis for loads before composite action.
- Beam analysis for loads after composite action.

a) Analysis for loads before composite action.

The actions applied to the basic beam are determined and the bending moments and shear forces established with the beam as a simply supported member from column to column. Unless the beam is supported on falsework these loads are usually the floor slab, topping screed and construction loads and should include any torsion from load eccentricity. If the beam is supported on falsework the supported loads do not contribute to the bending moments induced before composite action.

b) Analysis for loads after composite action.

The actions applied to the beam after the composite section is formed are determined. These actions are usually the superimposed dead and live loads. Torsional loading may or may not be applicable depending on the tie reinforcement detailing between beam and floor. The positive and negative bending moments and shear forces with the beam as a continuous member are then established. Because the beam-to-column connections are designed as non-rigid only the beam is considered and the influence of any columns ignored. Note that 'pattern or skip loading' on adjacent spans may produce the worst case bending moments and give maximum and minimum values at each location.

The final design requirements for bending and shear are a combination of a) and b). For the worst case negative bending, the maximum values obtained from the analysis for loads after composite action are used, case b). These may be redistributed in accordance with Section 6.2.7 of AS3600. For the worst case positive bending, the maximum values obtained from the sum of a) and b) are used, including any increase due to redistribution of the negative bending moments.

The above process can be carried out as a staged design as above but there are a number of software programmes that automate the process. Analysis and design as a single composite beam without taking into account the staged loading process can lead to erroneous results.

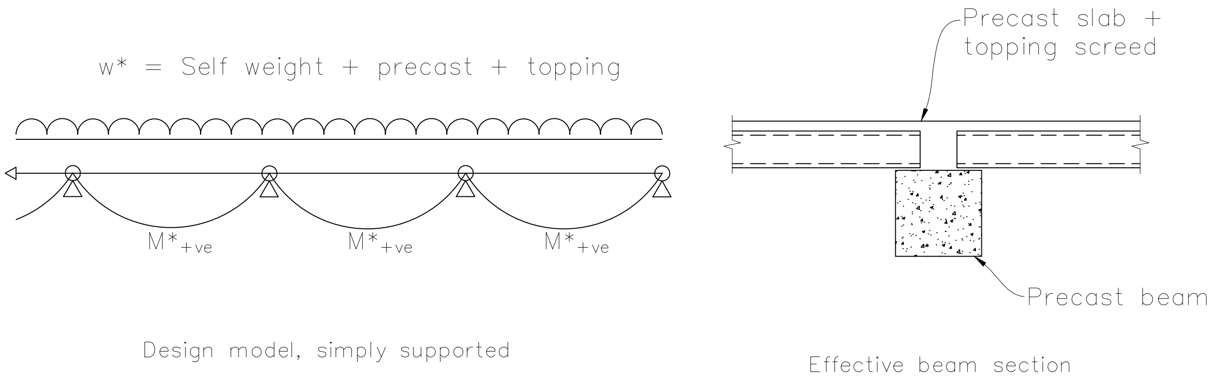


Figure 4.18 Design model, Beam before composite action

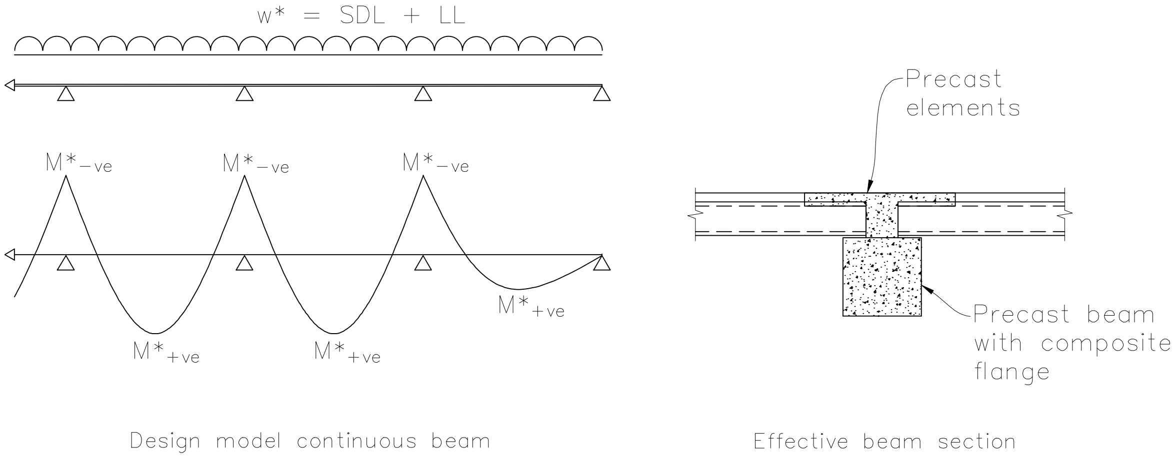


Figure 4.19 Design model, composite beam

4.4.3 Manufacture

Most beams are designed as pre-tensioned and therefore manufactured on a long line casting bed. Bed lengths of 40-60 metres are common and usually allow for two or three standard beam profiles and widths, and a range of depths and ledge heights.

Planning the manufacturing sequence to optimise the use of the casting bed can be complex. Changing the moulds to suit different beam profiles is expensive and time consuming so there is good economic reasons to minimise the number of beam profiles on a project. Also changing moulds to suit different beam profiles may not be compatible with the site erection process and can therefore result in scheduling issues.

Reinforced concrete beams can be cast in individual forms resulting in much more flexibility in the production process.

4.5 Floor slabs

4.5.1 General

Precast floors slabs offer many advantages over cast in situ floors. The principal advantages are short construction time, high structural performance, durability, large span capacity, absence or minimisation of falsework, large variety of types and economy.

Precast floors slabs are used extensively in all types of buildings, not only for totally precast structures, but also in combination with other materials. The choice of a flooring system varies in each building and depends on structural capacity as well as on transport and lifting facilities, availability on the market and building.

The main structural requirements of floors are load capacity, stiffness, transverse load distribution of concentrated loading and distribution of horizontal actions by in-plane diaphragm action. In addition, depending on their use, floors can also fulfil other requirements, such as thermal and acoustic insulation, fire resistance etc.

4.5.2 Types of precast floor elements

Precast floor slabs are described in detail in the Precast Concrete Handbook [2]. The main types of precast floor elements available on the Australian market are.

1. Hollowcore slabs.
2. Beam and infill slabs.
3. Permanent formwork slabs.
4. Composite solid slabs.
5. Double or single Tee beams.

The self-weight of the floor elements varies considerably from type to type. This can range from a few 100 kg for beam and infill to several tonnes for thick solid slabs and tee units. The choice of the most appropriate floor may therefore depend on the size of the project and the available lifting capacity.
The soffit of precast floors can be ribbed or flat and with or without thermal insulation. Thermal insulation may be required where the floor separates air-conditioned and non-air-conditioned spaces. Flat soffit floors permit slender floor structures. This is especially the case for prestressed hollow core slabs

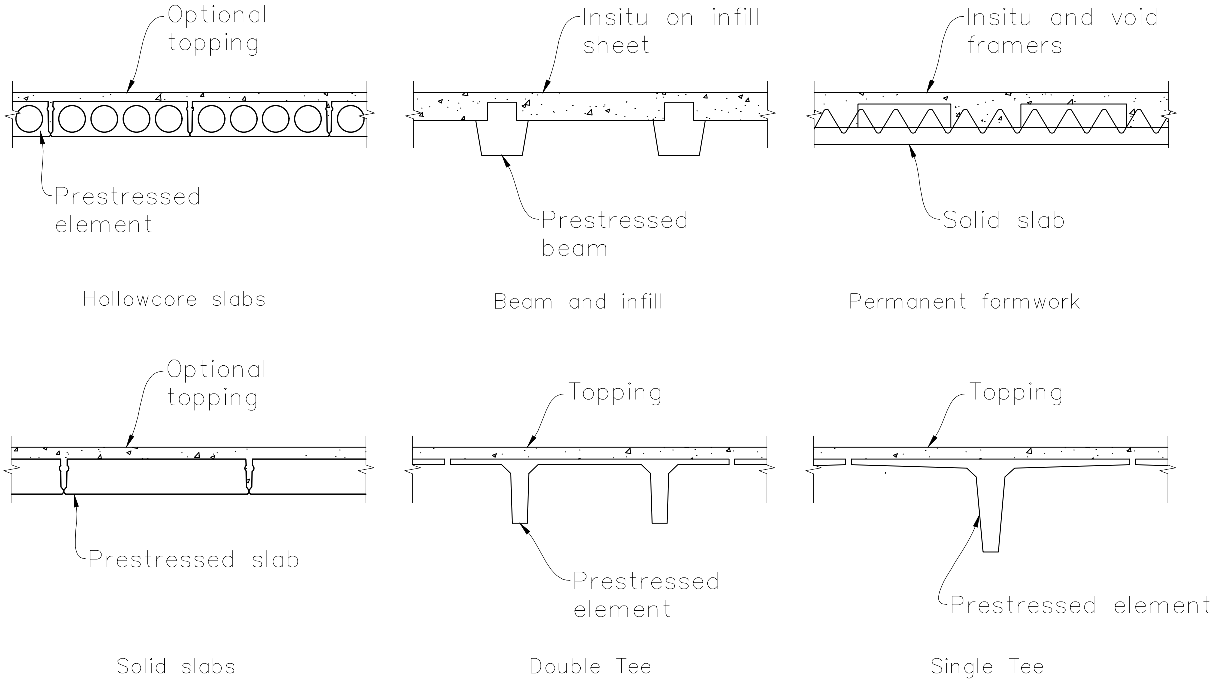


Figure 4.20 Precast floor elements

All floor types can be designed to cantilever to form projecting floors or balconies. Some types can be designed to cantilever directly over edge supports while others can combine separate cantilevering elements that are erected and supported on falsework until a reinforced topping screed is completed.

Acoustic property can be an important criterion in the choice of the floor type, especially in residential buildings. Concrete floors can easily accommodate the required performances for airborne capacity but like any other type of slab usually require additional measures for contact noise transmission.

This Design Guide will focus only on the use of hollowcore floor slabs. Information on the alternatives shown in Figure 4.20 is available from each of the manufacturers.

4.5.2.1 Hollowcore slabs

Prestressed hollow core slabs are usually 1200mm wide units with longitudinal cores to reduce the weight of the floor. They are prestressed and produced on long line casting beds using extrusion or slide form methods. After hardening, the elements are saw cut to the specified length. A rectangular end is standard, but skew or cranked ends, which are necessary in a non-rectangular floor plan, may be specified. Slabs can be cut to width after casting to suit the building dimensions. The edges of the slabs are profiled to ensure adequate vertical shear transfer across the grouted joint between adjacent units.

Thicknesses vary from 150-400mm and although they can be used as plain sections they are usually topped with a structural screed to give a composite slab. Span-to-depth ratios should be limited to about 35:1 but can be used up to 45:1 if serviceability requirements are checked.

Hollowcore slabs are designed to be erected and screeded without requiring temporary support.

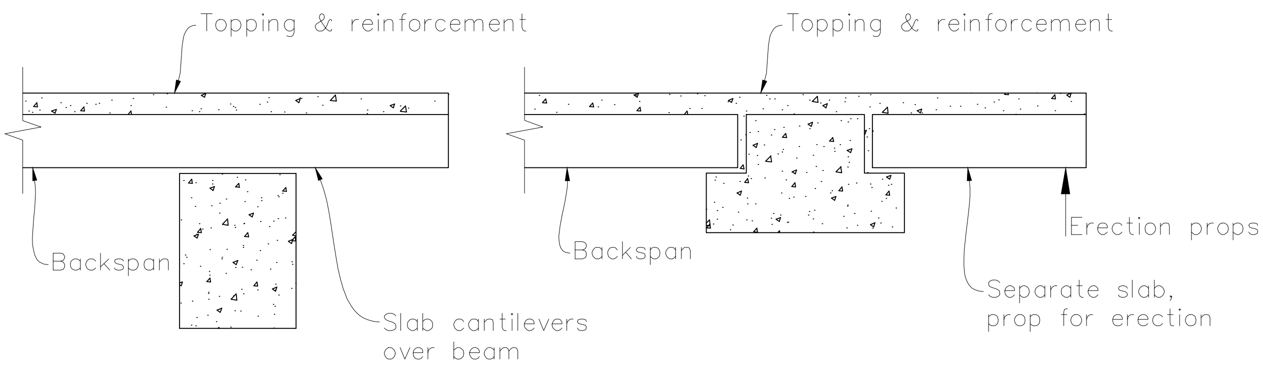


Figure 4.21 Cantilevering slab alternatives

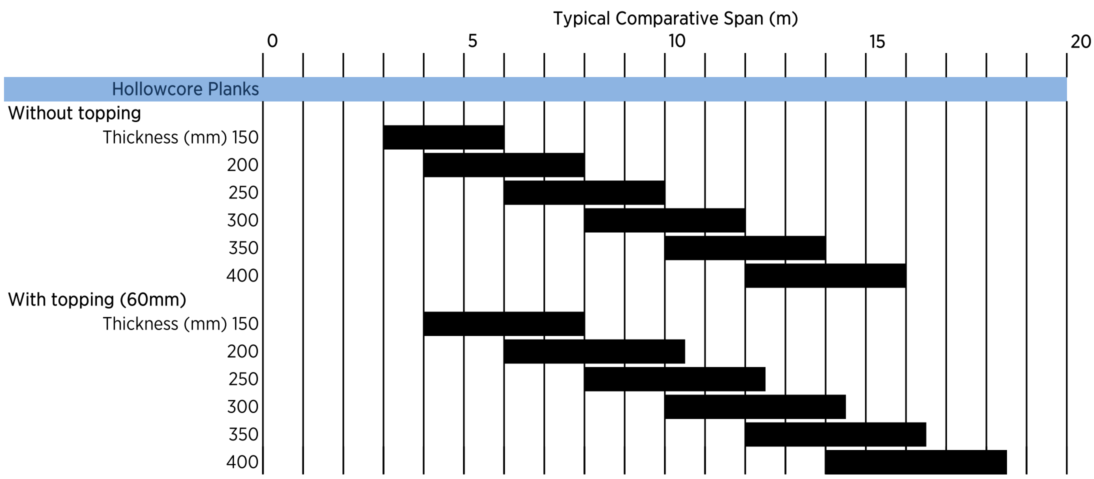


Figure 4.22 Comparative spans for hollowcore floor slabs.

Hollowcore slabs are mainly used in buildings with spans in the range of about 6 to 16 metres such as office buildings, hospitals, schools, shopping centres etc. Because of their flat soffit another common use is in apartment buildings.

Figure 4.23 Hollowcore floor

4.5.3 Design concepts for precast floor elements

The design and calculation of a precast floor is carried out in two steps:

- The individual slab elements
- The design of the whole floor.

Individual slab elements are dimensioned with respect to flexural capacity and shear resistance to the applied loads including torsion when relevant. Once elements are connected together serviceability criteria such as deflection, fire resistance, acoustic and thermal properties and durability are checked to ensure they meet recommended values. Other design criteria that may need to be considered include handling and construction methods.

The design of the individual elements is in accordance with AS3600 or other international standards and other selected literature. [4, 15, 16, 18]

Guideline information on performance is available in brochures and technical literature from each of the manufacturers. [3] These provide pre-calculated standard performance curves giving the allowable variable load in relation to span length and the reinforcement.

The following sections give information on specific design rules for hollowcore slabs in so far as they are not covered by the classical design procedures for reinforced and prestressed concrete members.

4.5.3.1 Prestressed hollowcore slabs

Prestressed hollowcore slabs are unique in that they have no reinforcement other than the longitudinal pre-stressing tendons anchored by bond. The detailed design of these elements is not covered by AS3600.

Owing to the absence of complementary reinforcement at the support and in the transverse direction, the tensile strength of the concrete has to be taken into account for the determination of the shear capacity and lateral load distribution. For calculation purposes each hollowcore slab can be modelled as a series of side by side 'I' sections. Shear transfer between adjacent 'I' sections is by aggregate interlock.

As in any prestressed concrete element, the design shear capacity is calculated for two conditions: the un-cracked section near the support, (shear tension) and the cracked section in flexure (flexural shear). Zones 1 & 2 in Figure 4.24.

Shear tension failure occurs near the support when the shear force combined with pre-stressing force exceeds the tensile capacity of the concrete and a diagonal or inclined crack initiates failure.

Flexural shear failure occurs when the shear force exceeds the shear compression capacity and a single flexural crack initiates the shear failure.

Because of web profile, the classic beam shear equations as given in AS3600 do not necessarily give accurate values for shear tension capacity of hollowcore slabs. This is particularly so for elements with depths greater than 300mm and with non-circular cores. For these elements the calculated capacity can considerably over-estimated the actual capacity.

Classic beam shear equations assume the critical section is at the centreline of the cross-section whereas with non-circular cores it is much nearer the bottom of the cross section where interaction with the pre-stressing forces can dominate. Unless an accurate shear analysis such as that given in European Product Standard EN-1168 [16] is carried out then a capacity reduction factor should be applied to the calculated shear capacity. As a guide the ACI [17] recommends that if designed in accordance with the ACI-318 beam shear equations, then an additional capacity reduction factor of 0.5 be applied to the calculated shear capacity of hollowcore slabs with depths greater than 320mm. Although this is a significant reduction, shear is seldom critical on

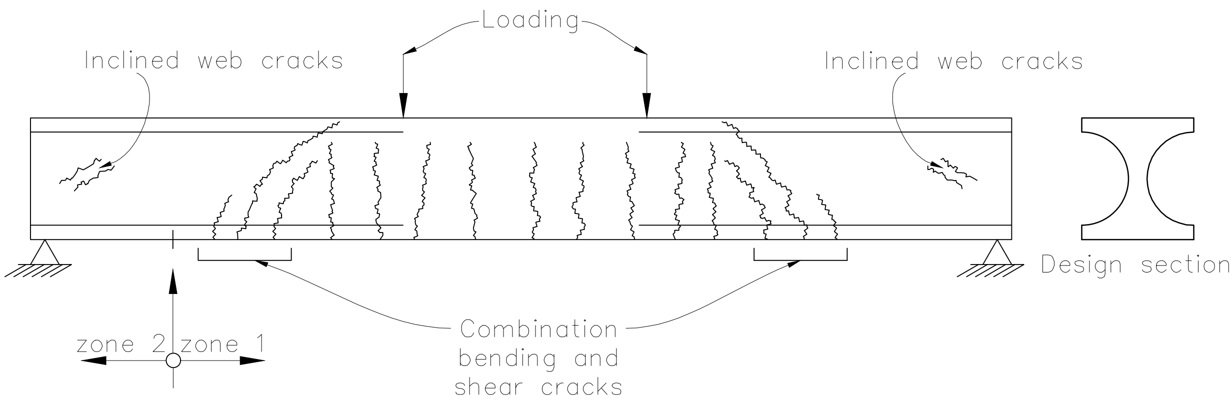


Figure 4.24 Cracking zones and pattern in a prestressed hollowcore

typically loaded slabs. Where it is critical, the shear capacity can be increased by filling the cores at the ends of the slab with concrete from the topping screed.

In most uses hollowcore slabs are designed as composite with a reinforced concrete topping. The topping is kept to about 60mm thick, the minimum required to contain the topping reinforcement. Thicker toppings do not necessarily increase the capacity of the composite section and on longer spans may actually reduce capacity. Surface roughness and surface preparation is important in ensuring full composite action occurs. Roughness is specified in AS3600 [7] and at the time of pouring the screed the surface should be 'surface saturated dry', damp but with no water pooling.

Because of the complexities in the design of hollowcore slabs most hollowcore manufacturers provide their own design service and the elements are supplied on a design and manufacture basis.

4.5.4 Design of the complete floor

The design of the complete precast floor structure concerns the connection of the individually designed and assembled elements to form a coherent and stable structure. The most important objectives are:

- Structural integrity
- Diaphragm action of the floor for the transmission of the horizontal actions into the stabilising walls or frames.
- Transversal distribution of concentrated loading
- Treatment of openings and cut-outs

4.5.4.1 Structural integrity

Floor systems, consisting of individual precast concrete units should be connected together to form a single entity by a tying system.

Most precast floor systems used in Australia are provided with structural toppings and these can be reinforced and used to tie the individual elements together. In the rare cases where no structural topping is used the individual elements need to be tied together by other methods such as welded connection plates or strategically placed reinforcement in the joints and across the ends of the elements to prevent lateral displacement and loss of integrity. This is discussed in greater detail in Chapter 5, Structural Stability

4.5.4.2 Diaphragm action

The diaphragm action of floors fulfils an important role in the stability of precast concrete buildings. It assures the transmission and distribution of all horizontal forces to the stabilising walls or frames.

Diaphragm action is usually realized with a reinforced structural topping screed, cast over the whole floor area. The connections of the screed with the stabilizing components should be designed accordingly. Force paths and design can be derived from strut and tie models.

If no structural topping screed is used then peripheral tie-reinforcement is required to take up the tensile forces arising from the in-plane bending. Shear forces are concentrated along the longitudinal joints between the floor slabs. Force paths for design can be derived from strut and tie models.

Diaphragm action of floors is described in greater detail in Chapter 5, Structural Stability

4.5.4.3 Transverse distribution of concentrated loading

Floors are usually designed not only to carrying uniformly distributed loads, but also concentrated line or point loads. The degree of load transfer between adjacent elements of hollowcore slabs depends on the torsion stiffness and the longitudinal and transverse flexural stiffness of the elements and how well the lateral displacement in relation to each other is restricted. Even in the case of cracked joints, shear forces will be transmitted across the cracks due to the presence of lateral compressive stresses originating from the torsion of the elements and the shear-friction mechanism.
Where the ability to distribute load between adjacent elements is high the concentrated loading is spread over a number of adjacent elements.

With a concentrated load on the slab, the element on which the load is applied deflects. Because slab elements are connected by grouted joints, transverse ties or an insitu infill the elements adjacent to the loaded slab are also forced to deflect and so the effect of a concentrated load is distributed to a wider area than the directly loaded slab element.

Much research has been carried out on the subject of transverse load distribution, particularly on hollow-core floors. The results show that the concentrated load is distributed over several adjacent units, almost as in a monolithic floor. The calculation model is based on the theory of elasticity. The elements are regarded as isotropic slabs and the longitudinal joints as hinges, in other words they can transmit shear forces but not bending moments.

Transversal load distribution may be taken into account if the following conditions are fulfilled:

- The longitudinal joints or infills between the elements are designed to take up vertical shear forces.
- The lateral displacement of adjacent elements is limited by a structural topping screed or by transverse tie reinforcement in the screed or at the support.

The determination of the possible load distribution can be done either with the help of graphs and tables or by more complicated analytical calculations.

In most cases, the use of graphs and tables showing the relationship between the element span, load configuration and load distribution will suffice. Finite element analysis can also be used but the complexity of the input rarely warrants this approach.

Load distribution graphs for hollowcore slabs are provided in the Precast Concrete Handbook. [2] These graphs and tables are based on both analytical calculations and tests.

4.5.4.4 Openings and penetrations

Openings in precast floors can be provided by a wide variety of methods and in a variety of sizes and positions. The methods depend not only on the slab type but on the size and location of the penetration. It is important to determine the locations of all major penetrations at the time of design so they can be incorporated prior to manufacture. This also applies to identifying wet areas where multiple plumbing penetrations frequently occur.

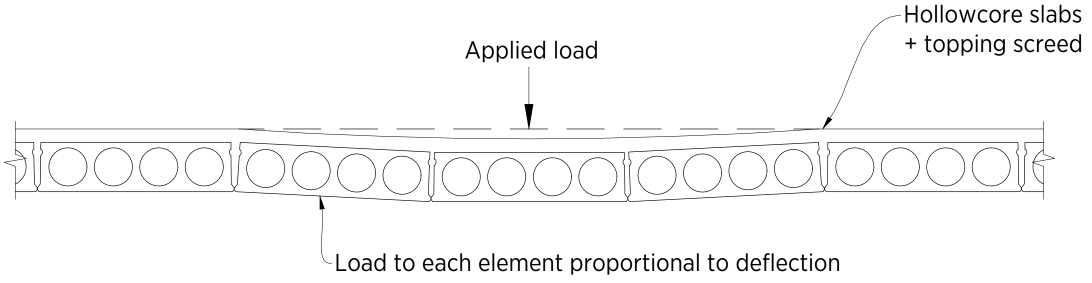


Figure 4.25 Load distribution, beam and infill slab

For hollowcore slabs penetrations are realised in one of two ways depending on the size. Small penetrations of less than about 300/400 mm in size may be formed in the element during the manufacturing stage and before the concrete has hardened or by saw-cutting the slabs after they are installed and grouted. The maximum size of the holes depends on the size of the voids in the slab and how much reinforcement may be removed without jeopardizing the strength of the unit. As a guide such penetrations should not cut more than one third of the total area of flexural reinforcement in any single slab. See figure 4.26.
No penetrations should be provided near the edge of unsupported hollow core slabs when any strands will be cut. For slabs with penetrations made before erection the behaviour of the slab during handling, transport and erection should be analysed. Small penetrations can also be added after erection of the slabs by drilling one or several individual cylindrical holes through the cores in slab. Coring or cutting of holes is not permitted without the approval of the designer.

Large penetrations or voids that cannot be incorporated within the hollow-core element can be accommodated by providing steel trimmer beams or cast in-situ trimmer beams to carry the supported element as shown in Figure 4.27.

The method involves designing the trimmer to carry the self weight and superimposed loading of the trimmed units and distributing the reaction from the trimmer beam onto the supporting slab and adjacent elements in accordance with the theory for a point load on the edge of a slab. The flexural and shear capacity of the supporting slab is checked taking into account the torsional effect of the load on the slab. This method gives relatively conservative capacities for smaller penetrations where the effect of the discontinuity caused by the penetration in the slab field is small. It gives more accurate capacities for larger penetrations where the supporting slab acts more like a single slab with edge loads than as part of a slab field. When large penetrations occur close to the support the effects of torsion and reduced shear width of the supporting slab must be taken into account. This topic is covered in detail in fib Precast prestressed hollow core floors manual. [30]

An alternative design method involves modelling the slab field and penetration using finite element methods (FEM calculations). This is rarely warranted for everyday slab-systems but can give a more accurate solution if large openings occur in unfavourable positions. To use FEM for the analysis of hollow core floor systems requires a detailed understanding of how individual hollow core slabs interact to form a slab field.

Figure 4.27 Use of trimmer angles or beams for large voids

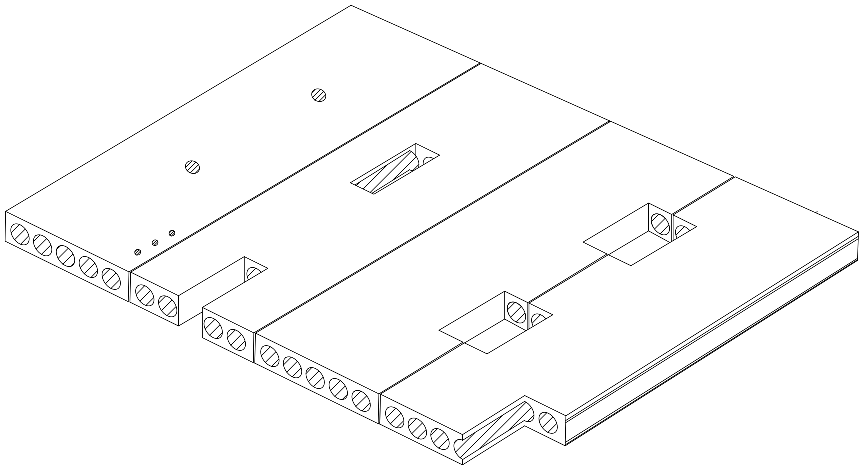


Figure 4.26 Example of possible penetrations in hollowcore elements

4.6 Balconies

Balconies are an integral part of residential buildings. They can be constructed within the floor plate or cantilevered beyond the wall line. In most cases a set-down in floor level is required to provide weathering. Similar situations with cantilevering floors occur on other types of buildings and the treatment is essentially the same.

For hollowcore slabs the treatment depends on the direction of span of the floor relative to the cantilever. Figure 4.29 shows a typical solution where the floor spans at right angles to the cantilever span. This frequently occurs on apartment buildings comprising loadbearing cross walls between apartments and non-bearing façade walls. A precast floor element that incorporates the cantilevered balcony spans between the loadbearing cross walls. The cantilevered portion is generally pre-finished and the element is tied into the floor by reinforcement in the screed.

Figure 4.28 Cantilevered balconies

Where the precast floor elements span in the same direction as the cantilever the usual solution is to erect a precast balcony element onto falsework and tie it back into the floor slab via reinforcement in the screed as shown in figure 4.30. Detailing of the reinforcement is particularly important to ensure that cantilever action is properly achieved.

An alternative to the above is to cantilever the precast floor over the loadbearing wall to form the balcony. The disadvantages with this solution are in accommodating the set-down in the floor and the usual need to erect formwork on the perimeter of the building to provide an appropriate finish on the face of the balcony.

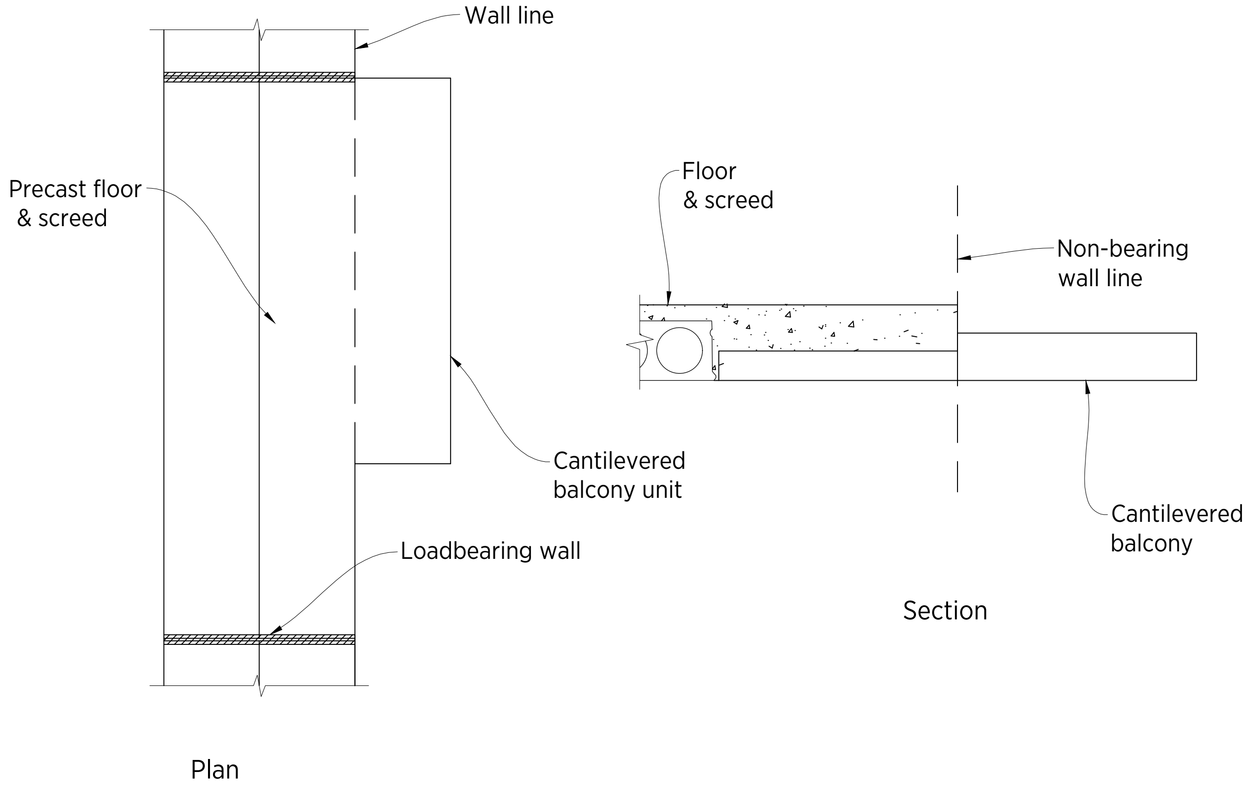


Figure 4.29 Floor element incorporating cantilevered balcony

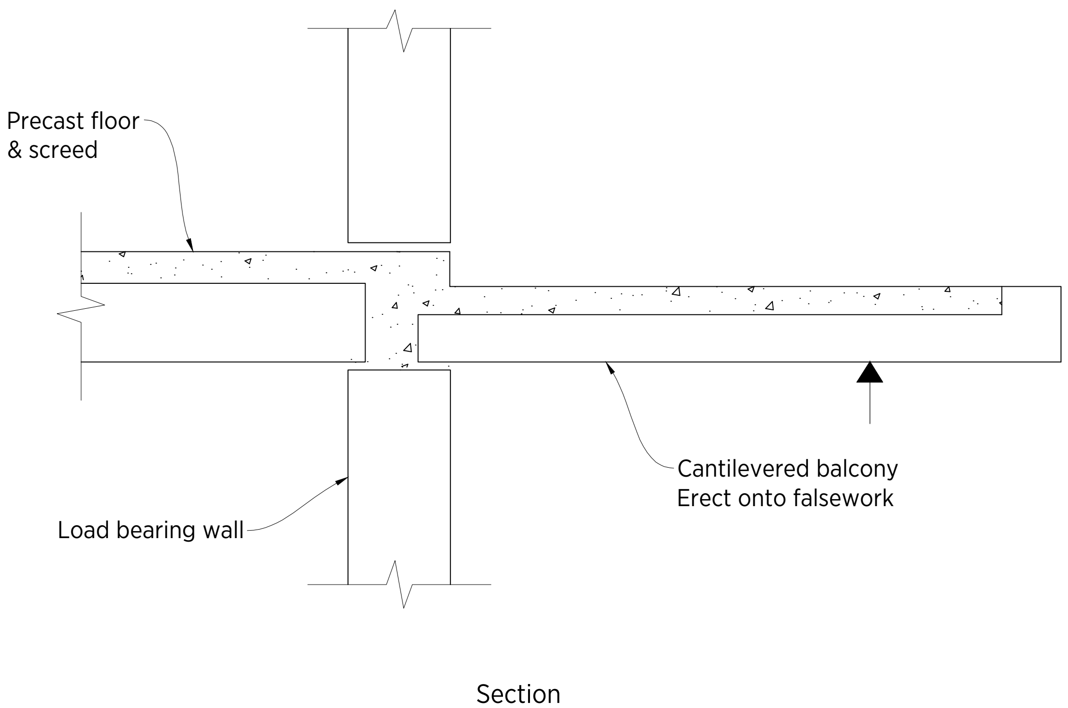


Figure 4.30 Cantilevered balcony element

4.7 Stairs

Although not strictly a structural element precast concrete stairs are often an integral part of precast or insitu concrete structures.

Precast concrete staircases are very popular because of the ability to erect a finished product that can immediately provide access for construction without the requirement for temporary access.

Traditional cast in-situ staircases are very labour intensive, additional finishing material is always needed and the effective total cost is often underestimated. Precast concrete stair units are industrialized products, with a high degree of finishing, ranging from smooth as cast, to polished concrete. Most manufacturers have a range of standard stair moulds covering the common tread and riser configurations and designers should endeavour to work within these limits.

Although there are a number of possible configurations for stairs the most common comprise straight stair elements. They are made out of either individual precast flights and separate landings or combined flight and landings. In the latter solution there may be differential levels at floors and half-landings, necessitating a finishing screed or other solution on the landing.

A more unusual type of stair is the spiral or self-supporting stair. The ability to manufacture these units off site means that they can be an economical solution compared with cast insitu.

Stair flights are usually not designed to be part of the lateral load resisting component of the building. Under lateral load, particularly seismic loads where significant lateral displacement can occur, it is therefore important to ensure that the bearing lengths of each element are adequate so they can accommodate the displacement without being subjected to axial load or collapsing into the shaft.

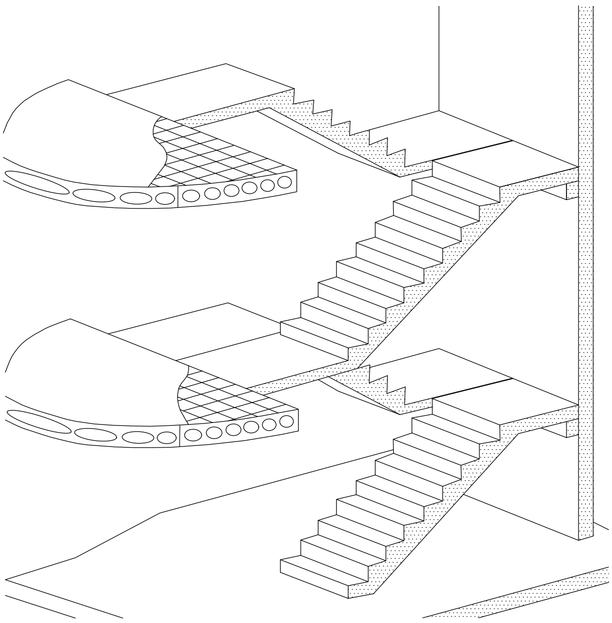


Figure 4.31 Combined landing, flight and half-landing elements

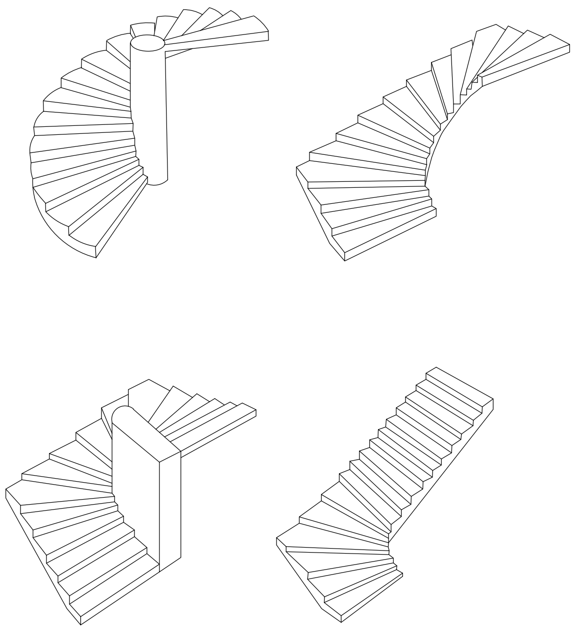


Figure 4.32 Spiral and self-supporting stair units

5 STRUCTURAL STABILITY

5.1 General

Structural stability is a crucial issue in precast concrete design. It involves a series of steps starting with the design of the individual elements, (as discussed in the previous chapter) the connections between them and finally the design of the framework in total.

Previous chapters have drawn on the importance of a correct design philosophy for precast structures and highlighted the difference from cast in-situ structures. In precast concrete a three-dimensional framework is seldom realized because of the difficulty in achieving moment fixed connections between discrete linear members.

In addition two design stages need to be considered with precast concrete. Firstly, the temporary stability during construction and secondly the permanent stability, which includes connecting the individual elements together and generating horizontal diaphragm action to transfer horizontal loading from the horizontal elements to the vertical bracing element or roof and thus into the foundations.

Provision of structural stability can be achieved by using the following concepts:

- Un-braced (or sway) structures, where stability is provided by the cantilever action of the columns or walls in the structure or by two dimensional frame action.
- Braced structures, where resistance against horizontal actions is provided by shear walls, lift shafts and central cores that provide horizontal support to the remaining parts of the structure.

In conjunction with the above concepts it is important to:

- Ensure that horizontal actions are distributed evenly across the structure by designing the floor or roof plates to act as diaphragms.
- Provide connections or ties between the individual precast elements to ensure that the assembled precast elements act as a coherent structure.
- Ensure that the structure can retain its integrity under accidental actions.

In this Chapter

5.2 Un-braced Precast Structures

5.2.1 Cantilever action

Resisting lateral loads by cantilever action is suited to low-rise structures that contain insufficient walls to act as shear walls. Low-rise commercial buildings and parking buildings are ideally suited. Single storied buildings where perimeter walls are required by regulation to cantilever from the foundations also fall into this category. [10]

The restraint of columns into the foundation is an easy method to stabilize buildings, but the maximum height of the structure is limited to about 10 m. The reasons are the limitations on the column size and the allowable deflections. Second order effects due to horizontal deflections need to be taken into account. For example, as shown in figure 5.1, the rotation between column and beam increases exponentially with column height. As the height increases the connections are required to accommodate prohibitively large rotations. Although the beam-column connections are usually designed as pinned, in reality they are semi-rigid due to the limited rotation capacity in the ultimate limit state. Design methods are not sufficiently understood to allow semi-rigid action in the post-elastic state to be considered in the design of the structure.

Geotechnical considerations also need to be addressed to ensure the ability of the footings and foundation to resist the bending moments at the column base.

The total horizontal force acting on the building can be distributed to all columns according to their stiffness, with roof or floors acting as diaphragms or through beams on the frame lines.

Combinations of the two methods, braced and un-braced, may be used in the same structure in orthogonal directions.

5.2.2 Frame action

When the restraint of the columns into the foundations does not provide the needed stiffness to the structure, for example in case of very slender columns or in response to excessive horizontal actions like in earthquake, additional horizontal stiffness may be obtained by providing rigid connections between beams and columns.

Provision of such rigid connections to produce a fully moment resistant frame in precast is difficult and should be avoided wherever possible. If it is required it is usually only possible to achieve with welded cast-in connection

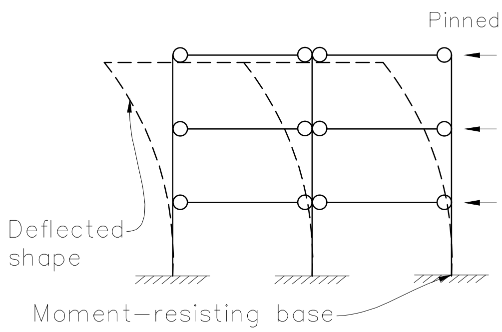


Figure 5.1 Deflections of unbraced (sway) frame using cantilever action in columns

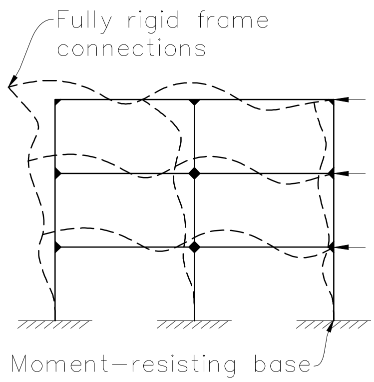


Figure 5.2 Deflections in a fully rigid frame structure

brackets or cast insitu concrete infills that emulate cast insitu construction. Provision of three dimensional frame action in precast structures is very difficult and is not recommended.

An alternative method of producing a moment resistant frame is by the use of 'T', 'TT' or '+' elements in the vertical plane as shown in figure 5.3. With strategic placement the connections between such units can be simple pinned joints. These types of elements are commonly used to form semi-rigid moment resisting perimeter frames on buildings in seismic areas.

Special precast frame systems utilising un-bonded post-tensioning tendons to provide continuity have been developed in the USA for earthquake resisting frames but they should only be considered where braced structures are not an option [19]. It should also be noted that un-bonded post-tensioning tendons are not allow under AS3600. [7]

Other systems exist that develop partially continuous moment connections between columns and beams. For example a beam column connection may be able to resist negative but not positive bending.

Provision of frame action is usually not necessary on the typical types of precast structures being built in Australia as most buildings have sufficient shear walls to develop a braced structure.

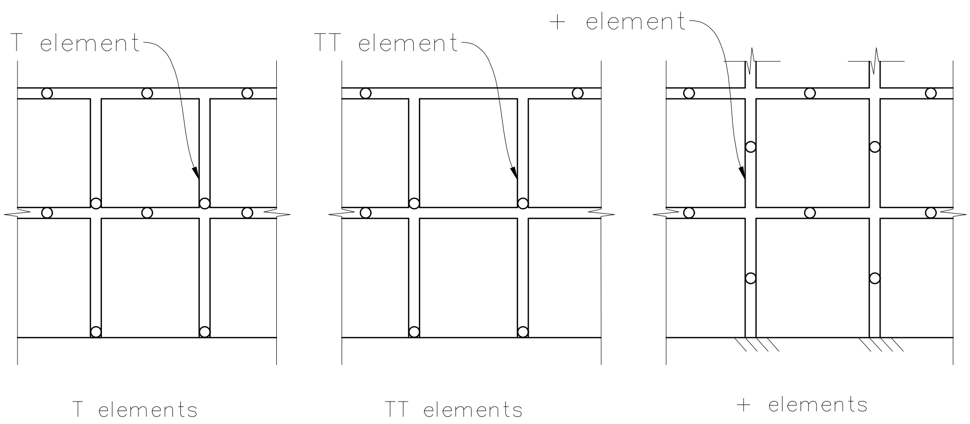


Figure 5.3 Elevation of multiple element semi-rigid frame structures

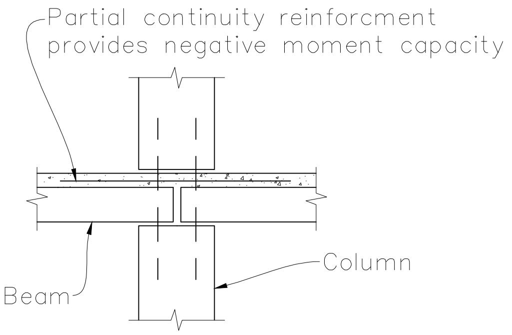


Figure 5.4 Partially continuous moment connection

5.3 Braced precast structures

5.3.1 Principle

In buildings where cantilever action of columns or walls are not utilised the horizontal actions may be resisted by shear walls formed from lift shafts, stairwells or individual walls. These are connected to the rest of the structure via the floor diaphragm to form a braced structure.

Walls forming these stabilizing elements are usually so substantial that the stiffness of the frame elements and their connections is not critical. Bending moments due to sway are small and columns can only deflect between floors as pin ended struts. The concentration of all horizontal actions to some selected members permits smaller columns and simpler connections. Furthermore the columns will in effect have horizontal support at each floor level, which reduces the slenderness of the columns.

5.3.2 Core walls and lift shafts

Braced systems are the most effective solution for multi-storey skeletal structures, because stair and elevator shafts are already present for functional reasons. The additional cost of utilizing them as stabilizing members is therefore negligible. Two dimensional walls are easy to manufacture and erect, and can be combined into three dimensional cores to provide greater stability.

The most usual precast solution for a precast concrete core is to construct it out of four or more precast wall elements connected to each other so that the vertical joints are able to resist shear forces. The subject of joint connections is dealt with separately. A precast box for a stair or lift shaft formed from individual walls will generally not have the same stiffness as an insitu concrete box. This is due to degradation of connection stiffness under load reversal. Never the less this is usually not a problem if properly considered in the design.

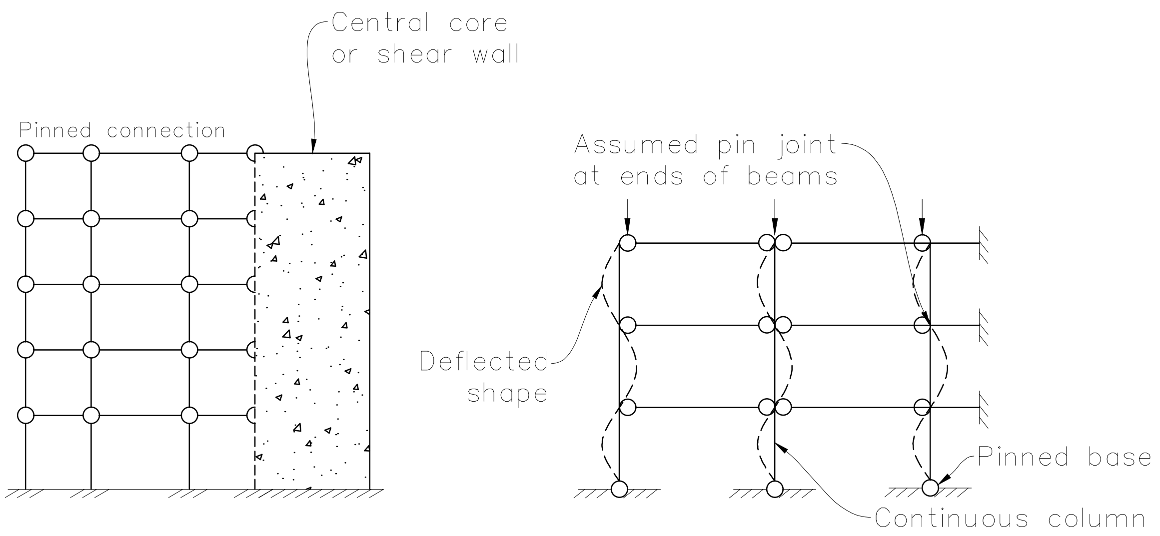


Figure 5.5 Example of a braced skeletal structure and deflection profile

Figure 5.6 Example of precast central cores

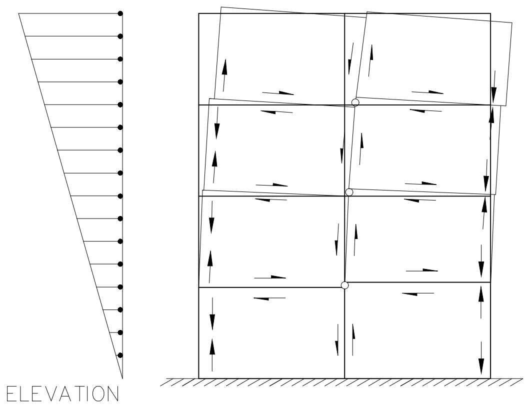


Figure 5.7 In-plane action of precast walls

5.3.3 Shear walls

Concrete walls have significant in-plane stiffness. For this reason they are commonly used as shear walls, both in precast and cast in-situ concrete buildings, to stabilize the structure against horizontal actions. The diaphragm action of the floors plays an important role in the transfer and distribution of the horizontal actions over the different stabilizing components.

Where possible precast concrete shear walls should be designed as individual elements to avoid the high cost associated with providing connections and the difficulty in ensuring that connection stiffness does not degrade with load reversal. Never the less with careful design, individual shear wall panels can be connected in such a way at the horizontal and vertical joints that the total wall can function as a single cantilevering unit. The interaction between the individual wall units is secured by connections and tying systems that transfer the necessary shear, tensile and compressive forces. If necessary, tensile reinforcement can be used to anchor the units to the foundation and to provide continuity between successive storey-height units.

Stepping or staggering the vertical joints between adjacent wall panels, as shown in Figure 5.8, is another very effective method of connecting walls to allow them to function as a single composite unit.

Individual shear walls are also often used to complement the horizontal stiffening action of cores, for example at ends of a long and narrow building with a central core, or where cores are placed in an eccentric position (Figure 5.10).

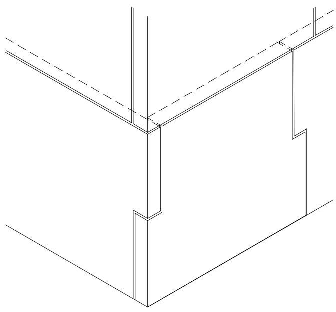


Figure 5.8 Staggered vertical joints in shear walls

Figure 5.9 Example of precast concrete shear walls

When walls have large openings, for example lift shaft doors, the wall should be checked to ensure that the link part of the wall above the door opening can contribute to the stiffness. If not, only that part of the wall each side of the door opening should be considered or the wall should be treated as a linked-wall system.

The distribution of horizontal loading between shear walls and/or cores depends on a number of factors as follows:

- Stiffness or in-plane deflection of the individual stabilizing elements.
- Position of the stabilizing elements. Stabilizing elements should be positioned according to their stiffness in order to minimise torsional effects on the structure.
- Movement joints in the floor diaphragms. Movement joints are usually provided at about 80 m intervals in floor diaphragms if the structure is rectangular on plan or at about 60 m intervals if the plan is non-rectangular. Movement joints effectively break the structure up into separate buildings.

Finally, when locating the stabilizing elements, due consideration should to be given with regard to dimensional changes. Long term shrinkage of the concrete and thermal expansion and contraction due to temperature change needs to be considered. Care should be taken that these deformations can take place without causing distress in the structure.

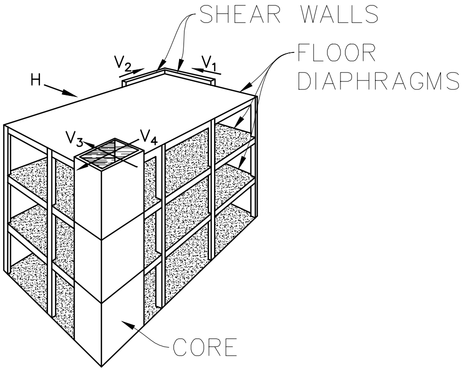


Figure 5.10 Shear walls are needed to balance the torsion induced by the eccentric position of the core.

5.3.4 Infill walls

An alternative type of shear wall may be provided with the so-called ‘infill’ wall. This is a wall that is constructed between columns and beams that when subject to horizontal forces through the floor plate, develops a diagonal compression strut resisted by the reactions in the beams and the columns. Infill walls can be precast concrete, masonry or even timber. Whatever the material, the wall needs to be constructed tight between the columns and beam.

For design, the infill wall is considered to act as a strut and the strut size and forces in each component are determined by an iterative process in a similar manner to that described for concrete in Section 7, AS3600. [7] Transfer of forces through the nodes is critical and can result in significant shear forces being introduced into the columns and beam.

This type of shear wall is not common in Australia as it is usually more economical to provide a loadbearing wall in lieu of a frame system plus infill wall. Infill walls are more likely to be used unintentionally, for example as infill spandrels or masonry infills. In seismic areas they are the causes of many failures by altering the intended response of the building. See figure 2.7.

5.4 Floor plate action

Horizontal actions from wind and earthquake are usually transmitted to shear walls or moment resisting frames through the floor or roof acting as horizontal plates. Because the floor is a relatively thin membrane this is commonly referred to as 'diaphragm action'. Diaphragms apply to both insitu and precast structures. Floors incorporating precast concrete elements, including those that do not have a topping screed, can act as horizontal diaphragms. Section 15 of AS3600 provides guidelines for the design of diaphragms including a clause on the use of cast-in-place toppings on prefabricated floor systems.

Most major texts, such as, Guidelines for the use of Structural Precast Concrete in Buildings [20], Multi-storey Precast Concrete Framed Structures [21], PCI Design Handbook [4], Precast Concrete Handbook [2] note the paucity of experimental data on the topic of diaphragm action of precast floors therefore caution designers to adopt conservative values for shear resistance.

The floor diaphragm must act in all directions, but is normally resolved into orthogonal directions to resist horizontal forces and the effects of drift. Load paths need to be practical and buildable. In many precast structures the configuration and behaviour of the diaphragm is simple with rectangular floors or roofs spanning between precast walls or frames. However some structures may include excessive horizontal spans between stabilising elements, large openings or discontinuities in the diaphragm, large torsional effects from eccentricity of the stabilising elements or lateral transfer requirements due to vertical discontinuities. Large openings at stair and lift shafts can make it difficult to transfer diaphragm forces into the stabilising elements. Likewise trenches and chases in the topping screed can disrupt diaphragm action.

5.4.1 Rigid and flexible diaphragms

For simpler structures the diaphragm can assumed to be rigid and be analysed by considering the floor as a deep horizontal beam and distributing the tension, compression and shear forces to the stabilising elements as shown in figure 5.11. Diaphragms that are assumed to be rigid should not have large openings or re-entrant corners and maximum distances between resisting elements should be about 40-50 metres for a typical in-plane depth of 6-20 metres. Where multiple stabilising elements occur the diaphragm is assumed to distribute the horizontal forces to the stabilising elements in proportion to their relative stiffness.

Where a topping screed is used the reinforcement in the floor diaphragm is calculated using classic reinforced concrete design. Where no topping screed is present, the tensile forces are resisted by peripheral tie reinforcement of the floor and the shear forces are resisted by shear friction, aggregate interlock and dowel action in the joints between adjacent elements. The most critical sections are the joints between the floor and the stabilising elements, because the shear forces are at their maximum at these locations.

The design of the diaphragm is essentially a connection design problem. The weak link in diaphragm action is always the connection between the diaphragm and the stabilising elements.

For more complex structures, particularly for those with large openings or discontinuities in the diaphragm or large torsional effects from eccentricity, the diaphragm may not act as a rigid element. In these cases, particularly in seismic areas, it is recommended that a topping screed be used and that a more detailed analysis is carried out that takes into account the deflection of the diaphragm. Strut and tie modelling can be used in these cases. In some instances the diaphragm forces may dictate the thickness of the topping screed.

Where seismic action is a major consideration special attention needs to be given to the robustness of the system and the detailing. This includes checking that support for floor elements is not lost due to elongation of supporting beams at plastic hinges and that the composite topping cannot de-bond from the precast elements. Designers should refer to the PCI Design Handbook [4] and Precast Concrete Handbook [2] for a full discussion on the subject.

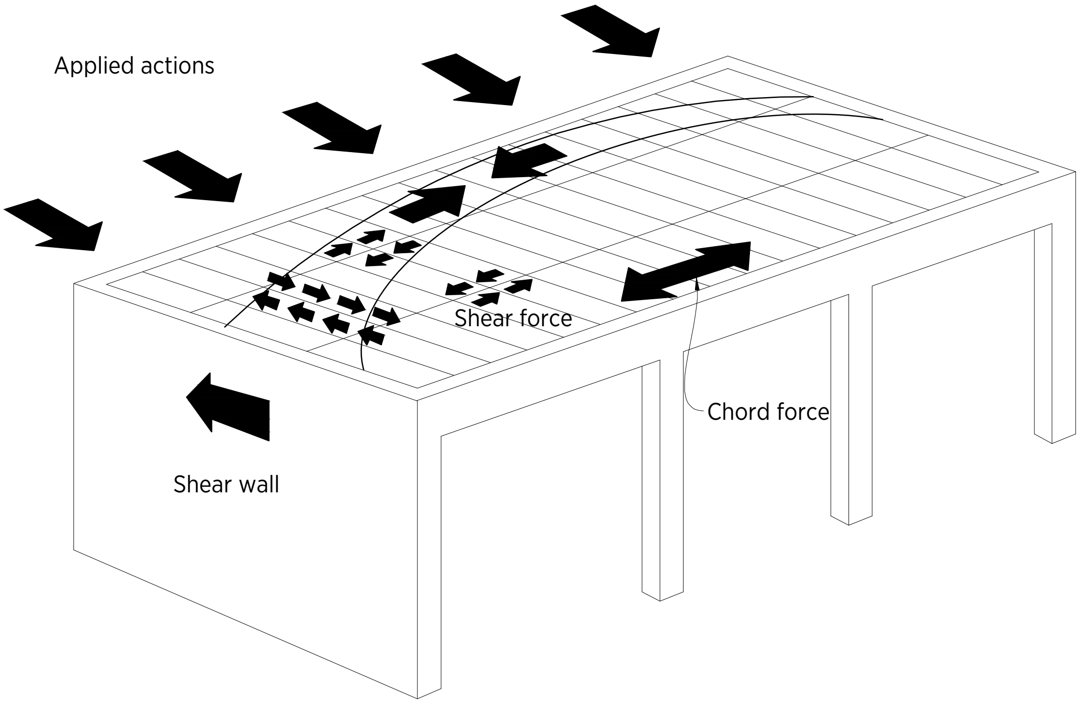


Figure 5.11 Force distribution in simple floor diaphragm

5.4.2 Shear transfer between elements

Floor diaphragms with a topping screed are generally designed on the basis that the precast concrete floor elements act compositely with the in-situ reinforced concrete topping to prevent buckling while the shear across joints is assumed to be carried entirely by the topping.

Floor diaphragms without topping screeds are not common in Australia and when used are generally on one or two storied residential buildings with relatively short floor spans. In these cases the shear transfer between elements is accomplished by shear friction, aggregate interlock or dowel action between adjacent elements. To resist these forces it is necessary that the units be tied together so that shear forces can be transferred across the joints even when they are cracked.

Because of their edge profile and the grouting of the longitudinal joints, floors made of hollow core units can be used to act as a diaphragm with or without a topping screed. The shear stress in the joints should be calculated using a section depth 30mm less than the overall precast depth to allow for the fact that the bottom of the joint does not fill with grout and to account for differential camber of the elements. AS3600 does not specifically cover interface shear capacity in joints between precast floor units but Section 8 provides some guidelines. It is recommended that a conservative value be adopted. Where used without a topping screed the hollow core elements must be restrained from moving apart.

Appropriate detailing and care in connection design is necessary to ensure that diaphragm forces can be transferred to the stabilising elements. Connections that transfer shear from the diaphragm to the shear walls or other lateral force resisting elements should be analysed in the same manner as the connections between adjacent precast elements. Care must be exercised, particularly where there are openings adjacent to shear walls, or to other elements which provide stability

The weak link in diaphragm action is always the connection between diaphragm and shear resisting elements.

5.4.3 Chord forces

Chord forces in the diaphragm are calculated from analysis as a deep beam or by a strut and tie model. Peripheral tie reinforcement is calculated and placed as appropriate round the perimeter of the diaphragm to resist these forces. As with any beam, this tie reinforcement needs to be lapped and anchored so that the forces are transferred to the shear walls or other lateral force resisting elements.

5.4.4 Movement joints

All buildings need to be designed to accommodate concrete shrinkage, long-term creep and thermal movements. One of the advantages of precast concrete buildings is that the majority of the concrete shrinkage occurs prior to the elements being installed on site.

Although the majority of concrete shrinkage and some of the long-term creep will occur before erection of a precast concrete building the thermal movement still needs to be considered. Where the plan dimension of a precast building exceeds about 60 metres the long-term movement can become critical and consideration should be given to providing permanent movement joints within the structure. Where movement joints are provided in seismic areas they should be detailed to accommodate the calculated worst case sway between the two separate parts of the building.

Movement joints can be provided in two ways:

- Columns and beams can be doubled at strategic locations to effectively break the structure into separate smaller sections. This is the preferred option for seismic areas.
- Sliding or slip joints can be provided at strategic locations to allow each side of the joint to move independently of the other.

Movement joints should also be provided at discontinuities in plan or elevation of a structure. A detailed discussion on this topic is given in the PCI Design Handbook. [4]

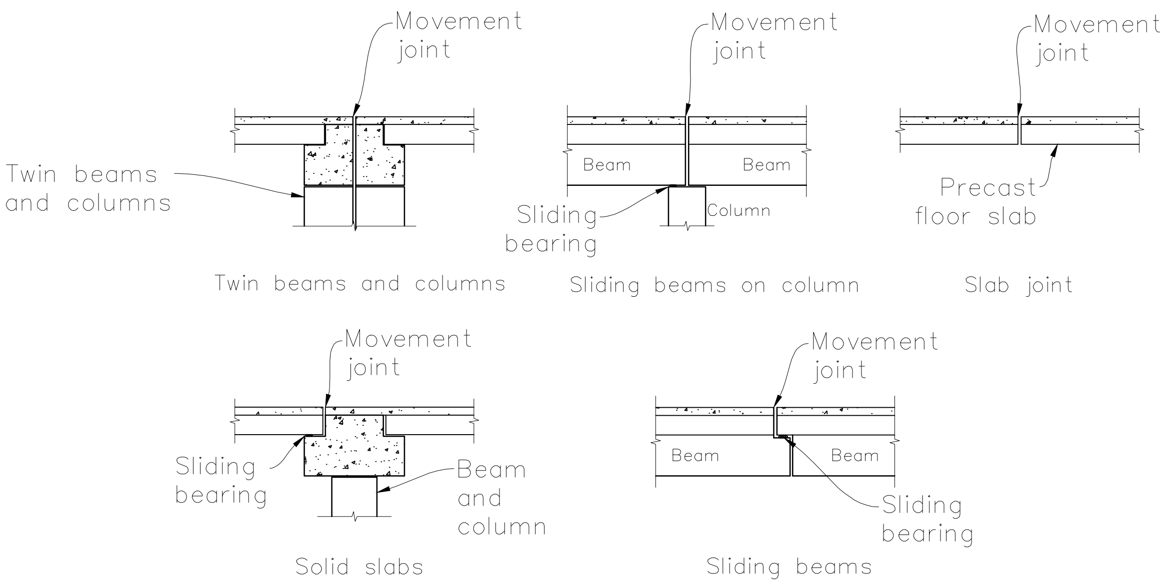


Figure 5.13 Typical movement joints

5.5 Connections

5.5.1 General

Connections are among the most essential aspects of prefabrication. Their role is to realize out of individual elements, a coherent and robust structure, able to take up all acting forces, including indirect forces resulting from shrinkage, creep, thermal movements, fire, etc. They also allow precast elements to be manufactured in manageable sizes and joined to form larger units. Precast concrete connections must meet a variety of design, performance and other criteria. Their principal function is to transfer forces across joints, so that interaction between precast units is obtained. This interaction can have several purposes:

- Connect units to the bearing structure;
- Secure the intended overall behaviour of each precast subsystem, including diaphragm action of floors, shear wall action of walls, etc.;
- Transfer forces from their point of application to the stabilizing structure

Other aspects concerning the function and appearance of connections may result in specific design and execution requirements, for instance durability and visual appearance. The design of connections is not only a question of choosing appropriate fixing methods but detailing them in a way that provides the appropriate degree of ductility and transfers the forces into the adjacent element.

In terms of this document the following definitions are used.

- Joint. The interface, or gap, between two discrete precast elements, or between a concrete element and some other portion of the structure.
- Connection. Method by which one or more concrete or other elements are joined together to transfer loads and/or provide stability.
- Fixings or fitments. The hardware components of all connections including brackets bolts, washers, weld plates and anchors.

Examples of standardized types of structural connections are usually listed in design handbooks or catalogues from precast manufacturers. The Precast Concrete Handbook [2] and the fib Bulletin 43, Structural Connections for Precast Concrete [27] have extensive coverage on this topic. However, the design of structural connections is not just a question of selecting an appropriate solution from listed standard solutions. The design of structural connections in precast buildings must consider a variety of criteria related to the structural behaviour, dimensional tolerances, fire resistance, manufacture, handling and erection. There are many types of connections but simple proven connections are always the best ones to use. They should be standardised and made as fool-proof as possible.

The basic principles and design concepts are given in this section to enable the designer to understand the design philosophy of connections in precast structures in general. Practical examples of connections for specific types of buildings are given in Chapters 6 to 9.

5.5.2 Strength

A connection should be designed to resist the forces to which it will be subjected. Some of these forces are apparent, caused by dead, live, wind and earthquake actions, and soil or water pressure. Others are caused by restraint of volume changes in the members, or additional forces that might appear due to unintended inclination of load bearing columns and walls and unintended eccentricities. The design should not only consider the actual connection and fixings but also the surrounding areas in the connected concrete elements where stresses can be high. Connections should be designed to be ductile so they yield before failure and have a degree of over capacity to ensure that they are not the weak line in a structure. As a guide, 15% to 30% should be considered, depending on how critical the connection is.

As discussed in Chapter 5.7 the design of connections should also consider the possibility of accidental actions that can cause severe damage to the building structure resulting in force redistribution and the requirement for alternative load paths that can bridge over the damaged area.

5.5.3 Volume change

The combined shortening effects of creep, shrinkage and temperature changes can cause tensile stresses in precast concrete components and in particular their connections. There are principally two ways to take care of volume change, either by allowing the displacements to occur at the connections, or by giving the connections the necessary restraint to prevent displacement. In the latter case the connection must be designed for quite considerable restraint forces.

In practice many solutions allow some relative displacement to occur. For example, elastic deformations of structural members or connection fixings will relieve restraint forces.

In some connections it is not only the force transfer capacity of the connections that is important but also the stiffness as measured by the load-displacement relationship and the deformability. This is particularly important in connections that are required to form a coherent structure from a series of individual elements. In this case, deformability can lead to a loss of stiffness, particularly under seismic loading.

5.5.4 Deformation

Movement between adjacent precast elements due to service load deflections, concrete creep and shrinkage and temperature variations needs to be considered or there will be a risk of damage to the connection zones. This can be satisfied by detailing the connection so that the corresponding movement can take place without restraint or by fully preventing movements between adjacent precast elements. In the latter case the connection and the elements must be designed to resist the corresponding restraint forces that will develop.

5.5.5 Ductility

Connections should be designed for ductile behaviour to avoid brittle failure. Ductility is the ability to undergo plastic deformation without a substantial loss of load capacity. In case of overloading, a ductile connection will reach yield and start to deform plastically. The plastic displacement will give the necessary relief to allow a new state of equilibrium in the connection. Large displacements will be the result, but the force transfer ability remains and brittle failure and damage of connection zones is prevented.

Ductility can be provided by confinement reinforcement in concrete connections or by anchor bars in steel connections. Methods of detailing the confinement reinforcement required to develop plastic hinges for ductility in concrete connections is discussed in detail in texts on seismic design. [37]

In connections using steel fixings the anchor bars are the ductile components. To ensure ductile behaviour the other components, mainly the welded fixings, should be designed to have higher ultimate capacity than the anchorage bars. The bar should project into the uncracked zone where it is properly anchored. Transverse confinement reinforcement should also be provided particularly close to edges and in thin sections.

5.5.6 Durability

Connections should be designed to provide the same level of durability as the individual precast elements as required by the BCA. [10] Exposed steel should be provided with permanent protection. This can be achieved by applying a layer of epoxy, rust proof paint or bitumen, or by encasing in concrete or mortar after the connection is secured. In more aggressive environments fixings should be hot dipped galvanised. In highly aggressive environments stainless steel should be considered even though the cost penalty is considerable. In the case of dissimilar metals the risk of galvanic corrosion should be taken into account.

5.5.7 Dimensional tolerances

Dimensional tolerances inevitably occur in the construction of a building and in the manufacture of the precast elements. These deviations will normally be concentrated at the connections and must be considered in the design of the connections, otherwise serious problems may occur during the erection of the structure. An important principle is to ensure that all fixings of whatever type allow for three-way adjustment to enable the units to be aligned and levelled.

5.5.8 Fire resistance

Connection details, which are vital parts of the structural system, should be protected to provide the same FRL as other structural members. Protection can be obtained by cast insitu concrete, mortar or fire insulating materials.

Many precast connections are not vulnerable to the effect of fire and require no special treatment. For example, the bearings between slabs and beams or between beams and columns do not generally require special fire protection. Steel connections partly embedded in concrete will have a lower temperature rise than non-embedded steel because of the thermal conductivity of the surrounding concrete and in some cases may not require additional fire protection.

In the case of walls and floors that have an important separating function with regard to thermal insulation and fire penetration the connections at wall joints and floors should be designed to prevent the passage of flames and hot gases.

More information on the design of fire resistant structures is given in Chapter 10.

5.5.9 Basic force transfer mechanisms

Structural connections are usually composed of a number of components and fixings that assure the transfer of forces through the whole connection. The transfer of forces from one component to another one, or within a connection as a whole, is based on a number of principles as outlined below.

5.5.9.1 Concrete infills

A connection can be provided by aligning one component with another and filling the remaining space with grout or fine concrete, or even with an adhesive. A classic example is the connection of precast elements with insitu concrete infills. Another example is connecting a precast column into a pocket foundation.

5.5.9.2 Anchorage of tension bars

Reinforcing bars in tension can be anchored by bond, hooks, bends or studs, or simply by the tensile capacity of the pull-out cone.

By providing sufficient development length and concrete cover to the bar, the anchorage capacity will exceed the tensile capacity of the bar. When this is not possible, due to limitations in geometry, the tensile capacity of the connection is determined by cone or splitting failure of the concrete or by pull-out failure of the bar.

In dowel connections subject to tensile loads the bond between the concrete of the element and the grout is unreliable if there is any shrinkage. Experience has shown that these dowel holes should be formed by corrugated ducts.

Bars can also be anchored by bar couplers. These are proprietary types of connectors that allow a loose bar to be connected to the cast in connector. They can be screwed couplers, grouted sleeves or swaged sleeves. All provide the full design capacity of the bar.

5.5.9.3 Dowel action of bars

Transfer of horizontal actions from one element to another can be provided in precast structures by means of dowel action. Depending on the strength and dimension of the steel bar and the position of the bar relative to the element boundaries, several failure modes are possible. A weak bar in a strong concrete element might fail in shear of the bar itself. A strong steel bar in a weak concrete element or placed with small concrete cover might result in concrete bearing failure or splitting of the concrete. However, when the bar is placed in well confined concrete with adequate concrete cover or when the splitting effects are controlled by properly designed confinement reinforcement, the dowel pin will normally fail in bending by formation of a plastic hinge in the steel bar at some distance above the joint face.

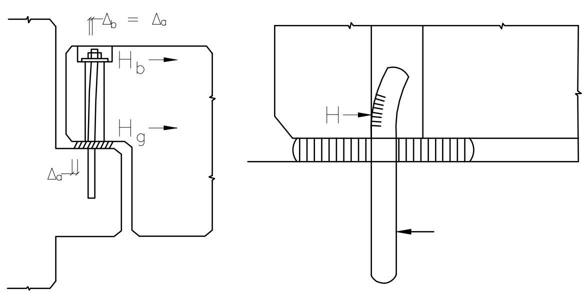


Figure 5.14 Principle of force transfer by dowel action

5.5.9.4 Bond

Connection by adhesion and bond between precast concrete and cast in-situ concrete is only appropriate for small interface stresses, for example in composite action between precast floors and topping screeds.

The factors which affect bond and shear transfer at the interface surface include: surface roughness, surface strength, cleanliness and moisture content. Test data indicates that the treatment of the precast surface is at least as important as the degree of roughness. Factors such as cleanliness, compaction, curing and wetting of the surface have a major influence on the shear strength of the interface. Of particular importance is the moisture content of the surface which should be wet with no surface water. This is known as 'saturated surface dry'.

Australian Standard, AS3600 [7] provides shear plane surface coefficients for various surface conditions.

5.5.9.5 Friction

Shear forces can be transferred between precast concrete and insitu concrete by friction, provided there is some interface roughness and a compressive force across the joint to create the frictional resistance. This concept is known as 'shear-friction'.

A permanent compressive force can be provided by gravity load or by pre-stressing. However, it is also possible to induce compressive forces by placing reinforcement bars across the joint which are strained when the connection is loaded in shear. Because of the roughness in the joint interface, a small joint separation will take place when the joint is loaded in shear and slip occurs along the interface. The joint separation creates tension in the reinforcing bars and the tensile force is balanced by a compressive force across the interface. Increasing the amount of transverse reinforcement increases the frictional coefficient and increases the shear resistance.

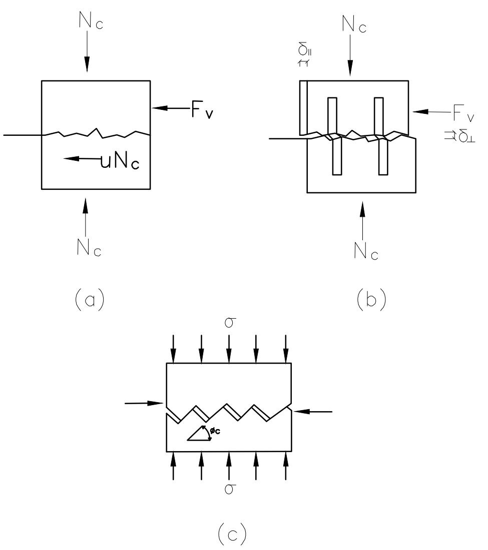


Figure 5.15 Transfer of shear forces by friction

In a similar manner shear forces can be transmitted across insitu infill joints between wall panels that have indented or roughened joint faces. Provided the elements are prevented from moving apart under shear loading the connection will be able to resist shear forces by shear-friction. Reinforcement ties can be provided at the top and bottom of the elements or as lapping reinforcement loops within the insitu infill.

5.5.9.6 Bolting

Bolting is used extensively in connections to transfer tensile and shear forces. Fixings such as, threaded ferrules, rails or captive nuts attached to the rear of plates are cast into the precast units to allow a bolt to form the connection. Tolerances can be accommodated by using over-sized holes in the connecting member and where necessary by welding heavy washers to the plates to limit movement.

Capacity of the connection will almost always depend on the type of cast-in fixing. Where a fixing such as a ferrule is provided with an anchor bar that can develop its full capacity the tensile capacity of the connection will be limited by the bolt capacity and the shear capacity by the lesser of the bolt or concrete bearing on the ferrule.

Where a ferrule, commonly called a foot anchor, has no anchor bar and only an enlarged base the tensile capacity of the connection will be determined by the pull out capacity of the ferrule and the shear capacity by the lesser of the bolt or concrete bearing on the ferrule.

In both cases the combined tensile and shear loads need to be checked against the combine tensile and shear capacities.

This topic is covered extensively in the Precast Concrete Handbook. [2]

5.5.9.7 Welding

Connections can be formed by welding directly to protruding steel fixings, for example reinforcement bars that overlap into an insitu infill. The disadvantage of this is lack of tolerance, particularly if the lapping bars have a short projection length. An alternative is to use an intermediate steel section, which is used as a link and welded between the protruding fixings.

Bars can also be welded to anchor plates or angles embedded in the precast element. In all cases welding should be carried out by qualified personnel. Site welded connections should be avoided where possible. Welding on site and usually at height is expensive and time consuming.

5.5.9.8 Post tensioning

Post-tensioning can be used in segmental construction and in shear walls of tall buildings. Ducts are installed into the units, and after erection, the pre-stressing cables or bars are placed in the ducts and post-tensioned. The joints between the units are able to resist tension and shear forces.

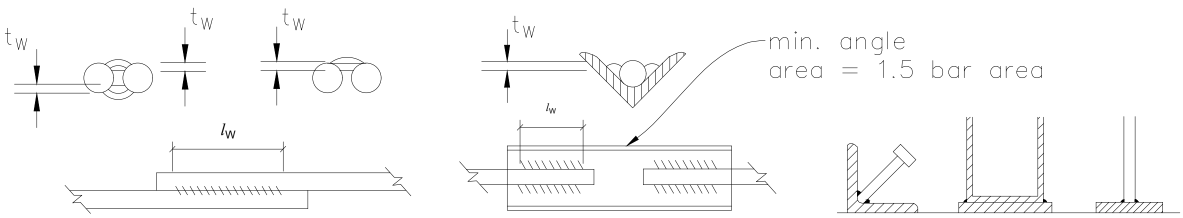


Figure 5.16 Types of welded connections

5.6 Tie systems

The most essential design purpose of precast structures is to realize a coherent entity out of individual precast elements. The principal method to obtain structural integrity and robustness in precast structures is through tying systems in the transverse, longitudinal and vertical direction. These tying systems effectively interconnect all the individual elements to ensure stability to the structure and to provide redundant load paths.

This issue is not specifically covered by AS3600, but BS 8110 [34] and Eurocode 2 [24] provide simple rules that are deemed to satisfy these robustness and integrity requirements. Elliott [21] also provides worked examples of ties required in a typical building. Ties are in effect the minimum reinforcement requirements between and within the elements in a precast structure in the same manner that AS3600 has minimum reinforcement requirements for insitu concrete elements.

Where it is deemed that the structure is to be designed for accidental actions additional ties should be provided in accordance with Chapter 5.7.

Ties are reinforcement bars or tendons, placed in longitudinal, transversal and vertical directions to provide continuous tensile capacity throughout the structure. Their role is not only to transfer normal forces between units, originating from wind and other actions, but also to give additional strength and robustness to the structure.

Although tie systems are necessary on all precast concrete buildings there are different requirements for loadbearing wall structures and skeletal frame structures. Loadbearing wall structures tend to have high numbers of lateral resisting elements and usually only require a minimum of tie reinforcement. Skeletal frame structures on the other hand with relatively few lateral resisting elements can require careful placement of reinforcement over and above that required for direct structural actions.

Chord and shear reinforcement required as part of a floor diaphragm or continuity reinforcement across beams can be included as part of the tie reinforcement and in some cases these requirements may exceed the deemed to comply requirements.

Tie reinforcement requirements can be met either by using individual continuous bars or by lapping bars or mechanically anchoring with the reinforcement within the precast elements.

5.6.1 Types of ties

To satisfy the deemed to comply requirements of BS 8110 [34] ties should be provided as shown in Figure 5.17. These requirements are based on fib Bulletin 74, Planning and design handbook on precast building structures. [18]

5.6.1.1 Internal ties (types 1 & 2)

These ties are also known as longitudinal and transverse ties. They are placed across the ends as well as perpendicular to the span of the floor elements. Internal ties may, in whole or in part, be spread evenly in the floor or may be grouped at or in the joints, tie-beams, along floor beams, walls or other appropriate positions.

Where a structural topping screed is provided the screed reinforcement can act as the internal tie.

The internal ties should be capable of resisting an ultimate tensile force equal to;

F_{tie} = 20kN/m x s

Where **s** = spacing of the ties.

If type 2 ties cannot be placed within the floor zone the ties may be concentrated and added to perimeter beams as ties type 5 and along the lines of the internal beams as ties type 5a.

5.6.1.2 Edge ties and Wall ties (types 3 & 4)

These ties are used in skeletal structures and loadbearing wall structures to tie the edge beams or perimeter walls into the floor system. They should be anchored into the structural topping or the precast floor elements and not into the joints between the elements.

The edge ties and wall ties should be capable of resisting the ultimate tensile force equal to;

F_{tie} = 20kN/m along length of edge or wall.

In skeletal structures these ties can be the torsion ties that are provided to resist torsion on the edge beam.

5.6.1.3 Peripheral ties (type 5)

These ties are required around the total precast floor, within a distance of 1.2 m from the edge. Peripheral ties are made continuous around external corners or by lapping the tie reinforcement with the longitudinal reinforcement in the precast component. In the latter case the precast component can act as the peripheral tie. At inner corners of the perimeter of structures, the tie reinforcement should be anchored straight inward on both sides.

Peripheral ties can also act as the tensile chord of the floor diaphragm. Peripheral ties should be capable of resisting an ultimate tensile force equal to;

F_{tie} = ℓ x 10kN/m but not less than 70kN.

Where ℓ = length of the longest floor span.

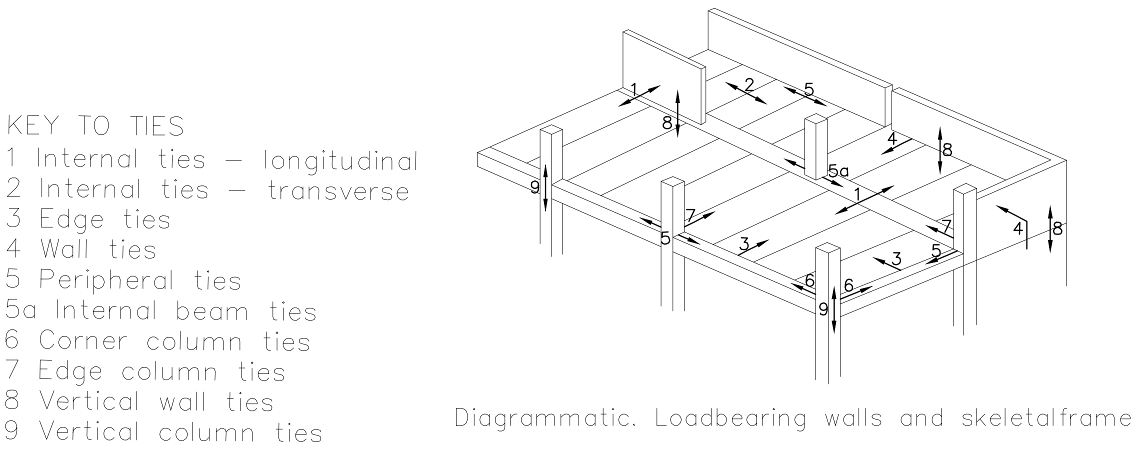


Figure 5.17 Tie locations and tie types

5.6.1.4 Internal ties (type 5a)

Internal ties are required along the internal beam lines and are provided either by passing the ties through a column to column connection, by passing ties through a sleeve in the column or placing ties symmetrically either side of the column. Internal ties also act as the internal tensile chord of the floor diaphragm.

Internal ties along beam lines should be capable of resisting an ultimate tensile force equal to;

F_{tie} = (ℓ1 + ℓ2) /2 x 20kN but not less than 70kN.

Where **ℓ1** & **ℓ2** are the span lengths in metres either side of the beam.

5.6.1.5 Corner column ties (type 6)

These ties provide horizontal restraint to the corner column by tying the floor system into the supporting structure. The corner column ties should be capable of resisting an ultimate tensile force equal to;

F_{tie} = 150kN in two perpendicular directions or equivalent placed diagonally.

Reinforcement provided for peripheral ties that is within 1.2 metres of the column can be used as part of this requirement

5.6.1.6 Edge column ties (type 7)

These ties provide horizontal restraint to the edge columns by tying the floor system into the supporting structure. The edge column ties should be capable of resisting an ultimate tensile force equal to;

F_{tie} = 150kN perpendicular to the edge.

Where it is not possible to provide a direct tie between the column and floor slab the tie force may be distributed for a distance of 1.2 metres each side of the column and anchored into the edge beam. Reinforcement provided for edge ties that is within 1.2 metres of the column can be used as part of this requirement

5.6.1.7 Vertical wall ties (type 8)

Vertical wall ties should be provided between superimposed walls to ensure there is a minimum tensile capacity vertically through a building. Normally, continuous vertical ties should be provided from the lowest to the highest level.

The vertical ties should be capable of resisting an ultimate tensile force per metre length of wall equal to:

F_{tie} = the maximum design ultimate vertical permanent and imposed load per metre length applied to the wall from any one storey.

Ties can be spaced equally along the length of the wall or grouped at not greater than 2.5 metre centres along the wall.

5.6.1.8 Vertical column ties (type 9)

Vertical column ties should be provided between superimposed columns to ensure there is a minimum tensile capacity vertically through a building. Normally, continuous vertical ties should be provided from the lowest to the highest level.

The vertical ties should be capable of resisting an ultimate tensile force equal to:

F_{tie} = the maximum design ultimate vertical permanent and imposed load applied to the column from any one storey.

5.6.2 Ties in structural toppings

Due to local construction techniques, most buildings in Australia are provided with a structural topping screed, although this is not always structurally necessary. However toppings should always be provided where there are heavy concentrated loads, where there are moving loads such as those from forklift trucks, where a fire rating is required, where seismic design governs and where design against accidental actions is specified.

Where structural toppings are provided the ties may be placed wholly within the concrete topping as part of the topping reinforcement. Structural toppings should always be continuously reinforced with a steel mesh or bars of sufficient cross section to resist shrinkage cracking and thermal movement or the requirement for diaphragm action. The area of mesh or bars should also be sufficient to resist the maximum tie forces but not less than the requirements of AS3600 [7] clause 9.5.3.

For edge beams it is important that the mesh is lapped with the tie bars projecting from or otherwise anchored to the edge beams. For internal beams the mesh can lap across the top of the beam.

Where composite action is required and ties are within the topping screed, bonding between the topping and the precast element is critical. Care needs to be taken to ensure that the precast surface is in accordance with design requirements before the topping is placed.

Figure 5.18 Example of internal and vertical ties

5.7 Design for accidental actions

5.7.1 Introduction

All building structures are designed to respond to normal load conditions without damage, but local and/or global damage cannot be avoided under the effect of an unexpected, but moderate degree of accidental overload. No structure can be expected to be totally resistant to actions arising from an unexpected extreme cause, but it should not be damaged to an extent that is disproportionate to the original cause. [40] Uncontrolled collapse or progressive collapse should not be allowed to occur. [41]

Types of hazards that can result in accidental loads include, design or construction error, overload due to misuse, gas explosion, bomb explosion, vehicle impact, storage of hazardous material, aircraft impact or fire. These events can occur randomly in space and time, and other than fire, data on their incidence, magnitude and structural effects are rarely available.
The design of a structure to mitigate the risk of progressive collapse after severe initial local damage requires a different approach to that of traditional building design. The reason lies in the large variation and magnitude of accidental actions and possible reaction of the building structure. Therefore the guidelines focus primarily on design philosophy rather than well-defined and exact design procedures.

Extensive research has been carried out overseas on this topic and one of the best sources of information is the National Institute of Standards and Technology, document NISTIR 7396, Best Practices for Reducing the Potential for Progressive Collapse in Buildings. [42]

Australia Building Codes currently have no requirements to design against accidental actions although some Transport Authorities have design guidelines for vehicle impact on structures.

American and European building standards prescribe a minimum level of protection of building structures against accidental actions as a function of possible consequences, primarily depending on the size (more specifically the height) and the occupancy of a building. Usually, buildings are classified in so-called consequences classes. Eurocode EN 1991-1-7 [22] has specific requirements for all building types and the fib Design of Precast Concrete Structures against Accidental actions [25] provides guidelines for precast concrete structures. Effectively the fib document recommends that all buildings other than single occupancy houses of less than 4 floors, agricultural buildings and buildings into which people rarely go should be designed for accidental actions. .

Designing a building for seismic action will, in most cases, unintentionally increase its resistance against accidental actions but seismic events are global and affect the whole building whereas accidental loads are usually severe and localised. In each case the building response is quite different.

Nevertheless, although there is some overlap between the disciplines in the area of prevention of progressive collapse, seismic resistant buildings are unlikely to resist totally the direct effects of blast loading acting on the exterior skin of a building and much less to resist completely an explosion or vehicle impact etc.

Figure 5.19 shows a comparison between the actions of an earthquake and an explosion on a structure.

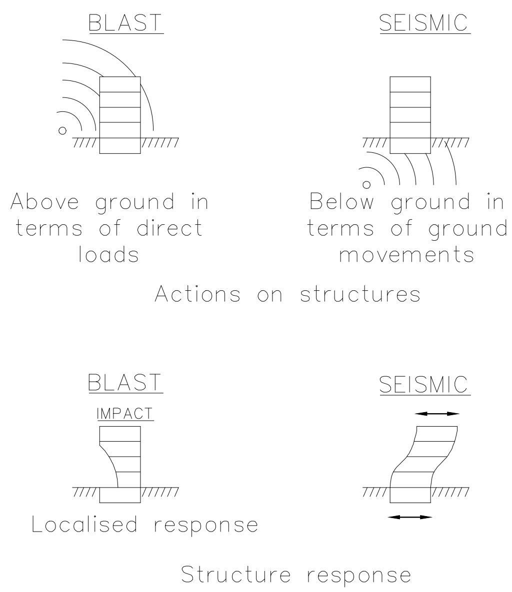


Figure 5.19 Seismic versus blast loading

The effects of accidental actions on multi-storey buildings are much more significant than on single-storey buildings but design requirements are equally applicable to both. The following sections will only discuss multi-storey buildings but the design philosophy for both is the same.

Where it is deemed appropriate to design for accidental actions there are three design alternatives that can be used to reduce the risk of progressive collapse. The choice of which alternative to use depends on the function of the building, the height of the building and the possible consequences of an accidental action and the level of protection required. The alternatives are;

- indirect design method
- alternative load path method
- specific load method.

It is allowable to apply different methods to different parts of the same structure but not permitted to mix the methods on the same part of the structure. In all cases it is recommended that a structural topping screed be provided

5.7.2 Indirect design method

With Indirect Design, also called 'Tie Force Method', resistance to progressive collapse is considered implicitly through provision of minimum levels of strength, continuity and ductility through the whole structure. The fully tied solution is based on the assumption that through a system of structural ties, a precast structure will have increased ability to prevent spread of local damage, facilitating alternative load paths and increase robustness, after a moderate degree of accidental action.

The approach is similar to that for the provision of minimum ties as described in Chapter 5.6 but for accidental loading the tie forces are higher than those given for normal loading. The types and location of the ties shown in Fig. 5.17.

This method can be used on buildings where the risks of and consequences of an accidental action are classed as low to medium and is limited to buildings of less than 15 storeys and to buildings that do not have large numbers of occupants.

Recommendations on building types and risk and consequence classifications are given in Eurocode EN 1991-1-7 [22] and the fib Design of Precast Concrete Structures against Accidental actions. [25]

Chord and shear reinforcement required as part of a floor diaphragm can be included as part of the tie reinforcement and in some cases may exceed the deemed to satisfy requirements.

The Eurocode, EN 1991-1-7 requirements for ties reflect the different responses between skeletal structures and loadbearing wall structures that are subjected to an accidental load and are treated differently.

5.7.2.1 Skeletal structures. Internal and Edge ties. (type 1, 2 & 3)

These ties are also known as longitudinal ties and transverse ties. They are placed across the ends as well as perpendicular to the span of the floor elements. Internal ties should be spread evenly in the floor but may be grouped at or in the joints, tie-beams, along floor beams, walls or other appropriate positions.

The internal ties edge ties should be capable of resisting an ultimate tensile force in the direction of the tie which is the greater of;

$\mathbf{F_{tie} = 0.8\ (G + \psi Q) \times \ell\ kN/m}$

OR

$\mathbf{F_{tie} = 75 kN/m}$

G and **Q** are respectively the permanent and imposed loads.

ψ is the load reduction factor for the accidental design situation. See AS1170. [23]

ℓ is the floor span in the longest direction.

5.7.2.2 Skeletal structures. Peripheral and Beam ties. (type 5 & 5a)

Ties are required around the total precast floor, within a distance of 1.2 m from the edge. Peripheral ties are made continuous around external corners or by lapping the tie reinforcement with the longitudinal reinforcement in the precast component. In the latter case the precast component can act as the peripheral tie. At inner corners of the perimeter of structures, the tie reinforcement should be anchored straight inward on both sides. Peripheral ties can also act as the tensile chord of the floor diaphragm.

Ties are also required along the internal beam lines and are made continuous at columns either by passing the ties through a column to column connection, by passing ties through a sleeve in the column or placing ties symmetrically either side of the column. Internal ties also act as the internal tensile chord of the floor diaphragm.

These ties are in addition to the internal ties and should be capable of resisting an ultimate tensile force along the line of the beam which is the greater of;

F_{tie} **= 0.4(G + ψQ) x ℓ kN/m**

OR

F_{tie} **= 75kN/m**

G and **Q** are respectively the permanent and imposed loads.

ψ is the load reduction factor for the accidental design situation. See AS1170. [23]

ℓ is the floor span in the longest direction.

5.7.2.3 Skeletal structures. Column ties (types 6 & 7)

These ties provide horizontal restraint to corner and edge columns by tying the floor system into the supporting structure. Reinforcement provided for edge ties within 1.2 metres each side of the column can be used as part of this requirement. Corner columns should be tied into the structure with the nominated tie force in each direction. Column ties should be capable of resisting an ultimate tensile force which is the greater of;

F_{tie} **= 0.03(G + ψQ) kN**

OR

F_{tie} **= 150kN**

G and **Q** are respectively the permanent and imposed loads.

ψ is the load reduction factor for the accidental design situation. See AS1170. [23]

0.03 represents 3%

5.7.2.4 Skeletal structures. Vertical ties (type 9)

Vertical column ties should be provided between superimposed columns to ensure there is a minimum tensile capacity vertically through a building. Normally, continuous vertical ties should be provided from the lowest to the highest level.

The vertical ties should be capable of resisting an ultimate tensile force equal to the largest design ultimate vertical permanent and imposed load applied to the column from any one floor.

F_{tie} **= (G + ψQ)kN**

G and **Q** are respectively the permanent and imposed loads.

ψ is the load reduction factor for the accidental design situation. See AS1170. [23]

5.7.2.5 Loadbearing wall structures. Internal ties (type 1, 2 & 4)

Internal ties can be divided into longitudinal ties, transverse ties and wall ties. They are placed across the ends as well as perpendicular to the span of the floor elements. Internal ties should be spread evenly in the floor but may be grouped at or in the joints, tie-beams, along floor beams, walls or other appropriate positions.

The internal ties should be capable of resisting an ultimate tensile force in the direction of the tie which is the greater of;

F_{tie} **= Ft kN/m**

OR

F_{tie} **= Ft(G + ψQ) x Z /(7.5 x 5)kN/m**

Ft is the lesser of 60 or (20 + 4 x N) where **N** is the number of storeys.

G and **Q** are respectively the permanent and imposed loads.

ψ is the load reduction factor for the accidental design situation. See AS1170. [23]

Z is the lesser of 5 times the storey height in metres or the longest floor span in metres.

5.7.2.6 Loadbearing wall structures. Peripheral ties. (type 5)

Ties are required around the total precast floor, within a distance of 1.2 m from the edge. Peripheral ties are made continuous around external corners or by lapping the tie reinforcement with the longitudinal reinforcement in the precast component. In the latter case the precast component can act as the peripheral tie. At inner corners of the perimeter of structures, the tie reinforcement should be anchored straight inward on both sides. Peripheral ties can also act as the tensile chord of the floor diaphragm.

These ties are in addition to the internal ties and should be capable of resisting an ultimate tensile force equal to ;

F_{tie} **= Ft kN**

Ft is the lesser of 60 or (20 + 4 x N) where **N** is the number of storeys.

5.7.2.7 Loadbearing wall structures. Vertical wall ties (type 8)

Vertical wall ties should be provided between superimposed walls to ensure there is a minimum tensile capacity vertically through a building. Normally, continuous vertical ties should be provided from the lowest to the highest level.

The vertical ties should be capable of resisting an ultimate tensile force per metre length of wall equal to:
$\mathbf{F_{tie} = (G + \psi Q)kN/m}$

$\mathbf{(G + \psi Q)}$ is the design ultimate vertical permanent and imposed load per metre length applied to the wall from any one storey.
ψ is the load reduction factor for the accidental design situation. See AS1170. [23]

5.7.3 Alternative load path method

The alternative load path approach method presumes that a critical element is removed from the structure, due to an accidental action, and that the structure is required to redistribute the gravity loads to the remaining undamaged structural elements.

The method involves notional removal of a critical element – e.g. a column, load bearing wall, etc. from the structure. It is an analytical exercise that ignores all other damage to the structure that may accompany the removal of a critical unit. For each plan location of a removed element, an alternative load path analysis is performed for every floor, one at the time and the remaining structure is checked to ensure it is capable of redistributing the applied static and dynamic loads and that the extent of any local collapse does not exceed allowable limits. The success of this approach depends on the assumptions of acceptable local damage.

This method should be used on all buildings of more than 15 storeys and on buildings where the risks of and consequences of an accidental action are classed as medium to high. Recommendations and design guidelines are given in Eurocode EN 1991-1-7 [22] and the fib Design of Precast Concrete Structures against Accidental actions. [25]

The alternative load path method implies that:

- the local damage must be bridged by an alternative load-bearing system and that the transition to this system is associated with dynamic effects that should be considered
- the connections and tie reinforcement should be designed to resist the resulting actions
- the structure as a whole must be shown to be stable with the local damage under the relevant load combinations.

An advantage of this method is that it is quite clear and can be used in the very beginning of the structural design process. Disadvantages are that the analytical procedure can be very tedious and that the demands on the remaining structure can be unrealistically large leading to possible over-mitigation and unnecessary additional costs. Where notional removal of a particular element results in damage in excess of acceptable limits then that element should be designed in accordance with the specific load approach.

5.7.3.1 Skeletal structures

The following mechanisms can be used to provide alternative load paths in multi-storied precast skeletal structures. These are covered in detail in the fib Design of Precast Concrete Structures against Accidental actions. [25]

Catenary action.
In the event of accidental damage to a column such that it can no longer carry any of its original load, the design force must be distributed to other members to avoid progressive collapse. The loss of support means that the beam has effectively doubled its span length. With appropriate tie reinforcement the excess forces in the system can be partly carried through catenary action of the floor beams.

Catenary action needs anchorage. Removal of penultimate columns one bay from an edge means that catenary action perpendicular to that edge will impose a large horizontal force on the edge column which must then span two or more stories vertically to transfer that load. The horizontal reaction from the catenary force could be taken up by the diaphragm action of the floor above, but not necessarily by the floors above due to the damage caused by the column removal.

Cantilever action.
In case of failure of a corner column the surrounding structure can be supported by cantilever action. For example the horizontal tie reinforcement on top of the floor beam can function as cantilever reinforcement. To this effect the tie-reinforcement should be inside the projecting stirrups at the top of the beams.

Suspension from the upper levels.
The intact upper structure above the damaged area can support the damaged area below. This is provided by vertical ties from the foundation to roof level in all columns and walls.

Membrane action of floors and roofs.
Floors and roofs can be designed to act as suspended membranes to span across damaged areas. As with catenary action large horizontal forces can be generated at the edges of the membrane and need to be appropriately anchored.

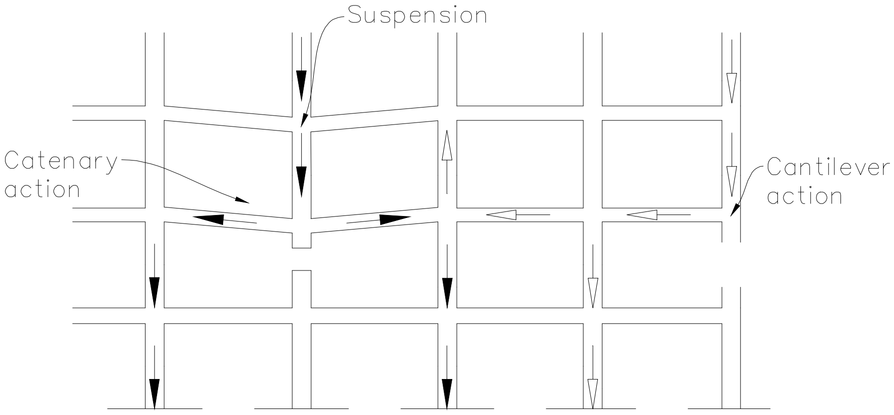


Figure 5.21 Alternative mechanisms for alternate load path in skeletal structures

5.7.3.2 Loadbearing wall structures

The following mechanisms are generally available in wall framed structures to provide an alternative load path in case of notional removal of a load bearing panel. These are covered in detail in the fib Design of Precast Concrete Structures against Accidental actions. [25]

Cantilever action of individual walls.
At the end of a series of wall panels the intact panels above the damaged area can be designed to individually cantilever through provision of horizontal ties between each panel and into each floor level.

Beam and arch action of the wall panels.
Where a panel within the length of a wall is damaged the panels above can be designed to span or arch across the damaged area through the provision of appropriate panel connections.

Vertical suspension of the walls.
Where a panel within the length of a wall is damaged, the undamaged panels each side can be designed to support the panel above the damaged area through provision of appropriate panel connections.

Cantilever action of the wall assembly.
At the end of a series of wall panels the intact panels above the damaged area can be designed to cantilever as a single assembly through provision of vertical ties between each panel and ties parallel to the wall at each floor level.

Catenary and/or membrane action of floors.
Successive spans of the floor elements above the damaged wall can be designed to carry the wall panels across the damaged area by provision of tie reinforcement in the floor with appropriate connections to the walls.

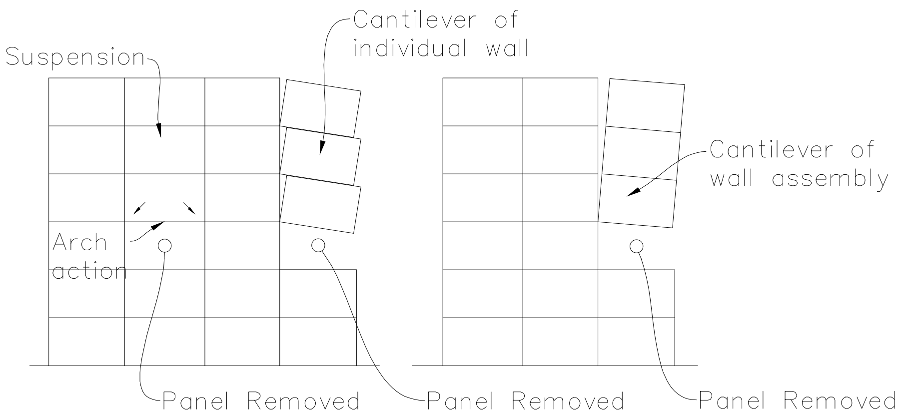


Figure 5.22 Mechanisms for alternative load path in wall frame structures

5.7.4 Specific load resistance method

The method of specific load resistance requires identification of all key gravity load-bearing elements and designing and detailing them to resist a postulated abnormal load. Key elements are defined as structural elements whose notional removal would cause unacceptable collapse.

As a guide, BS8110 [34] requires the key elements in all buildings not covered by the indirect design or alternative load path method be identified and that these elements be designed to resist a postulated abnormal load. The other parts of the structure should then be designed according to the alternate path method.

Key elements should be capable of sustaining an accidental design action applied in horizontal and vertical directions (in one direction at a time) to the member and any attached components having regard to the ultimate strength of such components and their connections.

The difficulty with strengthening key elements is that it must be done with a specific threat in mind. Many standards, however, do not define values for accidental actions, but leave it to the designer to nominate. The exception is structures subject to impact from vehicles where specific actions can be determined.

5.7.5 Risk elimination

A better strategy to cope with the problems of key element design or the difficulty of providing alternative load paths is to modify the structural arrangement of the building.

BS8110 [34] and the fib Design of Precast Concrete Structures against Accidental actions [25] recommend that for buildings with high occupancy such as stadia or for buildings where dangerous processes are carried out and for buildings where the consequences of accidental actions can be significant, a systematic risk assessment of the building should be undertaken and the required improvements based on this assessment be implemented.

6 BRACED WALL STRUCTURES

6.1 General

As described in Chapter 3, Braced wall structures consist of a series of wall panels connected to a lightweight roof structure. There are a number of variations on this concept but the most common is where the walls are loadbearing and support a lightweight roof structure. Wall panels are erected and braced and the roof structure is then erected to connect all elements.

Stability is provided by bracing the roof to transfer lateral load to the cross walls. A variation of this is where the loadbearing walls cantilever from the foundations to provide lateral stability. Alternatively stability can be provided by a complete portal frame system with the walls non-loadbearing cladding panels.

This document does not address the design of roof structure but assumes the designer will apply established engineering principles to ensure lateral loads from the walls can be transferred through the braced roof structure.

In this Chapter

6.2 Design Concepts

For most buildings the selection of the structural system will depend on the functional requirements, layout of the building and the wall heights. The roof framing module does not need to match that of the walls so the design of each should be optimised.

Broadly there are two structural options, loadbearing and non-loadbearing walls. Each can cover a number of variations with the choice usually depending on economics and buildability issues.

Wall panels are usually solid, but hollowcore or other profiles can be used. On loadbearing panels, slenderness is critical and the design needs to take into account P-Delta effects. In most cases axial loads are small and panels act more like flexural members than compression members. The design of the individual wall elements is covered in detail in Chapter 4.2.

Typically the most economical solution is where the walls are loadbearing and support the roof structure. This makes use of the inherent load carrying capacity of wall panels to support the roof without additional columns.

6.2.1 Loadbearing walls

Wall panels are full height and support the roof structure. Roof connections to the wall should be designed as pinned to minimise induced bending in the walls. Where necessary a steel eaves tie, connected to the rafters, can provide lateral support to the top of the wall panels. Wall panel thickness is determined by their height and the lateral and vertical actions. Lateral actions are transferred to the walls by roof bracing and walls can either resist these in-plane forces as a series of individual panels or be connected to act as a single element.

As an alternative, where the roof load requires an excessive wall thickness a thicker panel/column can be provided at that location.

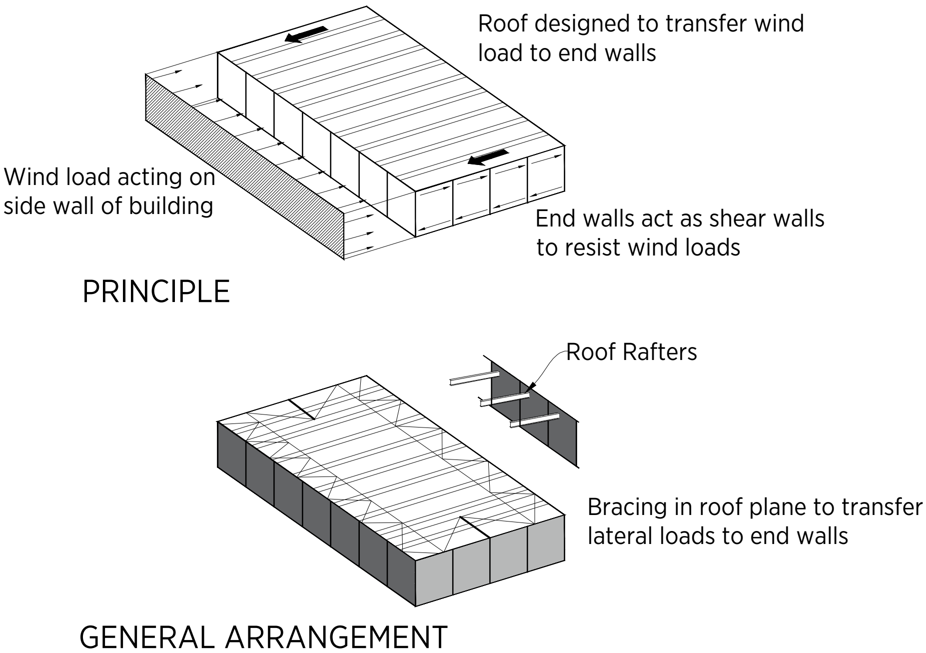


Figure 6.1 Loadbearing wall structure

6.2.2 Non-loadbearing walls

Where wall panels are vertical and non-loadbearing they are supported by a portal frame and eaves tie system. Vertical actions are carried directly to the footings. Lateral actions on the building are resisted by the portal frames and wall bracing or by shear wall action in the plane of the wall panels.

Where wall panels are horizontal and non-loadbearing they span horizontally between and are laterally supported on a portal frame structure. It is important to align the bearing pads between panels to ensure that vertical actions are carried directly to the footings. Where cladding panels are erected onto the edge of a suspended floor the erection load case of the panels supported on two discrete bearing pads needs to be checked.

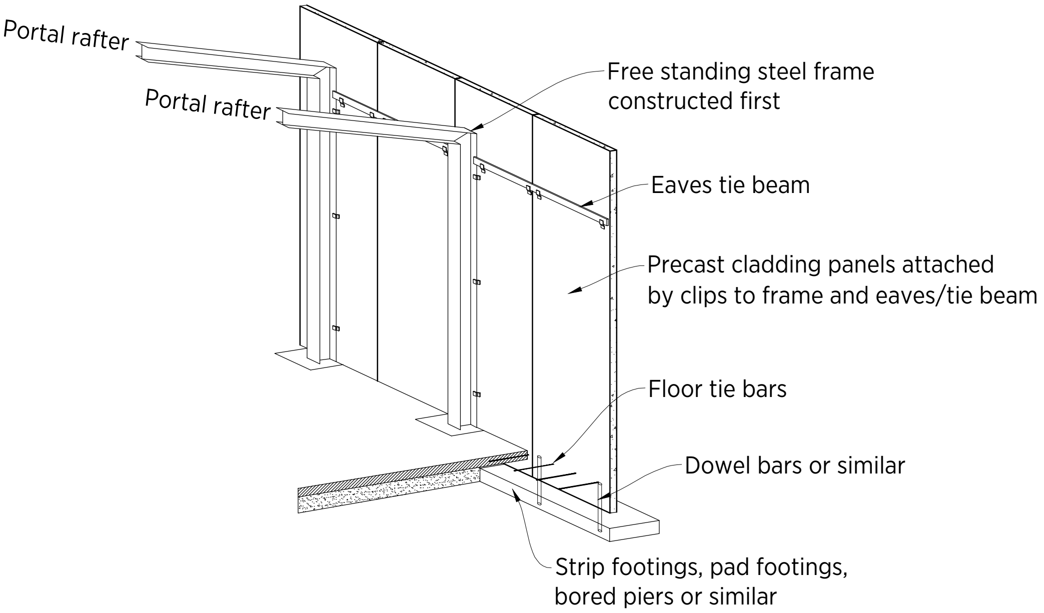


Figure 6.2 Vertical non-loadbearing panels

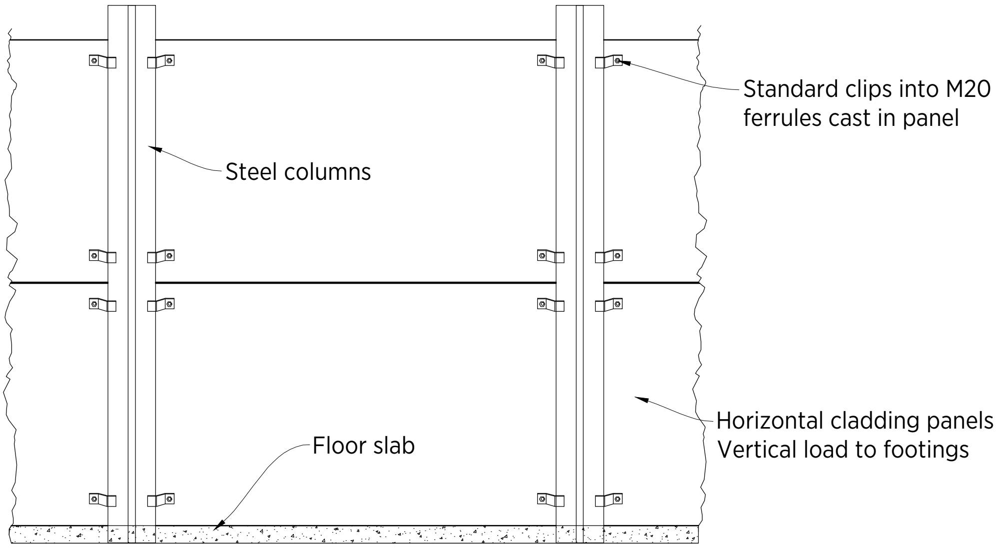


Figure 6.3 Horizontal non-loadbearing panels

As noted above, lateral stability of braced wall structures is usually provided by a roof diaphragm that transfers lateral actions to the wall panels. In-plane forces in the panels generally result from these lateral forces causing the panels to act as shear walls. In non-seismic areas it is rare for the shear stresses in the wall panel to be critical. The critical factor is usually stability of the compression edge of the panel and this should be checked for buckling due to induced axial forces. The simplest method of checking for lateral buckling is to calculate the reaction induced at the base of the compression side of the panel by the combined imposed axial forces and in-plane forces. This reaction is distributed up the panel as shown in Figure 6.4.

Where it is necessary to connect adjacent panels together to provide greater shear wall resistance the connection between panels needs to be designed to resist the induced vertical shear forces. These connections also need to take into account the forces induced by thermal and shrinkage along the length of the wall. These later forces can be quite significant and can be minimised by limiting the number of panels connected together.

In seismic areas careful attention needs to be given to the stiffness of the roof diaphragm to limit overall sway of the building. The resulting distortion of the connections to the panels on the tension and compression flanges of the roof diaphragm can lead to failure. Flexibility of the roof diaphragm is a common cause of failure of braced wall structures.

In all cases the roof diaphragm should be designed with redundancy to ensure that complete collapse of the building cannot occur due to failure or removal of a single member.

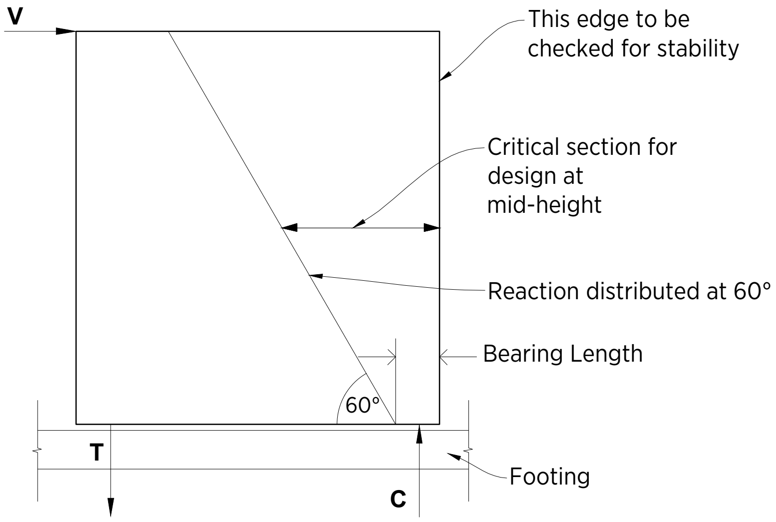


Figure 6.4 In-plane forces

6.3 Connection details

Some examples of typical connections in braced wall structures are given in this section. The intention is not to show a complete overview of all existing solutions, but to make the designer familiar with some common types of connections. The principles applied in the majority of solutions are valid both for low-rise buildings and the upper level of multi-storey buildings.

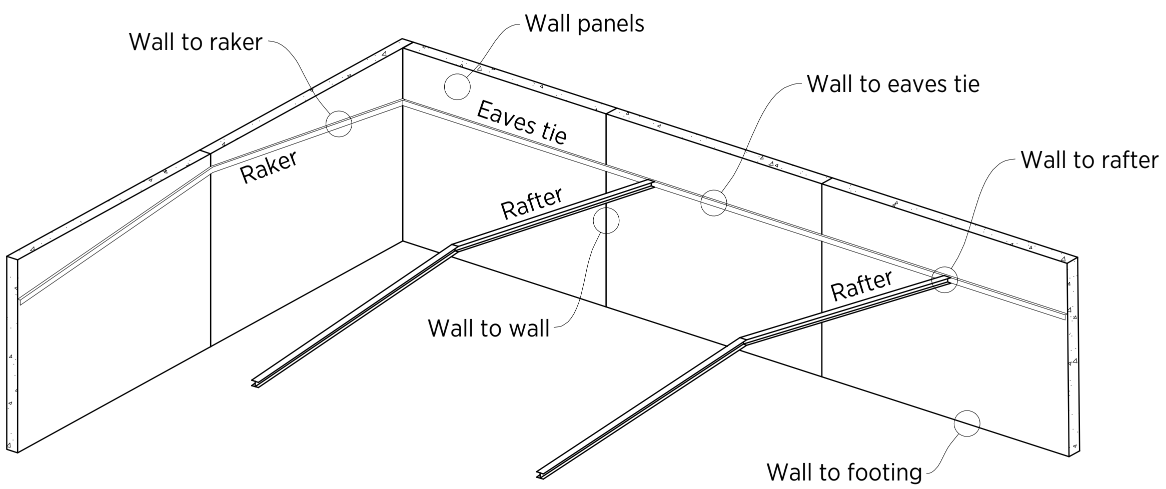


Figure 6.5 Wall connection types

6.3.1 Wall to footing connection

The most common wall to footing detail is shown in Figure 6.6 a) and can be used for both loadbearing and non-loadbearing wall structures. Care needs to be taken in detailing to ensure that the dowel is restrained by reinforcement to prevent side breakout particularly where dowels are drilled and driven into snug tight holes in the footing.

A similar detail can be used with cast-in dowels to provide base fixity for cantilevering panels. In this case it is recommended that the calculated development length of the dowels into the ducts be increased by 50% to account for the lack of confinement reinforcement.

An alternative connection is a welded fixing as shown in Figure 6.6 b). This is usually more expensive than the dowel but is often used in combination with dowels to provide base fixity to the wall panel.

Tie bars as shown in Figure 6.6 c) can be used where high lateral restraint is required or in combination with dowels to provide base fixity. This solution requires either the whole floor or an infill floor strip to be poured after the panels are erected.

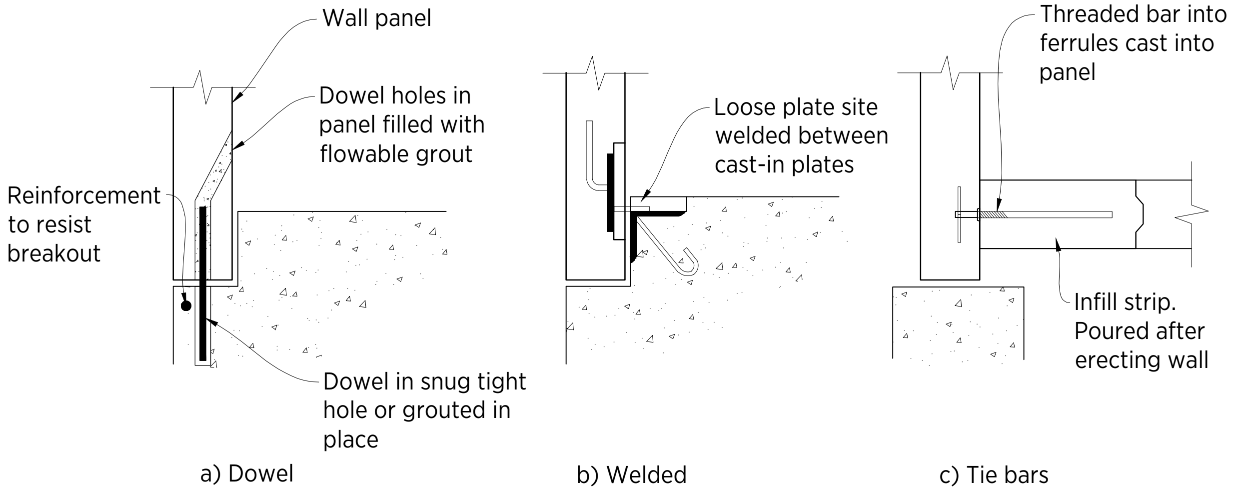


Figure 6.6 Wall to footing connections

6.3.2 Wall to roof connection

Wall to roof frame connections form a vital part of braced wall structures. They have a major influence on structural behaviour. Although it is possible to design connections within the full range from pinned jointed to fully rigid, simplicity of execution is far more important than complexity in design, and in this context pinned connections are the preferred choice.

There are two basic locations of wall to roof connections. They are rafter to a side wall panel and gable raker beam to an end wall panel.

Rafter connection.
With this connection the steel rafter is supported on the loadbearing wall panel by way of welded or bolted connection. See Figure 6.7. In either case the connection should be designed and detailed as pinned to minimise induced bending in the wall panel. This is accomplished by providing fixings only at one horizontal location over the depth of the rafter.

As noted above, the roof module does not need to match that of the wall panels so it is almost certain that the optimum roof module will be much greater than that of the wall panels. Whether the rafter connection should occur at panel joints or within a panel is open to some debate and the final decision on joint location may depend on buildability issues such as optimising wall panel widths.

If cast-in plates are used in the wall panel it is usual for a steel corbel to be provided at the appropriate height to act as support during erection of the rafter and as a bolting bracket for final fixing. See Figure 6.7 a).

The alternative directly bolted connection requires an end plate on the rafter with over-sized holes and heavy washers on the bolts to accommodate tolerance. This type of connection also requires some allowance between end plate and wall panel for construction and manufacturing tolerance. See Figure 6.7 b). In both cases it is difficult to avoid site welding, either to weld the corbel or to weld the heavy washers to the rafter end plate.

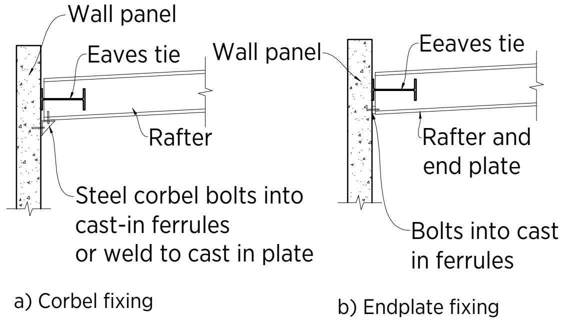


Figure 6.7 Rafter connections

Both types of connections are required to exhibit ductile behaviour and at the least this will require cast in plates or ferrules with anchor bars that can develop their full capacity after local failure of the concrete at the connection. Because of their limited ductility, drilled-in expansion or chemical anchors are not recommended as fixings for these connections.

Where rafters are pitched to an apex the connection needs to be able to accommodate the rotation due to the outwards deflection of the ends as the roof is loaded. Note that this is also an issue with the bracing of the wall panels during construction where the erection braces can be overloaded by outward deflection of the ends of a pitched rafter. Failures can occur as a result of this mechanism and care must be taken when designing the erection braces, particularly with long-span pitched rafters.

Raker beam connection

Rakers are steel members fixed to the end walls to support the end bay of the roof structure. Consequently the load on the raker beam is relatively small. The usual detail is for a continuous steel angle or channel to be fixed to the wall at regular intervals. Fixings are generally bolted or welded as shown in Figure 6.8. Even with bolting, welding is often required either to prevent the washer slipping in the oversized holes required for tolerance or to prevent the clip fixing rotating or slipping off the raker during a fire.

Various other types of proprietary fixings such as expansion anchors or cast-in toothed channels can be used in these connections but must be specifically manufactured for use as structural connections and have adequate capacity to resist unintentional overloading. They have the advantage of accommodating tolerance but can suffer from a lack of ductility. For this reason they should be used with care and are not recommended where design for seismic action is required.

Eaves tie to panel connection

Eaves ties are steel members spanning horizontally between rafters to transfer horizontal forces from the top of the wall panels to the roof bracing system. These ties are usually steel channel or universal beam sections. They need to be designed not only for horizontal load from the walls but for their self-weight in the vertical direction.

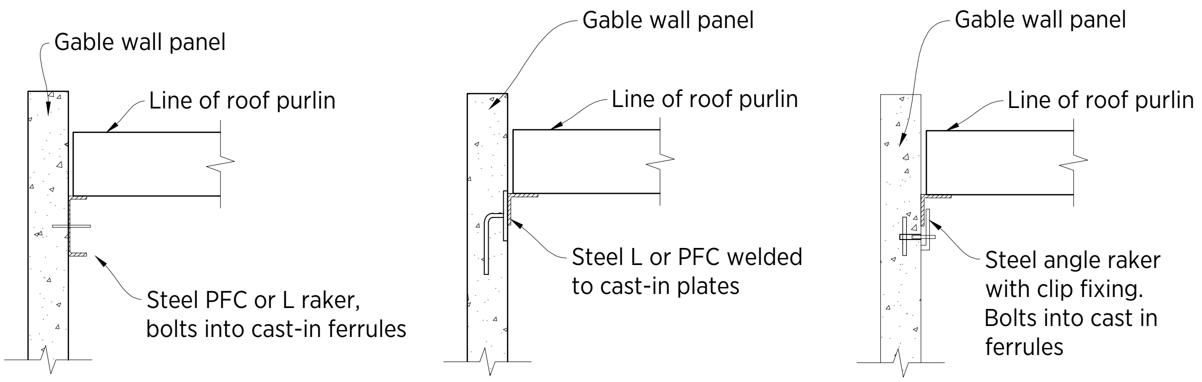


Figure 6.8 Raker beam connections

There are three basic types of fixings used in these connections.

- • Welded fixings formed between cast-in steel plates and the steel eaves tie. A disadvantage of this fixing is that temporary support maybe required until the weld is completed.
- • Bolted fixing through a cleat welded to the steel tie and into a cast-in threaded insert. This does not require temporary support for erection but tolerance requires over-sized holes in the cleat that may necessitate welding the washer to prevent movement after erection. A variation of this is to use post installed expansion or chemical anchors in lieu of the bolt into an insert. The disadvantage is the limited anchor capacity and the lack of ductility in this type of connection and as noted above such fixings should be used with care.
- • Clip fixings are often used because they provide ample tolerance but they need to be detailed to ensure that the clips cannot rotate or slip off the tie when subject to cyclic or fire load. This is best accomplished by welding the clip to the steel tie. The use of two bolts prevents the clip from rotating but does not ensure the clip cannot slip off the tie.

As with the previous connection details it is difficult to avoid site welding on these connections.

6.3.3 Wall to wall connection

Wall to wall connections are common within the precast industry and every consulting engineer and precast manufacturer seems to have their own favourite. The typical concept is to have a steel plate, complete with anchor bars, cast into the edge of each panel. After erection a stitching plate is bolted and/or welded across the joint to form the connection. Because of the cost of the plates and the need to site weld at height these are expensive connections and should be only used where needed for structural reasons. Each additional plate in a wall panel can add several dollars per square metre to the cost of the panel.

There are a number of proprietary panel connection systems available that allow the installation of precast concrete panels on site without the need for welding or bolting. These usually involve some type of dowel or bracket into a pocket that is grouted after erection.

Insitu concrete stitch joints between panels can be used but are seldom justified on braced wall structures. The exception is where large vertical shears need to be transferred across the panel joint. Insitu concrete stitch joints are expensive and time consuming to form and pour on site.

The concepts for corner panel connections are the same as for panel to panel connections. As shown in Figure 6.10 they are often required at mid-height of tall panels to restrain outward bowing of the corner due to thermal gradients. In such cases the forces can be very high and the connection should be designed accordingly.

Panel to panel connections are described in greater detail in Chapter 8.

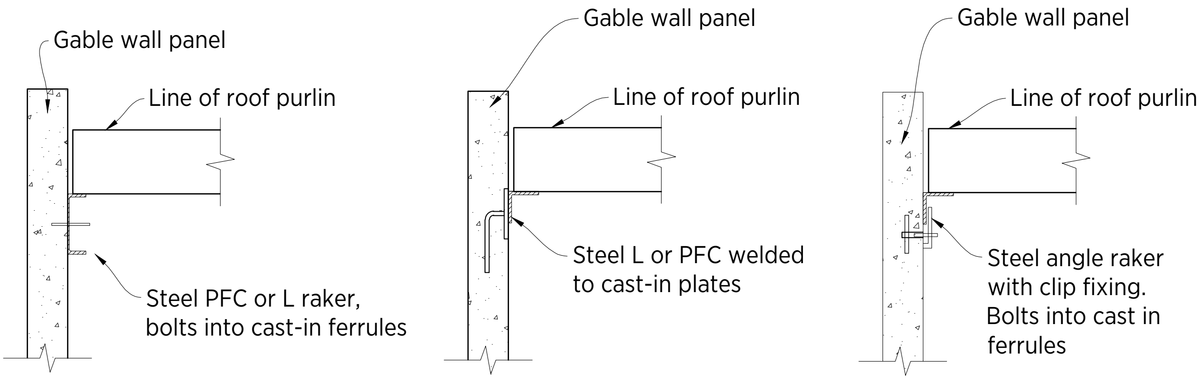


Figure 6.9 Eaves tie connections

6.3.4 Cladding panel connections

The most common connection for cladding panels involves a fixing between a steel portal column and precast wall panels that span horizontally between columns. In most cases the wall panels are stacked one above the other and the panel load is carried directly to the footings. The connections are therefore only required to carry horizontal forces.

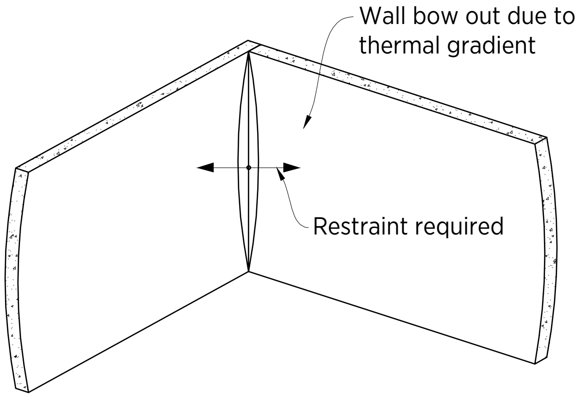


Figure 6.10 Corner panel connection

Although there are a range of connection types used, such as bolts through cleats or welded cast-in plates by far the most common is a steel clip fixing as shown in Figure 6.11. Clips need to be detailed to ensure that they cannot rotate and slip off the tie due to movement in the structure. This can be accomplished by using two bolts per fixing or by welding the clip to the column after erection.

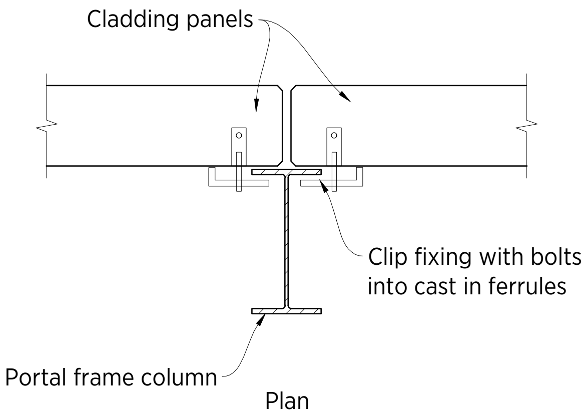


Figure 6.11 Horizontal panel connection

As required by the Building Code of Australia [10] these connections also need to be detailed to accommodate thermal movement and possible collapse of the roof structure during a fire.

Cladding panels can act as shear walls by positively connecting each panel to the steel frame or by providing grouted dowel connections in the horizontal panel joints so that a series of individual panels act as a single panel.

The remaining types of cladding panel connections, vertical panel to eaves tie, panel to panel and corner panel connection are all similar to those used for loadbearing walls as described in Chapter 8.

7 SKELETAL STRUCTURES

7.1 General

As described in Chapter 3 skeletal frames consist of series of columns, walls, beams and floor slabs, assembled and connected to form a robust structure able to support and transfer vertical and horizontal actions from floors and facades to the foundations.

The design of the individual elements comprising a skeletal frame structure is described in Chapter 4.

This chapter will focus on skeletal structures used for low to medium rise buildings. Technically there are no constraints to the height that can be constructed but currently in Australia material handling issues dictate a maximum practical height limit.

The most efficient skeletal frame solution irrespective of the number of storeys is in braced structures where the horizontal stiffness is provided by stair or lift shafts or shear walls. In this way, connection details between beams on columns can be designed as pinned, greatly simplifying the design and construction.

Figure 7.1 Precast skeletal frame structure with stabilising core

In this Chapter

For buildings up to 2 to 3 storeys the structural system can be based on the cantilever action of the columns or walls, which are clamped into the foundations.

Moment resisting skeletal frames are not common in Australia due to the high cost of the connections. Provision of such rigid connections to produce a fully moment resistant frame is difficult and should be avoided wherever possible. The use of cast-in and welded connection brackets or cast insitu concrete infills that emulate cast insitu construction is an option where a small number of rigid connections are required but is generally not an economical solution for a complete building.

Special precast frame systems utilising un-bonded post-tensioned tendons to provide continuity have been developed in the USA for earthquake resisting frames but for Australian conditions they should only be considered where braced structures are not an option. [19]

7.2 Design concepts

For most buildings the selection of internal frame elements is governed by the demands of the layout, such as the need for clear floor areas, the location, size and orientation of lift shafts and stairwells, and major subdivisions of the building. The choice of perimeter elements is governed by the facade, and the designer is able to specify an external frame that is, in the main, different from the internal arrangement, and to adjust the frame elements to suit both internal and external requirements. Beam alignments need not be parallel and changes of direction within a beam line can be used to accommodate offset column positions.

Columns can be floor to floor or multi-level. The choice depends on a number of factors related to cost of manufacture, cost of beam to column connection and erection costs per element. On low-rise buildings where the column loads are relatively small floor-to-floor columns are often be the best option.

To optimise the precast concept the walls are usually precast. They can be single or multi-level and act individually or connected to form lift and stair shafts to provide lateral stability to the structure. They can also be insitu concrete.

Beams can be of inverted Tee, L-shaped or rectangular cross-section. To provide minimum structural depth, the beams should be composite with the floor slabs and placed to span the shorter grid with the floor slabs spanning the longer grid. To further reduce the structural depth the beams can be designed to be continuous for imposed loads that are applied after composite action is formed. Structures using shell beams or solid beams with insitu concrete joints are not covered by this chapter. These types of structures emulate insitu concrete and the concepts and detailing of the joints is similar to that for insitu concrete.

Although there are several precast floor slab systems available that can be incorporated into a skeletal frame structure this chapter will focus on the use of hollowcore slabs. Each type of precast floor has its optimum range of spans and load capacities and although interchangeable to a certain extent hollowcore tends to dominate on longer spans. All precast floors have specific advantages and the choice often depends on features within the projects related to cost, availability and erection issues.

7.2.1 Framing grid

The basic concept in selection of the framing grid is to optimise the costs of the beams and floor slabs. Beams, with their supporting columns are relatively expensive compared with floor slabs so the objective should be to minimise the ratio of beam length to floor slab area. This requires an understanding of the relative performance as well as the cost of each of these elements.

Various beam profiles and design methods are discussed in detail in Chapter 4.4.
Figure 7.2 gives an example of load capacities verses span for a range of inverted-tee beams. This shows that for typical beam loads of about 150-250kN/m optimum spans are in the order of 8-10 metres. Greater spans are possible but beam depths and therefore cost can become excessive.

Establishing the optimum building grid is very much dependent on choice of floor slab. There is no simple solution, but maximising the span of the floor slab and minimising the span of the supporting beams will generally give the most economical outcome. In other words run the beams on the short span and the floor on the longer.

Applying the above guidelines will allow the internal grid of the building to be established. Unless constrained by the physical layout the traditional 8.4x8.4 grid is not the most economical for a precast structure. Certain types of buildings lend themselves to particular grids. For example on a building containing car parking on the lower levels a grid of 8.4x10.8m or 8.4x12.6m is a good solution. On a multi-level carpark building a grid of 8.4x16.8m is an ideal solution.

As noted above, the perimeter grid need not match the internal grid and should instead be optimised to suit the façade. If the façade is lightweight then a perimeter

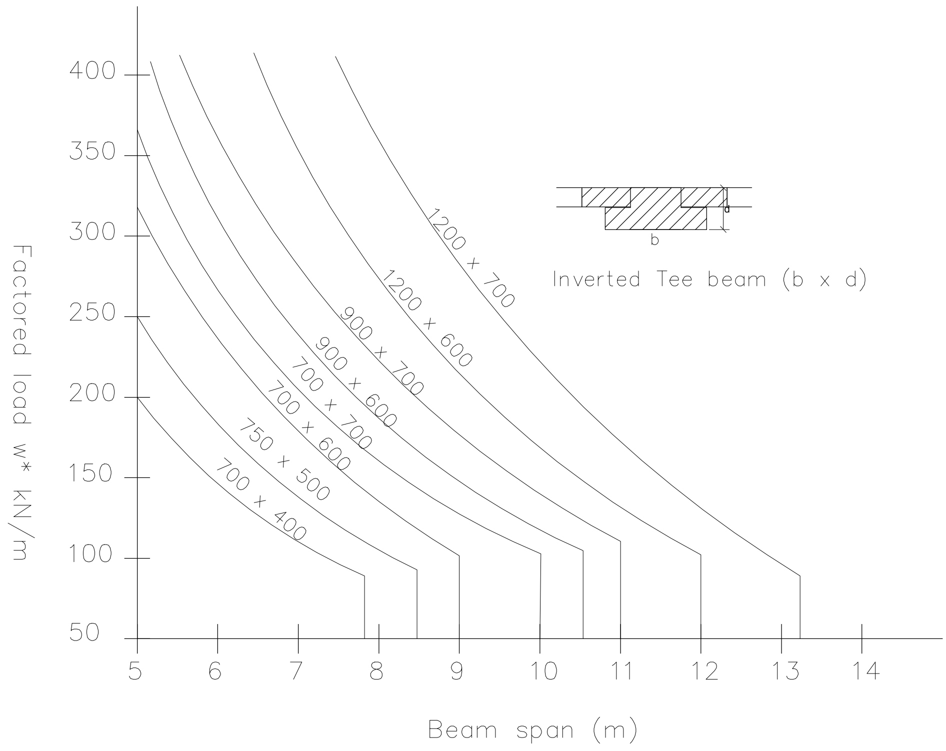


Figure 7.2 Indicative load verses span data for composite prestressed concrete inverted-tee beams

skeletal structure is the obvious solution and this can be dimensioned to suit the architectural requirements. If a concrete façade is nominated consideration should be given to using loadbearing precast concrete wall panels. Where this is not possible, façade or spandrel panels that are supported on a skeletal structure at each level can be used. A common solution is to have an upper loadbearing façade supported on a transition level above the ground floor. This can be an economical solution provided the spans of the transition beams are limited.

7.2.2 Establishing the grid layout

By way of example this section outlines the typical process that can be carried out by the project consultant to establish the grid layout for a building.

The building shown in plan in figure 7.3 has the following planning and architectural requirements.

- Building footprint set by site limitations.
- Internal column and core locations are set by basement carparking.
- North and East walls are glass curtain wall.
- South wall has horizontal glazing with up-stand spandrels.
- West wall is a blank wall on the boundary.

Given the above constraints the solution for the grid layout and framing is as follows.

- The column locations dictate longer grid dimensions in the east/west direction and determine the floor should span east/west and the beams north/south.
- The central core can provide lateral stability and allow the structure to be a braced skeletal frame.
- The blank west wall lends itself to a loadbearing precast concrete wall.
- Internal beams can be inverted Tee beams and splayed from the grid to be supported on the central core.
- The east perimeter beams can be L beams with spans to suit the façade module.
- The glass curtain wall can be supported off the floor slabs eliminating the need for perimeter beams on the north façade.
- Beams on the south side of the central core can either span from the south wall to the core or be supported on an internal column as shown. In the latter case the short beam can be very shallow to allow large service ducts to exit from the core.
- The south wall can be spandrel beams that span from grid to grid.
- On these spans hollowcore slabs with a topping screed is probably the most economical floor system.

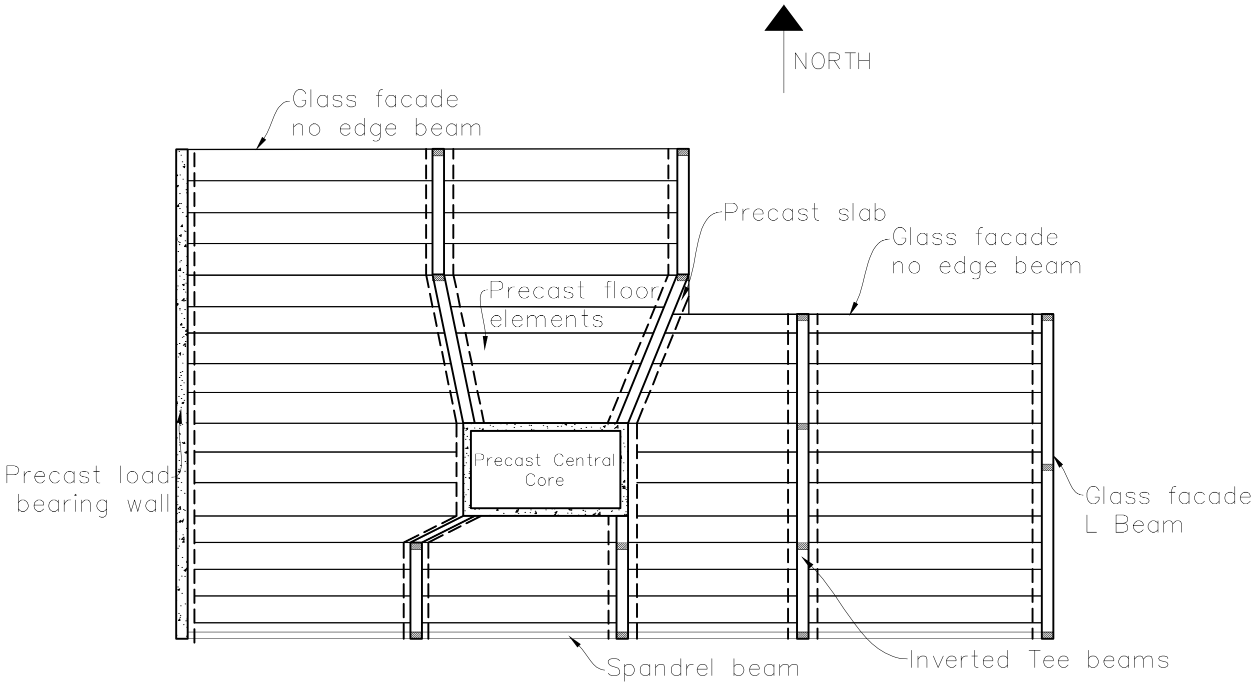


Figure 7.3 Grid layout for commercial building with lower level carpark

7.2.3 Bracing method

The bracing method used to resist lateral loads on a building will be governed by the demands of the layout. If the building is 2 to 3 storeys, with few walls the structural system can be based on the cantilever action of the columns, which are fixed into the foundations.

Taller buildings have escape stairs and lift-shafts that can be used as shear cores or shear walls to provide a braced frame structure in the same manner as for an insitu concrete structure. Braced frames require stabilising walls/cores in both orthogonal directions in the building plan, typically no more than 30-50m apart. This may present some problems in determining the number and the best positions for the shear walls/ cores that suit the architectural and building services requirements.

Placement of the shear cores within the structure is important to ensure that the torsional behaviour of the building is not compromised. The example given in Figure 7.3 is not ideal as the offset central core and west wall will introduce high torsional loads. This is described in detail in Chapter 2.3

7.3 Connection details

Examples of typical connections in skeletal structures are given in this section. The intention is not to show a complete overview of all solutions, but to make the designer familiar with common types of connections. The principles applied in the majority of solutions are valid both for low-rise and multi-storey buildings. In the total structure, these examples are complemented with other examples relative to floors, walls and façades as shown in Chapter 8. Additional connection details are available in manufacturers technical manuals.

Figure 7.4 shows a typical skeletal frame and identifies the main connection types as follows.

- Column to footing.
- Column to column
- Column to beam.
- Beam to beam
- Beam to wall.
- Floor slab to beam.
- Floor slab to wall

7.3.1 Column to footing connection

The most common column to footing connection is shown in Figure 7.5. The connection is realised by bars protruding from the footing and inserted into ducts in the bottom of the column to lap with the main reinforcement. The diameter of the ducts should be oversized to allow for placing tolerances and for the use of a flowable grout. The solution is also applicable for circular columns.

Where large numbers of protruding starters are required the projecting lengths should be staggered to facilitate positioning into the column ducts.

Additional tie reinforcement should be provided at the bottom of the column to resist splitting forces induced by the reinforcement discontinuity.

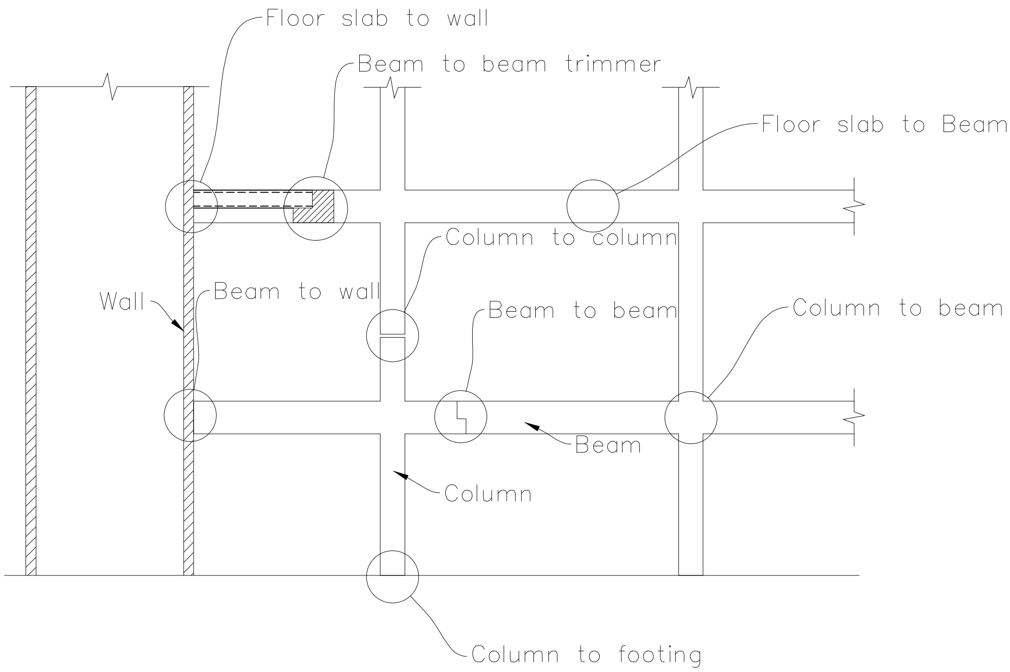


Figure 7.4 Skeletal frame connection types

There are several variations of this detail that can be used in different circumstances.

- In most cases of braced frame structures the column base is assumed to be 'pinned' and the protruding bars can be relatively short, allowing the bars to be driven into snug tight holes drilled into the footing after the footing has been poured. This simplifies the construction and the setting out process.

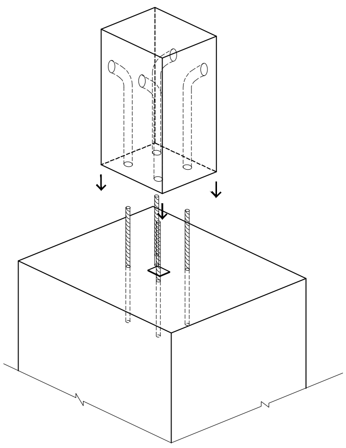


Figure 7.5 Protruding bars from the foundation into grout ducts in the column

- A moment-resisting connection can also be realised by casting or grouting the protruding starter bars into the footing and lapping them into corrugated metal ducts in the column. The lap into the column should be generous to ensure that full development is realised with the column reinforcement. Moment capacity can be assessed by modelling a section based on the geometry of the column base and the size and location of the grouted dowels.

An alternative column to footing connection is by grouting the base of the column into a pocket footing. This connection detail as shown in figure 7.6 is commonly used for a moment resisting connection to the footing. The solution is usually restricted to situations where concrete pad footings can be founded at shallow depths.

The pocket should be dimensioned as shown and large enough to enable good structural grade concrete or non-shrink concrete to be used as the infill.

When the inner surface of the pocket is roughened to expose the coarse aggregate or castellated with a shear key the vertical force and bending moments are transferred successively by shear at the interface. Such pockets may be considered to act monolithically with the column.

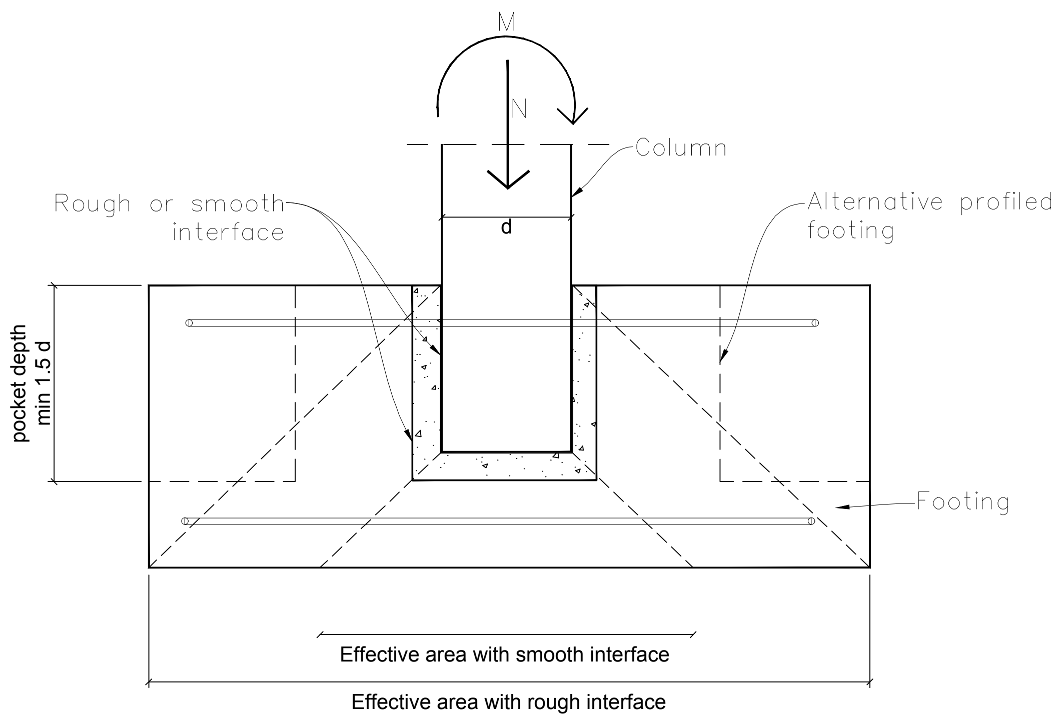


Figure 7.6 Pocket footing

When the inner surfaces of the pocket is smooth the vertical force is assumed to be transmitted directly under the column base and the depth of concrete below the column needs to be sufficient to transfer the column load to the foundations.

A less common connection between column and footing are bolted steel connections. This connection can also be used to provide a moment-resisting connection. The steel fitment is cast into the column and the force transfer is realized through overlapping of the column main reinforcement bars with steel bars welded to the fitment. There are various alternative solutions.

- Individual steel shoes as shown in Figure 7.7. Each shoe consists of a metal fitment with an attached void former and welded anchor bars. The number of shoes in the column depends on the dimensions of the column, moment capacity and type of column used. The solution is also applicable for circular columns.
- Base plate with starter bars welded to plate as shown in figure 7.8. The steel plate is either overhanging beyond the edge of the column, typically 100 mm or may be flush. If it overhangs, the disruption to manufacture of the precast column may be considerable because the plate cannot be contained within the internal confines of the mould.

Although the cost of bolted steel base plates is clearly greater than that of dowels, the potential cost saving is in the speed or erection, and the footing connection is immediately stable to allow the floor beam to be positioned almost immediately. The moment-resisting connection into the foundation is realized with anchor bolts in a similar manner to that for a steel column.

Where column bases transfer a bending moment into the footing, the geotechnical aspects of the foundations need to be checked.

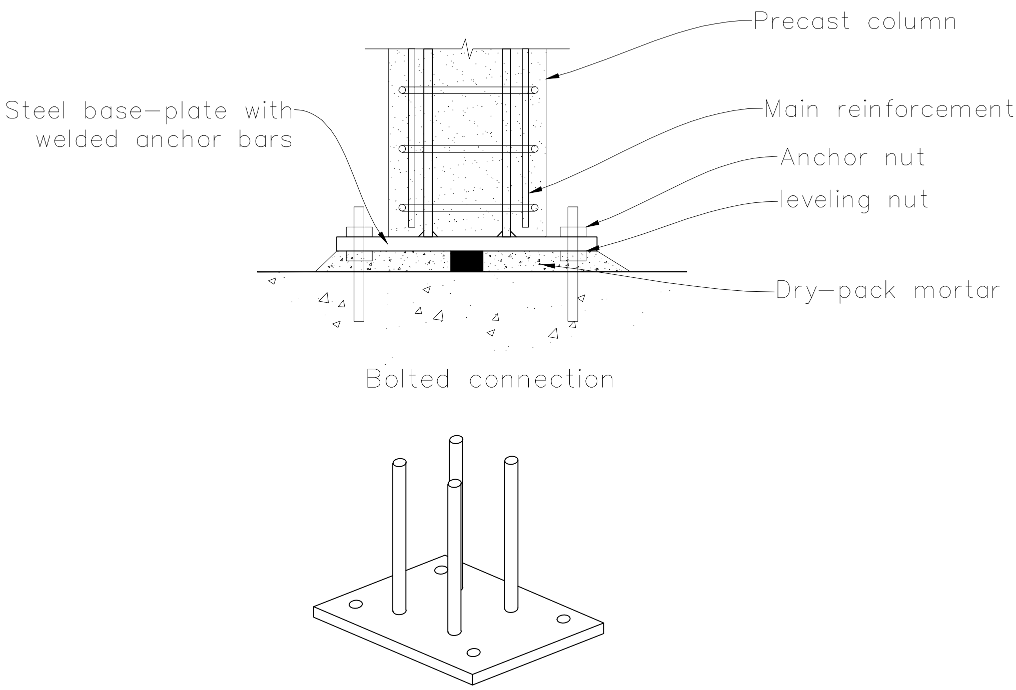


Figure 7.7 Columns bolted to footing: base plates

7.3.2 Column to column connection

If a full height column is not possible, column-to-column connections, known as 'splices' are required. These splices can be either within the beam to column connection as described in Chapter 7.3.3 or, where the column runs past the beam, they can be above floor level.

In the latter case where there is a direct column to column splice the connection is in principle similar to the dowel connection between columns and footings. The lower column is provided with projecting dowels which are grouted into ducts in the base of the upper column. As described in Chapter 5.6.2.4 the capacity of the dowels in tension should not be less than the reaction applied to the column from any single storey of the building.

In all cases of concrete to concrete connections care needs to be taken with detailing to ensure that edge spalling does not occur. Spalling can be avoided by using neoprene bearers or chamfering contact edges of the elements.

The National Precast Concrete Association Australia have produced a grouting guide for loadbearing joints that covers this topic in some detail. This document, 'Understanding Grouted Precast Joints, a guide for Engineers and Contractors', [43] highlights the need for careful design and detailing of the load transfer mechanism through grouted horizontal joints.

The load transfer area through a joint can be reduced significantly by chamfers on the column edges and by the use of foam strips as grout barriers. The effect of the construction process on site needs to be taken into account by the design engineer. Assuming that the full column area is available to transfer the load can lead to significant overestimation of the joint capacity.

The importance of establishing a work process that ensures the joints are grouted in accordance with the design requirements cannot be overemphasised. Foam 'donuts' around dowel bars should only be used where specifically detailed by the design engineer.

Structural failures have occurred due to improper grouting and this process should not be left to inexperienced or unskilled tradespeople.

AS3600 [7] requires minimum reinforcement in columns and Clause 17.7.3 requires minimum tension capacity across joints. Elliott [21] suggests that the splice zone should be checked as a 'short column' using the configuration of the dowels and the grout strength as the design model. As note above this design should take into account any reduction in effective area of the column due to the grouting procedure.

As with the column to footing detail additional column ties are required to resist splitting forces induced by the discontinuity of reinforcement.

7.3.3 Column to beam connection

Column to beam connections form a vital part of precast concrete construction. They have a major influence on structural behaviour. Simplicity of execution is far more important than complex design solutions, and in this context pin jointed connections as part of a braced frame structure are the preferred choice.

Figure 7.9 shows some of the typical column to beam connections that are commonly used.

Both type A and type B connections present advantages and disadvantages. Type B gives an excellent force transfer performance, the joints are easily accessible and large forces in the columns are readily transferred directly from one column to another. The disadvantage is in the cost of forming the corbels. Type A, with a beam passing over the column will give better moment distribution over the beams, but most probably a poorer performance at the column connection. The structural discontinuity in the column formed by the beam-column connection induces high splitting forces in the ends of the column and beam and it is difficult to provide appropriate reinforcement within the ends of the beams to resist these forces. On taller buildings transferring high column loads through this type of connection can become an issue. In this case the preferred option is the Type B connection with the columns running through the beam-column connection and spliced column-to-column.

With connection Type A, the beams are directly supported on the lower column. It is a simple connection to manufacture and erect and is the preferred option where the column loads are moderate.

The connection is realised by dowel bars from the lower column projecting up through ducts in the beams and into ducts in the bottom of the upper columns. These dowels are usually proprietary threaded bars to allow nuts and washers to clamp the beam to the top of the lower column. This clamping force can also be designed to resist any torsional effects due to out of balance loading on the beam, particularly during construction. Horizontal reinforcement to provide partial continuity can be easily provided across the top of the beams.

Figure 7.10 Connection type A

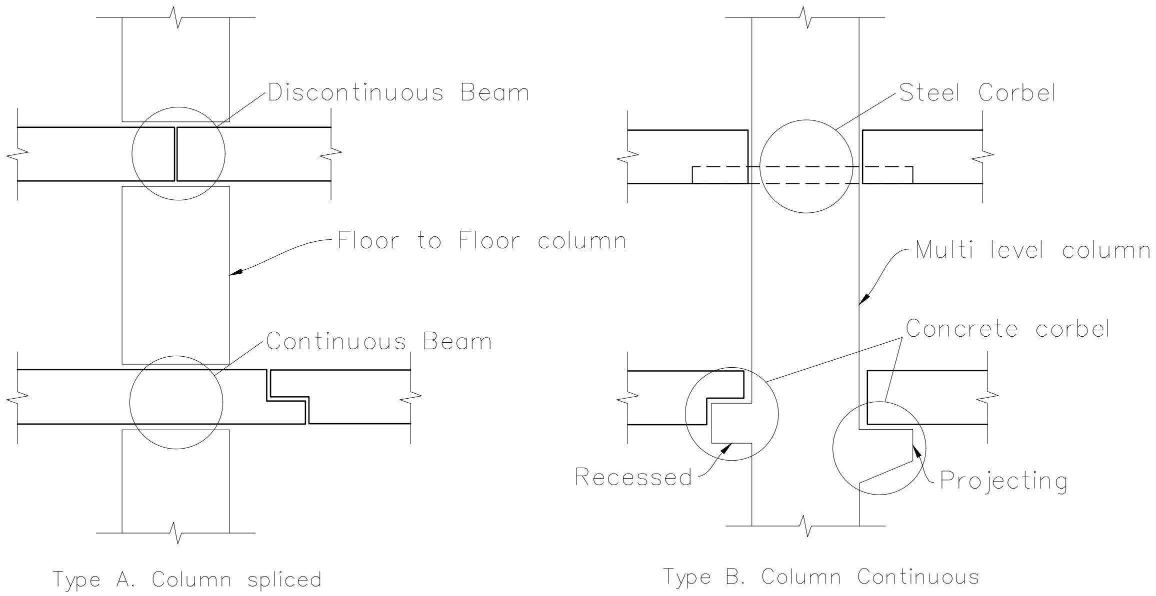


Figure 7.9 Alternative column to beam connections

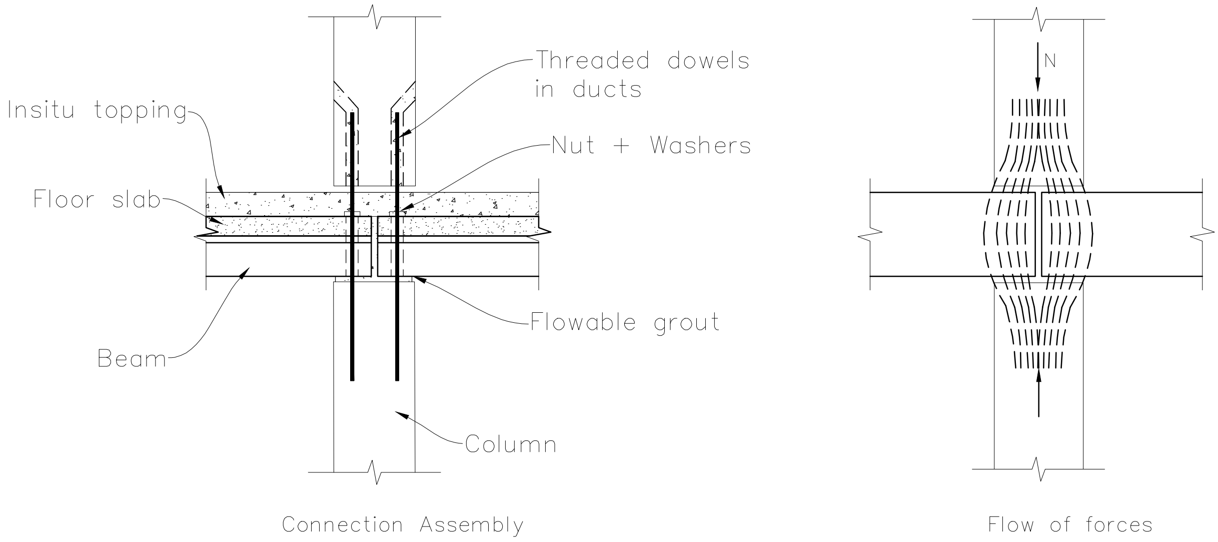


Figure 7.11 Connection Type A. discontinuous beam

The upper column loads are transferred down through the beam ends to the lower column. Although apparently a monolithic connection the failure mode depends on the relative strengths of the concrete and grout. Typically at ultimate limit state the grout 'extrudes' from the joint and induces high splitting forces in the ends of the elements. The flow of forces shows that the column ends are subject to a splitting force that must be resisted by horizontal tie reinforcement at the ends of the columns. The mechanism resisting the forces through the ends of the beams can be likened to a 'bottle strut' and horizontal reinforcement is required to resist the induced stresses. Added to these are the stresses induced by partial fixity of the beams and the discontinuity formed by the grouted dowels.

A variation of this connection is where the beam runs across the top of the lower column. This is a common detail where beams cantilever over a column or where beam continuity is used to reduce beam depths.

Connection Type B is the preferred option where column loads are high. With this type of connection the beams are supported by corbels on the column. Running the column past the beams avoids the issue of transferring high column loads through the beams and allows the column splice to be at any location. Corbels can be projecting or recessed and either concrete or steel.

Figure 7.12 Inverted-tee beams designed with a dapped joint to create a negative bending moment at the column that will reduce the depth of the beam

Figure 7.13 Connection type B

Concrete corbels are designed in accordance with strut and tie theory. The 'Precast Concrete Handbook' [2] has a detailed example of the design of a typical concrete corbel.

As with the Type A connections dowel bars from the concrete corbel project up through ducts in the beams. These bars are usually threaded to allow nuts and washers to clamp the beam to the top of the corbel. This clamping force can also be designed to resist any torsional effects due to out of balance loading on the beam, particularly during construction

Proprietary steel corbels as shown in figure 7.15 are not readily available in Australia but specifically designed steel insert corbels have been used on a number of projects. A variation of these is the hanger bracket described in Chapter 7.3.4, beam to beam connections.

7.3.4 Beam-to-beam connection

Beam to beam connections should be avoided if possible but where it is necessary, connections can be made as shown in Figure 7.16. There are two types of beam to beam connections. The first is where a joint is required along the length of a beam and the second is where two beams intersect at right angles.

Both types of connections can be formed either by concrete or steel corbels or frequently by a combination of both, with a concrete corbel on the supporting beam and a steel corbel on the supported beam.

Concrete corbels, or dapped ends, are difficult to detail on shallow beams as typically used on a skeletal frame structure. There are a number of dimensional limitations in design codes for the geometry of the dapped end.

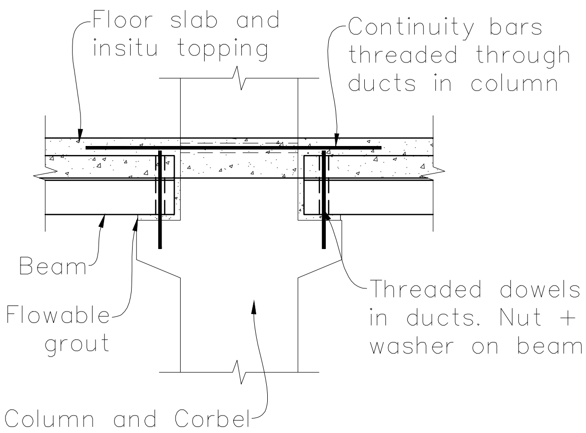


Figure 7.14 Connection type B. Concrete corbel

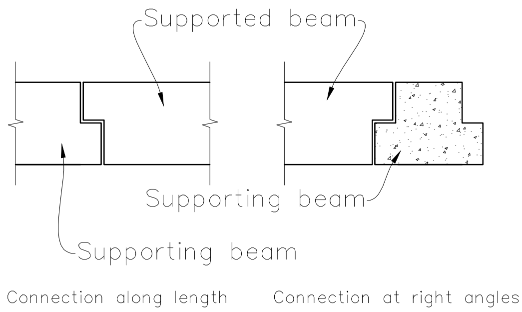


Figure 7.16 Beam to beam connections

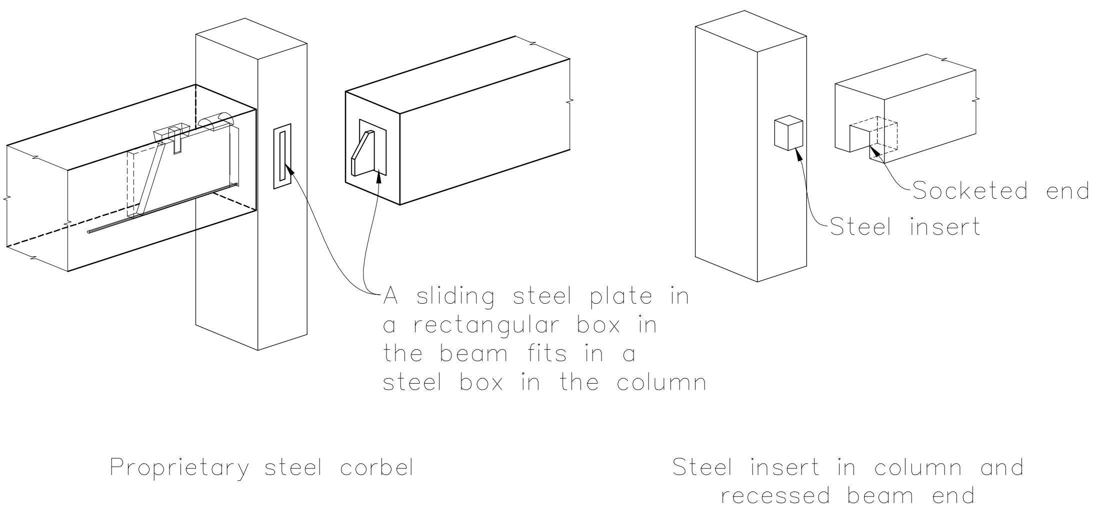


Figure 7.15 Hidden corbels

For example the PCI Design Handbook [4] requires the depth of the extending end to be not less than one half the depth of the beam. Strut and tie methods should be used to design dapped ends. The 'Precast Concrete Handbook' [2] has a detailed example of the design of a typical dapped beam.

Where beams intersect at right angles the connection design requires special attention, particularly in the supporting beam, where the combined effects of bending, shear, torsion and bearing stresses may cause problems within the shallow depth. Figure 7.17 shows the forces induced in the projecting corbel on the supporting beam.

Steel hanger brackets are the preferred option for beam to beam connections particularly with shallow beams. Although they have higher material cost, steel hanger brackets are preassembled and cause less disruption during manufacture and beams with hanger brackets are easy to install on site. Loads of several hundred kN can be supported by this type of connection.

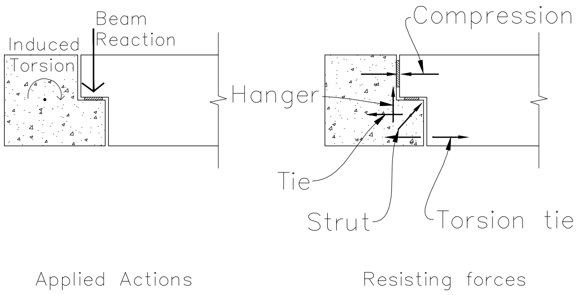


Figure 7.17 Typical intersecting beam connection

The assembled hanger bracket system comprises;

- Steel cantilever bracket(s) with anchor bars cast into the supported beam.
- Bearing bracket cast into the supporting beam ledge.
- Torsional brackets (if required) between beams.

Pairs of brackets are used with the load to each bracket being apportioned in accordance with direct and torsional loading on the beam. Brackets lengths should be least 1.5 times the beam depth. The design process is as follows;

1. Design the cantilever of the bracket to resist the applied combined bending moments and shear forces. The lever arm is taken from the centreline of the support to the centre of the anchor bars. Bracket sections are usually channel or rectangular sections.
2. Calculate the area of steel required for the anchor bars. The point of rotation is assumed to be the end of the bracket. The preferable reinforcement configuration is to provide a single bar each side of each bracket.
3. Check and design the supporting beam ledge. Bearing brackets are usually required and the ledge and the brackets and the associated hanger ligatures should be designed in accordance with strut and tie theory. The 'Precast Concrete Handbook' [2] has a detailed example of the design of a typical beam ledge.
4. Torsional effects induced in the supporting beam can be resisted by designing the beam for torsion or by providing torsion brackets in the bottom of each beam.
5. After erection the hanger bracket is usually welded to the bearing bracket and if present, torsion brackets are connected by a welded plate, usually before the beams are loaded.

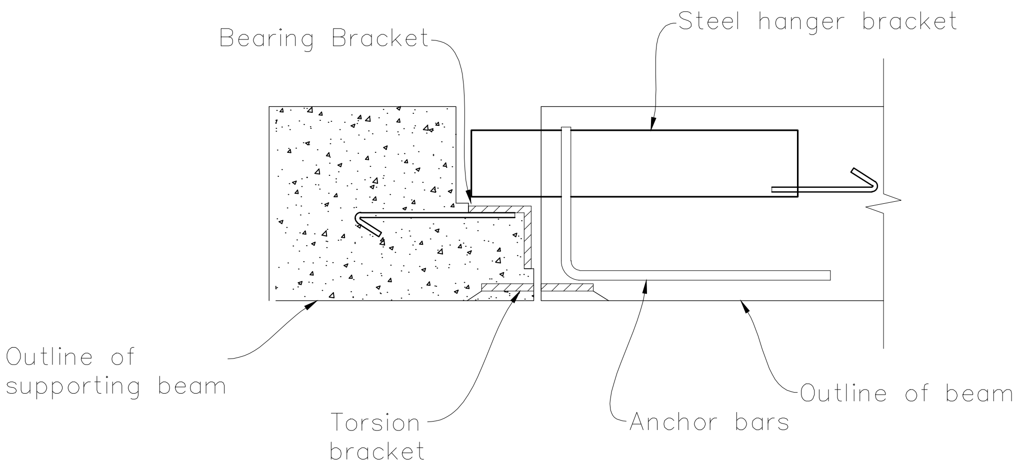


Figure 7.18 Steel hanger bracket

Figure 7.19 High capacity steel hanger brackets

7.3.5 Beam to wall connection

The type of beam to wall connection is dependent on the supporting wall. If the wall is precast concrete the options are to support the beam on a projecting nib on the wall or on a pocket in the wall. If the wall is cast insitu pockets or nibs may be difficult to provide and an alternative solution is to connect the beam to the wall by an insitu infill.

Concrete or steel corbels on the wall can also be used to support the beam but these become substantial and not economical because of the induced bending in the wall from the large beam reactions.
The nib on wall connection is effectively similar to a beam to column connection as described in Chapter 7.3.2 and the same concepts apply. To ensure adequate bearing and tolerance the nib projection should be at least 200mm. If the precast wall is multi-level the discontinuity in projecting nib above the beam needs to be such that the beam can be inserted and lowered over short dowels projecting from the lower nib. In this case the projecting nib should be the full width of the beam to prevent torsional overturning of the beam.

The pocket in wall connection is usually the simplest method of supporting beams, particularly with floor-to-floor height walls. Dowels project from the wall into ducts in the beam. Bearing on the wall needs to be checked and confinement reinforcement used if necessary. This can also dictate the wall thickness and if this is the case consideration should be given to the economics of providing a nib, or a combination of nib and pocket.

If the precast wall is multi-level the pocket clearance above the beam needs to be such that the beam can be inserted and lowered over short dowels projecting from the lower wall.

An insitu connection is frequently required where supporting walls are cast insitu and poured prior to erection of the precast. For example, where a precast skeletal structure is supported on an insitu concrete central core structure.

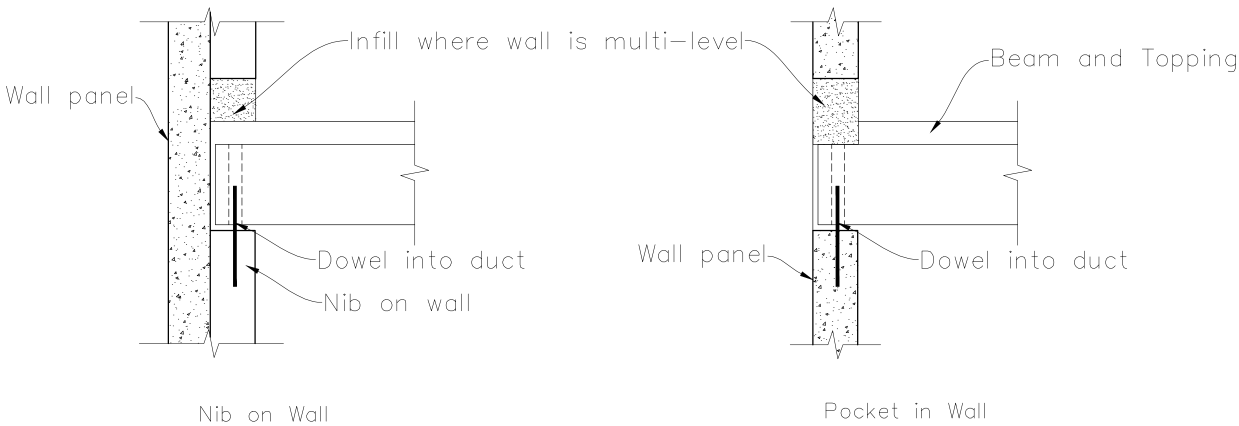


Figure 7.20 Beam to precast wall connections

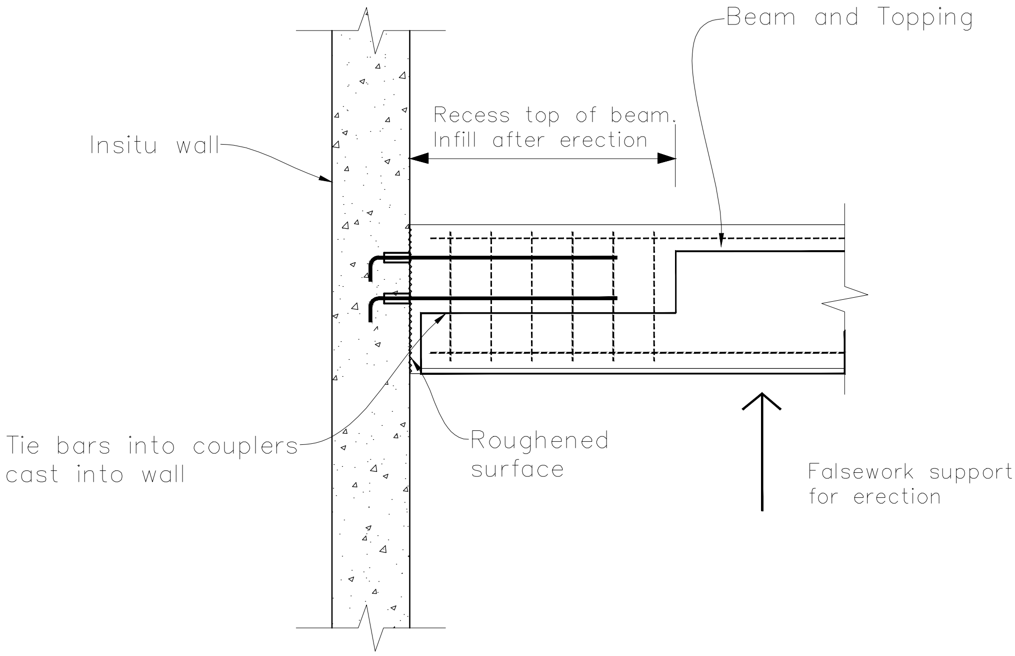


Figure 7.21 Beam connection to insitu wall

The top section of the beam is recessed for about 1 metre at the end with the bottom 'flange' containing the tensile reinforcement and ligatures continuing through to the supporting wall line as shown in figure 7.22. The beam is erected onto falsework, tie bar reinforcement is provided from the wall and into the pocket in the beam and the recess filled with insitu concrete. The design method is based on the concept of shear-friction and is covered in detail in the 'Precast Concrete Handbook'. [2] This connection effectively becomes the same as that used for connecting insitu concrete beams to previously poured insitu concrete walls.

7.3.6 Floor slab to beam connection

Precast concrete floor slabs are typically one-way spanning, and are therefore supported only at the two ends. Two support conditions occur, an internal beam and an edge beam. To provide even bearing, slabs are usually supported on neoprene bearing strips on the beam ledges.

Internal beams are typically inverted Tee or rectangular profiles with the precast floor units supported on ledges or on the top surface. Where structural screeds are provided the individual elements are tied together by

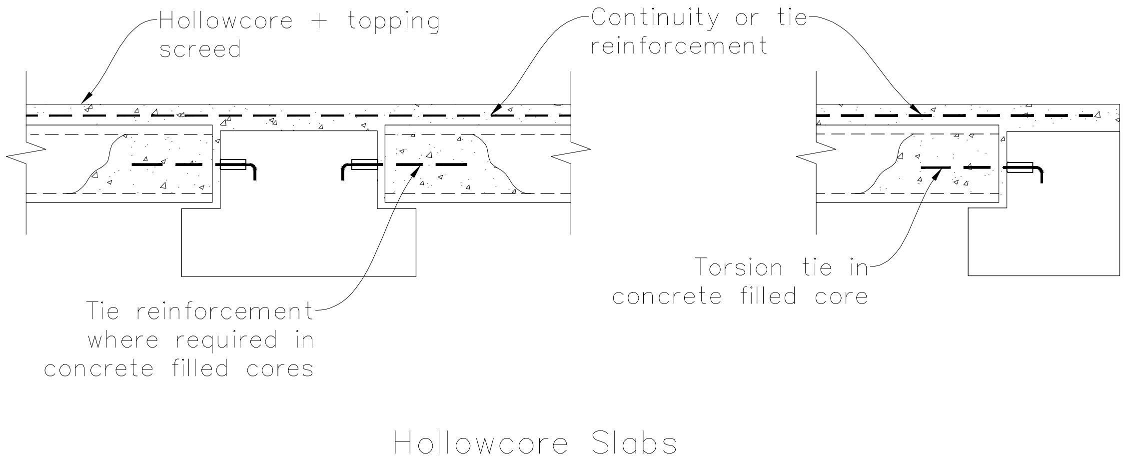


Figure 7.22 Floor slab connections to beams

reinforcement in the screed. If there is no screed, which can be the case with hollowcore slabs, tie bars are provided either through or screwed into the beam and into the cores of the slabs that are subsequently filled with concrete.

Torsion due to out of balance actions on internal beams during erection needs careful consideration. Although the beam can usually be designed to resist any induced torsion, the effect on the beam column connection is critical. This connection should either be designed to resist the torsion or the beam should be propped during erection.

For the service condition high torsion may occur with a long and short span either side of the beam. This is particularly the case with hollowcore slabs and torsion ties from the side of the beam and into concrete filled cores in slabs can be provided to resist the torsion.

Edge beams, or more precisely beams with floors supported on only one side, can be either L or rectangular and may project above floor level to form spandrels. The depth of edge beams is normally not restricted by headroom, in fact it is sometimes preferable to make the edge beams deeper in order to provide an envelope on to which cladding panels or the façade are attached. As such, edge beams depths of around 1200mm are quite common.

Torsion due to out of balance actions needs to be taken into account in the design of edge beams, particularly with long span floor slabs. Figure 7.23 shows the forces induced in an edge beam due to the applied loads from the floor slab.

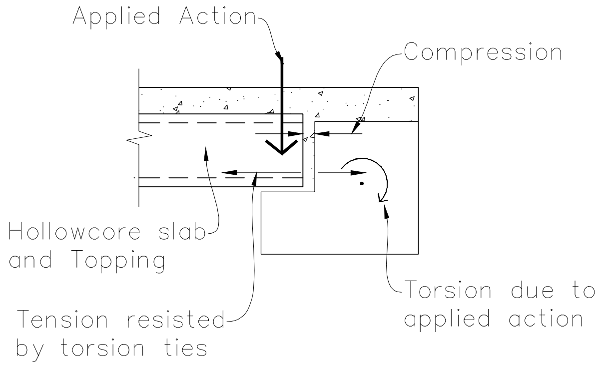
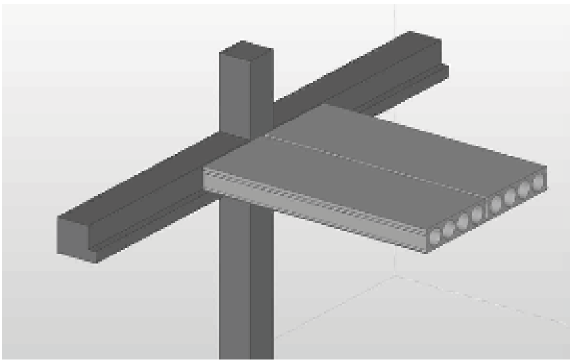


Figure 7.23 Forces inducing torsion in edge beams

For the erection case, the precast portion of the beam should be designed to resist the torsion induced by the floor slabs without any need for propping.

With hollowcore slabs it is relatively easy to resist the torsion due to the additional loads by providing tie bars into the bottom of cores that are filled with topping concrete. The applied loads used to calculate the torsion ties are those applied after the topping has been poured and cured.

Edge beams can be either the same width as the column, with the beam ledge projecting beyond the column face, or a smaller width with the ledge inside the column width. See Figure 7.24. In the first case the floor slab will pass in front of the columns. In the second case it may be necessary to cut notches in the slabs at the column and possibly provide support for erection and service loads with some form of corbel on the column.



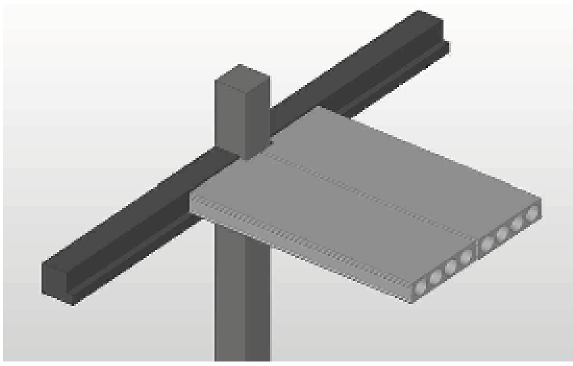


Figure 7.24 Floor slab detail at columns. (a) wider beam allows floor slab to be uninterrupted. (b) floor slab is notched at the column

7.3.7 Floor slab to wall connection

This section covers only the typical details that occur on a skeletal frame structure where walls are supported on or run parallel to shear wall core structures. In most cases the floor slab occurs only on one side of the wall. For a more detailed coverage of connections between floors and walls refer to Chapter 8, Loadbearing Wall Structures.

Broadly there are two types of floor to wall connections. The first where the floor is supported on the wall and the second is where the floor spans parallel to the wall. Because these cases usually involve tying the floor diaphragm to the shear wall structure, care is required in detailing to ensure that the ties can transfer the design forces into the precast floor. Ductility is an important requirement for this type of connection. Where very high diaphragm forces need to be transferred it may be necessary to locally thicken the insitu portion of the floor to accommodate the heavier reinforcement adjacent to the shear walls.

Figure 7.25 shows typical details where the floor slabs are supported on the wall. A common detail is to stop the wall at the underside of the floor and provide direct support for the slabs on the wall. Projecting dowel bars run up through the floor and into the upper wall. Hairpin bars are required to lap round the dowels and into the floor slab. Where the wall has sufficient thickness it is possible to support the floor on a rebate with a thin section of the wall running past the floor. This gives a tidier detail and avoids the need for formwork.

Where the wall is multi-level and runs past the floor slab, it is common to provide a concrete or steel corbel on the wall to support the floor. The wall may be precast or insitu concrete. Tie bars are required from the wall and into the insitu part of the floor slab. Where the wall has sufficient thickness the floor can be supported on a recess in the wall.

It is possible to support the floor slab without the provision of corbels by providing tie bars across the interface to transfer the forces in accordance with shear-friction theory. This detail is not recommended for hollowcore slabs unless the loads to be transferred are very small. As temporary support is required during erection and pouring the topping it is usually more economical to use one of the two previously described details.

Figure 7.26 shows the typical detail where floor slabs span parallel to the wall. Slabs are tied to the wall with bent-out bars or screwed-in starters into pockets notched in the side of the slabs. Because there is no transverse reinforcement in the hollowcore the lateral load capacity of this connection is limited unless there is a topping screed with lapping bars provided to tie the wall back to the screed. The number and length of these starters is limited by the amount that can be notched in the side of the hollowcore. Hooked bars are usually provided to lap with the starters and the topping reinforcement.

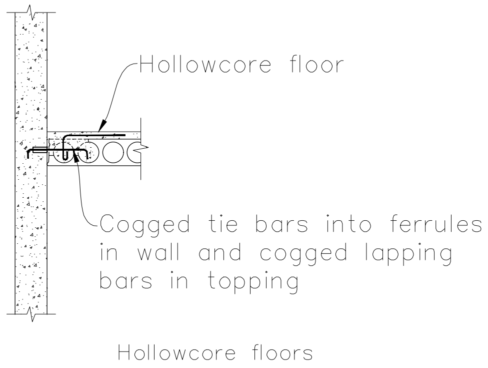


Figure 7.26 Floors spanning parallel to walls

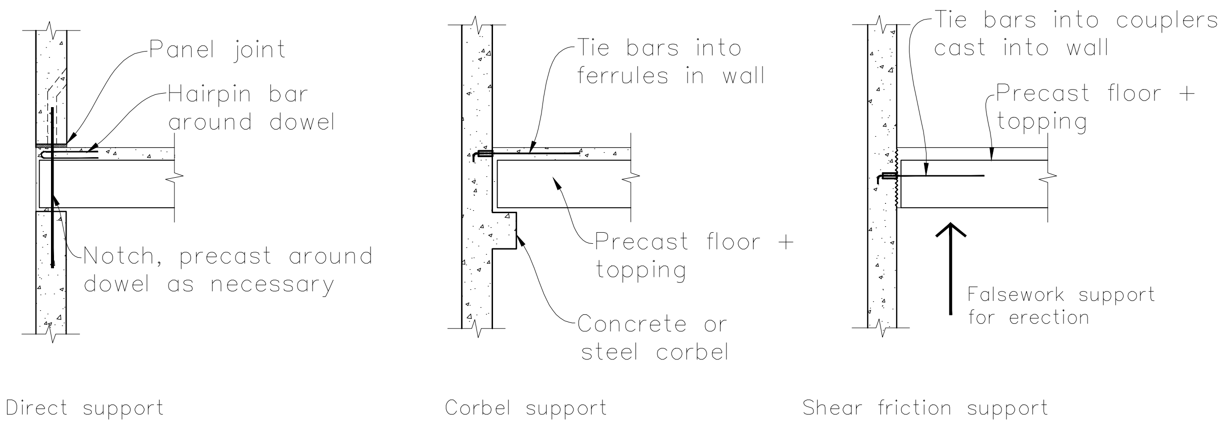


Figure 7.25 Floors supported on walls

8 LOADBEARING WALL STRUCTURES

8.1 General

Loadbearing wall structures are obviously ideal for buildings with many external and internal walls, such as apartment buildings. They are also suitable for multi-level commercial buildings where floors can clear span between loadbearing wall facades on each side. They can be used either with precast or insitu floor systems.

Although precast bearing walls commonly occur in combination with skeletal frame structures or in mixed construction this chapter will focus on their use in bearing wall structures. Similarly the use of bearing walls in cellular structures will not be discussed as the concepts are similar to those where individual wall panels are used.

In this Chapter

8.2 Design concepts

Any building that by its nature requires many internal or external walls is well suited to be constructed as a loadbearing wall structure. The most economical structure is the result of optimising the cost of the two main components, the walls and the floors. This usually results in minimising the number of vertical components and maximising the spans of the floor system. This will also depend on the type and occupancy of the building being considered, for example an apartment building with many walls or a commercial building requiring a minimum of walls.

8.2.1 Apartment buildings

Walls on apartment buildings can be floor-to-floor or multi-level, with the decision often dictated by the floor system. With precast floors the internal walls usually run floor-to-floor with the floor supported on top of the wall. External walls can be floor-to-floor or multi-level and usually run past the edge of the floor. Buildings with insitu concrete floors frequently have multi-level walls.

Apartment buildings can roughly be divided into two categories: Integral wall systems and envelope wall systems

- Integral wall systems are where all internal and external walls are in precast concrete. Some walls are load bearing, others perform only a separating function. The façades are also loadbearing and the floors can be precast or insitu concrete. This was a common form of construction in the early years of precast concrete as it mimicked traditional construction of loadbearing masonry.

- Envelope wall systems are where only the external or separating walls between the apartments are in precast concrete and the internal walls are in a lightweight partition wall system. The aim is to create large free spaces inside the apartment with a minimum of vertical loadbearing walls. As well as providing more flexibility for the internal layout of the floor plan it also allows the possibility of easy modification in the future.

For envelope wall systems the loadbearing wall configuration depends on the building layout. For a typical rectangular building the cross walls between apartments and possibly the corridor walls can be precast with a precast floor system spanning along the building and across the width of the apartment as shown in figure 8.1. and figure ??? The façade walls can be either precast or lightweight.

This configuration is appropriate for floor spans of up to about 16 metres between apartment walls. Where this span is exceeded and the total width of the building does not exceed about 16 metres the precast floors can span across the width of the building from façade to façade as shown in figure 8.2. Other alternatives are to span the floor across the width of the building from the corridor wall to an external loadbearing façade or to provide an internal loadbearing wall within the apartment.

There are also numerous variations with precast floors to accommodate cantilevered balconies and floor set-downs. These are described in Chapter 4.

With relatively large numbers of walls compared with the floor area the lateral stability of apartment buildings is usually not critical. In most cases the wall panels can be erected one above the other with only nominal tension and shear capacity across the horizontal joints and with no shear connections in the vertical joints. Where lateral stability is critical, individual wall panels can be connected to form larger shear walls as described in Chapter 5.

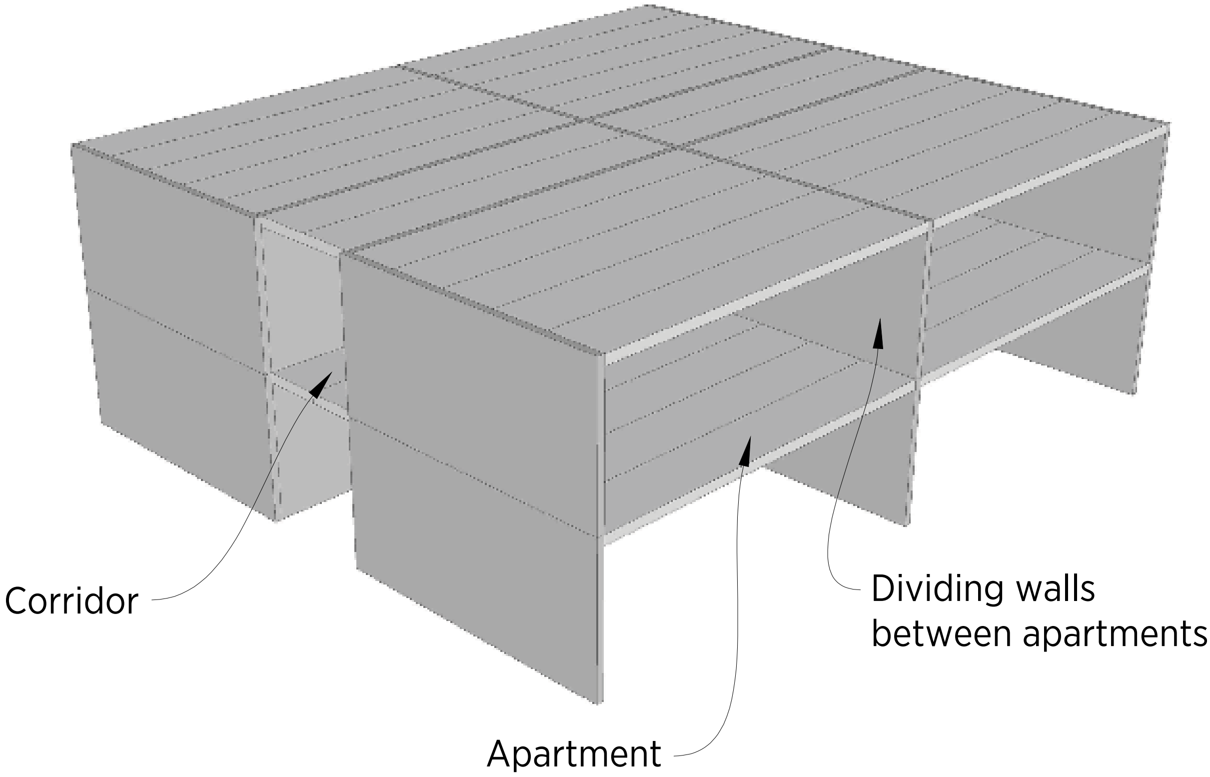


Figure 8.1 Outline of a building with load bearing cross-walls

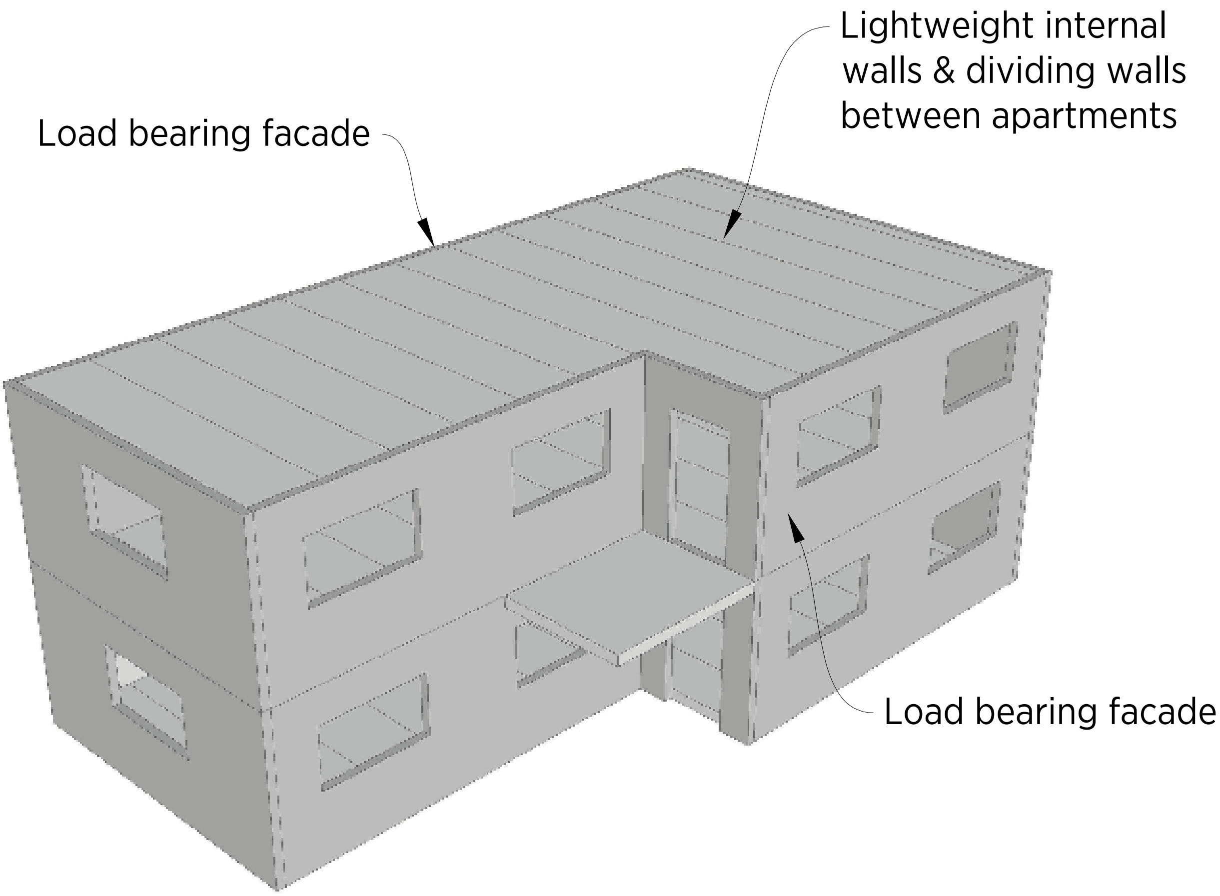


Figure 8.2 Outline of building with clear spans between facades

8.2.2 Commercial buildings

The use of loadbearing walls in commercial buildings can roughly be divided into two categories: Lift and stair shafts or external walls.

- Lift and stair shafts are an ideal use for precast concrete, particularly for low to medium rise buildings where overturning is not a major factor. Typically panels are connected after erection to form composite T, L, U or box-shaped sections. Tension and shear forces across horizontal joints are resisted by grouted dowel connections and vertical shear between panels by welded connections. Panels can be floor-to-floor or multi-level depending on the floor system be used.

As described in Chapter 5.2.2 staggering the vertical joints in a 'brickwork' pattern or overlapping the corner joints in alternate directions can provide significant shear transfer between panels and reduce, or eliminate, the need for vertical shear connections. Similarly staggering the height of the horizontal joints can minimise the requirement for temporary bracing for erection.

- External walls on a commercial building that have minimum fenestration such as a boundary wall or 'punched-hole' façades are ideally suited to loadbearing precast concrete. In these cases loadbearing walls eliminate the need for a separate support structure for the floor by making use of the inherent strength of the wall.

Floor-to-floor heights of commercial buildings are often greater than the maximum dimension for transporting panels so there is a tendency to favour the use of narrower multi-level panels. It is also common to locate the horizontal joint above floor level so that the wall acts as a safety barrier during construction.

Figure 8.3 Staggered panel joints

Figure 8.4 Commercial building with multi-level loadbearing walls

8.3 Connection Details

Examples of typical connections in loadbearing wall structures are given in this section. The intention is not to show a complete overview of all solutions, but to make the designer familiar with common types of connections. The principles applied in the majority of solutions are valid for all types of loadbearing wall structures and both for low-rise and multi-storey buildings.

Connections are classified with respect to location, direction and function, e.g. interior or external, horizontal or vertical, wall to footing, wall-to-wall or wall-to-floor. Within a typical loadbearing wall structure the main connection types as follows.

1. Wall to footing.
2. Floor to internal wall.
3. Floor to external wall.
4. Floor parallel to wall
5. Wall to wall.

Types 2, 3 and 4 are further sub-divided by use with either a precast or an insitu floor. Type four is further sub-divided into vertical and horizontal joints.

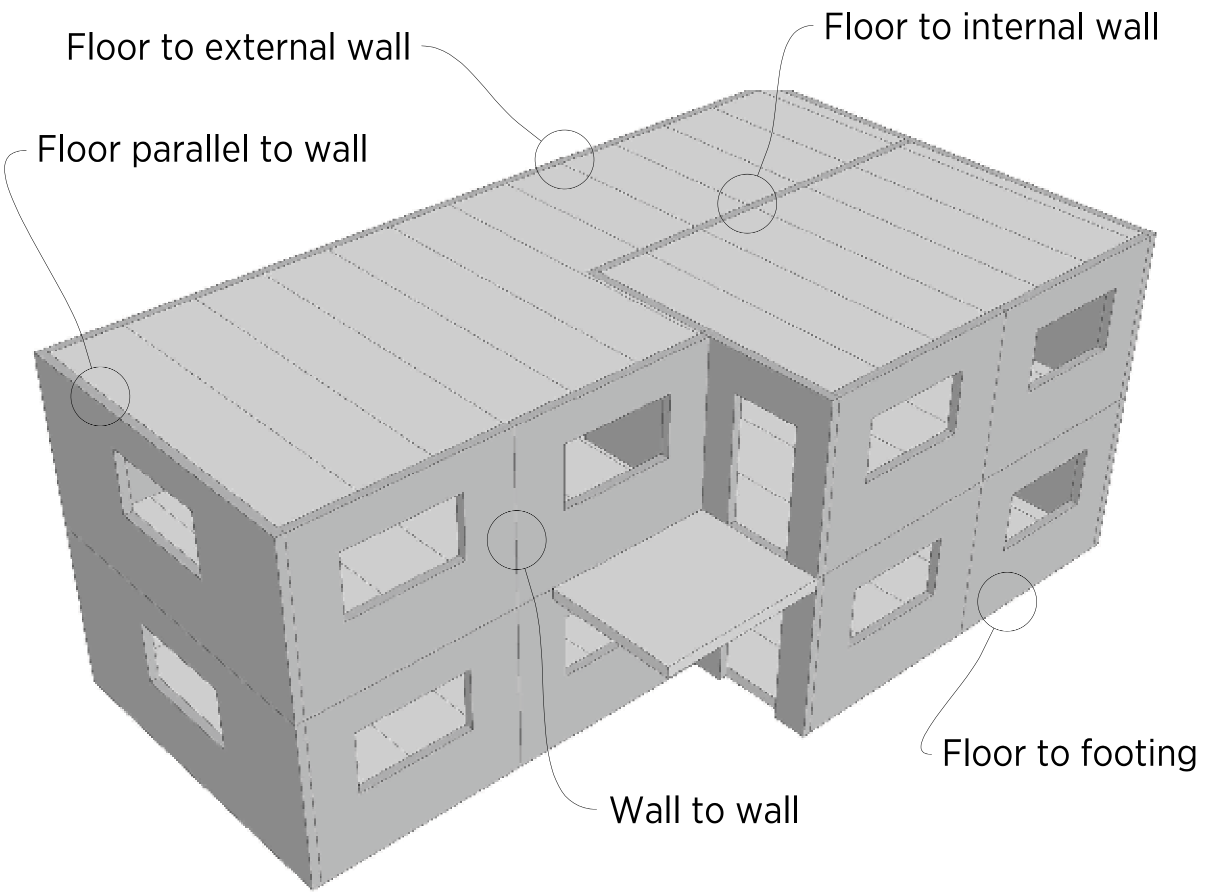


Figure 8.5 Connection locations loadbearing wall structure

8.3.1 Wall to footing connection

The most common wall to footing detail is a grouted dowel as described in Chapter 6.3.1. The number and size of dowels will be dependent on the axial forces and the presence or not of tensile or shear forces across the joint. Where compressive forces are high, confinement reinforcement may be required in the base of the wall panel.

8.3.2 Floor to internal wall connection

There are a number of variations of this detail that depend on the type of floor and the forces to be transferred from the upper to the lower wall.

Figure 8.7 shows connection details that are typically used between precast walls and precast floors.

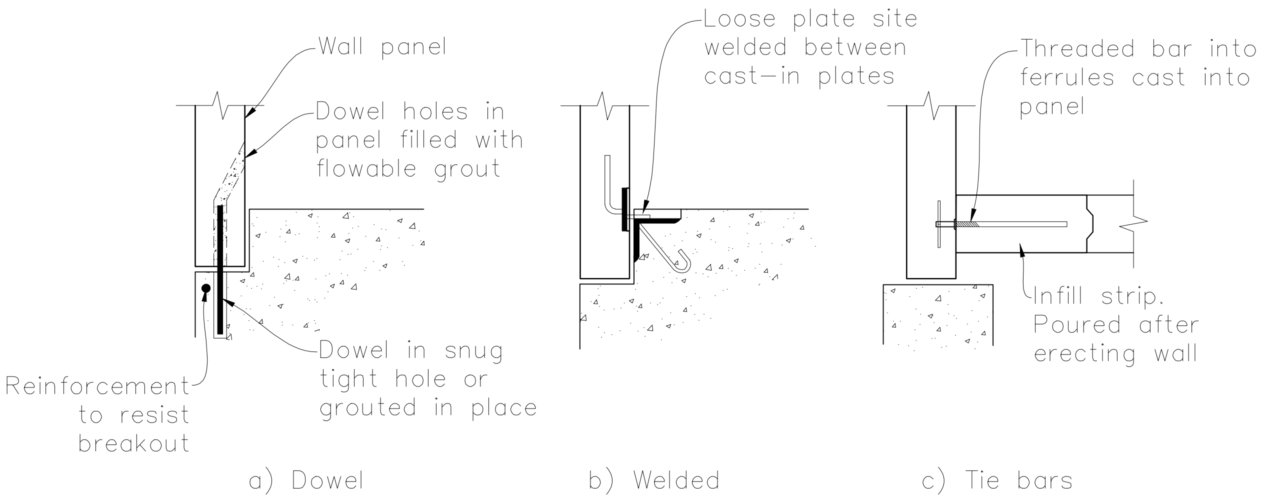


Figure 8.6 Wall to footing connections

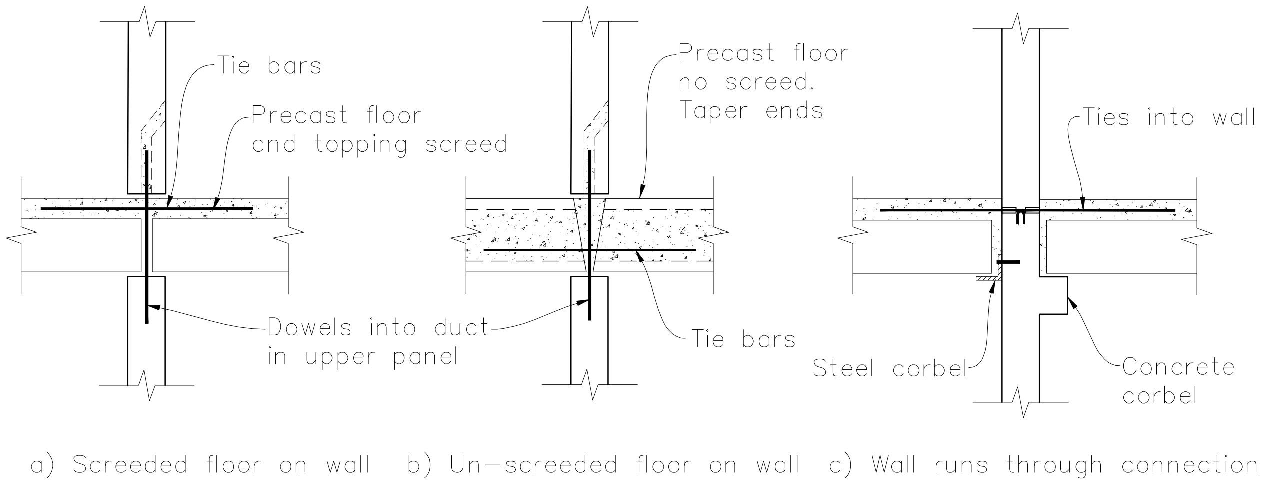


Figure 8.7 Precast floor to internal wall connection

Figure 8.7 a) shows the classic connection detail of a precast floor supported on the lower wall. Grouted dowels run from wall to wall through the floor and tie bars within the floor or structural topping tie the elements together. Consideration needs to be given to the clamping force due to axial forces from the upper wall and the resulting negative bending moment and reduced shear capacity that can be induced in the end of the precast floor elements. Where a structural screed is provided the reinforcement in the screed is usually sufficient to resist this bending moment. For un-screeded floor systems the ends of the precast floor elements can be tapered as shown in Figure 8.7 b) to ensure a true pinned support condition.

Bearing length of the floor elements and the effects of tolerances also need to be taken into account and this may dictate a minimum wall thickness. In multi-storey loadbearing wall structures, the ends of the slabs together with the jointing concrete or mortar transfer the forces from the upper wall element to the lower one.

Calculation of the load capacity through horizontal joints is very complex. Quality of the grouting is important and bearing is often non-uniform or eccentric and can result in high splitting forces in the panels even when the average stress is small. Even where bearing is uniform a reduction factor of 0.5 is often applied to the calculated vertical load capacity through the joint.

For horizontal joints with high loads and complex geometry it is recommended that a detailed finite element analysis be carried out and if necessary longitudinal and transverse reinforcement should be provided in the walls to resist cracking.

The National Precast Concrete Association Australia have produced a grouting guide for loadbearing joints that covers this topic in some detail.

Where loads from the upper walls are very high it may be necessary to eliminate the sandwich effect of the floor and provided direct bearing wall to wall as shown in Figure 8.7 c). The supporting corbels can be either steel or concrete as described below in Chapter 8.3.3. Concrete corbels each side of a wall panel is not a practical solution and should be avoided wherever possible. Where steel corbels are used, the FRL of the connection for moderate loading conditions can be achieved by providing dowels that project from the wall and into the precast floor system. Such dowels are designed in accordance with shear friction theory to carry the reduced fire load eliminating the need to fire protect the steel angle.

Information on the detailed design of wall connections and extensive literature is available on the subject.[4][27]

8.3.3 Floor to external wall connection

As with the internal wall connections there are a large number of variations of this detail that depend on the type of floor, the loads to be transferred from the upper to the lower wall and the presence or not of a horizontal wall panel joint. In all cases the eccentricity of load from the floor into the wall needs to be taken into account in the design. Deflection and rotation of the floor can also increase the bending moment in the wall. This is described in detail in Chapter 4.

Figure 8.8a) shows the connection detail of a precast floor supported by a concrete
corbel. The corbel is designed in accordance with strut and tie theory and eccentricity of load needs to be taken into account in the design of the wall. With this detail it is easy to produce a ductile connection. This corbel detail is probably the simplest and most economical of these support details but it is not favoured by many precast wall manufacturers because of the two stage pouring process and complexity of storing and transporting the element with a projecting corbel.

Figure 8.8 b) shows a similar connection detail of a precast floor supported by a steel corbel. The steel corbel can be either cast into the panel during manufacture or bolted or welded to the panel before or after erection. Casting in during manufacture is the most economical option as post fixing, particularly on site after the panel is erected is a time consuming and expensive process. Because of their lack of ductility, fixing with drilled-in expansion anchors is not recommended for such major and key structural connections. The steel corbel should be designed in accordance with the Steel Structures Code, AS 4100. [28] Due allowance should be made in the design for eccentricity of load from the floor. As noted for internal wall connections the FRL can be achieved without the need to fire protect the steel angle by using shear friction design methods.

In both cases tie bars from the wall into the precast elements or floor topping tie the elements together. If a horizontal wall panel joint occurs at this location it should be positioned at or above floor level. With a stepped joint for waterproofing, care should be taken to ensure that the wall panel has sufficient thickness to accommodate and provide adequate cover for all the fixings and edge reinforcement.

Where structural requirements dictate a thick wall, the top section of the wall can be rebated as shown in figure 8.8 c) to provide support for the floor elements. With minimum dimensions for the up-stand and floor bearing this detail requires a minimum wall thickness of about 200mm.

Bars projecting from the lower panel are bent down into the precast floor elements or screed and grouted dowels running from the lower panel past the floor and into the upper panel tie all elements together. The bent down bars can be eliminated if sufficient dowels are provided and each is confined by a hairpin bar anchored into the precast floor or screed.

As shown in Chapter 7.3.6 it is possible to support the floor slab without the provision of corbels by providing tie bars across the interface to transfer the forces in accordance with shear-friction theory. This detail is not recommended for hollowcore slabs unless the loads to be transferred are very small. As temporary support is required during erection and pouring the topping it is usually more economical to use one of the previously described details.

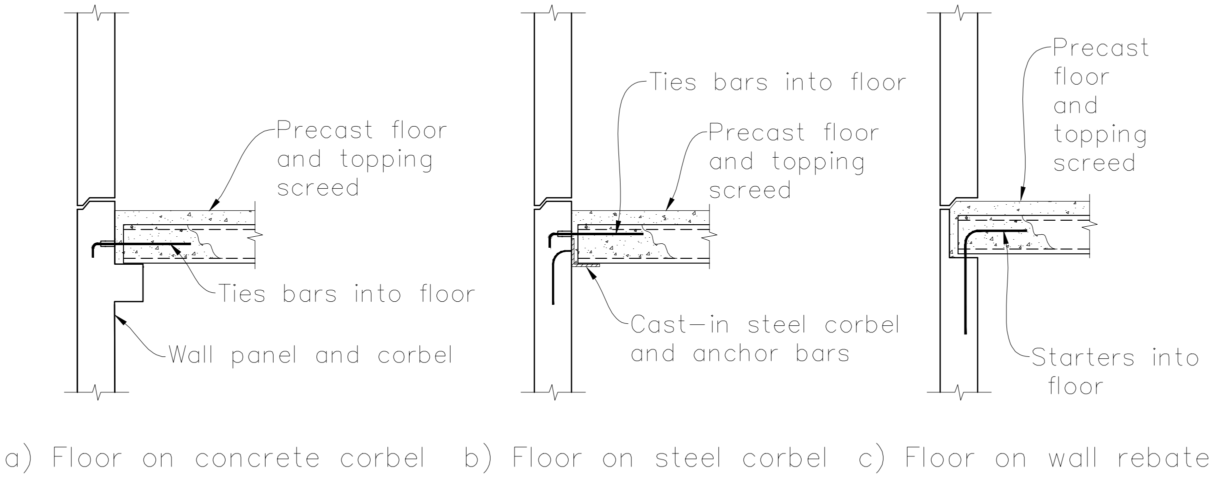


Figure 8.8 Precast floor to external wall connection

8.3.4 Floor connection parallel to wall

Where the floor system spans parallel to the wall the two elements should be connected together to eliminate any differential movement. Although the wall may not be intended to be loadbearing the fact that it is connected to the floor means that some load will be transferred. The magnitude will depend on the stiffness of the floor and the extent of axial shortening and shrinkage. For one-way prestressed floor elements it is usual to assume that the load from a one metre band of floor each side is carried by the wall. An alternative and more accurate load distribution assumes that only actions applied after the two elements are connected are distributed to the wall by two way action. Floors such as hollowcore slabs with high torsional stiffness will distribute higher loads than slabs that tend to have relatively low torsional stiffness.

Figure 8.9 a) shows a hollowcore slab connected to the side of an external precast wall. Threaded bars are screwed into ferrules cast into the wall and project into slots cut in the side of the hollowcore slab. Because there is no transverse reinforcement in the hollowcore the lateral load capacity of this connection is limited unless there is a structural screed and lapping bars can be provided to tie the wall back to the screed.

Figure 8.9 b) shows the detail where there is hollowcore on each side of an internal wall. The lower wall projects above the soffit of the floor. Dowels or starter bars project from the lower to the upper wall and the elements are tied together by and insitu infill.

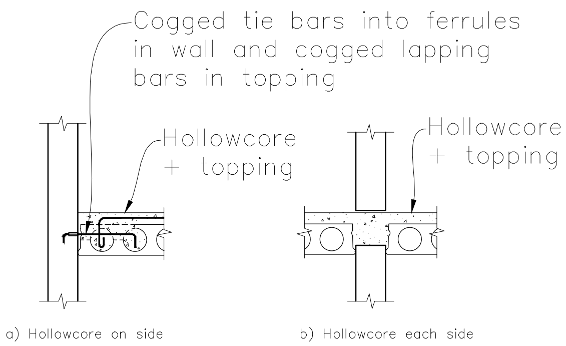


Figure 8.9 Floor connections parallel to wall

8.3.5 Wall to wall connections

There are two basic wall to wall connections. They occur at vertical joints and horizontal joints. The forces to be transferred by the connections will depend on the structural application of the wall but can include in-plane shear, out-of-plane shear, tension and compression. Ductile behaviour is a requirement of all connections and with relatively thin wall sections this is often difficult to obtain. Careful detailing of anchor bars and confinement reinforcement is required to prevent pull-out of the connection fixings. Where significant forces are to be transferred by the connection an insitu infill joint to emulate monolithic construction may be necessary.

The National Precast Concrete Association Australia have produced a grouting guide for loadbearing joints that covers this topic in some detail. This document, 'Understanding Grouted Precast Joints, a guide for Engineers and Contractors', [43] highlights the need for careful design and detailing of the load transfer mechanism through grouted horizontal joints. The importance of establishing a work process that ensures the joints are grouted in accordance with the design requirements cannot be overemphasised. Structural failures have occurred due to inadequate grouting and this process should not be left to inexperienced or unskilled tradespeople.
The use of stepped joints where high vertical loads occur should be given careful consideration as partial grouting the width of a joint introduces significant splitting forces that can limit the load bearing capacity of the joint. With thin wall sections it is often impossible to provide reinforcement to resist these splitting forces.

Figures 8.10 a) and b) show typical horizontal connections. Figure 8.10 a) is for an external wall where a waterproof joint is required and figure 8.10 b) is for a typical internal joint. In both cases the connection is formed by dowels in the top of the lower panel projecting into ducts in the upper panel. Dowels can be cast-in, screwed into ferrules or inserted into ducts in the lower panel. The ducts should be corrugated and be of sufficient diameter to allow filling with a flowable grout. The length and capacity of the dowels is dependent on the tension forces to be transferred but in all cases there is a minimum recommended as set out in Chapter 5.6. In some cases the most economical solution is to have ducts running the full height of the panels.

With a stepped joint for waterproofing, care should be taken to ensure that the wall panel has sufficient thickness to accommodate and provide adequate cover for all the fixings and edge reinforcement. Where out-of-plane forces occur confinement reinforcement should be provided to prevent side breakout of the dowels.

Tension forces can also be transferred by a bolted connection recessed in the base of the upper panel as shown in Figure 8.10 c) although this connection can be difficult to use where a waterproof joint is required.

Vertical post-tensioning with strand or bar is another option that can used to resist high tension forces that occur in lift or stair shafts. Vertical ducts are provided in the wall panels from base to top and strand or bar is threaded down and anchored into the footings. The strand or bar is tensioned after erection and grouting of the complete wall.

The horizontal joint between lower and upper panels needs to be filled to ensure axial forces can be transferred. This is usually accomplished by sealing each side of the joint and filling it with a flowable grout or by dry packing with mortar. The strength of the grout or mortar should be sufficient to carry the load. Where a waterproof joint is required the backing rod and sealer will reduce the load bearing area. Filling only part of the joint width can result in transverse splitting stresses being induced into the walls.

As noted in Clause 8.3.2 horizontal joints with high loads and complex geometry may require a detailed finite element analysis be carried out and if necessary longitudinal and transverse reinforcement be provided in the walls to resist cracking.

Connections at the vertical joints between wall units are normally designed to transmit in-plane shear forces. Figure 8.11 shows the three typical variations that can occur. In all cases the connection is formed by cast-in fixings and stitch plates. The fixings may or may not be recessed for protection or fire rating.

The basic types of fixings used are;

- Steel plates complete with anchor bars cast into the edge of each panel as shown in Figure 8.11. After erection a stitching plate is welded across the joint to form the connection.
- Threaded inserts with appropriate anchor bars are cast into each panel and a loose plate is bolted across the joint. Bolt holes need to be oversized to accommodate tolerance and this may require the plate to be welded to complete the connection.
- A variation of the above fixings is to bolt the stitching plate to the cast-in plates as a temporary fixing for erection and then weld the plate at a later date to complete the connection.

Because of the cost of the plates and the need to site weld at height these are expensive connections and should be only used where needed for structural reasons. Each additional plate in a wall panel can add several dollars per square metre to the cost of the panel.

The use of post installed expansion or chemical anchors is not recommended for this type of structural connection due to the difficulty in providing ductility.

There are a number of proprietary panel connection systems available that allow the installation of precast concrete panels on site without the need for welding or bolting.

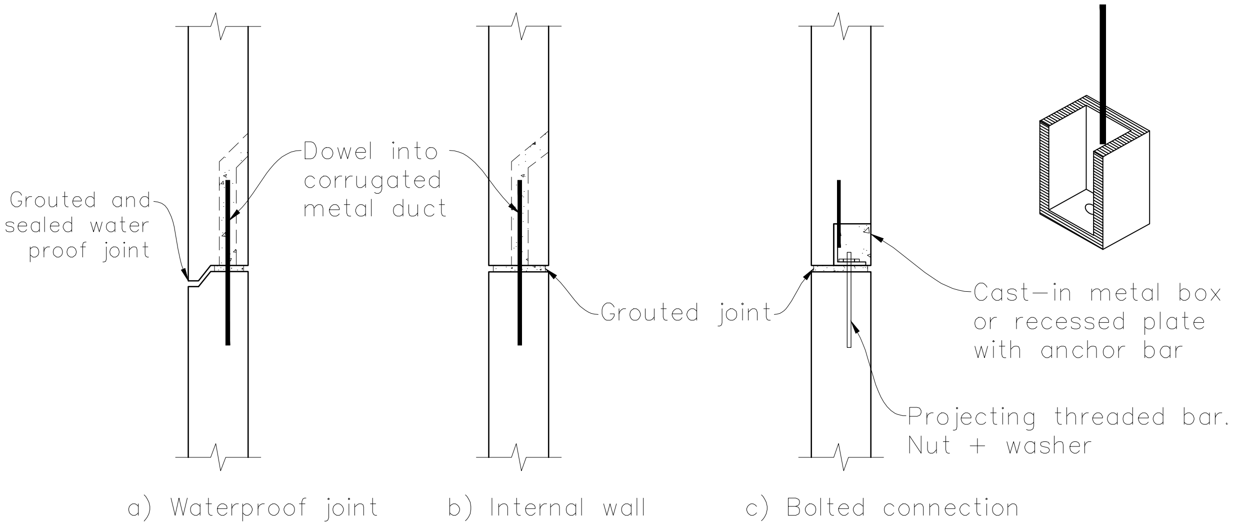


Figure 8.10 Horizontal wall to wall connections

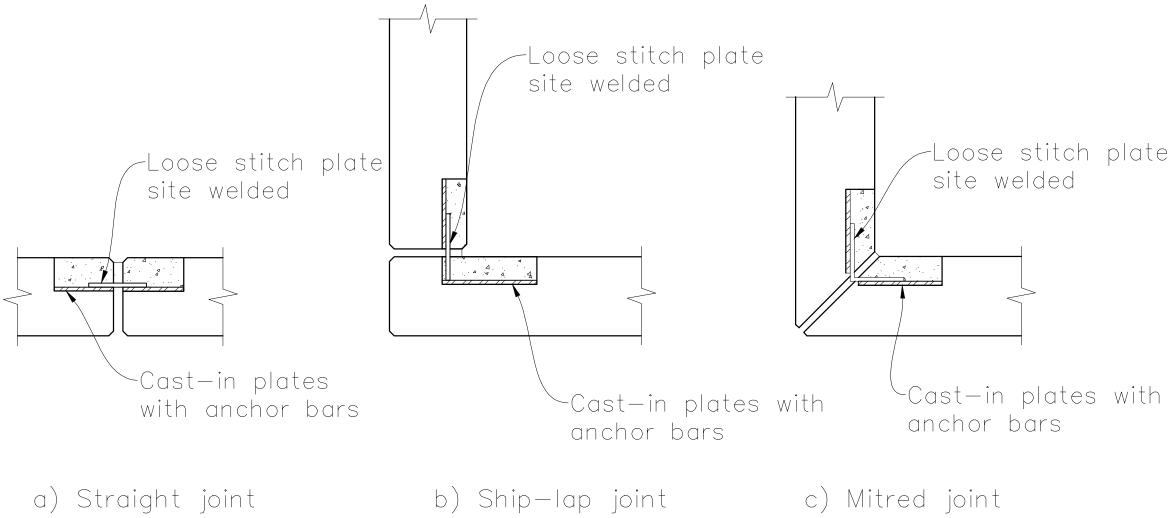


Figure 8.11 Vertical wall to wall connections

Figure 8.12 shows a detail for a ship-lap joint that is suited to connecting individual walls to form lift or stair shafts. The advantages are high load capacity, ease of providing ductility and ability to work on the connection from floor level outside the shaft.

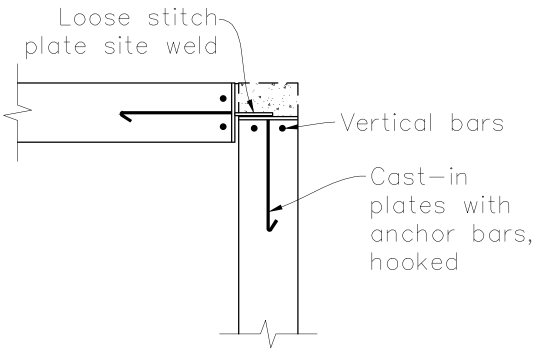


Figure 8.12 Vertical Wall to Wall External Fixing

Figure 8.13 shows variations of an insitu concrete infill connection that is intended to emulate monolithic construction.

The vertical joint faces are usually indented or intentionally roughened to increase the shear capacity of the joints. The size or width of the infill needs to be sufficient to allow it to be filled with a flowable grout or concrete. Projecting hairpin ties over the height of the panels are held together by a continuous vertical bar. The shear capacity of the connection can be calculated using shear-friction theory.

A range of proprietary components are available to form these type of connections that include the recess former as well as the starter bars.

Shear and in-plane forces can also be transferred across the vertical joint by staggering the ends of the panel joint as shown in Figures 5.8 and 8.3.

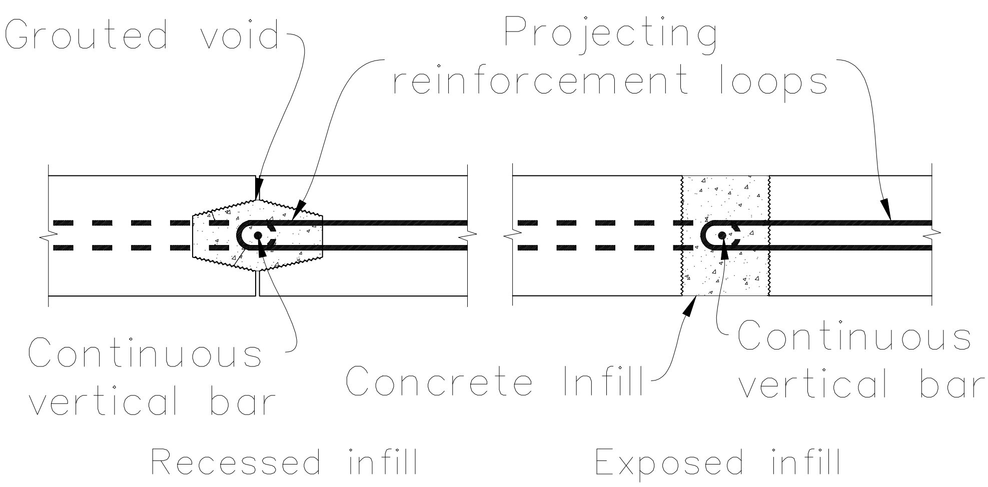


Figure 8.13 Insitu concrete infill connection

8.4 Design of welded plate connections

8.4.1 Introduction

Welded connections between walls are common within the precast industry and every consulting engineer and precast manufacturer seems to have their own favourite. In many cases details have been used for many years and repeated from project to project without being subject to a rational design method to verify capacity.

This section will set out a design philosophy that gives a rational design method for welded connections. Given the critical nature of these connections the approach is based on a conservative lower-bound design case.

An alternative is to carry out a finite element analysis, but given that the material cost is only a small part of the cost of weld plate connections this is rarely justified.

8.4.2 Design concepts

All connections should be robust so that they have the ability to withstand accidental events such as fire, explosions, impact and consequences of human error without causing damage to an extent disproportionate to the original cause.

Robustness infers that the connection is stable, has reserve strength and stiffness, exhibits ductility and has redundant capacity. These requirements are often difficult to provide in connections in relatively thin concrete sections. Concrete failure is usually the critical and limiting factor in establishing the capacity of the connection. With a concentration of forces in such a small area it is difficult to provide sufficient reinforcement that can anchor and prevent localised splitting or spalling of the concrete around the fixings. For example additional reinforcement other than a long anchor bar has little influence on the concrete cone failure of a cast-in ferrule.

Given the above, the design concept is to provide a connection that has sufficient capacity to ensure that localised concrete failure does not occur under design actions. In addition the design should ensure that under overload the anchor bars on the fixings have sufficient capacity and ductility to sustain the forces without collapse.

8.4.3 Design method

Figure 8.14 shows a diagram of a typical welded plate connection between two panels that are subject to vertical shear. The fixing plates are provided with anchor bars and a headed stud on the rear. The anchor bars need to be of sufficient length to develop full capacity and can be either perpendicular to the joint or splayed as shown. The fixing plates are recessed to ensure the anchor bars occur within the middle third of the panel thickness. A welded stitch plate forms the connection between the two fixing plates.

Vertical shear between panels produces direct shear as well as a rotational moment in the connection that is resisted by the anchor bars acting in tension and compression. The vertical shear is assumed to act at the face of the wall joint and the design forces in the anchor bars are calculated on the assumption that each plate can rotate about a point where the anchor bars are fixed to the plate.

Tension or transverse forces that occur across the joint are additive to the above. Splaying the anchor bars provides a more efficient connection than perpendicular bars, particularly with high shear forces, and use of a headed stud is important where transverse forces can occur. Stresses in the anchor bars should be limited to ensure that concrete cracking under service loads is minimised. The Codes give little guidance on this but limiting the calculated steel stress in the anchor bars to 0.6fsy should be considered.

The stitch plate and associated welding should be designed to resist the same forces as those calculated above.As required by the Building Code of Australia [10] these connections also need to be detailed to accommodate thermal movement and possible collapse of the roof structure during a fire.

Cladding panels can act as shear walls by positively connecting each panel to the steel frame or by providing grouted dowel connections in the horizontal panel joints so that a series of individual panels act as a single panel.

The remaining types of cladding panel connections, vertical panel to eaves tie, panel to panel and corner panel connection are all similar to those used for loadbearing walls as described in Chapter 8.

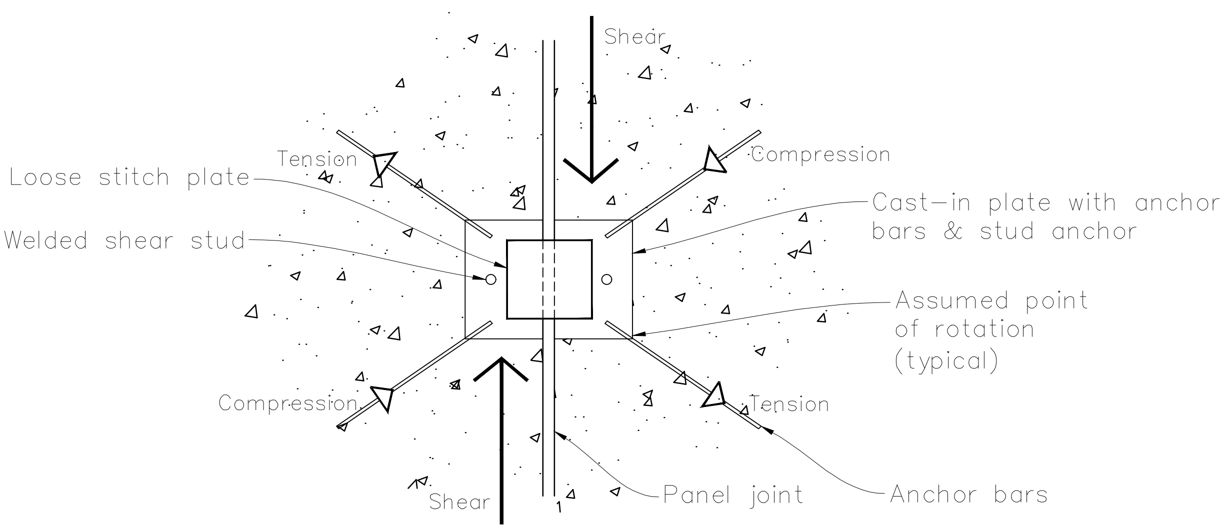


Figure 8.14 Forces in welded plate connection

9 MIXED STRUCTURES

9.1 General

As noted in Chapter 3 the term mixed construction is used to describe a type of construction where precast concrete is used in combination with other building materials, such as cast in-situ concrete, steel, masonry or timber. Common examples are; steel frames supporting precast floors, masonry walls supporting precast floors, precast walls or beams supporting metal deck floors, precast walls supporting insitu floors and insitu lift or stair shafts combined with precast skeletal frame systems.

Most precast buildings will contain some form of mixed construction and frequently the most economical structural solution involves mixing precast concrete with other structural materials for at least some part of the building.

This chapter will look at the various options available using a mixture of structural materials. Detailed information on mixed construction is given in fib Bulletin 19 “Precast concrete in mixed construction” [9].

In this Chapter

9.2 Design concepts

The optimum structural design of any building is based on selecting the most appropriate type of structure for each major part of the building as distinct from allowing a single part to dictate the complete structure. Within this selection process the optimum structural material should also be selected for each component.

The following sections will give an overview of the use of precast concrete with other materials such as steel, insitu concrete and masonry. Many of these concepts have already been referred to in previous chapters.

9.2.1 Steel and precast

Uses of structural steel and precast include steel beams supporting precast floor elements, steel permanent formwork combined with precast beams and steel elements used to support precast concrete.

Figure 9.1 shows steel beams supporting precast concrete floor elements. The beam may be an individual steel beam or part of a steel frame structure. The floor elements can be beam and infill, composite floor plates or hollowcore. In all cases it is usual, and structurally more efficient, to detail the two elements so that they act compositely.

Setting the precast elements below the top of the steel beam can make erection difficult and does not suit all types for precast floors.

Composite action with the steel beam is facilitated by the provision of stud anchors on the beam projecting into an insitu infill between the ends of the precast elements. The design process is similar to that for an insitu concrete floor except for staged loading of the beam by the precast floor before composite action is taken into account. Where hollowcore floor elements are used, as shown in figure 9.2, the flange width and/or effective slab depth are modified to account for the voids in the hollowcore slabs.

The effect of the bottom flange of the hollowcore is ignored and the width of the infill above the steel beam can be varied by filling all cores with concrete for a distance each side of the beam. The thickness of the concrete flange is the sum of the concrete thickness above the core plus the screed thickness. Because the flange is relatively thin the effective flange width should be limited.

Detailed guidelines are given in the fib 'Guide to good practice, composite floor structures.' [14]

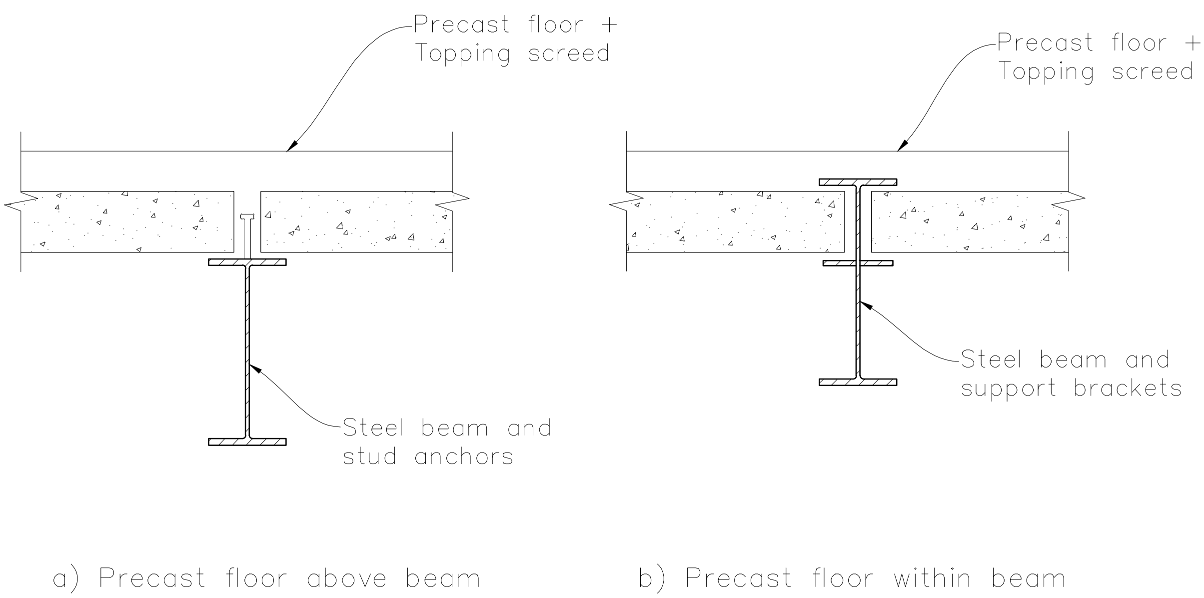


Figure 9.1 Steel beam and precast floors

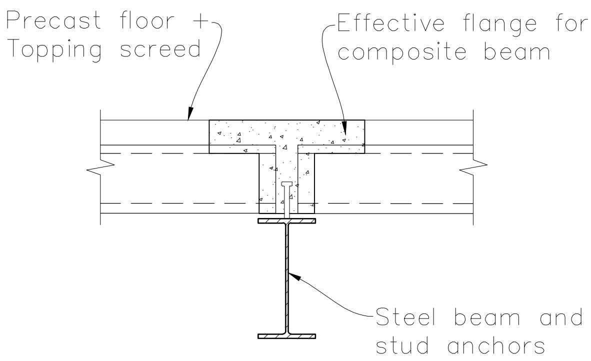


Figure 9.2 Steel beam and hollowcore floor

Figure 9.3 Composite steel beams and hollowcore floor

The design method for edge beams is similar to the above methods but ties are required from the steel beam into the precast or screed to prevent horizontal movement of the beam.

In all cases, particularly edge beams, the stability of the steel beam during erection and pouring the screed can be critical and some type of flange restraint or temporary propping or bracing may be required to prevent buckling or rotation. Friction between the steel beam and precast elements should not be relied upon to provide restraint.

Where a shallow floor profile is required, a steel beam that supports the precast floor elements on its bottom flange can be used as shown in Figure 9.4. The main use in Australia has been as trimmer beams at openings within floor slabs where height considerations preclude the use of a down-turned supporting ledge.

The steel beams can be fabricated sections with a profile to suit the particular use. PFC's with a flange added are common but they can also be UC or RHS sections with flanges added. They can be detailed as either composite or non-composite.

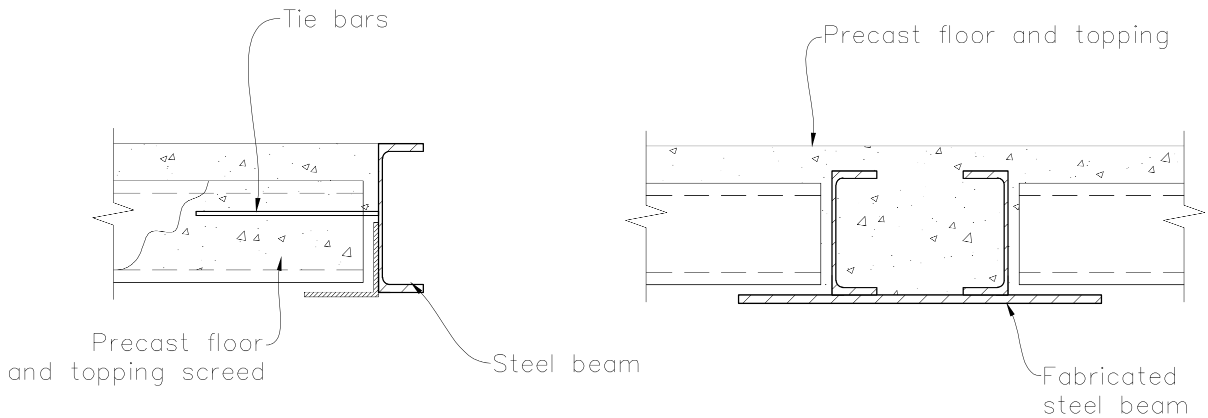


Figure 9.4 Steel trimmer beams and precast floor

Use of these types of beams with hollowcore floor elements can result in reduced shear capacity of the hollowcore due to the fact that deflection of the supporting beams induces transverse stresses across the webs of the hollowcore. Where the design is based on full composite action between beam and hollowcore, and reinforcement is provided to transfer all forces, the reduction in shear capacity can be small. For non-composite action the reduction in shear capacity can be significant. In both cases the shear resistance can be increased by filling the cores with concrete at the slab ends over a length of at least the slab depth. Extensive research has been carried out on this topic and is available in a number of fib publications. [14, 30]

Where precast floors are supported on insitu concrete walls a steel angle as described in Chapter 8.3.3 is commonly used. Eccentricity of load needs to be taken into account when designing the angle and fixings. In all cases bars from the wall into the precast elements or floor topping tie the elements together.

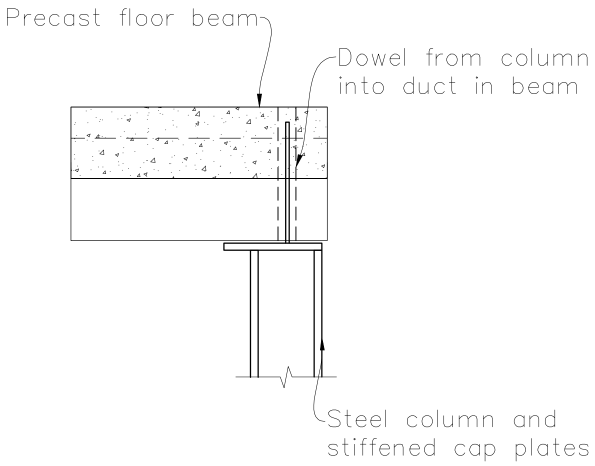


Figure 9.5 Steel support for precast beam

Precast concrete beams can be similarly supported on steel corbel brackets or on steel columns as shown in Figure 9.5. Where beams are supported on steel columns the eccentricity of load can be an issue. The beam will almost always be wider than the supporting column and the effect of loading the beam on one side during erection needs to be considered. In addition the erection and location tolerances need to be taken into account in the design of the column and fixings.

9.2.2 Insitu concrete and precast

Insitu concrete flooring can be used with precast walls. The precast walls are erected and braced in position and an insitu or metal deck formwork concrete slab is poured.

Figure 9.6 a) shows the typical situation where an insitu floor slab is poured over the lower wall. Starter bars project from the wall and lap into the slab and dowels are cast or drilled into the slab to provide a connection to the upper wall. Note the minimum tie force requirements as set out in Chapter 5.

A variation is shown in figure 9.6 b) where a multi-level wall runs past the floor. In this case the load from the floor is transferred to the wall by using the concepts of shear-friction. The interface surface is roughened and reinforcement is provided across the interface to tie the insitu and precast together. This tie reinforcement can be pull-out bars or starters screwed into ferrules cast into the wall. There is no benefit in providing a recess in the wall as this is not taken into account in the shear friction design. The most important criteria is the roughness of the interface. The shear-friction design method is set out in detail in the Precast Concrete Handbook. [2]

Figure 9.8 shows an insitu floor supported on an external wall. This detail is essentially the same as that shown in Figure 9.6 b). The load from the floor is transferred to the wall using the concepts of shear-friction.

A similar concept is use where an insitu beam is supported on a precast wall. Tie bar reinforcement as shown in Figure 9.9 is provided from the wall and into the beam. These are usually screwed into couplers cast in to the wall. The design method is based on the concept of shear-friction and is covered in detail in the 'Precast Concrete Handbook'. [2] With appropriate roughening of the interface and heavy tie bars, loads of several hundred kN can be accommodated.

Figure 9.7 Insitu floor and precast wall

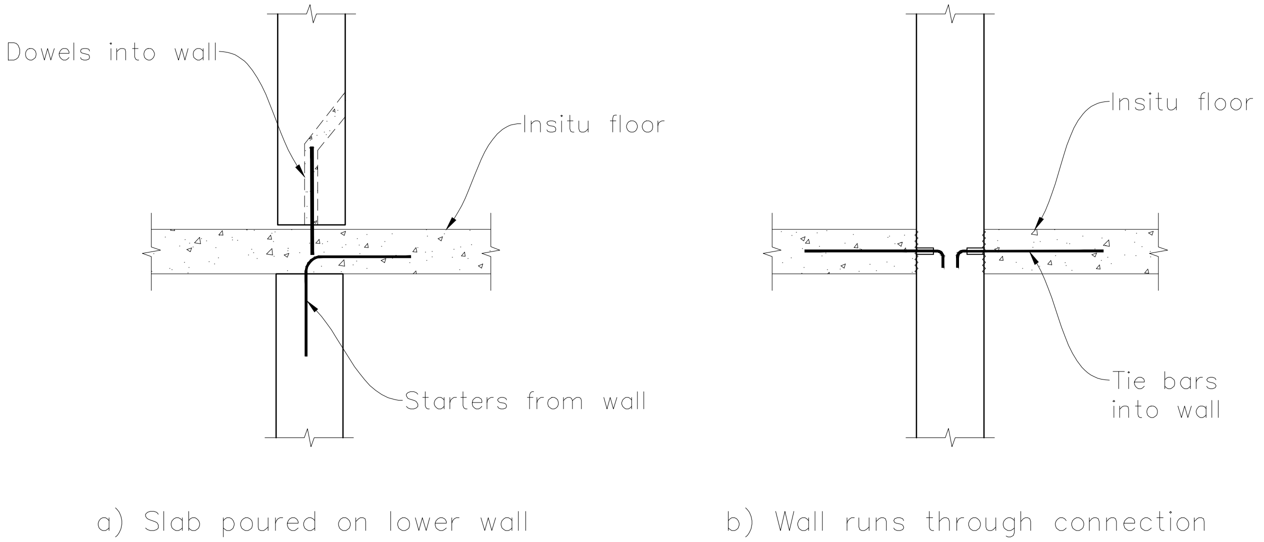


Figure 9.6 Insitu floor to precast walls

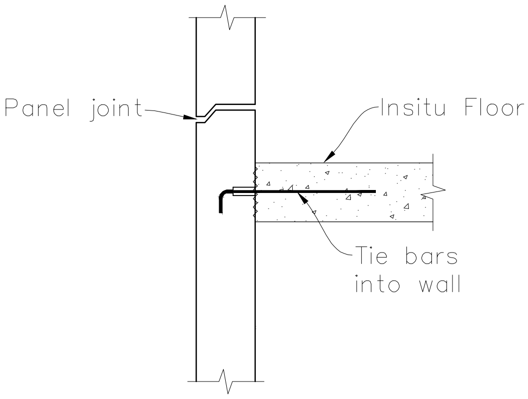


Figure 9.8 Insitu floor to external precast wall

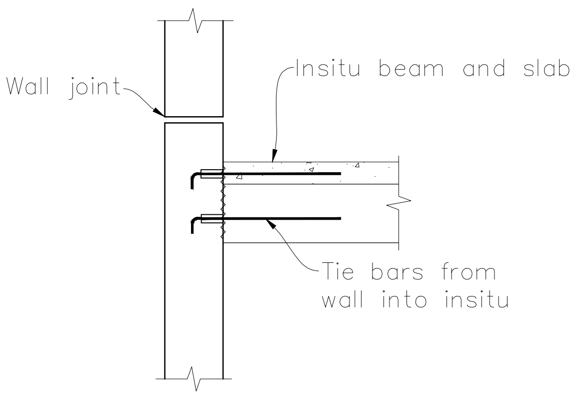


Figure 9.9 Insitu beam and precast wall

Insitu concrete beams can be combined with precast floors. This combination only becomes economical if there is considerable repetition of beams or where beam depth or span precludes the use of precast beams. Where this occurs the logical option is to erect the precast floor elements onto the edge of the supported beam formwork and then pour the beam and floor topping in a single operation as shown in Figure 9.10. The beam is designed as a monolithic section with the flange width as described in Chapter 4.4.2. A precast shell beam acting as permanent formwork is a variation of this concept.

The insitu beam allows the design to be based on full continuity, either as reinforced or post-tensioned concrete and is ideal where beam depth is critical.

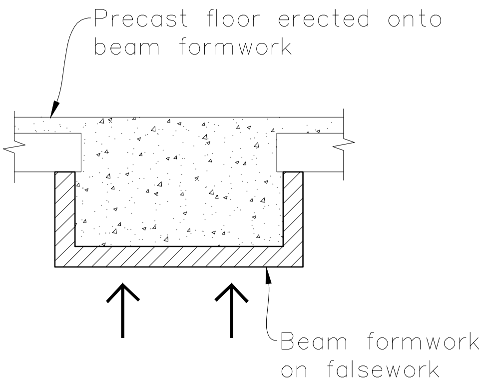


Figure 9.10 Insitu beam and precast floor

9.2.3 Masonry and precast

The use of masonry walls to support a precast floor is common in low-rise domestic construction. Construction speed is much greater than for an insitu floor but buildability can be an issue with respect to crane access. Sufficient walls need to be constructed to allow erection of enough precast flooring to justify mobilisation of a crane.

As shown in figure 9.11, the precast floor system can be supported directly on top of the masonry wall in the same manner as an insitu floor slab. An isolating bearing strip is usually used on top of the wall and in most cases there is no need to tie the walls into the floor system. The exception is where the floor is required to act as a diaphragm and lateral loads are transferred between the floor and walls or where lateral support is required at the top of the wall.

The use of precast floors supported on unreinforced masonry walls is not recommended in areas subject to seismic actions.

Figure 9.11 Masonry wall and precast floors

10 FIRE RESISTANCE

10.1 General

The structural behaviour of a concrete building exposed to fire is a complex phenomenon and due to the large number of intervening parameters the design and calculation methods related to the analysis of the global structure during a fire are still under development.

The purpose of this chapter is to give the designer more insight into the behaviour of a building structure exposed to fire, so that they understand what direct and indirect actions are taking place and how the concrete structure is reacting as a whole. It should enable the application of specific design philosophy extending beyond the simple check of the fire resistance period (FRP) of single concrete elements, as it is often the case.

The requirements with respect to the performance of a building subjected to fire are set out in regulations such as The Building Code of Australia [10] and are expressed as the Fire Resistance Level (FRL). They specify how long a structure shall resist a normalized fire - generally the ISO Standard fire curve. Figure 10.1 shows the typical ISO time temperature curve compared with that for a hydrocarbon fuelled fire.

The Building Code of Australia requires each of the individual elements of a building to have a fire resistant period (FRP) for structural adequacy, integrity and insulation that is not less than the required fire resistant level (FRL).

In this Chapter

There are two aspects to producing a building with the required FRL as nominated by the Building Code of Australia. They are the 'passive design' and the 'active design'.

The passive design refers to the design of the building structure and envelope. The active design refers to measures that are put in place within the building to enhance its performance in a fire. For example provision of a sprinkler system may allow a reduction of the FRP of all or some of the elements in the building.

Common methods of meeting fire code requirements often include both passive and active design solutions. This document does not cover active design measures.

Section 5 of AS3600 provides for the FRP of a structural member to be established by empirical methods based on member dimensions and reinforcement cover. The designer must check whether the member is able to meet these requirements. Calculation methods are acceptable, but no guidance is given other than a reference to Eurocode-2. [24] The performance of concrete buildings subject to fire is discussed in detail in the Cement Concrete & Aggregates Australia, Fire Safety of Concrete Buildings. [36]

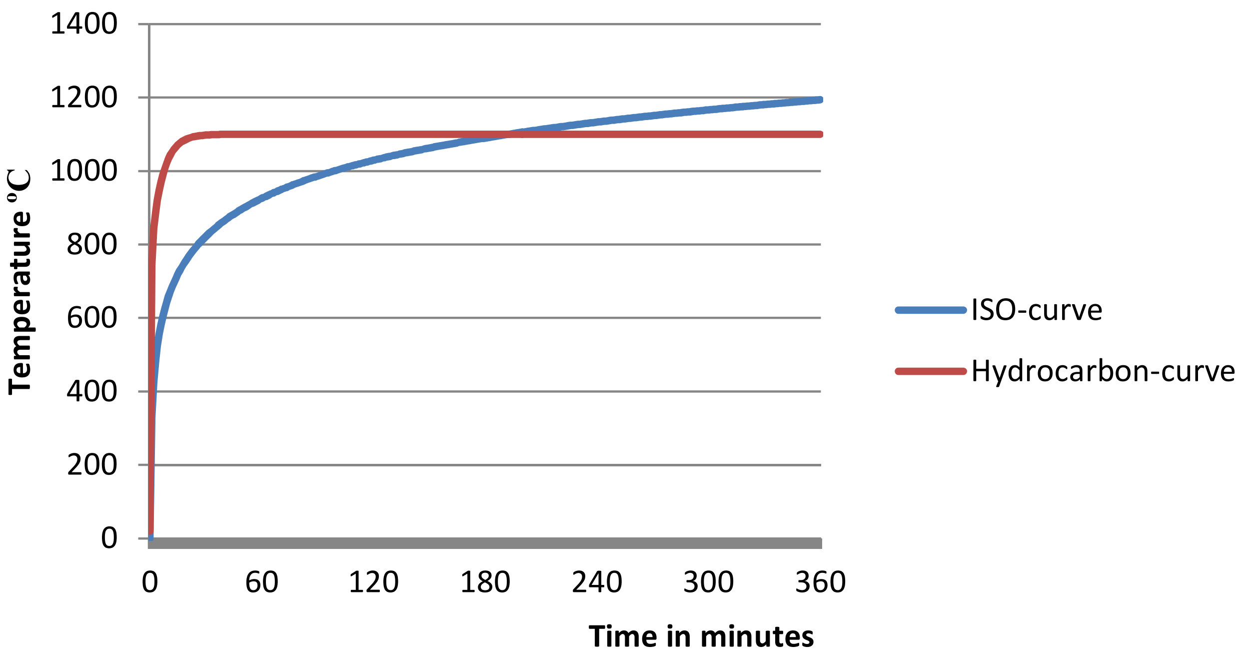


Figure 10.1 Standardised fire temperature curves

10.2 Basic requirements

In accordance with Eurocode-2 [24] requirements the capacity of a concrete structure to maintain its load bearing function during the relevant fire exposure, is expressed as follows:

$\mathbf{E_{d,fi}(t) \leq R_{d,fi}(t)}$

where $\mathbf{E_{d,fi}(t)}$ is the design effect of actions in the fire situation at time '**t**'.

$\mathbf{R_{d,fi}(t)}$ is the corresponding design capacity at elevated temperatures.

The basic criteria for a concrete structure, to comply with the above conditions for structural adequacy, integrity and insulation are as follows:

- Structural adequacy is satisfied where the load bearing function is maintained during the required time of exposure.
- Integrity, or flame tightness is satisfied where the separating function, namely the ability to prevent fire spread by passage of flames or hot gases or ignition beyond the exposed surface is maintained during the relevant fire. Practically this means that precautions must be taken to avoid the passage of fire through cracks, joints and other openings.
- Insulation is satisfied where the average temperature rise over the whole of the non-exposed surface is limited to 140oC, and the maximum temperature rise at any point of that surface does not exceed 180oC. These temperatures are to be seen as serviceability limit states related to the occupation.

10.3 Fire actions

10.3.1 Reduction of material performances

When a fire occurs in a building, the temperature rises fast, at least when there is enough combustible material and oxygen. The exposed structural members will heat according to the thermal conductivity of the materials: very fast for unprotected steel, rather slowly for concrete. Two phenomena occur simultaneously: a reduction of the material performances and a thermal dilatation (expansion). Data about material performance as a function of the material temperatures are available in Eurocode 2. [24]

In unprotected steel structures the temperature will rise rapidly over the whole cross-section, because of the high thermal conductivity of the material. Depending on member sizes and fire temperature a critical limit state will be reached in which the material strength is reduced to about half, and the safety margins disappear. Plastic hinges appear everywhere and the structure collapses. For light steel structures this can occur within about 15 minutes.

In concrete structures, the situation is completely different due to the low thermal conductivity of the material. The heating progresses much slower, and in a non-uniform way over the cross section and length of the member. For example, after one hour of ISO fire, the temperature in a plain concrete floor slab can be 600°C at the bottom and only 60°C at the top. The reinforcing steel in the lower part of the cross-section warms up and gradually loses its strength. At a certain temperature the reinforcing steel is no longer capable of taking up the stresses and failure occurs. This temperature is called the critical temperature.

For example, with a concrete cover to the reinforcement of 25mm, the critical temperature of the reinforcing steel (500 °C) will occur after about 90 minutes ISO fire exposure. For a concrete cover of 35mm the critical temperature will be reached after 120 minutes. For prestressing steel the critical temperature is 100 to 150 °C lower than for normal reinforcing steel. As a consequence the cover on the prestressing reinforcement should be increased by 10 to 15 mm to obtain the same FRP.

Figure 10.2 Example of a fire damaged car park

In addition to the decrease of the material performance, the structure will be subjected to thermal expansion. Beams and columns will expand mainly in the longitudinal direction while floors and walls will expand in both longitudinal and transversal directions.

The above considerations show an important difference between the behaviour of concrete and steel structures at fire. For steel structures, the stability of the structure depends on the resistance of the individual members whereas for concrete structures the global behaviour of the structure is governing, and the fire resistance of the individual members is seldom critical.

Another consideration is that high-strength concrete (HSC) is more sensitive to high temperature than normal strength concrete. Spalling becomes an issue when HSC is subject to fire. Spalling results in loss of surface material, reduction in section size and early exposure of the reinforcement to excessive temperatures. This can result in the actual fire performance of the element being less than that predicted. Codes do not cover this specific aspect of the performance of HSC but further information can be found in the Concrete Institute of Australia publications. [26]

Braced wall structures consisting of loadbearing concrete wall panels and lightweight steel roof structures are a special case where removal of the braced roof by fire can result in the instability of the wall panels even though they may not be directly affected by the fire. For this reason the Building Code of Australia [10] has specific requirements that apply to these types of buildings to ensure stability of perimeter walls after a fire.

Figure 10.3 Example of heat damaged concrete

10.3.2 Thermal expansion

During a fire, concrete needs time to warm up so the effect of thermal expansion will be much less during a short intense fire than during a longer lasting less intense fire. For this reason the ISO fire curve may not necessarily be the most unfavourable thermal action. The time temperature curves on Figure 10.1 show that a hydrocarbon fire can be much more intense than a standard ISO fire.

The most critical situation is when the fire covers a wide surface, resulting in large accumulated deformations. Thermal expansions of 2-3mm per lineal metre can occur in concrete structures during a fire and parts of the structure remote from the fire can be subjected to significant displacement and even failure. In precast structures cooling and contraction after a fire can also result in failure by subjecting connections to load reversals. For example precast floor elements slipping off supporting ledges.

When a fire occurs locally in the centre of a large building, the thermal expansion will be restrained by the surrounding cold concrete structure, and very large compressive forces will generate in all directions. When the fire occurs at the edge of the same building, the horizontal "blocking" will be much lower. The most critical situation is when the fire covers a wide surface, resulting in large accumulated deformations.

The longitudinal expansion of beams or ribbed slabs will be considerably larger than for plain slabs. Beams are exposed on three sides to the fire so the thermal gradient will be more uniform over the whole cross-section.

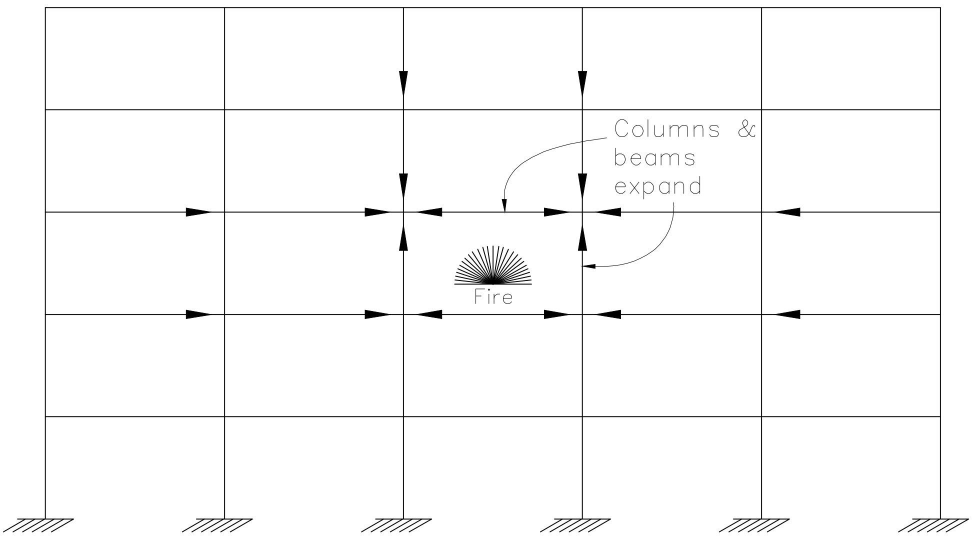


Figure 10.4 Forces in a frame structure due to restrained thermal expansion

10.3.3 Thermal deformation of the cross-section

In addition to the longitudinal and transverse expansion, elements subjected to fire on only one side, for example flat floors, will also undergo a deformation of the cross-section. Because of the temperature gradient over the cross-section, the exposed underside will expand much more than the cooler upper side. This will force the member to deflect downwards. However, the deformation will not be directly proportional to the temperature gradient, since the latter is generally not linear over the cross section.

In a continuous floor structure the downwards thermal deflection will be resisted by the continuity reinforcement at the top of the slab, and the support moment will increase. The increased support moment will create additional compressive stresses at the bottom of the floor and tensile stresses in the top reinforcement above the support will increase and in some cases even yield. In this case the increased shear induced into the floor unit at the support as well as the flexural capacity needs to be checked.

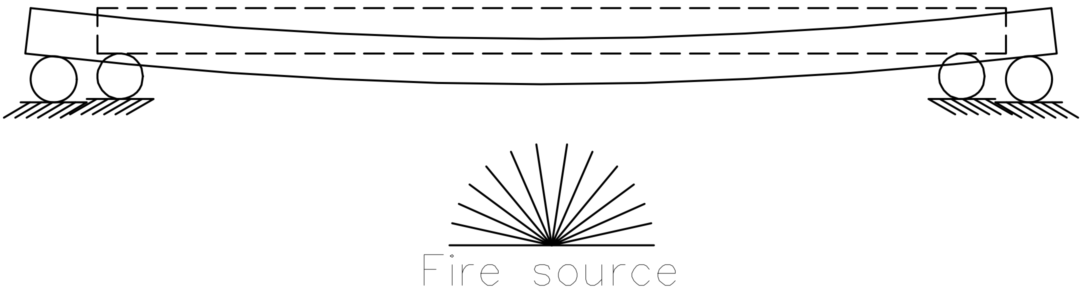


Figure 10.5 Thermal deformation of simply supported floor slab

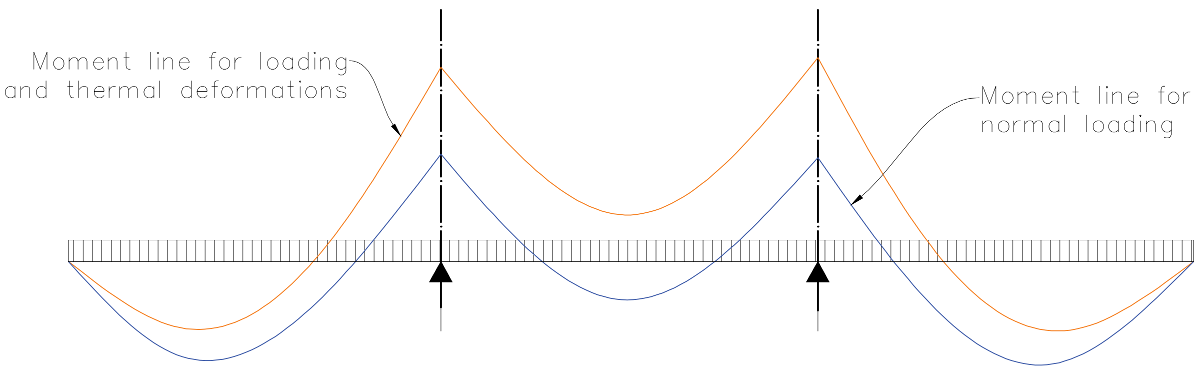


Figure 10.6 Increase of support moment for continuous structure

10.4 Global structural analysis

The analysis of the global structure of a building or parts of it, should take into account the relevant failure mode due to the fire, the temperature-dependent material properties and member stiffness and effects of thermal expansion and deformations. Australian Codes do not stipulate how to include these effects and the analysis for fire safety is restricted to the verification of single structural members to ensure they comply with the cross section dimensions and the minimum concrete cover on the reinforcement for a given ISO fire exposure time.

Experience in real fires shows that instability of concrete structures seldom occurs due to the decrease of material performances at elevated temperatures, but nearly always because of the incompatibility of the structure to take the imposed thermal deformations. Fortunately, concrete structures not only have a high inherent fire resistance but they also have large redistribution capacities due to robustness and structural integrity. Consequently, failure of concrete buildings due to fire seldom occurs.

Based on research it is possible to outline a design philosophy and guidelines based on practical experiences of real fires and model simulations.

- More attention should be given to the overall behaviour of the building when exposed to fire, rather than looking only at the individual components affected by the fire.
- The shape and dimensions of the building, as well as the static system are very important in the behaviour of a building during a fire, especially the effects of thermal expansion, deformations and blocking forces. For example in small buildings the thermal expansion will cause much smaller blocking forces than in large buildings with a fire in the centre.
- In buildings with large floor areas and insufficient movement joints very large expansion may occur. Not only should the distance between the movement joints be considered but also the width of each joint. As a first approximation the effect of expansion due to a temperature rise of 100-150°C should be checked.

In a multi-storey building with a stabilizing stair and lift shaft, the most favourable situation will be met when the stabilising core is placed in the centre of the structure, enabling an even expansion of the surrounding floors in all directions. Figure 10.7 shows a structural layout typical for a precast building, which would probably react favourably to the thermal expansions during a fire. The central core will take up the horizontal actions. All other components are connected to it with hinged joints. During a fire, the restraining forces will be limited, and the connections between the core and the other structural components (columns, beams, and floors) have a statically determined character. Slender columns or hinged connections will deform together with the structure, without causing large blocking forces and shear failure. The design should allow for movement where possible, to avoid incompatibility of deformations due to thermal expansion.

The most important part in 'fire engineering' a concrete building is to assess how the building as a whole will deform under elevated temperature and ensure that these deformations can be accommodated without failure occurring. Attention to the design of the connections is important to ensure that they can accommodate not only the thermal expansion during the fire but also the contraction due to cooling after the fire.

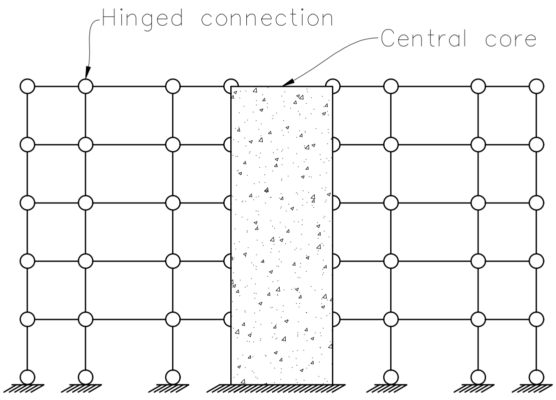


Figure 10.7 Favourable stability lay-out with respect to thermal expansions

10.5 Member analysis

According to AS3600, Section 5.3, [7] the FRP of a building element can be determined by tabulated data from design codes or methods of calculation.

Testing in accordance with AS1530.4. [31] can also be used but establishment of the FRP from tabulated data is the most commonly adopted method.

AS3600 provides solutions for the standard fire exposure up to 240 minutes. The data has been developed on an empirical basis confirmed by experience and theoretical evaluation of tests. Minimum dimensions for the cross-section of the components and a minimum concrete cover on the main reinforcement is given. The information in AS3600 is mainly taken from Eurocode 2 - Part 1-2. [24]

For reinforcement levels which are larger than strictly needed at ambient temperature, the minimum axis distances given in the tables may be adjusted

For prestressing tendons in beams and slabs, AS3600 requires axis distance to be increased by 10mm. The reason lies in the fact that the reduction of steel strength as a function of temperature is much faster for prestressing steel than for normal reinforcing steel.

Note that by using tabulated methods in assessing the FRP of beams and slabs the applied actions on the structure are not taken into account.

10.5.1 Beams

Using AS3600, Section 5 [7] the FRP for a beam can be obtained from a series of tables and charts plotting axis distance of the reinforcement against the beam width. Axis distance is based on the geometry of the reinforcement and beam width is measured at the axis height. Tables and charts are given for both simply supported and continuous beams and for beams exposed to fire on all four sides and for the more typical precast beam case where the top flange is protected by a slab.

Holes through the webs of beams do not affect the fire resistance provided that the remaining cross-sectional area of the member in the tensile zone is not less than Ac = $2b^2$min where bmin is the minimum beam width.

The FRP of beams can also be verified by a simplified calculation based on the Ultimate Limit State method. The load bearing capacity is calculated with reduced material characteristics corresponding to their temperature at a given fire exposure time. The calculation method is based on the assumption that concrete at a temperature of more than 500°C is neglected in the calculation of the load-bearing capacity, while concrete at a temperature below 500°C is assumed to retain its full strength. In other words any concrete with a temperature above 500°C is neglected in establishing the cross section dimensions.

This method is extensively described in Eurocode 2 Part 1-2 [24] and in CEB Bulletin 208 [32]. These documents are referenced in AS3600 Section 5.

This method using the reduced cross-section may be applied for bending, shear and torsion in the design of beams and slabs, where the loading is predominantly uniformly distributed and where the design at normal temperature is based on linear analysis. This is not the case with hollowcore floor slabs where the shear capacity is dependent on the tensile strength of the concrete. This issue is discussed separately in further detail in Chapter 10.5.4.1

Fire is considered to be an accidental action. As a result a reduction factor may be the applied to the imposed actions. In accordance with AS1170.0 [23] the applied action for fire is (G + ψ1Q) where ψ1 ranges from 0.4 for residential to 0.6 for storage occupancies.

10.5.2 Columns

The FRP of reinforced and prestressed columns is influenced by several parameters:

- Size and slenderness of the columns.
- Magnitude of applied load
- First order eccentricity
- Concrete strength and aggregate type
- Reinforcement size and configuration
- Axis distance of the reinforcement

Complex computer programs now exist, enabling the calculation of the FRP of columns, taking account of the above parameters, inclusive of buckling. However, it is not possible to include all this information in tabulated data.

AS3600 provides two tabulated methods to establish the FRP of braced columns but both are restricted in their application. Because the majority of precast buildings are braced structures this is a common method of designing precast columns for fire capacity.

The values in the tables apply to normal weight concrete (2000 to 2600 kg/m^3) made with siliceous aggregates. Unlike Eurocode 2, AS3600 makes no distinction between normal and lightweight concrete.

Unbraced or sway columns and braced columns that fall outside these restrictions require an alternative approach such as that provided by Eurocode 2 Part 1-2. [24]

10.5.3 Walls

The FRP of walls is established in a similar manner to that for columns with a further provision for a minimum effective thickness to comply with the insulation requirements.

The requirements are set out in Tables 5.7.1 and 5.7.2 in AS3600. This is based on the concept of insitu concrete and is not always applicable to precast concrete wall panels. Precast wall panels usually have pinned ends that determine the effective length, sway and P-Delta effects.

On braced wall structures with panels designed to cantilever from the footings the P-Delta effects induced by curvature due to thermal gradients through the wall can become critical.

10.5.4 Floor slabs

AS3600, Section 5.5, [7] allows for the FRP of one and two way solid, ribbed and hollowcore slabs to be established from tabulated data. The approach is similar to that for beams with a further provision for a minimum effective thickness to comply with the insulation requirements. For solid and ribbed slabs this is the most effective method of establishing the FRP of the element.

Because of the complex interactions that affect the fire performance, particularly in the area of the support, the AS3600 requirements do not necessarily reflect the true behaviour of hollowcore slabs, particularly in shear.

The topic of flexural and shear capacity of hollowcore slabs subject to fire is dealt with separately in Chapters 10.5.4.1 and 10.5.4.2

The reference documents for hollowcore slabs subjected to fire are Eurocode-2 Part 1.2 [24] and European Product Standard EN1168. [16]

10.5.4.1 Flexural capacity of hollowcore subject to fire

As for all types of floor slabs the FRP of hollowcore slabs in flexure is governed by the decrease of the strength of the prestressing tendons as a function of the temperature. Full-scale fire tests have shown that the temperature within the vicinity of the prestressing reinforcement is practically independent of the slab thickness and slab profile. For this reason the tabulated data in AS3600 Section 5.5 can be used to determine the FRP of reinforced and prestressed hollow core slabs in flexure. The tables give the equivalent thickness of the slab and the values of the axis distance for reinforcement as a function of the required FRP for simply supported slabs in normal weight concrete. Note that these values are not applicable to shear capacity.

The FRP of hollowcore slabs with respect to flexural failure can also be determined by using simplified calculation methods as described for beams in Chapter 10.5.1 and Eurocode-2 Part 1.2. [24]

10.5.4.2 Shear capacity of hollowcore subject to fire

Prestressed hollowcore floor slabs generally have no shear reinforcement and very short bearing lengths at supports and this governs the shear capacity of hollowcore slabs subject to fire. Elevated temperatures increase the tensile stresses in the profiled webs and increase the transfer length of the prestressing tendons due to possible strand slippage.

Calculations show that after about 20 to 40 minutes ISO fire exposure, the tensile stresses in the central zone of the web exceed the tensile capacity of the concrete. At further fire exposure horizontal cracks originate in the weakest zone of the cross-section due to shear stresses from thermal origin, self-weight, imposed actions, prestressing and thermal expansion. For circular cores the weakest section is situated in the middle of the cross-section, for more rectangular cores, it is situated towards the bottom of the slab. The failure occurs when the horizontal cracks meet the vertical cracks. The phenomenon is of course influenced by the slab thickness, level of the imposed forces, level of prestressing and the total web width of the slabs.

Figure 10.8 (a) shows the appearance of vertical cracks due to differential thermal deformation over the cross-section. Figure 10.8 (b) shows propagation of the vertical cracks into horizontal cracks due to thermal effects, self-weight and imposed actions. To assure adequate shear capacity the FRP is limited so that possible cracks in the webs of the hollow core slabs are kept closed to enable shear transfer by aggregate interlock mechanism.

The European Committee for Standardisation (CEN) in EN1168:2005+A3:2011 [16] has published an empirical formula that has been validated by means of FEM calculations and fire tests to calculate the shear and anchorage failure for hollowcore slabs subject to fire. The method is derived from the shear formula in Eurocode-2 for prestressed members at ambient temperature. By using this formula it is possible to compare the shear and anchorage capacities of hollowcore slabs under fire conditions with those under ambient conditions.

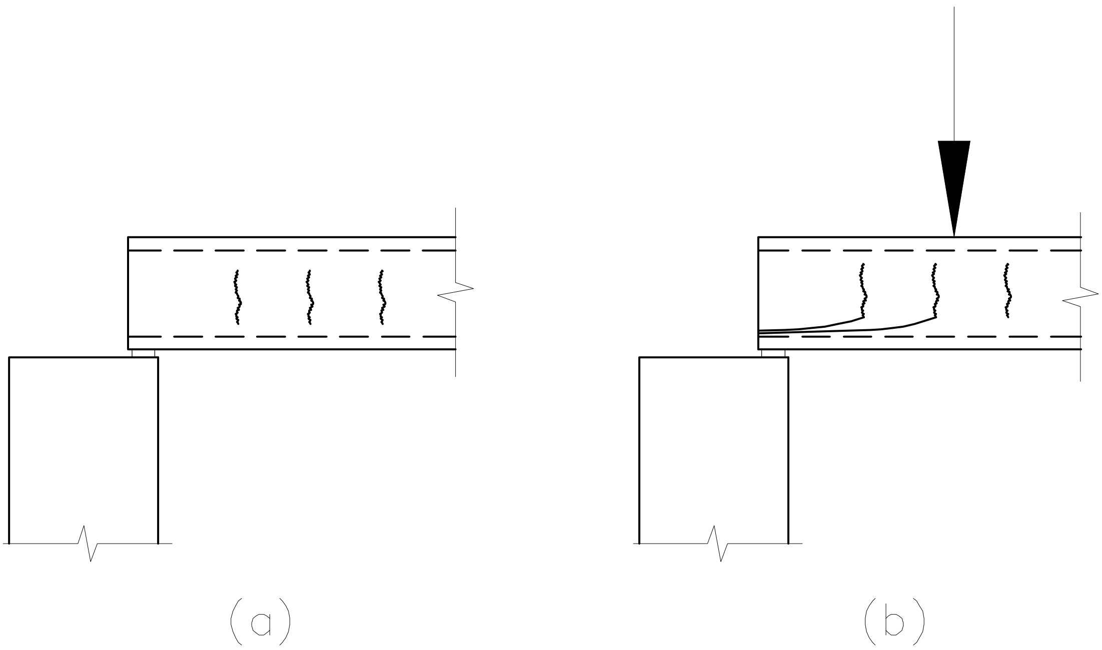


Figure 10.8 Failure mode of a hollow core slab during fire

Table 10.1 expresses the shear capacity under fire conditions (VRd,c,fi) as a percentage of the shear capacity in ambient (cold) conditions (VRd,c,cold) for a range of slab thicknesses and FRL. This shows that under fire conditions the effective shear capacity of un-topped hollowcore slabs can be reduced by over 50% and indicates that great care should be taken in assessing shear capacity of hollowcore slabs in fire particularly with deeper slabs and rectangular cores. For example a 200mm slab with a FRL90 has only 60% shear capacity under fire compared with the calculated value at ambient temperature.

The values in the table should be used as a guide only and are based on the following relatively conservative assumptions.

- slabs are un-topped, no structural screed.
- hollow core slabs are pre-stressed with strands cut at the ends of the elements.
- a support length of 70 mm.
- longitudinal tie reinforcement of approximately 200mm2/m is provided and placed at approximately mid-height of the slab in joints between the slabs and/or in filled cores.
- the influence of the concrete in the filled cores with embedded tying reinforcement is neglected.

Numerous fire tests in different laboratories have shown that the FRP of hollowcore floors can be increased by enhancing the aggregate interlock across cracks. The objective is to provide reinforcement and/or clamping forces that restrict the spread of the vertical and horizontal cracks shown in Figure 10.8. Methods used include the following.

- a reinforced structural topping to tie the slabs to the support, and during a fire prevent horizontal cracks in the lower part of the slab from opening. Structural toppings improve the fire resistance of hollowcore floors and are recommended where an FRP is required.
- tie bars in joints or in filled cores between the ends of slabs. In order to decrease the lever arm of the induced force from the thermal curvature of the floor, tie bars should be placed in the centre of the hollowcore and not in the structural topping.
- a shear ligature cage of reinforcement in filled cores at the ends of the slabs. The ligatures are designed to carry the full design shear forces.
- peripheral ties that contribute to the preservation of the shear capacity of the hollowcore slabs when exposed to fire by forming perimeter beams that resist directly and indirectly the expansion of the floor and hold adjacent slabs together.

The effectiveness of most of these methods is difficult to evaluate and should not be relied on without verification.

$V_{Rd,c,fi}/V_{rd,c,cold}$ [%]	Un-topped slab thickness [mm] and core profile				
Fire resistance Level [min]	150 circular	200 circular	240-300 circular	300 rectangular	360-400 rectangular
FRL 60	70 %	65 %	60 %	60 %	55 %
FRL 90	65 %	60 %	60 %	55 %	50 %
FRL 120	60 %	60 %	55 %	50 %	50 %
FRL 180	45 %	50 %	50 %	45 %	45 %

Figure 10.8 Example of the shear capacity under fire conditions ($V_{Rd,c,fi}$) as a percentage of the shear capacity in ambient (cold) conditions ($V_{Rd,c,cold}$)

10.6 Fire resistance of structural connections

The principles and solutions applied for the fire resistances of structural components are also valid for the design of connections, namely, minimum cross-sectional dimensions and sufficient cover to the reinforcement. The design philosophy is based on the large insulating capacity of concrete. Most concrete connections will normally not require additional measures. This is also the case for supporting details such as bearing pads, since they are protected by the surrounding components. Other considerations are related to the ability of the connection to absorb large displacements and rotations due to thermal movements. Some considerations regarding specific connections are given below.

10.6.1 Beam to column connections

Typically beam to column connections are pinned and perform well during a fire because of their high rotational capacity. Pinned connections are a good solution to transfer horizontal forces in simple supports. They need no special considerations since the dowel is well protected by the surrounding concrete. In addition, dowel connections can provide additional stiffness to the structure because of their semi-rigid behaviour. This is normally not taken into account in the design, but provides a reserve in safety. Care should be taken to ensure that rotation can occur at the connection without concrete to concrete contact causing spalling.

Although all of the connection types shown and described in Chapter 7.3.2 are inherently fire resistant, secondary damage may occur within the connections if the structure is subject to large thermal movements.

10.6.2 Beam to beam connections

Beam to beam connections formed by concrete corbels are inherently fire resistant. Steel hanger brackets as shown in Figure 10.9 are fire resistant due to the fact that the steel fixing is fully enclosed within the concrete. Where this type of connection requires a torsion tie this should be designed to accommodate rotations due to thermal movement and be recessed to provide protection.

10.6.3 Floor to concrete beam connections

The connections tying precast floors to supporting beams are within the depth of the slab and in the colder zone of the structure, and hence not affected by the fire. The position of the longitudinal tie reinforcement (longitudinal means in the direction of the floor span) should preferably be in the centre of the floor thickness.

In case of slabs that are continuous across the support sufficient continuous tensile reinforcement should be provided in the floor to cover possible induced positive and negative moments due to thermal movement.

All of the connection types shown and described in Chapter 7.3.6 are inherently fire resistant.

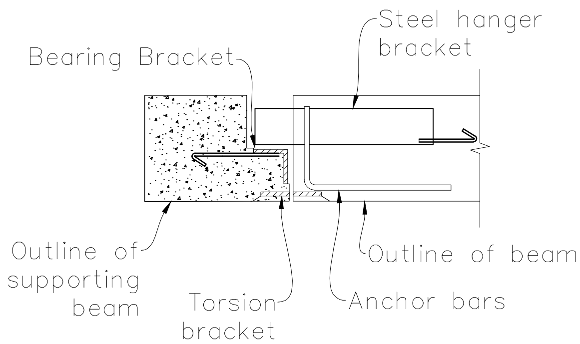


Figure 10.9 Steel hanger bracket

10.6.4 Floor slab to steel beam connections

In the typical case where a precast floor structure is supported on top of a steel beam, the beam will require some type of fire protection to meet any FRP requirements. The connection between the beam itself and the precast floor is protected within the depth of the floor and hence not affected by the fire.

In case of partially encased steel profiles, for example in slim floor structures or trimmer beams as shown in Figure 10.11, the temperature rise in the steel profile within the depth of the floor will be slower than in non-encased unprotected profiles. This is due to the effect of the thermal conductivity of the surrounding concrete. For low to moderate design loads, fire protection of the exposed steel ledge can be avoided by provision of reinforcement ties between the floor and the beam. These ties are designed to support the design actions for fire by activating shear-friction at the interface.

For hollowcore slabs with high design loads or high FRP it is recommended that the exposed steel flange be protected by a fire insulating material. The reasons are that thermal deflection of the steel beam will introduce transverse stresses in the webs of the hollow core slabs which can reduce the shear capacity of the slab.

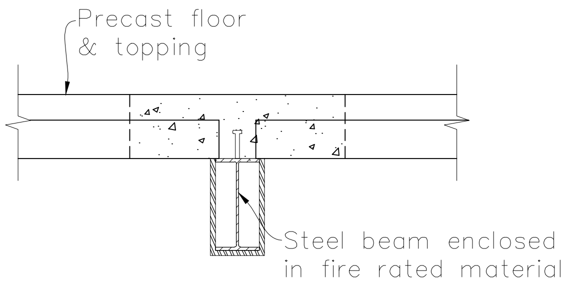


Figure 10.10 Steel beam to floor connection

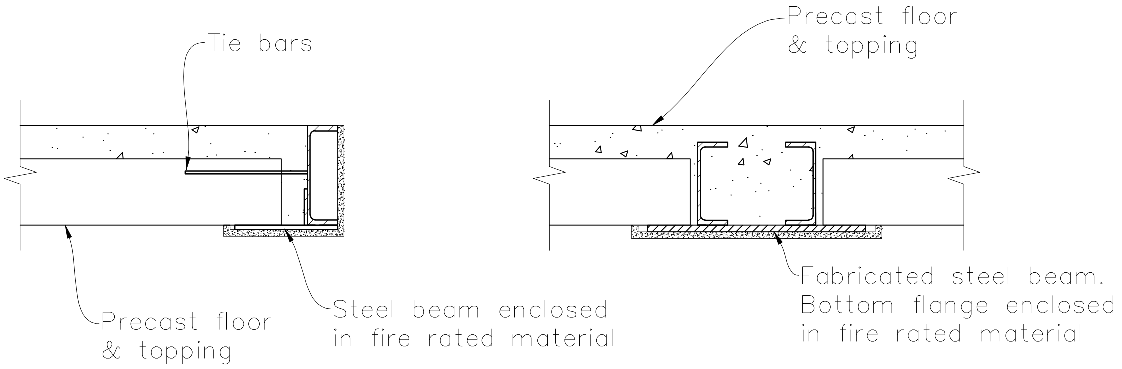


Figure 10.11 Examples of steel beams within floor structure

10.6.5 Floor to wall connections

As shown in Chapter 7.3.6 there are many combinations of floor to wall connections with different precast floor types and different wall types. All except the steel corbel detail are inherently fire resistant.

Where the precast floor is supported on a steel corbel as shown in Figure 10.12 the tie reinforcement and vertical leg of the corbel are protected by the concrete.

For low to moderate design actions, fire protection of the exposed steel ledge can be avoided by provision of reinforcement ties between the floor and the wall. These ties are designed to support the design actions for fire by activating shear-friction at the interface.

In the case of hollowcore slabs the cores containing tie bars should be filled with concrete over the critical transfer length at the support and for hollowcore slabs with high design loads or high FRP it is recommended to protect the exposed steel flange by a fire insulating material.

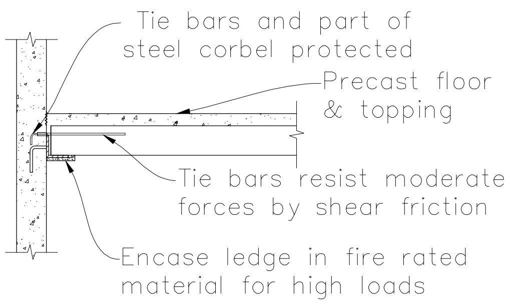


Figure 10.12 Floor to wall connection with steel corbel

10.6.6 Inserts and fixings

Mechanical fixing devices should be protected to the same degree as other structural members. Steel parts embedded in concrete will have a lower temperature rise than non-embedded steel because of the thermal conductivity of the surrounding concrete. However, it is always recommended to provide sufficient protection to exposed parts of the connecting items, such as bolts, steel angles etc. to ensure that yielding due to degradation from the fire cannot occur.

Fixings should also be detailed to ensure that they can accommodate any forces or rotations that result from thermal movement.

Chemical fixings should not be used in connections where an FRP is required.

10.6.7 Joints

Joints between precast elements must be detailed in such a way that they comply with the required criteria for structural adequacy, integrity and insulation.

Longitudinal joints between precast floor elements generally do not require any special protection. The condition for thermal insulation and structural integrity is a minimum section thickness according to the required FRP. Joints must remain closed and this can be ensured by provision of tie-reinforcement or topping screeds.

Adjacent panels in firewalls and columns should be connected to ensure that differential horizontal movement does not occur at the joints due to thermal gradients through the wall.

Depending on the required FRP, the joint can be sealed with a fire rated field moulded sealer or by providing a special profile within the cross-section of the joint.

10.7 Establishing FRP by testing

Tests are usually performed on simply supported elements, with imposed actions corresponding to the imposed (reduced for fire) action combination. Over the past 30 years, many tests and associated research work on precast beams, columns, double T elements and hollow core slabs, have been carried out. The work covered a wide range of cross-sections together with variations in reinforcement quantities and position, cover to soffit and lateral surface, etc.

Because of the limited size of test furnaces (normally up to 4 to 6 m length), and the fact that spans of precast elements are ranging from 6 to 20 m, tests are not really relevant to the global performance of the structure and other methods are needed for the assessment of the overall FRP of concrete buildings. This usually requires the designer to look at the issues from first principles and to assess how the building structure will perform as a whole.

10.8 Conclusion

As previously noted concrete buildings have an inherently high FRL and that the global performance of the building is much more important than the FRP of individual elements. This is particularly the case for precast concrete buildings where connections between discrete elements can be subject to significant unanticipated load reversals under fire conditions.

In this respect the most logical approach to determining the FRL of a precast concrete building is to establish the FRP of individual elements by tabulated methods and to then calculate the dilation (expansion and contraction) of the structure during a fire. As a first approximation a temperature rise and fall of 150-200oC within the area of the fire can be used to determine the movement.

Connections between all elements should then be designed to accommodate the calculated movement.

11 DEMOLITION AND DISMANTLING

11.1 General

All structures should be designed with consideration being given to future demolition and possible re-use or re-cycling. This is particularly important for precast concrete structures where the demolition process can result in instability of the partially demolished structure.

A structure should be design to enable demolition using existing techniques. Designers of new structures are ideally placed to influence the ultimate demolition by designing-in features that facilitate the demolition process. Associated with this, detailed drawings showing the structural design should be archived for future reference.

Detailed information on demolition and dismantling of structures is given in the. Code of Practice: Demolition Work, [39] and in AS3850. [35]

In this Chapter

11.2 Structural assessment for demolition

An integral part of the safe dismantling or demolition of a precast concrete building is the assessment of the structure by a designer with expertise in building construction and demolition. Where possible this should include an assessment of the original structural design drawings as part of the evaluation process before prescribing the method of demolition. Where original drawings are not available it may be necessary to carry out an investigation of the structure to determine the as built details. Even with access to the original structural design drawings the engineer should satisfy themselves that the drawings reflect the actual as-built structure.

The engineer responsible for assessing the demolition process should consider each concrete element individually as the scope and nature of the work for each concrete element may be different. A written statement should be produced that outlines how best to ensure the stability and overall integrity of the building or structure during demolition along with specific processes or procedures to follow during demolition. The engineer should be satisfied that the prescribed method of demolition is safe.

11.3 Safe design for demolition

Based on the proposed demolition sequence a detailed design and associated work method statement should be prepared by the engineer and the demolition contractor that clearly defines the demolition process and method.

The engineer is best placed to advise on how to treat connections and associated temporary works requirements and should produce detailed step-by-step drawings showing each phase of the demolition process including how each element is to be lifted from the structure.

To eliminate or minimise work health and safety risks that may occur during the demolition process, a hazard identification and risk assessment should be carried out by the engineer and demolition contractor during the demolition design process.

A detailed work method statement should then be prepared that incorporates the demolition design and step-by-step work sequence.

Further guidance on the safe design of buildings and structures, including demolition, is available in the Code of Practice: Safe Design of Structures. [38]

11.4 Demolition or dismantling process

During the demolition or dismantling process adequate control measures should be maintained to ensure that all aspects of the structure are as anticipated in the work method statement. No deviations should occur without the written approval of the demolition engineer.

For further guidance on the demolition of concrete elements, refer to the Code of Practice: Demolition Work. [39]

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