Bulletin 74

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Planning and design handbook on precast building structures

manual - textbook

Planning and design handbook on precast building structures

Manual / textbook prepared by
Task Group 6.12

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category	minimum approval procedure required prior to publication
Technical report	approved by a task group and the chairpersons of the commission
State-of-the-art report	approved by a commission
Manual, Guide (to good practice) or Recommendation	approved by the technical council
Model code	approved by the general assembly

Any publication not having met the above requirements will be clearly identified as preliminary draft.

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Foreword

The main aim of *fib* Commission 6 *Prefabrication* is to further the progress of precast structural concrete by promoting research and development internationally and transferring knowledge to practical design and construction.

Since the 1950s, first under the FIP and then, after the CEB and FIP merged in 1998, under the *fib*, the commission has published many technical reports, guides to good practice and design recommendations on various precast concrete design and construction topics. One of these was the successful *Planning and Design Handbook*. Edited in 1994, it had a print run of about 45,000 copies and was also published in Spanish and German.

Nearly 20 years later this new handbook brings the first publication up-to-date. It offers a synthesis of the latest structural design knowledge about precast building structures against the background of 21st century technological innovations in materials, production and construction. With it, we hope to help architects and engineers achieve a full understanding of precast concrete building structures, the possibilities they offer and their specific design philosophy.

Ultimately, it aims to provide information to architects and engineers who have never created a project in precast concrete so that they can produce a sound initial design. Professors and teachers at universities and technical institutes should be able to find in it enough basic information to assist them in preparing lectures and tutorials on precast concrete design and construction. Masters students who have a background in structural concrete should easily be able to extend their knowledge into the field of prefabrication with its help.

This handbook was mainly produced by Arnold van Acker, the former commission chair and task group convener. The task group members contributed valuable comments and feedback. Stef Maas helped extensively with the final edition.

fib Commission 6: *Prefabrication* thanks the task group for this accomplishment and is confident that the new handbook will provide impetus to the dissemination of information about and knowledge of precast concrete structures.

Marco Menegotto

Chair of *fib* Commission 6: *Prefabrication*

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Introduction

The use of precast concrete in construction is widely regarded as an economical, durable, environmentally friendly, structurally sound and architecturally versatile form of construction. The precast concrete industry is continuously making efforts to keep in line with the demands of modern society: economy, efficiency, technical performance, safety, labour circumstances and environmental friendliness.

The changes in construction and civil engineering works over the next decades will undoubtedly be influenced by developments in information processing, global communication, industrialization and automation. These developments will almost certainly already be implemented in prefabrication. However, when looking into the efficiency of present building processes, we should try to find a smoother transition from the design of the construction to its completion. The only way to move forward from the traditional labour-intensive methods to the modern approach of prefabrication is to apply an industrial philosophy throughout the entire building process.

Sustainability is a major issue in any industrial activity today. In the building industry, prefabrication offers the best approach to this end. Indeed, the whole life cycle is controlled better. The industrialized production of elements allows materials and energy to be saved during construction. In all phases, time, waste, noise and dust, as they impact the environment, are reduced.

The prefabrication of concrete structures is an industrialized process with great potential for the future. Uninitiated designers often see it as an execution technique that is a mere variant of cast in-situ construction, where the notion of prefabrication means that parts of the construction are precast in specialized plants, to be assembled afterwards on site in such a way that the initial concept of cast in-situ structures is obtained again. This is incorrect. Every construction system has its own characteristics that, to a greater or lesser extent, influence structural layout, span and width, the stability system, and so forth. For the best results a design should, from the very outset, respect the specific and particular demands of the intended structure.

This handbook is intended to dispel these misguided impressions by providing a detailed review of the subject and, thereby, to promote a greater awareness and understanding of precast concrete buildings. It gives a synthesis of the work carried out by a focus group of industrialists, consultants, contractors and academic members of *fib* Commission 6 *Prefabrication* over the past decades. It has been written particularly for those less familiar with this form of construction but will also be of interest to all engineers, architects and others concerned with the design and construction of buildings. Students of technical universities and high schools will also find valuable information in this handbook.

It predominantly covers non-seismic structures. Brief information on how to design precast concrete mainly for moderate seismic loading is spread over several chapters.

We hope that the best-practice examples for buildings and structures made of precast concrete will persuade readers to consider this construction method for future projects.

The first chapter of the handbook highlights best practice opportunities that will enable architects, design engineers and contractors to work together towards efficient solutions, which is something unique to precast concrete buildings. It evaluates the precasting capacities of a project based on the choice of construction method and suggests that most buildings are suitable for precasting, either in whole or in part, depending on the architectural and / or structural requirements. This chapter does not hide that prefabrication is an industrialized building process but shows how it is now at the cutting edge of modern design. The most prominent features of the system are described, together with some of its limitations.

The second chapter explores the various routes together with the rationale that lead to the development of precast building projects. With the emphasis on *solutions*, and since structural form follows function, the precast projects develop intrinsically. The essential principles underpinning good engineering design in precast concrete are discussed: the respect for basic design philosophies, the importance of standardization and modulation, dealing with dimensional tolerances, the integration of different building techniques, and so forth.

Chapter 3 illustrates the aesthetics of precast concrete, enabling the reader to visualize precast solutions for the most common types of buildings, such as offices, sports stadiums, residential and apartment buildings, hotels, industrial warehouses and car parks. Different application possibilities are explored for an understanding of the types of precast units that are commonly used in all of those situations. Although there are apparently a large number of technical systems and solutions for precast buildings, they all belong to a limited number of basic structural systems of which the design principles are more or less identical: portal and skeletal structures, wall frame structures, floor and roof structures and façades in architectural concrete. After a detailed description of the basic structural systems, the chapter continues with guidelines to and recommendations on the selection and application of these systems for different market segments.

Chapter 4 discusses the basic design principles of stability from both theoretical and practical standpoints. Precast concrete structures should be designed based on a specific stability concept that differs from the one used in cast in-situ structures. The chapter explains the various systems that are used to stabilize a precast structure: braced and unbraced frames, the restrainment of columns into the foundations, wall-frame action, portal-frame action, multiple or central cores, and floor and roof diaphragm action. Essentially, all these systems contribute to the design of a coherent entity made of individual precast elements. This is what is called structural integrity and comes from many years of collaborative work in research and development. It has been documented for the first time in this handbook and another recent *fib* publication [14]. Structural integrity is obtained through a three-dimensional network of ties. The different types of ties and design rules are described in detail, together with current code prescriptions.

Chapter 5 discusses structural connections. In prefabrication, connections are some of the most essential parts. Their role is to create out of individual elements a coherent and robust structure able to resist all acting gravitational and sway forces as well as indirect actions resulting from, among other causes, shrinkage, creep, thermal and ground movements and fire. This chapter gives the basic principles and design criteria for structural connections in precast concrete constructions. First, basic criteria that govern the design are discussed: the strength of the connection, the influence of volume changes, movements, the ductility and durability. It explains the basic mechanisms for the transfer of forces through the components of a connection: encasing, lapping of reinforcement, dowel action, bond, friction, shear interlock, bolting and welding, and post-tensioning. A detailed description of structural connections and their applications is given, for example, connections that transfer compressive, tensile and shear forces, and torsion, individually or collectively. The chapter ends with a survey of design criteria related to production, transport and erection.

Chapters 6 to 9 address the four most commonly used systems or subsystems of precast concrete in buildings, namely:

- Portal and skeletal structures
- Wall frame structures

- Floor and roof structures
- Architectural concrete façades

Practical information on each of the above is given so that the architect and the engineer may prepare a first draft scheme on any one of them or a combination of them. This may be submitted to the specialist prefabricator for detailed discussion. The optimum use of each method is discussed with regard to building co-ordination, element sizes and location, and load transfer systems for gravity and horizontal loads. The chapter gives an overview of a wide range of connections with their serviceability performances.

In Chapter 10 the design and detailing of a number of specific construction details in precast elements are discussed, for example, supports, corbels, openings and cut-outs in the units and special features related to the detailing of the reinforcement. For each topic the requirements, solutions and design aspects are dealt with in detail and recommendations are given for the design.

Precast buildings generally possess excellent fire resistance. Chapter 11 explains the various actions a fire exerts on a concrete structure. Different methods of how to analyse the precast structure and determine the fire resistance of the precast elements and the assembled building are given in terms of: assessment by tabulated data, by simplified calculation method or by laboratory tests. For each of the methods, information is given on how to apply them to the different precast concrete elements in a building. Lastly, the fire resistance of the connections between precast elements is discussed.

All the information given in the present handbook is evidence of normal precast construction practice. The user should always bear in mind that special designs can be created to meet specific requirements.

The handbook concludes with a list of references to good literature on precast concrete construction.

1 Suitability of precast concrete construction

1.1 General

At the initial design stage the first requirement is to identify whether the building project or certain parts of it are suitable for construction in precast concrete, and what the specific advantages and inconveniences are compared to other building systems.

Figure 1.1 gives an example of the possibilities of modern precast concrete in the domains of architectural design, product application, surface finishing and more. The building pictured is the headquarters of Decomo in Belgium.

Fig. 1.1: A precast solution for the headquarters of a leading precasting company in Belgium.

The building structure of Figure 1.1 comprises standard beams and columns completed with prestressed hollow-core floors. The façades are in precast architectural concrete sandwich panels and 3-storey-high slender rectangular columns. The monumental entrance includes stairs and landings.

The main advantages of precast concrete are the speed of erection, the quality of the concrete, long-span prestressed concrete floors, a controlled working environment and the economy of the project. However, many misunderstandings occur around, for example, the lack of flexibility, the multiplicity of precast building systems and the long lead-in times needed for a complete study.

Precast concrete has many more advantages than are mentioned above, and the excessive repetition of products or long design and manufacture periods no longer exist in contemporary practice. On the contrary, thanks to modern production techniques and computer aided manufacture, in combination with Building Information Modelling (BIM), its flexibility combined with its short delivery times have become a major commercial asset for prefabrication.

Therefore, over the past thirty years the latest generation of precast concrete buildings have become high-specification buildings. Architectural structural precast concrete elements are being used on an increasing number of prestigious commercial buildings, and steelwork, timber, plastics and masonry are being combined for the holistic benefit of the entire building process. Designers are becoming more aware of the high-quality finishes available for prefabricated units, and changes are now being made to the way that the traditional precast structures are conceived and designed. The construction industry is calling for multi-functional design, where the optimum use of all the elements forming the building must be maximized. In the initial phase of study, precasting cannot be ignored any longer, either as a solution for the whole structure or for parts of it.

In this chapter information is given concerning the possibilities precast concrete offers, its advantages and limitations, and the principles of quality assurance and plant certification.

1.2 Advantages and limitations

Most buildings are suitable for precast concrete construction. Buildings with an orthogonal plan are, of course, ideal for precasting because they exhibit a degree of regularity and repetition in their structural grid, spans, member size, and so forth. When designing a building, one should always strive for standardisation and repetition to ensure economical construction while still attempting to respect the individuality of the architectural design. This rule applies to all design and not only to precast concrete construction. The concourse structure shown in Figure 1.2, where a set of nearly identical elements was configured to create a challenging solution in exposed, grit-blasted concrete columns, beams and slabs, illustrates this philosophy.

Fig. 1.2: Precast elements consisting of circular columns, beams and vaulted roof panels provide the finished structure to the concourse at Paddington station, London

On many occasions irregular ground layouts are, if not totally then certainly partially, suitable for precasting. It is a misunderstanding that precast concrete offers no flexibility. Modern precast concrete buildings can be designed safely and economically, with a variety of plans (see figures 1.1 to 1.5) and with considerable variation in the treatment of the elevations, generally to heights of about twenty to thirty metres but also up to forty floors high.

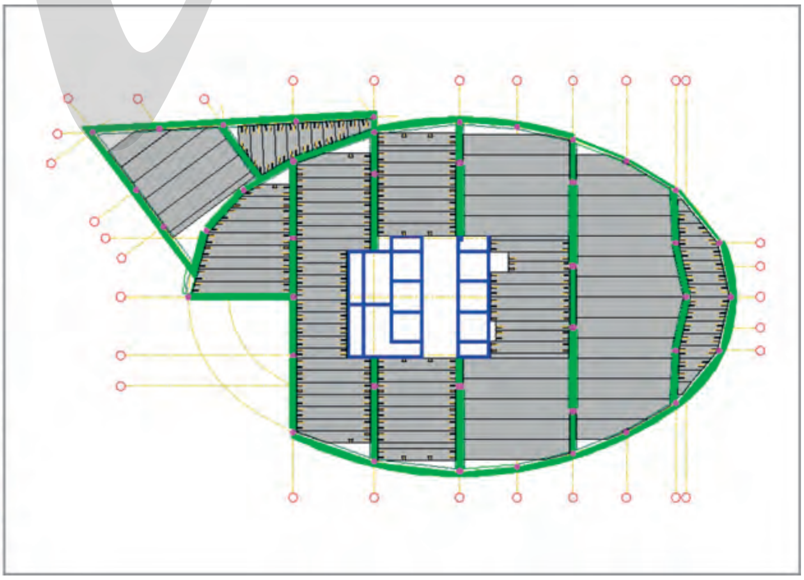


Fig. 1.3: Flexibility in design and construction enables this oval shape building in Brussels, Belgium, to be prefabricated with mainly rectangular concrete elements

Figures 1.3 and 1.4 illustrate market trends in the architectural layout of all kinds of non-orthogonally shaped floor plans, for example, ones that are elliptic, rounded and have sharp edges. This is now possible in the precast industry thanks to great flexibility in the production technique.

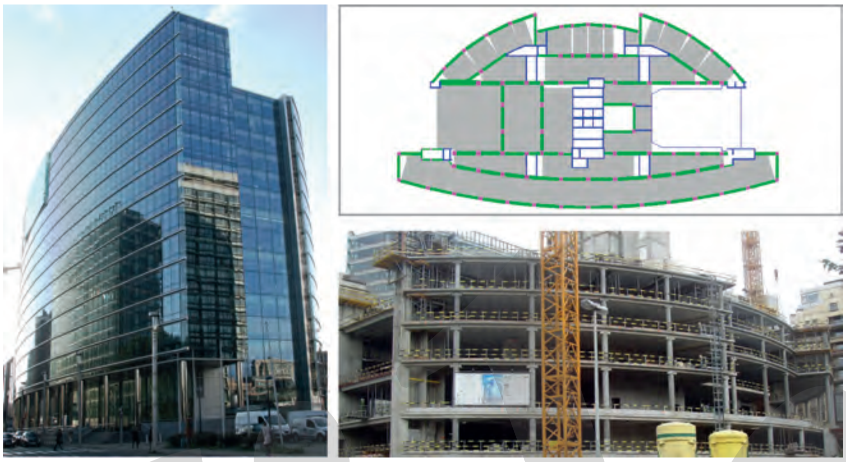
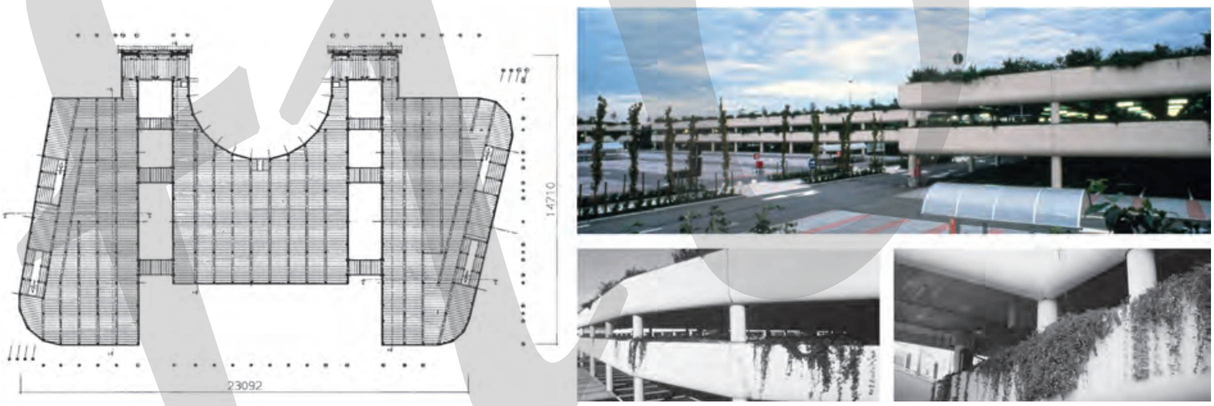


Fig. 1.4: Freedom in design of the floor layout of the conference building of the European Union, Brussels, with its 7 large conference rooms

Figure 1.5 shows the example of an entirely precast car park with 2,500 places in Bologna, Italy. Despite the large number of different units, most of the elements were cast in the same basic moulds. This enabled not only economical production, but also flexibility in the casting.



a. Plan view *b. View of façades*

Fig. 1.5: Layout of the floor and a view of the façades of an entirely precast car park with 2,500 places in Bologna, Italy

Precast concrete offers considerable scope for improving structural efficiency. Longer spans and shallower construction depths can be obtained by using prestressed concrete for beams and floors. For industrial and commercial halls, roof spans can be up to forty metres in length or even longer. When precast concrete is used in parking garages, more cars can be fit into the same construction space because of the large span possibilities and slender column sections. If precast units were used to their full potential, office buildings could have even larger open spaces divided only with partitions. Using such units leads to greater flexibility as well as an extended lifetime because of a building's greater adaptability. Thus, a building can retain its commercial value over a longer period.

1.3 Differences in precast and cast in-situ structures

Apart from a production in weather-controlled conditions, precast concrete offers a number of other differentiating features:

Cast in-situ concrete structures behave as three-dimensional frameworks. The continuity of displacements and the equilibrium of bending and torsional moments as well as shear and axial forces are achieved by reinforcing the joints so that they have equal strength as members (Fig. 1.6a). However, in precast concrete structures in non-seismic areas, the creation of a three-dimensional framework is seldom applied because it is typically not necessary to achieve moment-fixed connections between linear members. The stability of precast structures should be assured by appropriate systems that are easy to achieve on site: by the restrainment of columns in foundations (Fig. 1.6b), the in-plane stiffness of shear walls or cores, diagonal bracing, floor and roof diaphragms, or combinations of these systems.

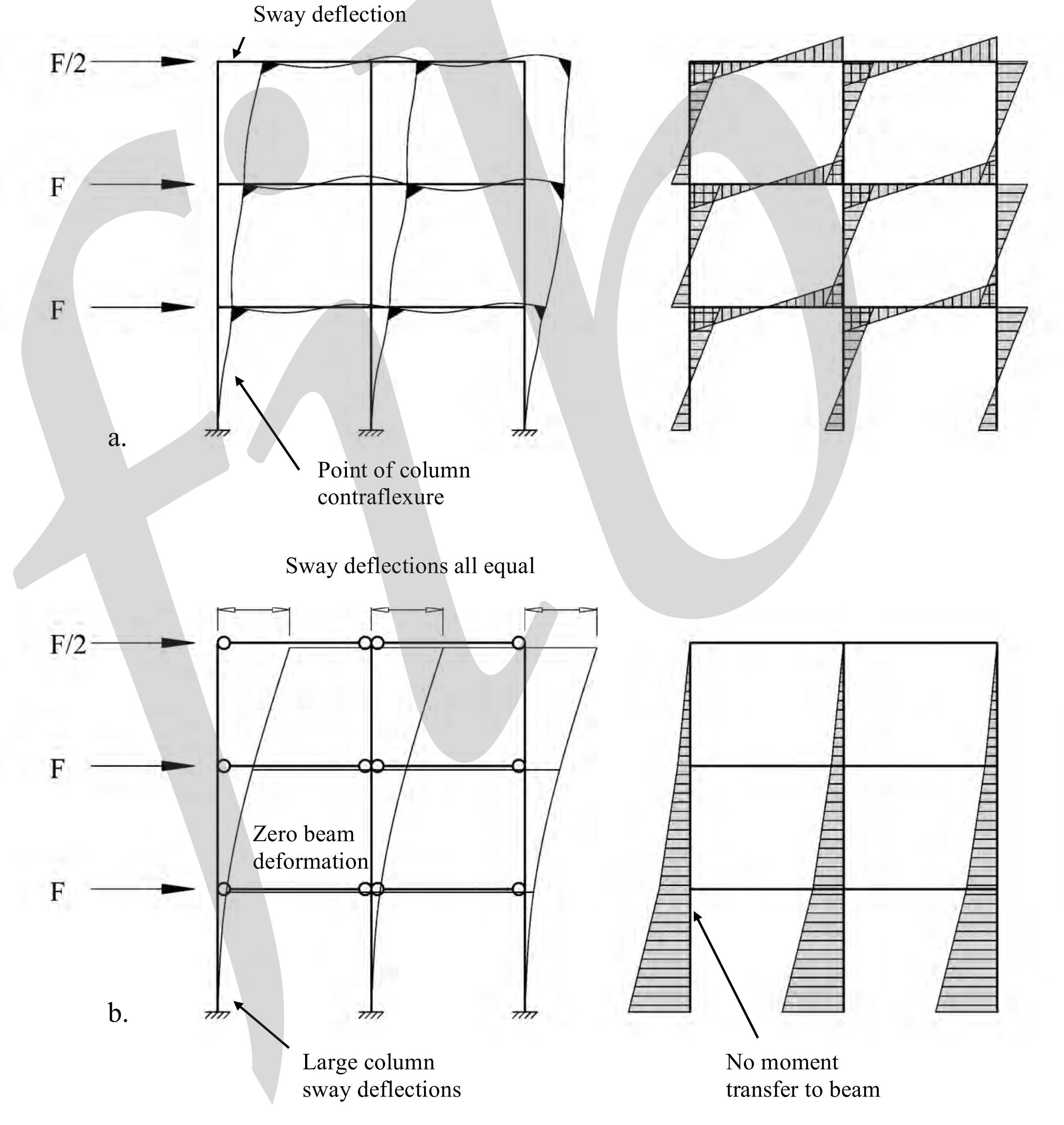


Fig. 1.6a 2-D and 3-D continuous frame action in cast in-situ frames is replaced in Fig. 1.6b by moment-resisting columns in a precast skeletal frame

Precasting works use computer-controlled batching and mixing equipment. Additives and admixtures are used in the mix design to obtain the specific mechanical performances needed for each product. The casting and compaction of the concrete is performed in indoor working conditions, with optimum equipment. The water content can be reduced to a minimum and compaction and curing are performed in a controlled environment. The result is that the grade of concrete used can be exactly suited to the requirements of each type of structural element in order to eliminate the use of more expensive and exhaustible materials. Usually, the concrete cube strength of precast concrete elements is 50 to 60 N/mm². In addition, the optimisation of the mix is beneficial for durability.

High performance concrete, with cylinder strength exceeding 100 N/mm², is common in prefabrication and widely used by precasters. The major benefit for building structures is an improved structural efficiency that contributes to more slender products and the optimum use of materials. Another advantage of high-strength concrete is its smaller ecological footprint.

Self-compacting concrete (SCC) is a new and very promising solution for prefabrication. Whereas high-strength concrete essentially focuses on improved product performances (strength and durability), self-compacting concrete also provides significant benefits to the production process. It needs no vibration and thus offers many advantages, such as a low noise level during casting, less mould pressure and formwork wear, rapid casting, easy casting when using dense reinforcement or for thin or complicated cross-sections, fewer air pores at the surface and easy pumping.

Figure 1.7 shows a new application achieved using SCC for circular columns. Beforehand, circular columns were cast in a vertical position using cardboard moulds with an inner lining in aluminium folio. The column height was usually limited to one storey. The inconvenience of this limited height, compared to multi-storey columns, lies not only in the additional cost for the production, handling and erection of a larger number of columns, but it also requires more connections. With self-compacting concrete, it became possible to cast circular columns in closed horizontal moulds via the openings left for the corbels. Hence, the problem of limited column height at manufacture is solved.

The application of SCC in the precast concrete industry is already widely in use and it is expected that in a few years from now a large part of daily production will use this technique.

Fig. 1.7: Two-storey-high circular columns in self-compacting, high-strength concrete

Prestressing is often applied in precasting because of the possibility of using prestressing beds and tendons anchored by bond. This technique does not only offer all the constructional advantages of prestressed concrete but also economical manufacture because of the low labour input and the absence of the expensive anchorage devices needed in post-tensioning.

The major difference with cast in-situ concrete is that the planning around the units to be precast has to start at an earlier stage. Both the architect and services engineer must be ready to define their requirements in time for the precaster to prepare his/her drawings. Therefore, the final study of the building services has to be made earlier than usual; this could equally well be seen as an advantage.

1.4 Opportunities in 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. Prefabrication is an industrialized construction technique with inherent advantages:

1.4.1 Factory-made products

The only way to industrialize the construction business is to shift the work from the site to modern, permanent factories. Factory production means rational and efficient manufacturing processes, skilled workers, repetition of actions and lower labour costs per square metre due to the automation of the production process, as is the case, for example, with the manufacture of hollow-core floors and the activity of quality surveillance. Factory products are process-based and lean manufacturing principles are used in manufacturing. Competition and the social environment are forcing the industry to continuously strive for greater efficiency and better working conditions through the development and innovation of products, systems and processes. Automation is gradually being implemented. Examples already exist in the domain of reinforcement preparation, the assembly of moulds, concrete casting, the surface finishing of architectural concrete, and so forth. Other operations will follow.

1.4.2 Prestressing

The pretensioning of steel tendons (either small diameter wires of 5 to 9 millimetres or compound helical strands, such as steel ropes with a diameter 9.3 to 15.7 millimetres) is often applied in precasting because of the possibility of using prestressing beds and tendons anchored by bond, as shown in Figure 1.8. The technique not only offers all the constructional advantages of prestressed concrete but also economical manufacturing because of low labour input and the absence of the expensive anchorage devices used in post-tensioning.

Fig. 1.8: Prestressing methods use long-line casting beds of between 100-150 metres in length

1.4.3 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 and optimal use of the materials. This is obtained through modern manufacturing equipment and carefully studied working procedures. Precasting works use computer-controlled batching and mixing equipment. Additives and admixtures are used in the mix design to obtain the specific mechanical performances needed for each product. The casting and compaction of the concrete are performed in indoor working conditions, with optimum equipment. The water content can be reduced to a minimum and curing is also carried out in controlled circumstances. As a result, the grade of concrete used can be exactly suited to the requirements of each type of element in order to eliminate the use of more expensive and exhaustible materials. In addition, the mix efficiency is better than in cast in-situ concrete.

1.4.4 Quality

The term 'quality' is broad in meaning but its end purpose is the supplying of products and services that meet customer expectations. Quality control starts with the preliminary study and preparation of the project and continues through the production of the elements and adherence to delivery and erection deadlines.

Quality control during manufacture is based on four principles:

- People
- Plant installations and equipment
- Raw materials and operating processes
- Quality control of the execution

Quality is usually based on a self-regulatory system, with or without supervision from a third party. Factory quality control depends on procedures, instructions, regular inspections, tests and the utilization of the results to inspect equipment, raw materials, other incoming materials, the production process and the products. The results of the inspections are recorded and made available to clients. Most precasting companies have obtained the ISO-9000 label.

Prefabrication offers clients marked advantages with respect to building use, lifetime and ecology, in line with present trends in construction.

1.4.5 Architectural freedom

The design of buildings is not limited to the application of rigid precast elements and almost every building can be adapted to the requirements of the builder or the architect. Architectural elegance and variety on the one hand and increased efficiency on the other are not mutually exclusive. The days are gone when industrialization led to large numbers of identical units; an efficient production process combined with skilled workmanship allows for modern architectural design within a budget.

1.4.6 Structural efficiency

Precast concrete offers considerable scope for improving structural efficiency. Longer spans and shallower construction depths can be obtained by using precast prestressed concrete for beams and floors. For industrial and commercial halls, roof spans can reach 50 metres. In car parks, precast concrete enables greater parking densities because of the large span possibilities and more slender column sections, for example, a double-parking bay and driveway may use 16m-long prestressed slabs of a minimum depth of 400 millimetres. In high-rise buildings, an additional storey can be added thanks to thin, precast flooring solutions at the same total height of the building. In office buildings, the trend is to construct large open spaces, separated by demountable partitions. Precast concrete offers not only flexibility in the building but extends its lifetime because of greater adaptability. In this way, the building retains its commercial value over a longer period.

The major benefit for building structures is an improved structural efficiency that allows for more slender products and the optimum use of materials. Another positive aspect is improved resistance to frost and chemicals. The greatest advantages are achieved for vertical components, especially load-bearing columns, where the load bearing capacity increases by between 100% and 150% for concrete strengths ranging from 30 to 90 N/mm².

1.4.7 Flexibility in use

Certain types of buildings are required to frequently adapt to the user's needs; this is especially the case with office buildings. The most suitable solution to this effect is to create a large, free internal space without any restriction to possible subdivisions with partition walls.

1.4.8 Adaptability

In the past and even today buildings are conceived for a clearly defined purpose that does not truly take future developments and possible refurbishments into account. However, over the course of time, buildings no longer respond to their users' changing requirements or are not suited to new occupants and this results in complex renovations or even demolition. These latter solutions are expensive, time-consuming and environmentally unfriendly. In the future they will become even more problematic because of the likelihood of stricter regulations on the creation of noise, dust, demolition waste, traffic congestion and other inconveniences.

The solution to this problem can already be found at the design stage of a new building. The preliminary study should anticipate future renovations or repurposing so as to eliminate the necessity to demolish the structure. The basic idea is to clearly differentiate between the structural part of the building and its finishing. The structural part comprises all principal functions such as the load-bearing structure, principal circulations, principal conduits, load-bearing façades, and so forth. The finishing comprises partitions, technical equipment and non load-bearing façades, amongst others.

Today precast concrete structures are already being conceived using this approach. One of the mainstays of prefabrication is the large span capacity it allows for beams and floors and this leads to large open spaces inside the structure.

More details about the applications of this design approach are given in the following chapters.

1.4.9 Fire-resistant construction

Precast building structures in reinforced and prestressed concrete normally have a fire resistance of 60 to 120 minutes or more. For industrial buildings, all types of precast elements without special or additional inflammable characteristics will attain the normally required fire resistance of 60 minutes. For other types of buildings, a fire resistance of 90 to 120 minutes is easily obtained by increasing the concrete cover to the reinforcement. In general, production processes in precast factories allow for a more efficient and better controlled use of rebar spacers than in site-cast concrete.

1.4.10 Environmentally friendly construction

Preserving the environment is becoming increasingly important on a global scale. Since some of the most basic needs of all generations are housing and mobility, the building sector plays a central role. Even today most construction inflicts damage on the environment in terms of energy consumption, the draining of resources, pollution, noise and the production of waste.

However, the precast concrete industry is making headway in environmentally friendly construction by reducing material use by up to 45 per cent, energy use by up to 30 per cent and waste at demolition by up to 40 per cent. Several plants recycle all fresh and hardened concrete waste; future precasting plants will work as a closed production system in which all waste material is processed and reused.

1.4.11 Appearance and surface finishing

Precast concrete elements 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 by using beams and columns with special shapes and with high quality finishes. The designer can inspect and accept the units before they are transported to the site and fixed in place.

Precast architectural concrete offers a wide range of top quality finishes in an array of colours and textures, such as limestone or granite, complex brickwork detailing and masonry profiles reproduced in reconstructed stone or simulated stone – all features which would be prohibitively expensive if carried out on site using conventional methods.

1.4.12 Transport and site erection

Transportation is normally carried out by trucks. The maximum economical distances vary between 150 and 350 kilometres, depending on the type of products transported, the traffic infrastructure, the population density, and so forth. When rail or maritime transport is used the maximum cost-effective transport distance can reach 2,000 kilometres.

The erection procedure can affect the maximum weight of the units depending on the accessibility of the site and the capacity of the lifting crane. This should be taken into consideration at the start of the final design work.

1.4.13 Building services

The installation of building services can be integrated into the building system. A major advantage of this is that the precast structure can be designed according to the specific requirements of the building equipment. Elements can be provided with a variety of holes, and fixings can be cast in the units; many additional means are available on site after the erection of the precast building.

Precasting offers marked advantages for building services. For example, the thermal mass of concrete is used efficiently to store thermal energy in hollow core floors, which leads to substantial savings in heating costs. Another example is the possibility of using cast-in ducts, boxes or chases for electrical fittings. Internal rainwater pipes are sometimes cast in columns or façade units. Large prefabricated conduits for ventilation and other pipes can be installed inside double ceilings or along projecting spandrel-façade units during the erection of the precast units.

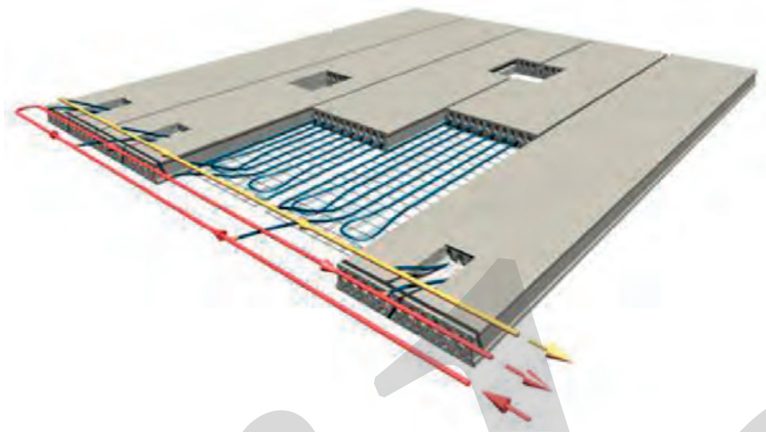


Fig.1.9: Incorporation of floor heating ducts in hollow core slabs to activate thermal mass

1.5 Quality assurance and product certification

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

The quality assurance and the quality control of precast concrete members are initiated at two levels: an in-house quality-assurance programme with continuous internal control, and plant certification with quality-control supervision by an independent body.

The certification of precast concrete production plants offers confirmation by an independent inspecting body that quality products can be manufactured and that the in-house control system functions smoothly.

Confirmed capability means that a plant is well equipped and the people who operate it are competent to produce quality products. They achieve this through the inspection of their production operations, materials, equipment, personnel and products for conformity to the Plant Certification Program. This means that the producer has the capability, by virtue of personnel, facilities, experience and an active quality assurance programme to manufacture quality products. Plant certification evaluates a plant's overall ability to maintain sound production procedures and an effective in-house quality-assurance programme.

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

- Completeness of work orders and product drawings
- Testing and inspection of the materials selected for use
- Accurate manufacturing equipment
- Proportioning and adequate mixing of concrete
- Handling, placement and consolidation of concrete
- Curing
- Dimensions and tolerance control
- Handling, storage, transportation and erection of members

The procedures to be followed for quality control are normally based on ISO 9001 or EN 29.001 standards. Specific quality assurance and certification programmes formulated by precast concrete federations and institutes also exist. The Precast/Prestressed Concrete Institute, USA, has published quality control manuals for plants and the production of precast and prestressed concrete products. The FIP Commission on prefabrication published a guide to good practice on *Quality assurance of hollow core slab floors* [20] in 1992 and the *fib* published *fib* Bulletin 41 *Treatment of imperfections in precast structural elements* [11] in 2007.

1.6 Best practices in precast concrete

Precast concrete structures deliver excellent solutions worldwide. To offer an idea of the immense potential that precast concrete possesses and to inspire enthusiasm for this versatile product, a wide range of precast concrete projects have been listed below:

1.6.1 Residential buildings

Fig. 1.10: A residential project in the Netherlands. The shape, layout and colour of the project illustrate the versatility of precast concrete and its adaptability to the requirements of the builder and the architect.

Fig. 1.11
Left: The restoration of a traditional, classical natural-stone-masonry façade with architectural concrete elements in Brussels. There is no visual difference between the new façade in precast concrete and the traditional neighbouring façades.
Right: A precast single-family house in Finland

Fig. 1.12

Left: Strand Apartment building, London

Right: Apartment building South Street, London

The precasting industry has developed concrete mixes as well as moulding and surface finishing techniques that give concrete units a highly refined aspect. The technique is called architectural concrete to indicate that the material and the way it is produced and applied contribute to the architectural and aesthetic character of the project. What strikes one most in the current conception of buildings is the wide variety in the design of façades. This greater freedom has also affected a change in the choice of the materials used. Instead of internal concrete structures exposed through water washing, sandblasting or other techniques popular in the past, the finishings seen today are reconstructed natural stone, polished concrete and brick or stone veneering, amongst others. Currently some of the key concepts in façade design are aesthetics, the use of natural or traditional materials, flexibility, individualism and comfort.

1.6.2 Office buildings

Fig. 1.13: The headquarters of Decomo, Belgium

Precast concrete is a cost-effective and yet versatile resource in the design and construction of office buildings.

Decomo is a manufacturer of precast architectural concrete façade units in Mouscron, Belgium. The elements of the company's headquarters (Fig. 1.13) are in a light grey, fine-washed or polished concrete and the surface is self-cleaning due to the application of white cement with a titanium dioxide admixture.

The DYB building (Swatch Group) in Cormondréche, Switzerland, has a surprisingly delicate facade. From a distance the building appears powerful and yet airy.

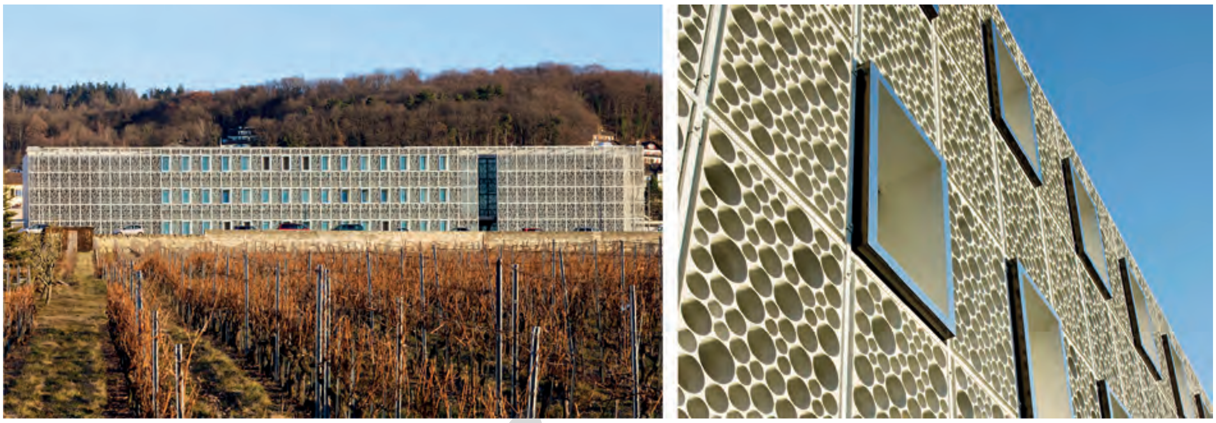


Fig. 1.14: The DYB building (Swatch Group) in Cormondréche, Switzerland

The elongated wing of the building is fronted on its south side by a semi-transparent concrete wall that has a foam-like appearance. The façade appears as a single wall that is, at the same time, a prestigious entrance, a privacy screen and a glare shield. The thickness of the wall and the distance to the glass façade behind it were calculated so that direct sunlight in summer would not enter the building, thus protecting the building from heat, while in winter the rays would penetrate to serve as an additional source of energy.

The façade is assembled from rectangular precast concrete elements that create a repeating pattern. The three-by-15 rectangular cut-outs with mirrored frames act as window openings. Otherwise, the 11-by-74-metre wall only has one narrow slit - the main entrance - as an opening.

A more detailed description of this building can be found in Journal Opus C – Concrete Architecture & Design [48].

Fig. 1.15: A free-standing façade made of precast concrete elements serving as a sun filter, in Commondrèche, Switzerland

The Swift Bank building (Fig. 1.16) in La Hulpe, Belgium, is an entirely precast concrete project with a precast architectural split-wall sandwich façade in a neo-classical style, and architectural wall and ceiling units.

The precast office buildings (Fig. 1.17) at Airport Garden near Brussels airport are ten storeys high and completely precast. The circular columns are in high-strength, self-compacting concrete. The floors are hollow-core slabs with a span of 6.8 to 11.5 metres and the total construction depth of the floor, including the beams and structural topping, is only 405 millimetres.

Fig. 1.16
Left: Precast mullions and ring beams in cruciform units provide the architectural and exterior structure of this office building in London
Centre and right: The Swift Bank building in La Hulpe, Belgium

Fig. 1.17: Precast office buildings at Airport Garden near Brussels airport

Fig. 1.18: Shopping centre, Guimaraes, Portugal. The precast façade panels are finished with a special technique to give them the appearance of weathered corten steel.

1.6.3 High-rise buildings

Fig. 1.19
Left: Galaxy towers, Brussels, has 36 floors
Right: The MG tower in Ghent has 27 floors. The tower buildings are characterized by a cast in-situ central core with a climbing-mould technique, surrounded by a complete precast structure comprising load bearing columns, prestressed floor beams and prestressed floors. The central core comprises all utility functions, while the office spaces are located in the outside section in the precast part of the building.

The buildings in Figure 1.19 illustrate a new development in prefabrication in Belgium. Since 2006 more than 20 projects have adopted the new design. Buildings are generally comprised of two-storey-high circular columns in high-strength, self-compacting concrete (80 N/mm² cylinder strength) cast in closed horizontal moulds. In most of these projects the columns have the same slender cross-section over the complete height of the buildings. The architect very often decides on the maximum column size, which, in many cases, is of only 500 or 600 millimetres. The latter also often establishes the maximum construction depth of the floors, which includes the floor beams. Because of the move away from classic in-situ construction, it has become possible to add one more storey to the maximum building height imposed by the city planning directives in Brussels. The floors are in prestressed TT or hollow-core slabs with a span of 6.80 to 11.50 metres and the total construction depth of the floor, including the beams and 60-millimetre topping, is only 405 millimetres. The erection time was four to five working days per floor.

1.6.4 Hotels

The façades of the hotels in Figure 1.20 are in precast architectural concrete.

The Grand Hotel Djibloho Kempinski and its congress centre are in the city of Oyala, Equatorial Guinea. The entire structure and all the façades of the hotel are in precast concrete. The project was built in the tropical forest but it has already been absorbed by urban expansion.

Fig. 1.20

Top left: A hotel in Scheveningen, Netherlands
Top right: A hotel in the Herault region, France

Left: Grand Hotel Djibloho Kempinski, Equatorial Guinea

1.6.5 Public services buildings

The German embassy (Fig. 1.21) in Warsaw, Poland, is one of the most important German diplomatic representations and also one the largest. Above a plinth made from green precast concrete elements with an ivy relief, sits an overhanging T-shaped structure that houses the consular offices.

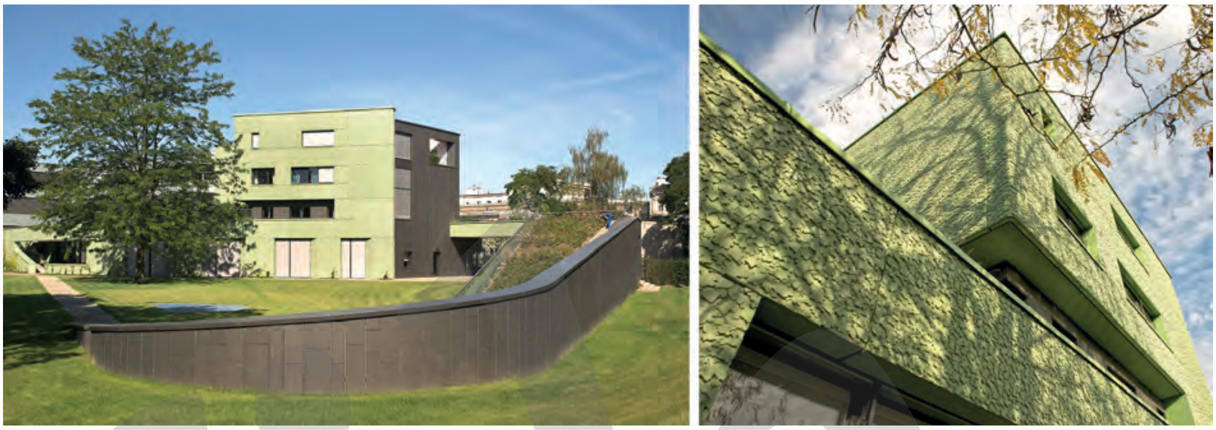


Fig. 1.21: German embassy in Warsaw, with its coloured textured façade panels

Under the consular wing is the residence, which is made up of a reception area and the ambassador's private rooms. It is encased in a façade of green-pigmented precast concrete elements with an ivy relief and rounded corners. The façade anticipates a future plant covering of natural ivy and either the artificial replica or the natural vegetation will dominate in a few years time, depending on the season.

Fig. 1.22: The façade panels were produced using silicone moulds

The single-storey Community Centre (Fig. 1.23) in Mannheim, Germany, has a striking white architectural concrete façade. The form, created using precise, made-to-measure moulds, is inspired by blades of grass and gives the building a uniform appearance. The material is a white as-cast concrete, whose fine-pored surface was treated to be water repellent. The design is the winning entry from a 2003 architectural competition in which 444 consultancies took part.

Fig. 1.23: White precast concrete panels form the façade of the Community Centre in Mannheim, Germany

1.6.6 Educational and cultural buildings

Fig. 1.24: The European School, Uccle, Brussels

The European School (Fig. 1.24) in Uccle, Belgium, is an entirely precast building with load-bearing, precast sandwich-brick-faced façades. The project was initially designed for traditional on-site construction. However, after finishing the foundations in January, the contractor went bankrupt. The only way to meet the deadline for the start of school in September was prefabrication. Even though all the plans had to be redrafted and the production started over, the project was ready on the requested opening day.

The Steinmürli schoolhouse and sports hall (Fig. 1.25) in Dietikon, Switzerland, with its geometrical architectural style and variously textured façade structure suggests both orderliness and an acrobatic lightness.

Fig. 1.25: The structural façades of the Steinmürli schoolhouse in Dietikon, Switzerland

The precast elements in a white-coloured concrete had to be staggered with precision to ensure that the joints and indent patterns were in harmony and that the butt joints created a uniform surface. This paired structural effect gives the Steinmürli schoolhouse in Dietikon an exceptionally attractive appearance. The intermediate negative of the master indent mould simply provides the 'inverted' texture – two textures with one mould. The form liners used consist of polyurethane and are capable of taking extremely fine detail. The sturdy elastic properties of PU allow rounded edges with tight radii and can be used in high numbers of 100 and more studs.

The Institute of Physics building (Fig. 1.26) on the campus of the Chemnitz University of Technology, Germany, has a façade comprised of precast architectural concrete panels in three different shades of grey, whose surfaces are finely washed and hydrophobed.

Fig. 1.26: White cement and dark granite gravel were used for the façade panels of the Institute of Physics at the Chemnitz University of Technology, Germany

The aggregate used was dark granite gravel, with white cement creating light grey shades. The elements, which are 150 millimetres thick, 1.12 or 1.42 metres high respectively and up to 8.80 metres long, emphasise the elongation of the building with their horizontal formats.

There are 30-millimetre-wide, open joints between the rear-ventilated concrete elements, which make the building seem lighter.


Fig. 1.27

Left: The courtyard theatre in Hereford, Sweden, with its precast columns, beams and gradins

Right: Cité de la Musique, Paris, with its wall cladding in architectural concrete panels

1.6.7 Retail buildings


Fig. 1.28
Left: Werre-Park, a shopping centre in Bad Oeynhausen, Germany, has a façade in precast architectural concrete panels
Right: Precast concrete circular columns, edge beams and façade units inside a shopping centre in Leicester UK

The façade of the Alexa department store (Fig. 1.29) in Berlin is a visual highlight of as well as a tremendous engineering challenge in construction. The warm shade of the architectural concrete façade is reminiscent of the clay bricks as used in earlier edifices, for example, in the construction of viaducts for adjacent elevated railroads. With its rear-ventilated curtain wall façade with three-dimensional surface structure, it had to meet the highest architectural standards of excellence. Because of the considerable demands placed on the quality of the architectural concrete and the complex geometric shapes of the forms, the entire façade was conceived as precast units to be installed later as a perfect-fit curtain wall into the skeleton on the construction site.

The surface structure of the architectural concrete façade was not handed over as a shop drawing on paper, as is the custom, but as a three-dimensional CAD data model. Based on the CAD for construction made by the architect, the shapes were manufactured as the reverse image of the shapes of the manufactured façade units. The sample façade for this became the pilot project in the three-dimensional, CNC-supported formwork construction.

Fig. 1.29: Three-dimensional, CNC-supported formwork construction for the Alexa shopping centre in Berlin, Germany

To ensure the durability and serviceability of the architectural concrete surface, the precast units that were later hung at the base of the façade were provided with graffiti protection up to a height of 3 metres. The remaining precast units for the central and upper areas of the façade received a hydrophobic coating, Up to 50 precast units were delivered to the construction site and installed daily.

1.6.8 Sports arenas and stadia

Fig 1.30
Left: José Alvalade Stadium, Lisbon
Right: Ghelamco Stadium, Ghent, Belgium

The José Alvalade Stadium (Fig. 1.30, left) in Lisbon, which also houses a shopping centre with a cinema, a health club, the club's museum, a sports pavilion, a clinic and offices, was built with precast seats and hollow-core floors.

The Ghelamco football stadium (Fig. 1.30 right) in Ghent, Belgium, can seat 20,000 people for matches. The total amount of precast concrete used for the project was 18,000 m³, with 10,360 precast units.

1.6.9 Car parks

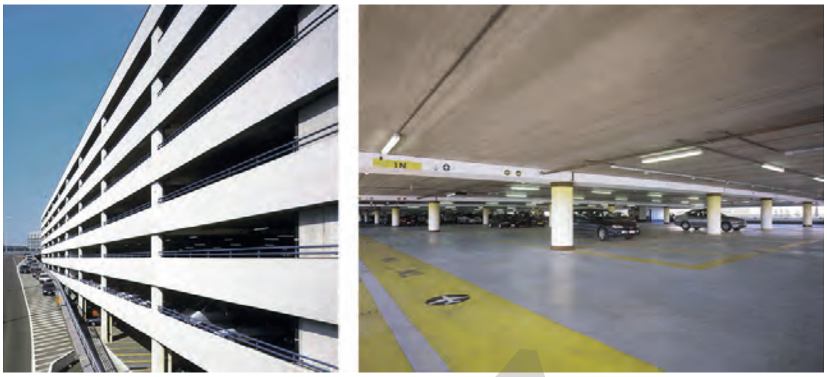


Fig. 1.31: Car park, Brussels Airport, Belgium

Car parks are an interesting market segment for prefabrication because they require large spans and short construction periods. Façades are often made up of spandrel units in architectural concrete. Lift shafts may be finished in cladding panels. Virtually any design can be moulded into the exterior of precast concrete panels.

Fig. 1.32

Left: 164th Street car park, New York, USA in 2010

Right: Orlando Health Centre car park, Florida, USA

1.6.10 Industrial buildings and warehouses

Fig. 1.33: Industrial buildings in Belgium, with a portal frame consisting of rigid founded columns and simply supported rafters

Industrial buildings and warehouses are often built with precast concrete for its short construction time, long span capacities and fire resistance. Two main solutions for roof decking exist: precast concrete units and lightweight profiled steel decking. The first is often used in cold climates because of its load-bearing capacity for snow and ice. It is also applied in hot climates because of its thermal capacity. The second solution is often used in moderate climates because of its low self-weight.

1.6.11 Other structures

Fig. 1.34: The Jubilee Church in Rome, where the 'sails' are composed of double-curvature precast panels in white concrete with marble aggregate, assembled by post-tensioning

The traffic control tower (Fig. 1.35) at Arlanda airport, Sweden, has a tower shaft that was cast using a climbing-mould technique. The surfaces of the superstructure and shaft were finished with architectural concrete cladding. The white circles in the shaft bear calligraphic texts from the work of the writer and pilot Antoine de Saint-Exupéry. The project was original in that the shaft cladding was erected by using the platform of the climbing mould at the end of casting. However, since the platform was at the top of the shaft, the erection sequence had to be top-down instead of the normal down-up. A special type of connection was developed by the precaster, which worked satisfactorily.

Fig. 1.36: 'The Dream'

Fig 1.35: The traffic control tower at Arlanda airport, Sweden

The Dream' (Fig. 1.36) is a 22-metre-high, precast concrete statue built in 2008 to commemorate the closure of the Sutton Manor Colliery near St Helens, UK. It was designed by the Spanish artist Jaume Plensa and manufactured using segmental precast construction.

2 Preliminary design considerations

2.1 Approaches to design

There are several options available to designers as they develop building projects. In all of these the architect, the structural engineer and the services engineer are responsible for all the construction design and detailing - work that can be fully described, specified, measured and competitively tendered for. However, the contractor alone is responsible for construction, under the supervision of the designers, and is not normally required to contribute to the design.

When a more traditional route is taken, the architect is usually responsible for satisfying the user's (client's) functional requirements as well as for the aesthetics of the final structure. In the first design phase, s/he prepares the preliminary plans and elevations with indications on the arrangement of the structural members. In the second phase, the architect collaborates with the structural engineer to finalize the arrangement of the structural members as well as their form and size. They then proceed to the final and detailed design phase of the project, after collaboration with the electromechanical engineer.

Some changes in the design production methodology should be provided for due to the architectural and structural particularities of precast construction, as well as certain economic-technical considerations surrounding it. During the first phase of the design process several decisions have to be made that go beyond the project's functional and aesthetic requirements in order to accommodate prefabrication. From the start of the project and throughout the design process (conceptual design, analysis, dimensioning and verifications) a design team dedicated to prefabrication should be established and should include the precasting engineer, who will ensure that the precaster's input is properly integrated into the overall design.

Another reason why this collaboration is essential is that the precasting industry is continuously improving and adapting its construction techniques to reflect increasing market demands and competition. This is beneficial to the client.

Those involved should bear in mind that when it comes to prefabrication, making any major additions or changes on site can complicate construction considerably. Therefore, all relevant architectural, structural and electromechanical studies should be fully completed beforehand. Nowadays the BIM design approach constitutes a good platform for collaboration, resulting in nearly faultless design.

2.2 Basic design recommendations

Designers should consider the possibilities, restrictions and advantages of precast concrete, and also plan the detailing, manufacturing, transportation, erecting and serviceability stages before deciding that a design is complete. Choosing a good project team and organizing the design routines well are essential.

So as to give qualified guidance to the entire design team, it is recommended that precast concrete organizations make design and production information available to clients, architects, consulting engineers, services engineers and engineers in the other disciplines. In this way, all the parties are made aware of the particular methods they should use in the various phases of the project, which will lead to maximum efficiency and benefits. This is particularly true for manufacturing and erection since many consulting engineers may not be familiar with some of the methods required.

To obtain the best design, designers must plan for a precast structure from the very outset and not merely adapt their design from the traditional cast in-situ method. The following phases of the conceptual design stage should be respected for all to reap the considerable benefits of a precast-concrete solution.

2.2.1 Respecting a design philosophy specific to precasting

One of the most important objectives of this handbook is to explain the design philosophy specific to precast structures since it is the key to efficient and economical construction. (Chapter 3 explains how to follow the basic guidelines). They are:

- Using inherent stabilizing systems
- Ensuring structural integrity
- Adopting large spans and minimizing structural depth

During the conceptual design of the building the design team should aim to develop a structural system characterized by simplicity and morphological clarity, a system that provides short and direct paths for the transmission of the vertical and horizontal actions, and reduces the uncertainties that arise during the modelling, analysis, dimensioning and detailing of the construction.

The structural simplicity, in turn, should be characterized by the uniformity and regularity of its geometrical configuration, both in plan and elevation. The configuration should be compact and clear: in-plan setbacks, such as re-entrant corners and edge recesses, should be avoided or otherwise limited and given special attention.

The correct approach to precast-concrete design is to see the structural frame as a whole and not as an arbitrary set of elements whose interaction is limited to two elements being connected at one time. If the construction is conceived of as a whole, the designer visualizes all the aspects of component design and structural stability simultaneously. The main aspects at the preliminary stage include:

- Structural form
- Frame stability and integrity
- Component selection
- Connection design

These items cannot be dealt with in isolation. For example, the strength of the floor slab that bears gravity load also influences the design of the floor plate that suffers horizontal force, such as wind pressure or weakness due to sway. The frame design is therefore an integrated process in which many of the iterative steps are apparent during the ordinary development of the design, detailing and site erection procedures. This will be dealt with in later sections.

2.2.2 Using standard solutions whenever possible

Standardization is an important cost-saving factor in prefabrication. It enables repetition and builds experience, thereby lowering costs and assuring better quality, reliability and faster execution.

Standardization is applicable in the following areas:

- Modular design, as relevant to flexible design (Section 2.2.f)
- Standard products (Section 2.2.g)
- Internal standards for detailing and working procedures

There is a clear distinction between 'modular coordination' and 'standardization'. The former means the interdependent arrangement of dimensions based on a primary value of 300 millimetres for a module. Standardization means the adoption of these module quantities and configurations without necessarily limiting architectural freedom. This freedom is evident if we compare two buildings adjacent to Vauxhall Bridge in London (Fig. 2.1). Both of them were built using a modular grid of approximately 300 millimetres but the conventional geometry and lack of individuality of the older building on the left compared to the more recent building on the right is striking.

Fig. 2.1: Modular coordination and standardisation at work in examples of past and present use of precast concrete at Vauxhall Bridge, London

2.2.3 Details should be simple

A good design in precast concrete should use details that are as simple as possible. Details that are too elaborate or vulnerable should be avoided. Figure 2.2 shows the use of single and multiple column corbels supporting floor beams, and the connection between floors and supporting beams. The simplicity of the design, manufacture and construction is evident.

Fig. 2.2: Achieving simplicity of design and construction for beam connections (ABCIC Brazil)

2.2.4 Taking dimensional tolerances into account

There will be inevitable differences between the specified dimensions and the actual dimensions of the precast elements and the finished building, some of which are accepted in national codes and standards and are known as permitted deviations. These deviations must be recognized and allowed for. Precast concrete elements are generally manufactured with relatively small deviations, known as tolerances, but designers should nevertheless 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 precaster. Dimensional tolerances are discussed in Section 10.6.

Tolerances occur both at the precasting plant and on site. Production tolerances at the plant include dimensional deviations of the products, their non-linearity, non-flatness, lack of the orthogonality of the cross-section, the camber deviations of prestressed elements, the position of inserts, and so forth. Site tolerances include deviations on the setting out of construction axes and levels. In addition, during erection, deviations will occur with respect to the position and alignment of the elements.

A study carried out by nine European research institutes at twenty factories in 2002 [7] found that the manufacturing accuracy of precast concrete elements, in terms of dimension, the placement of rebars, concrete strength and self-weight, was sufficient to propose reducing partial safety factors by ten per cent for the self-weight and six per cent for the compressive strength of precast concrete.

Information concerning allowable tolerances can be found in specific literature issued by precast concrete federations, national and international product standards, and catalogues from precast element producers.

2.2.5 Taking advantage of the industrialization of the process

Precast concrete production should be based on industrialization. The industrialization is partly influenced by the design, for example:

- Prestressing allows long-line production (Fig. 1.8)
- Standardization of elements and details enables standardization of the process
- Adequate positioning of details, such as waiting bars, decreases labour time
- Simplicity of documents lowers the risk of mistakes
- Last minute modifications disturb production planning, lead to mistakes, and so forth

2.2.6 Modulation is recommendable

Modulation is worthwhile considering for the design and construction of buildings, both for cast in-situ and precast structures. It is already widely used for the structural elements of precast buildings.

Columns could, for example, be positioned centrally on the modular axis grid, at the highest possible spacing to yield the greatest lettable floor area, but remain within the limits of the depth of the beams that span between them (see figure 6.24). For this reason, internal column spacing is often 7.2 to 9.6 metres. Corner and edge columns may be implanted with the grid axis along the face of the column but this solution is recommended less than the previous one. The spacing of the edge columns does not have to be the same as for the internal columns, which is typically 6.0 to 7.2 metres, and is controlled more by the architectural coordination than structural requirements. In the first solution, all beams are of the same length and the gap at the edge of the floor with the façade can be easily filled with strips of cast in-situ concrete or match plates.

Central cores and lift shafts are positioned so that the modular axis in the direction of the floor span coincides with the outside of the core or shaft. In the other direction, the implantation should preferably be such that all floor elements of the bay are of the same length.

The length of floor elements is, in principle, completely open to choice. Modulation is certainly desirable but will have little impact on the cost of the floors. It should also not constitute an obstacle to the architectural concept of the building. Most façades are designed as one-off projects for which new moulds always have to be made.

Modulation in connection with industrial production is certainly not imperative but does have an influence on the cost of the elements. Modulation should always be considered as an aid, not as an obligation.

2.2.7 Standardization of products and processes

The standardization of products and processes is widespread in prefabrication. It constitutes an important economic factor because of the lower costs for moulds, the industrialization of the production process with high productivity, great experience in execution, and so forth. Precast manufacturers have standardized their elements by adopting a range of preferred cross-sections for each type of element.

Standardization is generally limited to details, cross-sectional dimensions and geometry but seldom to the length of the units. Typical standard products for buildings are columns, beams and floor or roof slabs. Standard products are cast in existing moulds. The designer can select the length, dimensions and load bearing capacity within certain limits. This information can be found in catalogues from the precast element producers.

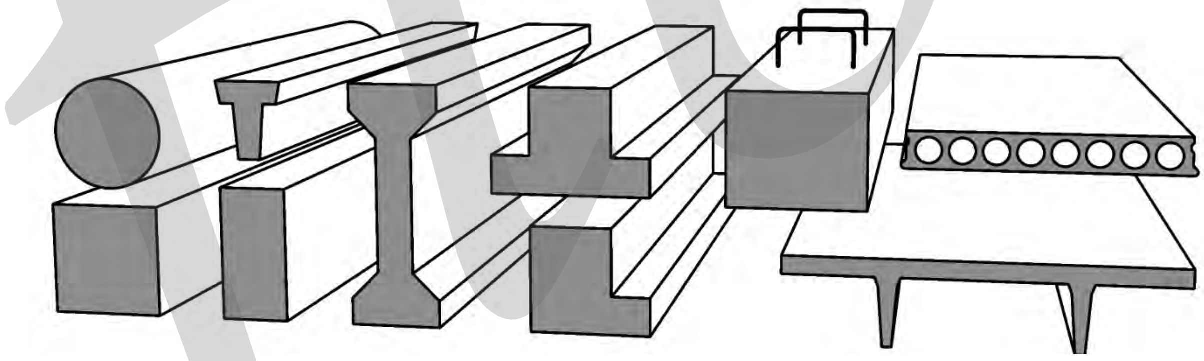


Fig. 2.3: Examples of standard cross-sections

Wall elements usually have standard thicknesses but the height and width is free of restriction, within certain limits. Openings for windows and doors are also normally free of restriction. Façades are always designed individually for each project. Cladding panels for utility buildings are sometimes available in standard dimensions.

Precasters also produce non-standard elements. In addition to the already mentioned façade elements in architectural concrete, the precast industry produces purpose-designed elements, for example stairs and landings, balconies, specially shaped elements, and so forth.

Precasters have developed design routines and organization manuals that help the design staff to elaborate projects. Standardization of, for example, systems, products and connections does not only mean the industrialization of the component production but that the subsequent repeated handling will lead to fewer errors and bad experiences. Standardization also encourages the use of BIM in precast construction. Standard products and details have already been programmed in BIM software to enable the improved standardization of the whole design process.

2.3 Basic conceptual design principles in earthquake regions

Experience from past earthquakes has shown that the behaviour of precast structures that were conceived with basic design principles of the conceptual design in mind adequately satisfied the fundamental requirements of non collapse and damage limitation, within acceptable costs.

In seismic areas the structural engineer should carry out a more in-depth analysis in terms of the loads and structural behaviour of the building. However, at the conceptual design stage it is essential that the following points be considered:

2.3.1 Structural simplicity

The structural simplicity and morphological clarity of a structural system are essential from the early stages of the design of a precast building and provide direct or alternative paths for the transmission of seismic loads. The uncertainties in the modelling, analysis, dimensioning, detailing and prediction of the seismic response of the structure are minimized with a simplified structural system.

Structural simplicity is characterized by the uniformity and regularity in configuration of the structural system in plan and/or elevation (Fig. 2.4).

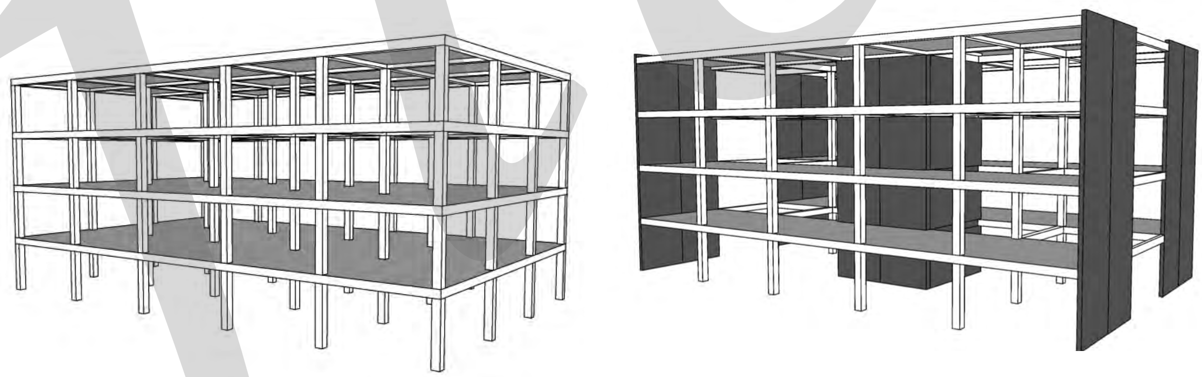


Fig. 2.4: Two schematic examples of structural simplicity [37]

2.3.2 Regularity and uniformity in plan

Regularity in plan is greatly affected by the geometrical configuration of the building, so:

- The configuration should be be compact and clear. Re-entrant corners, edge recesses and non-uniform shapes (L, Π, E, ⌉L) should be avoided (Fig. 2.5) or otherwise limited and given special treatment (Fig. 2.6). In this respect, and based on the size and the shape of the entire structure in plan (imposed by architectural aspects), it may be necessary to subdivide the entire structure into dynamically independent units by means of seismic joints (Fig. 2.6). Special care should also be given to the width of the separation seismic joints in order to avoid a dangerous hammering effect on the individual units.

- The building's maximum length to width ratio should not be higher than 4.

- The distribution of the lateral stiffness and mass should be closely symmetrical in plan with respect to the two orthogonal axes (Fig. 2.7 and 2.8).

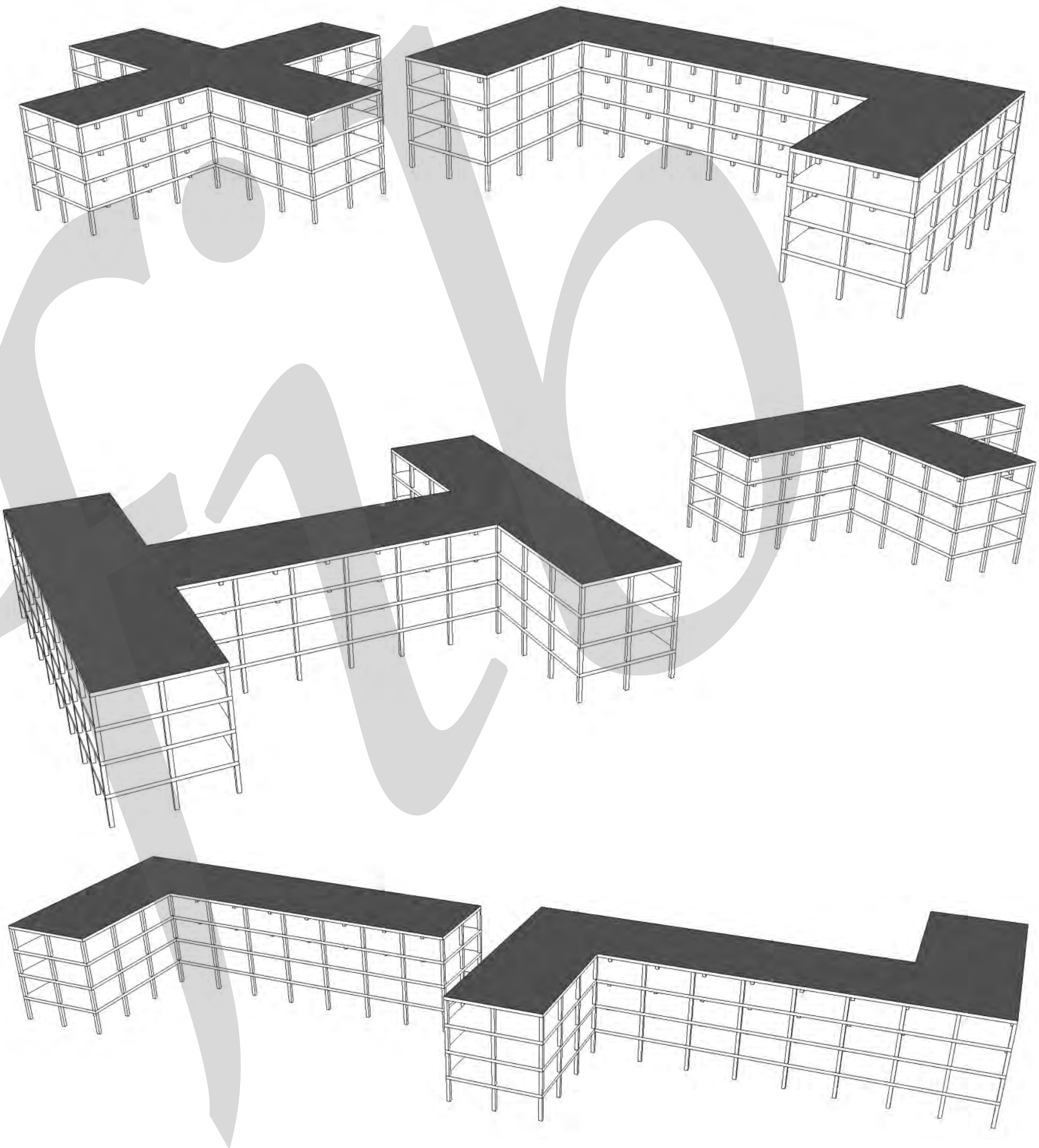


Fig. 2.5: Building configuration in plan to be avoided or limited and given special attention [37]

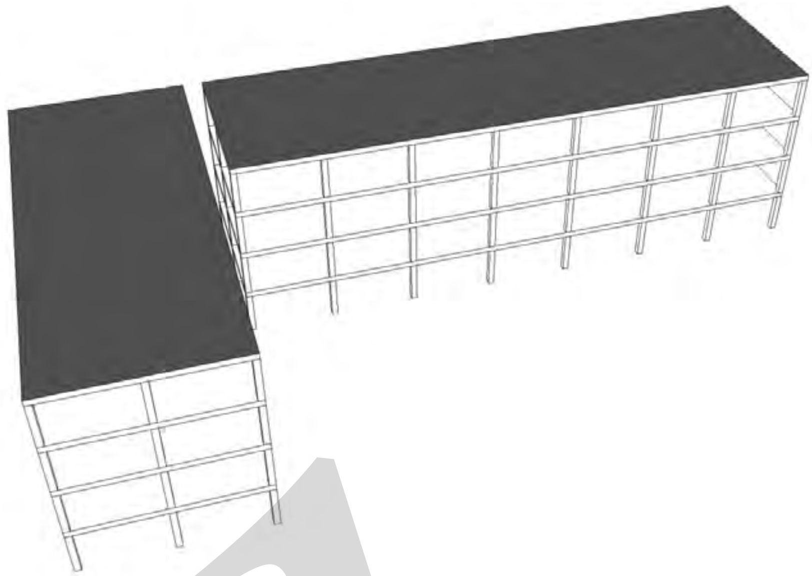


Fig. 2.6: Solution for the re-entrant corner condition through separation by means of seismic joint [37]

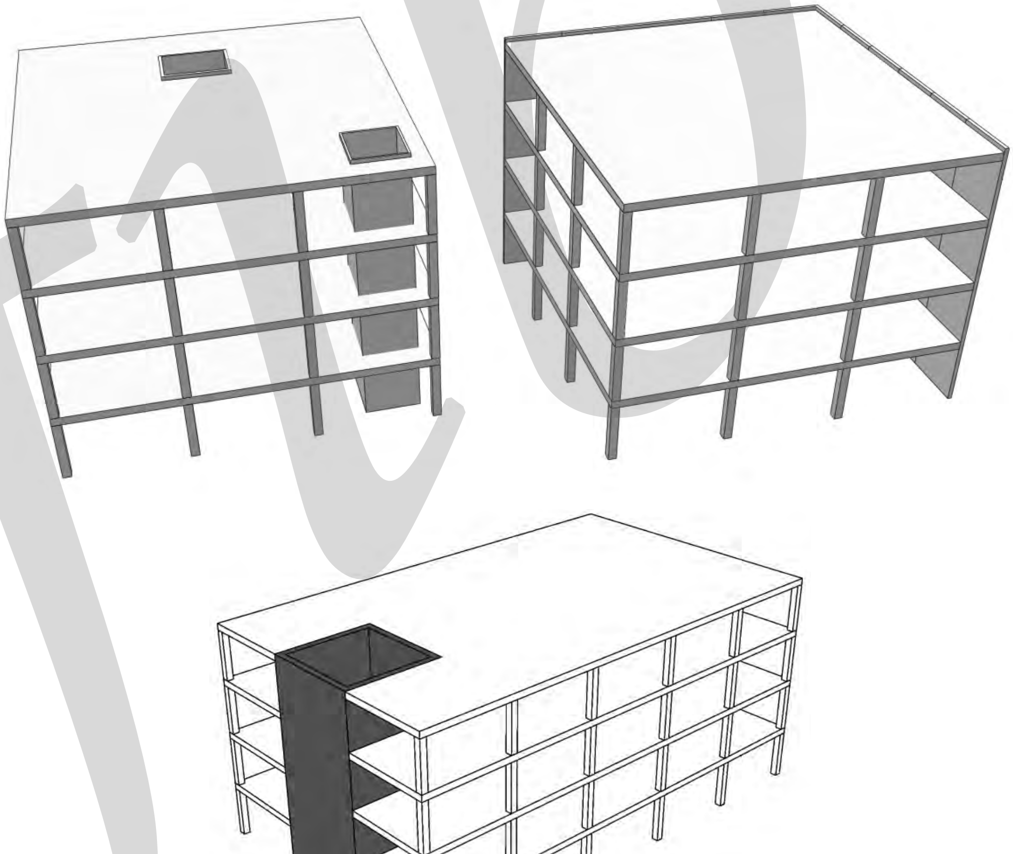


Fig. 2.7: Shear wall configuration should be avoided since such walls are not symmetrical in plan on the two orthogonal axes [37]

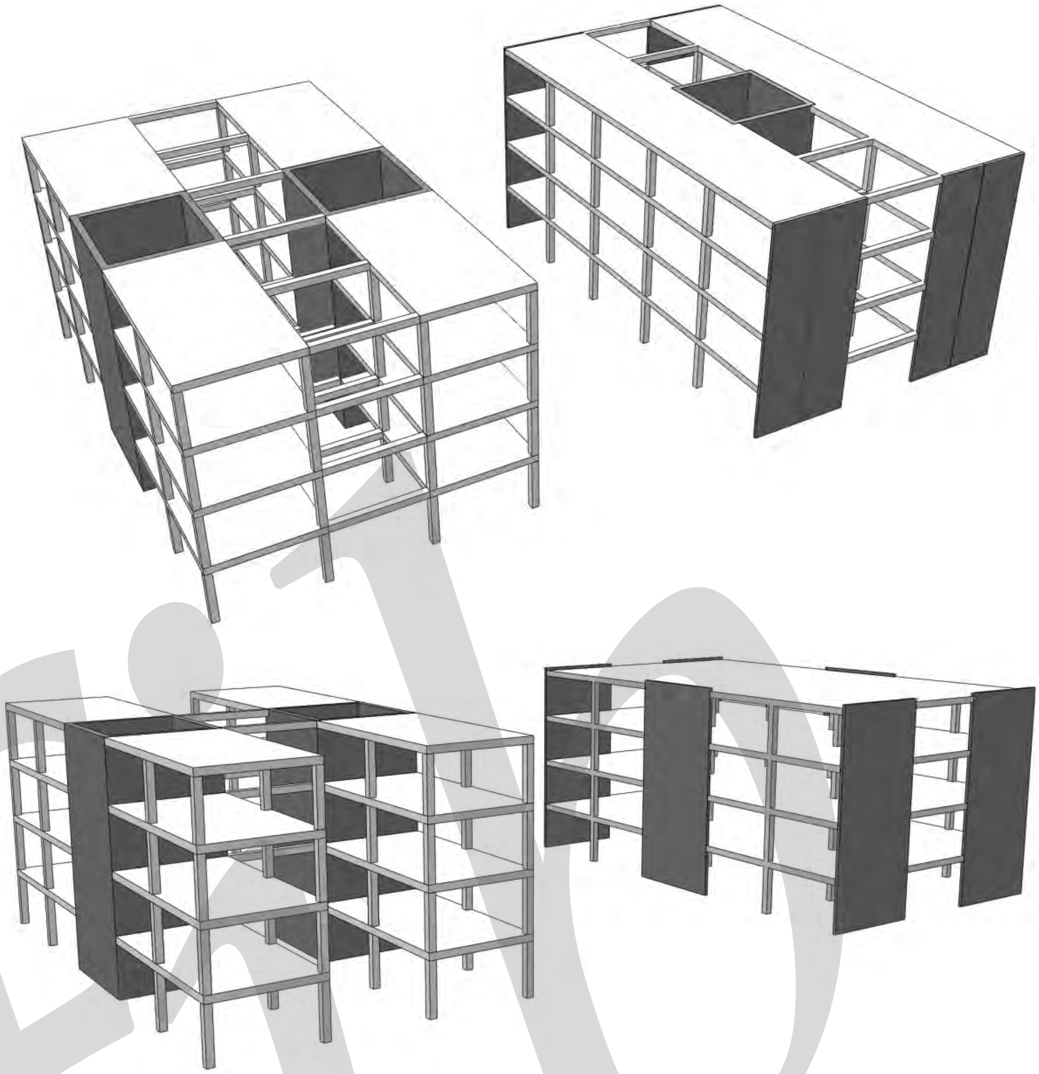


Fig. 2.8: Building examples of the symmetrical distribution in plan of the lateral stiffness and mass [37]

Uniformity/regularity in plan:

- leads to the uniform distribution of the structural elements in plan, which contributes in turn to a better mass and stiffness distribution of the structural system
- increases redundancy and contributes to well-distributed energy dissipation across the entire structure
- reduces possible torsional effects, a short distance between the centre of mass and the centre of rigidity being the best way to avoid undesirable torsional effects

2.3.3 Regularity and uniformity in height

For a building to be characterized as regular in elevation, the following design guidelines should be taken into consideration:

- Almost 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 (Fig. 2.9)
- Both the lateral stiffness and the mass of the individual storeys should remain constant or reduce gradually
- A natural flow of forces should be ensured by avoiding staggered beams, or worse, staggered columns

The analysis and observation of structures damaged in earthquakes have shown that degradation at times resulted from a disregard for the basic principles of conceptual design. Where possible, the following should be avoided:

- Soft-storey ground floors (Fig. 2.9 and 2.10)
- Soft-storey upper floors (Fig. 2.9)
- Asymmetric bracing
- Discontinuities in stiffness and resistance (Fig. 2.10)
- Short columns (Fig. 2.12)

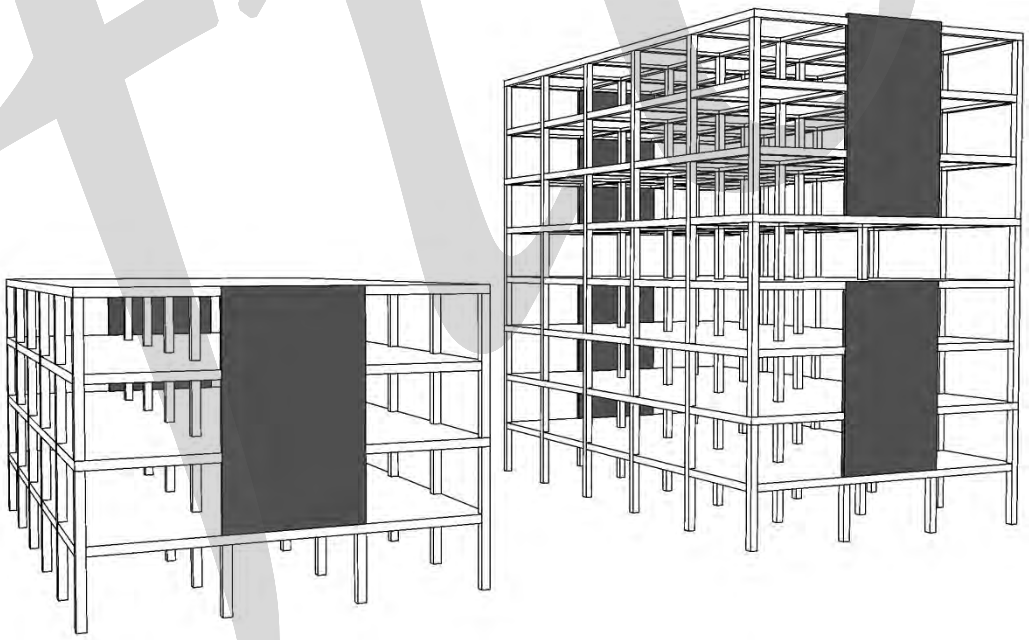


Fig. 2.9: Soft-storey ground floors and soft-storey upper floors, which should be avoided [37]

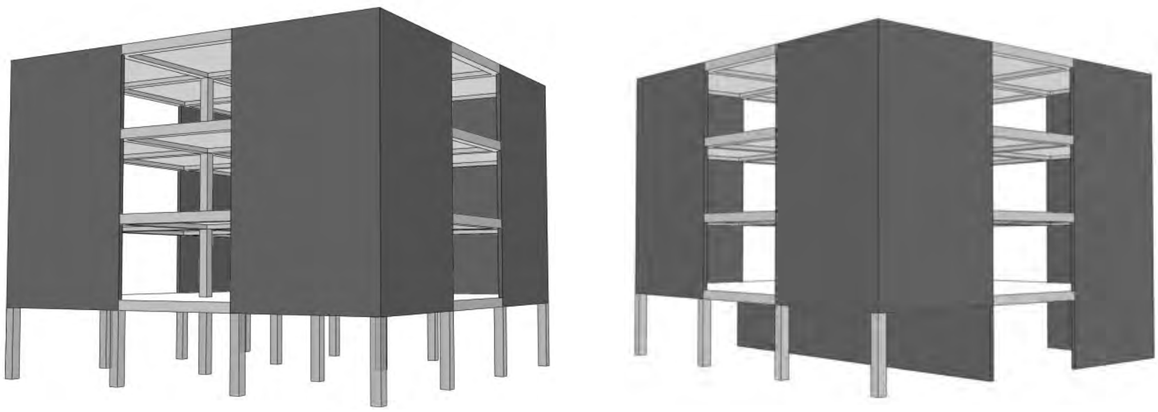


Fig. 2.10: Discontinuities in stiffness and resistance in height, which should be avoided [37]

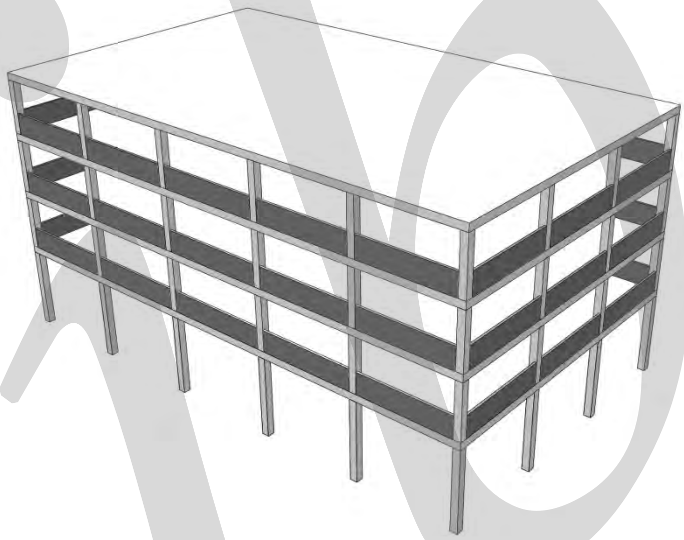


Fig. 2.11: Short columns [37]

2.3.4 Bi-directional resistance, torsional resistance and stiffness

Each structural system should be able to resist seismic actions in both directions. This can be obtained by the proper arrangement of the structural elements (columns and/or walls) in an orthogonal grid, which provides quasi-similar resistance and stiffness in both directions (Fig. 2.8). Torsional irregularities should be avoided as much as possible (Fig. 2.7).

In all cases, special care should be given to the position of the elevators and staircases in the structural system. These parts of the building, which are usually provided by structural walls around their area (in order to prevent their collapse and ensure rapid evacuation after an adverse seismic event), affect the torsional behaviour of the structural system. This leads to regions of significantly large tensile stresses in the floor connections, depending on their position in the layout and the arrangement of the other vertical structural elements (Fig. 2.7).

2.3.5 Adequate and secure connections in precast buildings

The connections in precast buildings are often designed for construction and imposed gravity loads only. Such buildings can therefore be vulnerable to earthquakes since they lack ductility and the ability to absorb deformations without failure. Short support lengths, weak or missing dowels, and unsatisfactory overturning restraints of girders are frequently the cause of collapse. Therefore, mobile bearings must have a minimum support length in accordance with seismic building codes and fixed bearings must have dowels designed for the forces that account for the over-strength of plastic zones (the so-called capacity design method). Additionally, the beams should usually be secured against lateral overturning movement. In the case of precast floors, adequately reinforced in-situ concrete must cover and connect the floor elements in order to guarantee the horizontal floor plate action, known as diaphragm action. More especially, in precast one-storey frame buildings, such as industrial buildings, columns should be designed to be connected at the top by beams and/or roof elements in both directions.

2.3.6 Adequate foundation

In seismic situations, the interaction of the soil with the superstructure should be carefully studied. Generally, the design and construction of the foundations should ensure that the whole building is subjected to uniform seismic excitation. In the particular case of precast frame systems, the foundation is usually provided by individual footings.

With the exception of rocky soil conditions, the following rules enable the satisfactory behaviour of the structure:

- Tie-beams should be able to act in tension and compression should be provided between individual elements in both main directions
- Foundations laid at different levels should be avoided (Fig. 2.12) and when this is not possible, their common horizontal displacement should be ensured

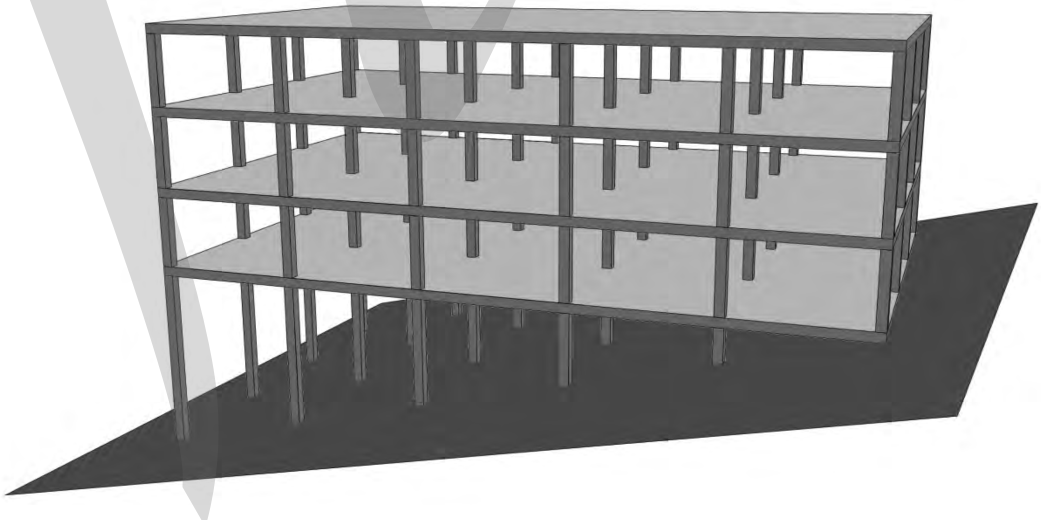


Fig. 2.12: Buildings with different levels of foundation (should be avoided) [37]

2.3.7 Effects of the contribution of infills, partitions and claddings

When designing precast structures, and particularly frame systems for the interaction of secondary elements such as infills, partitions and claddings, particular attention should be paid to the seismically induced irregularity (especially in ground floors) caused by these secondary elements. Appropriate measures should be taken:

- Generally, secondary elements have to be connected to structural elements so that, during a seismic event, they respond as planned for in the structure's design.
- In the particular case of masonry infill walls, which, as a rule are in contact with the columns, the possible shear failure of these columns under shear forces induced by the diagonal strut action of the masonry infill should be taken into account. In addition, short column effects, which may develop in the columns of the structural system due to the configuration of masonry infills, should also be taken into account.
- The contribution of the infill walls and panels to the lateral resistance of the structural system is generally neglected in analyses. Nevertheless, particular attention should be given to the mechanical characteristics of the infill and its connections with structural members. Sometimes, depending on the infill's nature and connections, it may alter the expected behaviour of the structural system if its contribution to the infill walls is neglected.
- The infill's possible torsional effects on structural response depend on its nature and position in the structure, which should be taken into account.
- The possible consequences of the irregularity in plan and in elevation of the infills, partitions or claddings should be taken into account.
- Appropriate measures should be taken to avoid brittle failure of the secondary elements mentioned above and special care should be given to the detailing of the connections of cladding panels to the structural elements in order to avoid partial or total out-of-plane collapse of the cladding panels. Thus, in order to avoid hazardous situations, the connections of cladding panes to the structural elements must be designed and detailed not only for gravity loads but also for horizontal cyclic loads. Additionally, the connections to the structural elements and between the façade elements should be able to follow the expected deformations of the structure.

Experience from past earthquakes has shown that inadequate connection details between cladding panels (mostly of rather rigid sandwich type) and the corresponding structural elements lead to the out-of-plane collapse of these cladding panels during the seismic event, putting human life at considerable risk.

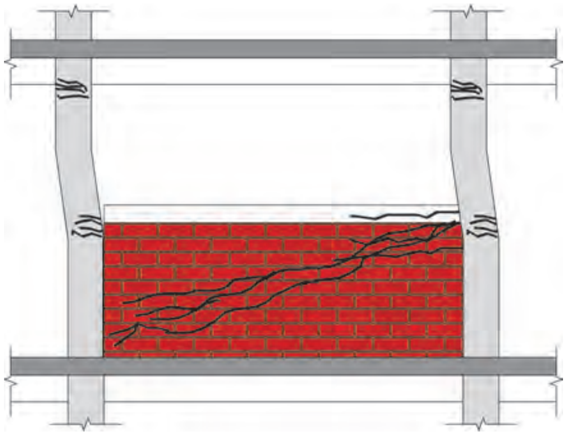


Fig. 2.13: Shear failure of columns under shear forces induced by the inadequate arrangement of infill wall

Fig. 2.14: Out-of-plane collapse of cladding panels during the L'Aquila earthquake, 2009 [49], due to inadequate connections of cladding to the bearing members

The connections of cladding panels to the structural elements must be designed and detailed not only for gravity loads but also for horizontal cyclic loads. Additionally, the connections to the structural elements and between the façade elements should be able to follow the expected deformations of the structure.

2.4 Schematic design in initial stage

As soon as the requirements of the project are fixed, and a general idea of the building has been shaped, the following stages in the elaboration of the schematic design are recommended. They are exemplified in an illustration of a simple office building, given in Figure 2.15.

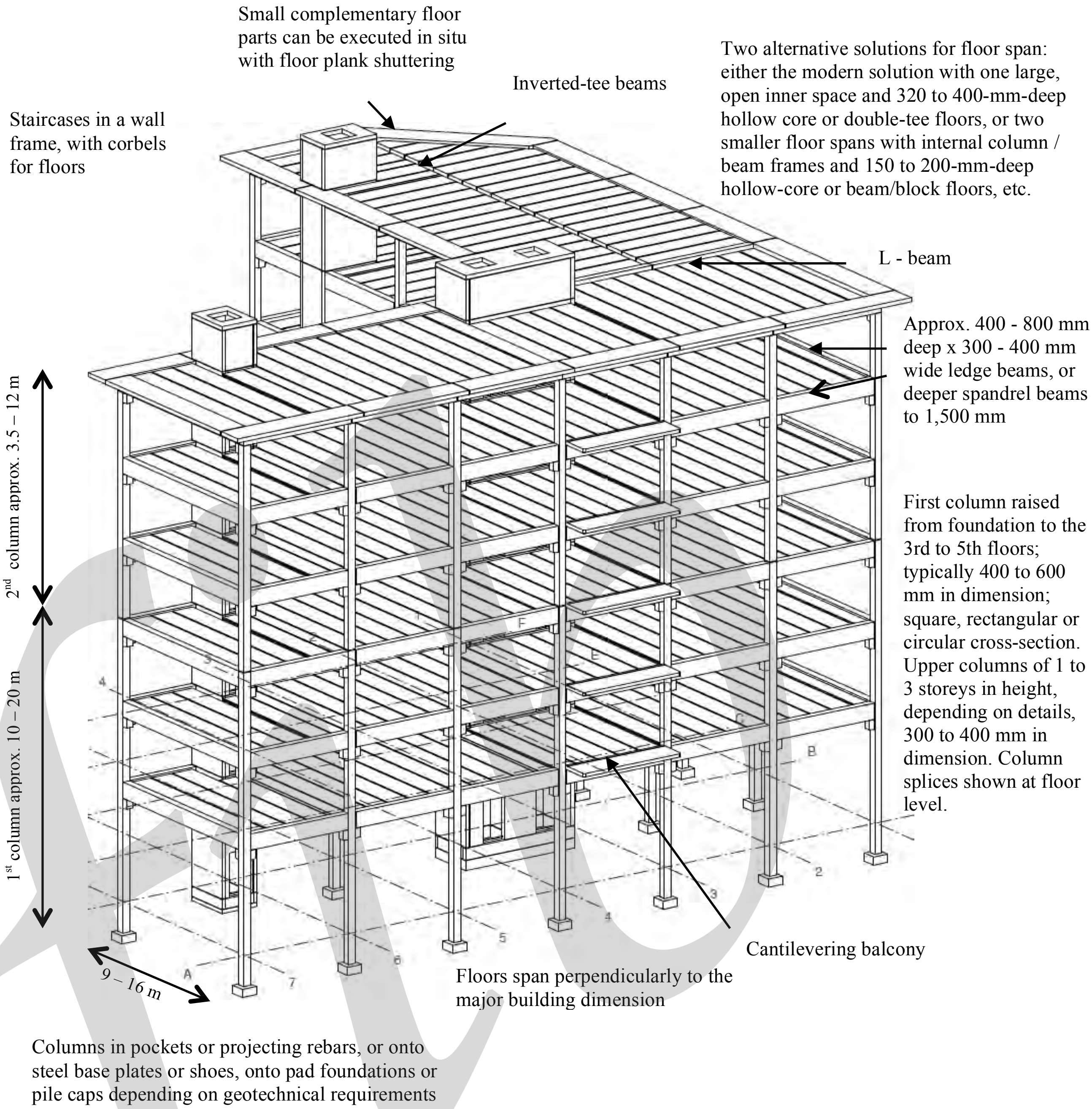


Fig. 2.15: Example of possible floor layouts for an office building

Below are the stages in the elaboration of the schematic design:

- *First step:* General layout of the floor plan and vertical and horizontal circulation
  In the example of Figure 2.15, the office spaces run along the façades, with a corridor alongside the staircases or the central line of the columns.

- *Second step:* Selection of the precast structural system
  Information and guidelines are given in Chapter 3 and throughout the rest of the handbook.

- *Third step:* Choice of the column grid and floor span
  Guidelines are given in Sections 2.2, 7.3 and 8.5.

- *Fourth step:* Choice and implantation of the stabilising components
  In Figure 2.15, horizontal stability is obtained by lift shafts. Detailed guidelines are given in Chapter 4.

- *Fifth step:* Choice and preliminary dimensioning of the precast beam and floor units
  Here the column cross-sections are 400 to 600 millimetres, the beams are spandrel units and inverted T and L-beams chosen to reduce the overall construction depth of the floor; the floors are prestressed hollow-core units typically 200 to 400 millimetres in thickness. Information about the preliminary design of precast elements with regard to load and span is available in the catalogues and technical brochures of the precasters.

- *Sixth step:* Choice of the façade cladding

2.5 Selection of a structural precast system

2.5.1 General

In the precast concrete industry, the notion of 'structural system' is often considered to be an element of business competitiveness. Manufacturers claim to have their own specific systems that offer the greatest benefits to clients.

This is often a matter of confusion for and a source of distrust in designers who are not familiar with precast concrete and consider it an insurmountable handicap to design. However, the reality is, in most cases, perfectly simple. In the building market alone, a large number of commercial solutions for more or less completely precast buildings can be found but they all have a limited number of basic structural systems, of which the design principles are all identical.

Consequently, the designer does not need to know all the existing commercial systems to design a project in precast concrete but only the basic principles of the structural systems. This knowledge is provided in the handbook. Certain details are not covered but they are not essential to the initial design stage. The precasting engineer can offer assistance with further issues at the final stage of the project's elaboration.

The most common basic precast-concrete structural systems are:

- Portal and skeletal systems
- Bearing wall systems
- Façade systems
- Floors and roofs
- Cell systems

Several of these systems can be combined in the same precast building. In the following sections, some general guidelines are given with respect to the choice of system. More detailed information about the various systems is given throughout this handbook.

2.5.2 Portal and skeletal systems

Portal and skeletal structural systems are greatly suited to buildings that need a high degree of flexibility, mainly because they make it possible to adopt large spans and to achieve open spaces without interfering walls. This is very important in industrial buildings, shopping centres, parking structures, sports facilities and large office buildings but is also valued today in residential buildings. The skeletal structural concept also offers greater freedom in the planning and disposition of floor areas since it eliminates the issue of load-bearing walls.

Since the load-bearing system of skeletal structures is normally independent of the complementary sub-systems of the building (mechanical, electrical, ducting and partition walls, and so forth) the buildings are easily adaptable to changes in use and function and to technical innovations.

The skeletal concept also gives a large degree of freedom to the architect in the choice of façade cladding. The individual structural units are well suited to rational production and erection processes.

Detailed information about frame and skeletal systems is given in Chapter 6.

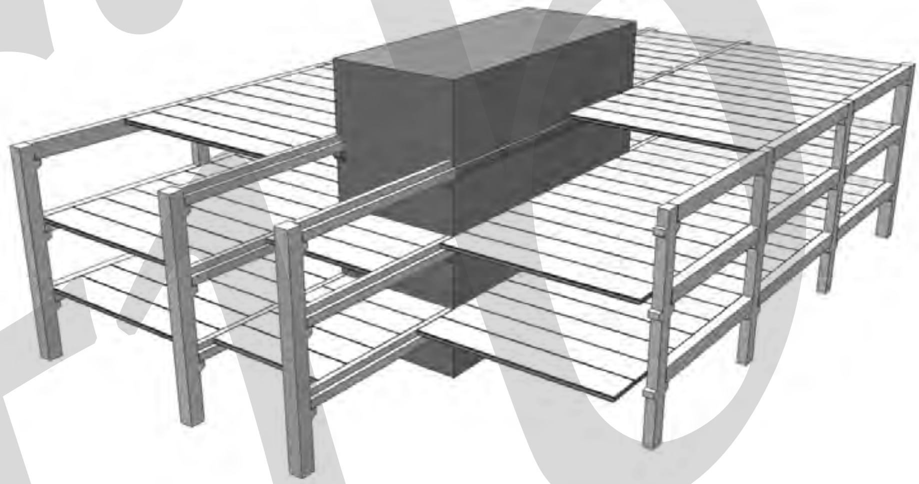


Fig.2.16: Outline of large, open skeletal structure

2.5.3 Bearing-wall systems

Precast bearing walls can appear as cross-walls, walls in shafts and cores, and load-bearing façades. Precast cross-wall systems are mostly used in domestic construction, both for individual housing and for apartments. It is based on classical buildings with brick or block masonry walls.

Precast walls offer the advantages of speedy construction, ready-to-paint surface finishing, acoustic insulation and fire resistance.

Modern systems belong to the so-called open-construction technique, which means that the architect is free to design the project according to the requirements of the client. The trend is to build free, open spaces between the load-bearing walls and to use light partition walls for the internal layout. This makes it possible to change the interior layout without major costs.

Detailed information about load-bearing-wall systems is given in Chapter 7.

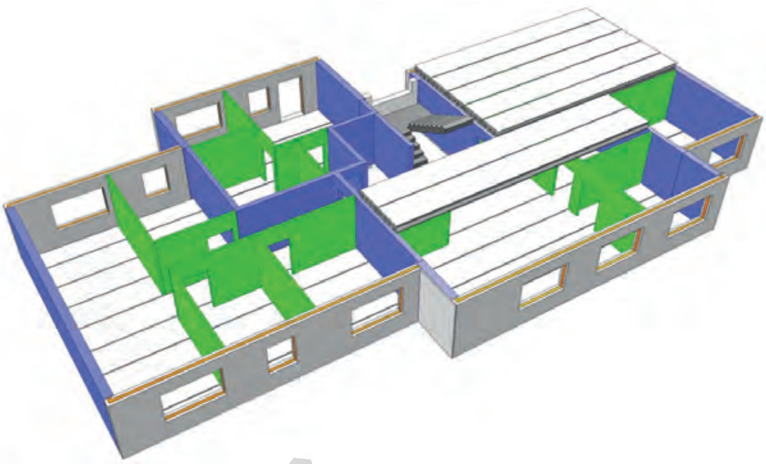


Fig.2.17: Apartment building with load-bearing cross-walls and large floor spans

2.5.4 Façade systems

Precast façades are suitable for any type of building. They can easily be executed in a great variety of colours other than concrete grey and can be designed as load-bearing or simply cladding units.

Load-bearing façades have a dual function in that they are both decorative and structural. They support the vertical loads from the floors and the structure above. The system constitutes an economical solution since it dispenses with the need for external columns, beams and shear walls. Another advantage of load-bearing façades is that indoor conditions are achieved at an early stage on the building site.

The façades are often used in combination with skeleton structures. The internal structure is composed of columns and beams. The modern trend is to build offices without internal columns. The precast floors span from one façade to the other, over a length of 16 to 18 metres (Fig. 2.18).

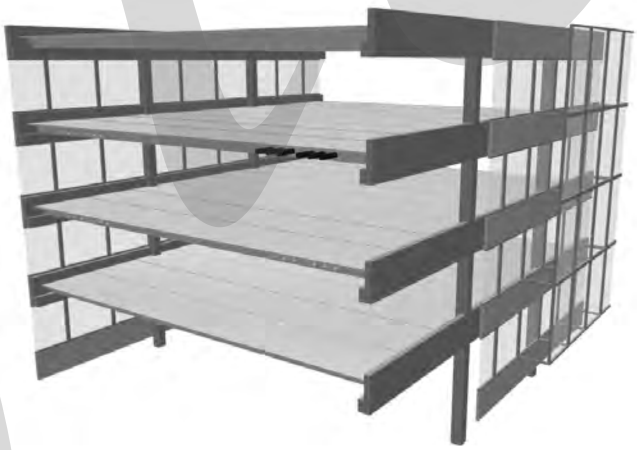


Fig. 2.18: Schematic view of a modern construction concept with large, open internal spaces

Non-load bearing façade panels perform a decorative and enclosing function only. They are fixed to the building structure, which can be either in precast concrete, cast in-situ concrete or steel.

Detailed information about architectural concrete façade systems is given in Chapter 9.

Fig.2.19: Outline of façade with non-load-bearing, precast-concrete cladding

2.5.5 Floors and roofs

Precast floors are one of the oldest precast products. The market offers a large variety of precast floor and roof systems, from which two main types can be distinguished: hollow core floors and ribbed soffit floors. Precast floors offer the following main advantages: speed of construction, the absence of scaffolding, a large variety of types, a large span capacity and economy.

Precast floors are used extensively in all types of buildings. The choice of a flooring system varies in each building and from country to country and depends on transport and lifting facilities, availability in the local market, building culture, and so forth.

There are different types of roof elements, such as ribbed units and saddle lightweight slabs, among others. Concrete roof elements are mainly used, for instance, for industrial and commercial buildings, warehouses and sport halls.

2.5.6 Cell systems

Cell units are mainly used for certain parts of a building, for example, bathrooms and kitchens, and occasionally for complete housing structures, hotels, prisons, and so forth. The advantage of the system lies in the speed of construction and the industrialization of the manufacture, since the finishing and equipment of the cells are executed entirely at the precasting plant.

Fig. 2.20: Precast concrete cells

Left: Bathroom cell

Right: Electrical cabin

2.6 Mixed precast construction

The term 'mixed' is used to describe a type of construction where precast concrete is used in combination with other building products, such as cast in-situ concrete, steel, masonry or timber. The term must not be confused with 'composite' construction, which also uses both precast and another material, but where structural performance relies on the interaction between the two.

Precast concrete systems are compatible with most other forms of construction such as in-situ concrete, steelwork frames, masonry walls, steel or timber roofs, façade cladding in other materials, and so forth. Precast floors, roofs and façades are often combined with steel frames or cast in-situ concrete.

Figure 2.21 shows a common solution that uses concrete filled tubular steel columns and prefabricated and rolled steel beams to support long-span, prestressed concrete floors on the bottom ledge of the steel beam. The floor-span depth ratio for this type of construction is around 40.

Structural masonry can be combined with precast floors and roofs but is seldom combined with a precast concrete frame because the frame is normally erected too quickly for the load-bearing masonry to keep pace. A deformation compatibilities issue may also arise at times since frame systems are sensitive to lateral displacements (due to horizontal loading), depending on the height of the structure. Masonry is, however, sometimes used to provide infill walls.

It is common for in-situ concrete to be used solely in the foundations and in parts of the substructure, for example, in access ramps and retaining walls in underground car parks. Structural compatibility is seldom a problem. Apart from considerations of stability, it is the design and construction of the joints that require the greatest attention.

Joining precast to in-situ concrete demands particular accuracy in the in-situ work because of smaller tolerances in the precast units. There is more latitude in joining in-situ to precast concrete because inaccuracies can be taken up in the in-situ concrete work.

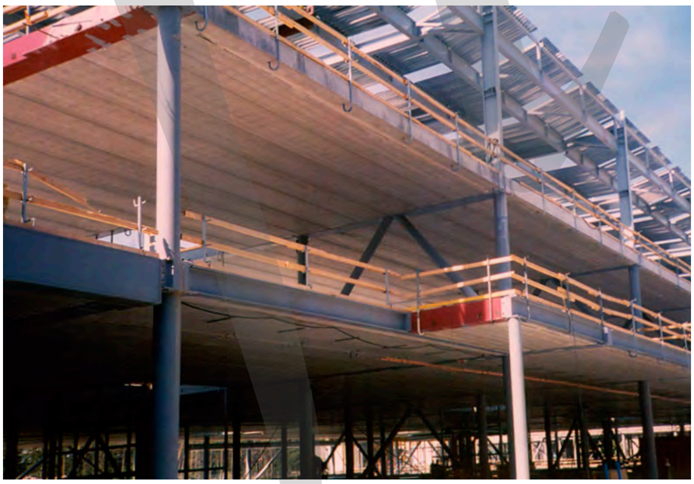


Fig. 2.21: Mixed steelwork and precast concrete at the Big Apple retail centre near Helsinki, Finland

Detailed information about mixed precast systems is given in *fib* Bulletin 19 *Precast concrete in mixed construction* [10].

2.7 Review of precast concrete structural systems

The following table summarizes the different structural options available for various types of buildings and lists their relative attributes.

Table 2.1: Review of precast concrete structural systems and applications

Type of building	**PRECAST STRUCTURAL SYSTEM**			
	Skeletal structure	**Wall frame structure**	**Loadbearing façades**	**Mixed construction**
Houses and apartments	• Gives flexible layout but beams are inside ceiling space	• Suitable for low and medium-rise buildings	• Minimizes structure • Good acoustics • Free choice of façade design	• Generally suited to multi-use buildings
Offices	• Large open spaces; offers flexibility. • Free choice of façade	• Suitable for low-rise buildings and upper levels with lightweight roofs	• Large open spaces • Offers flexibility • Often combined with skeletal frames	• Suited to steel frame system • Façade flexibility
Hotels and hospitals	• Offers flexibility • Free choice of façade	• Suitable for areas with many walls	• Economical solution • Often combined with load-bearing walls or skeletal frames	• Suited to steel frame system • Façade flexibility
Educational buildings	• Large open spaces • Offers flexibility • Free choice of façade	• Suitable for low-rise buildings and upper levels with lightweight roofs	• Large open spaces • Offers flexibility • Often combined with skeletal frames	• Suited to steel frame or loadbearing masonry system
Sports halls and gymnasiums	• Large open spaces • Offers flexibility • Free choice of façade	• Suitable for single-storey buildings and upper levels with lightweight roofs		
Industrial buildings	• Large open spaces • With/without intermediate floors • Lightweight roof • Simple façades			• Suitable for intermediate floors with/without high floor loading
Shopping centres	• Large open spaces • Offers flexibility • Free choice of internal layout	• Suitable for single-storey buildings and upper levels with lightweight roofs	• Often combined with skeletal frames	
Car parks	• Large open spaces and slender floor structure			• Suited to steel frame or loadbearing masonry system
Grandstands	• Complex building structure • Free choice of layout			• Suitable for upper level seating areas using steel beams
General performances, such as fire resistance, are valid for all types of buildings and, therefore, not mentioned				

3 Precast building systems

3.1 Introduction

Every construction material and system has its own characteristics which to a greater or lesser extent influences 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 concept as a cast in-situ one. However, this is a wrong approach and one which is very labour intensive and costly.

If all the advantages of precast concrete 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 precast concrete.

3.2 Structural systems

It would seem that there is a large number of technical systems and solutions for precast buildings in the precast concrete industry; however, they all belong to a limited number of basic structural systems, of which the design principles are more or less identical. The most common basic types of precast concrete structural systems are:

- The portal frame, which consists of columns and roof beams, is used for industrial manufacturing facilities, warehouses, commercial buildings, and so forth.
- The skeletal structure, with columns, beams and slabs for low to medium-rise buildings and with a small number of walls for high rises. Skeletal frames are used chiefly for offices, schools, hospitals and car parks, amongst others.
- The wall frame, which consists of vertical load-bearing wall and horizontal slab units, is used extensively for houses and apartments, hotels, schools, etc.
- Cell structures composed of completely precast concrete cells are sometimes used for bathrooms, kitchens and garage boxes, amongst others. In the past, the system has been used sporadically for complete buildings, for example, for hotels, prisons and similar buildings.

The above structural systems are completed with a number of complementary precast systems for the creation of floors, roofs and façades:

- An extended range of precast floor elements and systems exist. The finished floor structure is able to distribute vertical concentrated loading and to transfer horizontal forces to the stabilizing components. Precast floors are extensively used for all types of buildings and with different structural frame materials.

- Roof structures can be set up with the same floor units but there are also a number of specific roof products in precast concrete. These are lighter than normal floor units but usually have larger spans and widths.

- A large variety of possible façades exist. The architect often plays an important role in shaping the façade, which may consist of single or double skin units, with or without a structural function. They appear in all sorts of shapes and executions, from simple cladding to the most luxurious applications in architectural concrete for offices and prestigious façades.

Several of the systems can be combined in the same precast building. In the following section, general guidelines are given with respect to the choice of the system.

3.2.1 Portal-frame systems

Portal-frame systems consist of columns and beams of different shapes and sizes, combined to form the skeleton of a building. A portal frame is composed of two or more columns that are clamped into the foundations. They support the roof girders. The columns in the raftered portal frame in figure 3.1 have rigid and moment-resisting foundations, such that the frame sways in a vertical cantilever action from the foundation to the eaves, where pinned jointed rafters are simply supported over a span of typically 15 to 40 metres. The complete skeleton of the building is produced through a series of successive portal frames that support the roof and wall cladding.

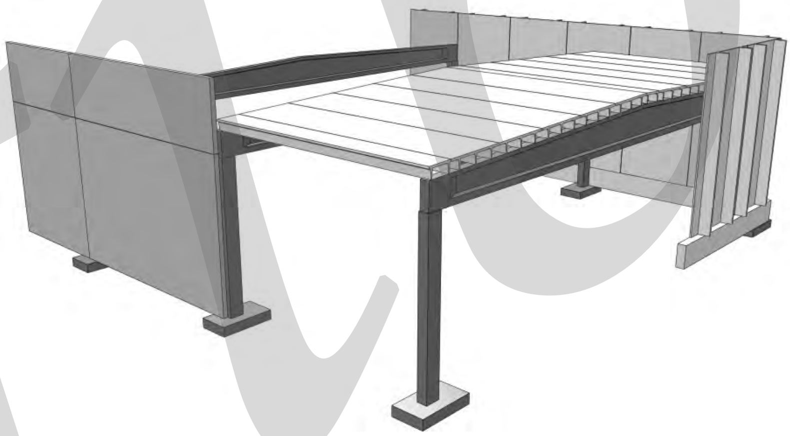


Fig.3.1: Precast portal frame system with columns and prestressed rafters

In buildings basically constructed as single-storey structures, it is possible to insert intermediate floors in some parts of the building or over the whole surface. This is commonly achieved by adding a separate beam/column assembly to carry the intermediate floor slabs.

Fig. 3.2: Portal frame with intermediate floor

3.2.2 Skeletal frame systems

3.2.1.1 General

Precast skeletal structures consist of rectangular or circular columns and rectangular or inverted-tee-shaped beams that are assembled and connected to form a robust structure by the use of small but ductile strips of cast in-situ infill between the precast elements. The latter are called tie-beams and can support and transfer vertical and horizontal actions from the floors and façades to the foundations. Skeletal structures are most commonly used for low to medium-rise buildings, with about 90 per cent being between 3 and 10 storeys high, where concrete compressive strength is about 50 N/mm^2. However, recent developments in high-strength concrete, defined as up to 100 N/mm^2, have enabled precast columns to compete, and beat, equivalent structural steel universal columns in buildings up to 30 to 40 storeys high, when the combined advantages of strength, section size, self-finish and fire resistance are all taken into account.

3.2.1.2 Low and medium-rise building

For buildings of up to 3 or 4 storeys high, the structural system is based on the cantilever action of the columns, which are clamped into the foundations. They are normally one single element of up to approximately 12 to 20 metres in height for column depths of 300 to 500 millimetres respectively.

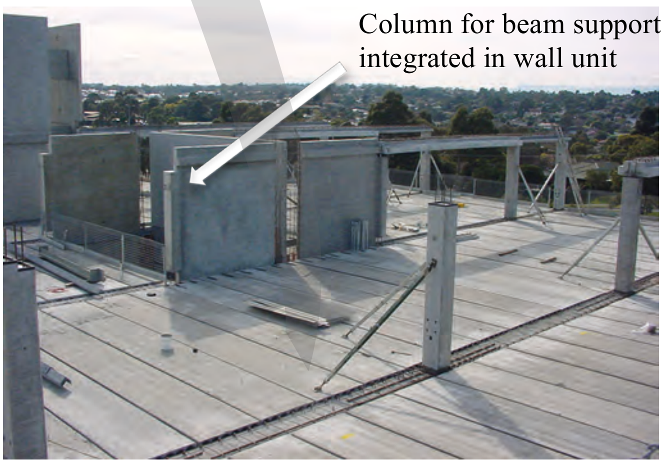


Fig. 3.3: Precast skeletal frame braced by shear walls and elevator shaft

For multi-storey skeletal structures, braced systems are the most effective solution irrespective of the number of storeys. The horizontal stiffness is provided by staircases, elevator shafts and shear walls. In this way, connection details and the design and construction of foundations are greatly simplified. Central cores can be cast in-situ or precast.

3.2.1.3 High rise buildings

Precast tower buildings are generally characterized by a cast in-situ central core surrounded by a complete precast structure comprising load-bearing columns, prestressed floor beams and prestressed floors. A typical recent development is that the architectural layout of floor plans features all kinds of non-orthogonal shapes: ellipses, circles, sharp-edged shapes, and so forth. This is a clear tendency in the current market.

The skeletal structure is often built with circular columns in high-strength, self-compacting concrete with a compressive cylinder/cube strength of 80/95 N/mm². The floor beams have an L-shaped or inverted-T-shaped cross-section with a slender-booth (i.e. a downstand below the floor slab) height, the latter varying in most projects from 80 to 200 millimetres. The 80-millimetre boot is designed as a composite-bearing nib with a steel angle anchored in the prestressed beam and covered by a 70-millimetre-thick concrete layer for fire protection. Boot depths of around 200 millimetres are designed as normal reinforced-concrete nibs. Two types of precast-prestressed floors are used in tower buildings: hollow-core slabs and ribbed floors. Both systems have specific advantages and the choice often depends on specific features within the projects.

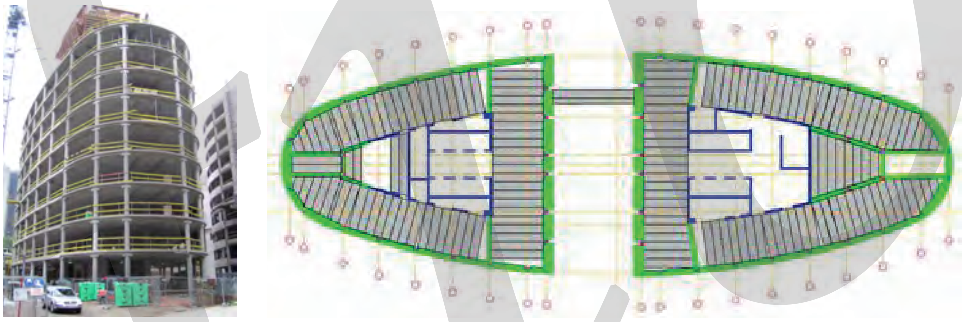


Fig. 3.4: Precast skeletal frame structure of 26 storeys with elliptic floor layout

3.2.1.4 Guidelines for the selection of a structural system

The selection of a portal or skeletal-frame structure for a given project is based on a number of parameters related to the type and importance of the project, the spans and grids, the solution for the façades, the required fire resistance, the available lifting capacity during erection, and so forth.

- Project type

Portal and skeletal frame structures are especially well suited to buildings that need a high degree of flexibility. Since the load-bearing structure is independent of the completing sub-systems, such as for the electrical equipment, conduits, partition walls, and so forth, the buildings can easily be adapted to changes during their lifetime, or to new functions and technical innovations.

Skeletal structures facilitate large spans, often between 8 and 16 metres and even up to 20 metres, and, henceforth, greater freedom in the planning and disposition of floor areas since there are no hampering load-bearing walls or many internal columns. This is very important in office buildings, shopping halls, car parks and sporting facilities. The internal space can be further subdivided using non-load bearing demountable partition walls, for example, in large office buildings.

- Floor grid and imposed loading

The most appropriate structural system is obviously influenced by the necessary grid dimensions and the imposed loading. Portal structures are practically the only solution for industrial buildings with large open spaces on the ground floor. The self-weight of the roof will determine the maximum span of the unit. For this reason, light roofing elements are often used when large spans are required.

Portal structures will further be used for buildings with one intermediate storey, for example, for the storage of goods or for office spaces within an industrial hall. The existing portal columns can be used to support the floor beams on corbels or haunches. Since the span width of the roof beams is much larger than for floor beams, intermediate columns will be needed inside the building.

For multi-storey buildings, the choice of the structural system will depend on the building function and the importance of the imposed loading. Skeletal frames are currently used for large, clear floor areas, such as in office buildings, car parks, shopping centres, and so forth.

Normally, columns and beams can easily be designed for their required size and reinforcement quantity. While for roof beams the span/bearing capacity is usually determining, for floor beams the maximum construction height may be the limiting factor.

Skeletal systems are also sometimes used in combination with precast walls and traditional infill masonry walls.

- Façade systems

When the façade is constructed with non-bearing cladding panels, they are normally fixed to a skeletal structure and designed in precast concrete or another material. In the case of precast concrete cladding, it is better to use a precast concrete skeletal structure as well so that only one team of experts is involved in the erection of the superstructure. The skeletal system gives the architect more freedom in the choice and design of the façade cladding.

Skeletal systems can also be combined advantageously with load-bearing façades in architectural concrete. The application is economical for low and medium-rise buildings and still makes a prestigious appearance possible since it provides the architectural design with flexibility. (Also see Section 3.2.6)

- Fire resistance

Precast-concrete columns in reinforced or prestressed concrete normally have a fire resistance of a minimum of two hours. In the case of beams, their shape and size determine fire resistance. Slender roof beams and secondary beams normally have a fire resistance of one hour, while floor beams can easily be designed for a fire resistance of two hours or more.

In most cases, greater fire resistance can be obtained by simply increasing the concrete cover on the main reinforcement.

- Maximum lifting capacity

The self-weight of columns or roof beams may vary from about one to 20 tonnes, exceptionally even 30 tonnes. The heaviest units are usually adopted in large buildings, where the erection is carried out with mobile cranes. Today very heavy lifting cranes are available, which makes the weight parameter less critical when determining the choice of the structural system.

3.2.3 Wall-frame systems

Precast walls are usually made with reinforced concrete. The elements are mostly storey height with a length of four to 14 metres. The thickness varies between 80 millimetres for non-load-bearing walls and 150 to 200 millimetres for load bearing walls, exceptionally reaching 300 millimetres for special applications.

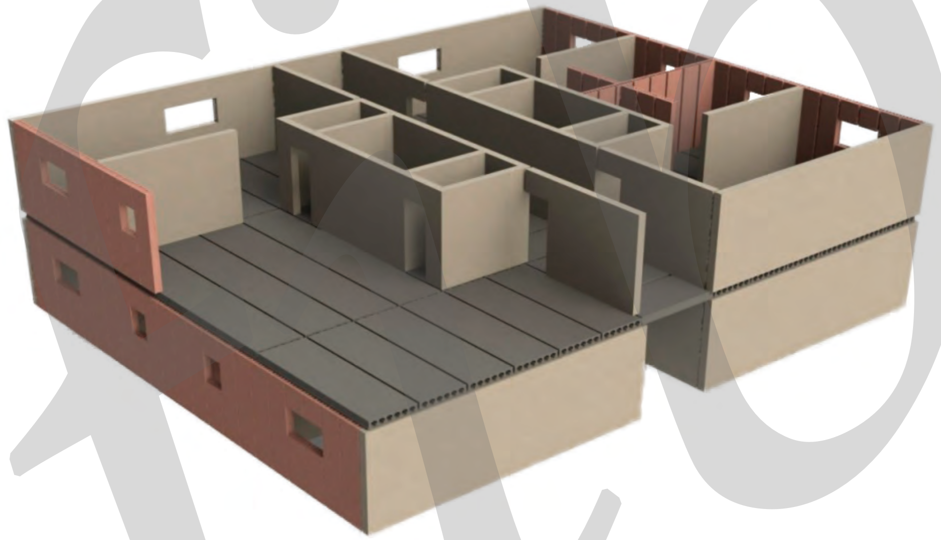


Fig. 3.5: Load-bearing wall structures

Precast walls are used for internal and external walls, lift shafts, central cores, and so forth. The system is mostly used in domestic construction, both for individual housing and for apartments. The precast walls can be load-bearing or non-load bearing. Precast walls offer the advantage of speed of construction, smooth surface finishing, acoustic insulation and fire resistance.

Modern systems belong to the so-called open construction technique, which means that the architect is free to design the project according to the requirements of the client. The latest trend is to build free open spaces between the load-bearing walls, and to use partition walls for the internal layout. It allows for the anticipation of different end-user demands in the matter of the layout of the internal space without major costs added.

The most important parameters for the choice of a precast wall frame system are the type of project, the required surface finishing, thermal and acoustic insulation, and fire resistance.

3.2.3.1 Type of project

Wall frame structures are appropriate for buildings with many external and internal walls, such as housing and apartment buildings. Indeed, the solution can be considered as the industrialized form of cast in-situ walls or classical brick or block masonry walls.

3.2.3.2 Self-weight of the units

The self-weight of precast-concrete wall units varies from two to 10 tonnes, exceptionally reaching even 20 tonnes, depending on the length and the thickness of the elements. The length is determined by the project. Solutions in lightweight concrete also exist.

3.2.3.3 Surface finishing

The surface of precast wall elements is usually smooth on both sides and ready for painting or wallpapering. This will, of course, considerably reduce construction time, which may also be a decisive parameter in the choice of the system.

3.2.3.4 Acoustic insulation

Concrete walls have excellent acoustic insulation because of their mass. If needed, the thickness of a wall can be increased to meet exceptional requirements. In general, walls that are 180 millimetres thick are adequate for the acoustic insulation of most projects since they achieve attenuation levels of around 50 dB and, thus, exceed national requirements by approximately 45dB.

3.2.3.5 Fire resistance

Concrete walls have a fire resistance of two to six hours, depending on wall thickness and loading. They are currently used for fire separation walls.

3.2.4 Floor systems

Precast-concrete flooring offers an economical and versatile solution to ground and suspended floors, and to long-span roofs in any type of building construction. Approximately half of the floors used worldwide in commercial and domestic buildings, sport stadiums and shopping centres are made of precast concrete. Precast concrete offers both design and cost advantages over traditional types of flooring that use in-situ concrete, steel-concrete composite and timber.

A wide range of flooring provides economical choices for all types of loading and spans of up to 30 metres in length. Precast-concrete floors offer maximum structural performance with minimum weight and may be used with or without structural topping concrete, non-structural finishes (such as tiles and granolithic screed), or with raised ‘computer’ floors. There are five main types:

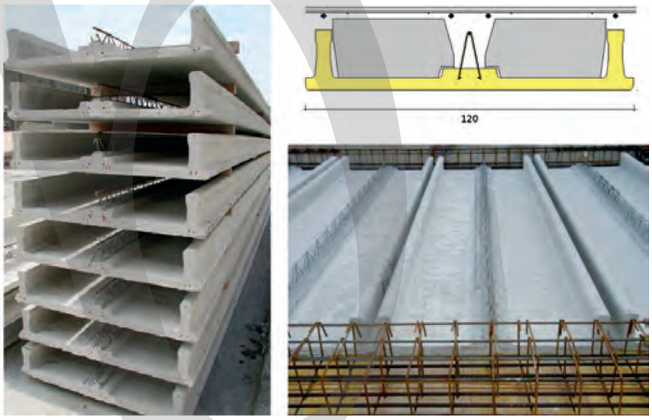
- Hollow-core floors
- Ribbed soffit floors
- Massive slab floors
- Composite floor-plate floors
- Beam-and-block or beam-and-infill-plate floors

Prestressed hollow-core floor

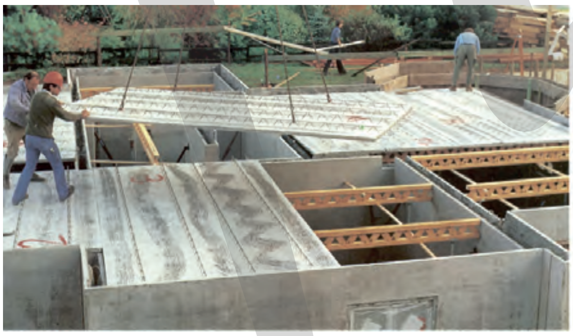
Prestressed hollow-core units in the stockyard

Prestressed double-tee floor

Prestressed inverted-U floor



Prestressed ribbed-plank floors



Composite floor-plate floor

Beam-and-infill-plate floor

Fig. 3.6: Examples of precast floors in long span utility buildings

The principal advantages of precast floors are speed of construction, absence of scaffolding for most of the systems, a large variety of types, large span capacity, low self-weight and economy. Some of these parameters are summarized in Table 3.1.

Table 3.1: Indication of the size and weight of the main types of precast floors

Floor and roof type	**Maximum span (m)**	**Floor thickness (mm)**	**Most common unit width (m)**	**Unit weight (kN/m²)**
	20	120 - 500	600 – 1,200 – 2,400	2.2 – 5.2
	30	200 - 800	2,400	2.0 – 5.0
	12	175 - 355	2,400	1.2 – 1.8
	6	100 - 250	300 - 600	0.7 - 3.0
	10	100 - 400	600 – 2,400	2.4 – 7.2

Precast floors are used extensively in all types of buildings, not only for totally precast structures, but also in combination with other materials, for example in steel structures, cast in-situ concrete, and so forth. The choice of a flooring system varies in each building and from country to country depending on transport and lifting facilities, availability on the market, building culture etc. The selection of the most appropriate type of floor for a building project is governed by a number of factors, related for example, to the availability of a floor type on the market, available transport and lifting capacity and level of average labour cost, among other factors. The most important criteria are analysed below.

3.2.4.1 Span - bearing capacity

Ribbed floors, such as double-tee units, are well suited to large span/load performances, such as those required in industrial buildings, warehouses, shopping centres and distribution centre. They are used mostly with a cast in-situ structural topping.

Hollow-core floors are often used for large spans and moderate loading, for example, in offices, apartments, car parks, shopping centres and circulation areas in grandstands.

Composite floor plates are comprised of precast and cast in-situ concrete, and may be used for smaller spans and moderate loading such as for housing, apartments and hotels.

Beam-and-block floor systems are principally used for small spans and small loading, mainly in housing. They are very well suited to renovations because of the reduced weight of the units. They may require a cast in-situ topping or be completed using a finishing screed depending on building requirements.

Figure 3.7 gives an indication of the span vs. load carrying capacity of the above floor types. Limiting spans are usually a function of the maximum practical manufacturing or handling length, which varies depending on the manufacturer's facilities. Therefore, they may be greater or less than those shown in the figure. Hollow-core and beam-block data do not include cast in-situ toppings – a 10 per cent increase may be added to the span if a topping is used. Beam-block data is based on the prestressed beams being placed at about 500 millimetres apart, with lightweight infill blocks in between.

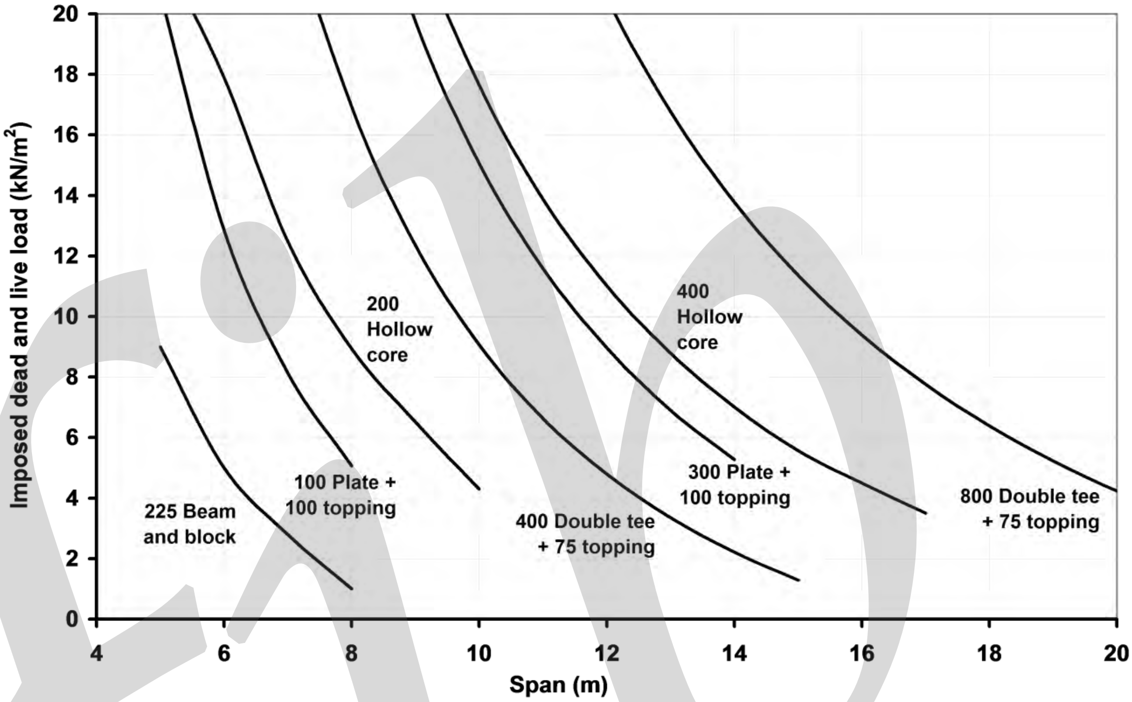


Fig. 3.7: Chart showing imposed load vs. span for the initial estimate of the load-carrying capacity of precast-prestressed floors. Self-weights of the precast unit and the necessary topping are already included in the data.

3.2.4.2 Type of soffit

The soffit of precast floors can be ribbed or flat, smooth or rough for plastering, with or without thermal insulation. Ribbed floors allow ducts and pipes to be placed between the ribs. Flat soffit floors make slender floor structures possible, especially in the case of prestressed hollow-core slabs. Beam-and-block floors have a rough and irregular soffit requiring suspended ceilings or classic plasterboard surfaces. Finally, prestressed hollow-core and ribbed units can be fitted with a thermal insulation layer at the bottom. This solution is widely applied in colder regions for ground floors and floors above crawl spaces.

3.2.4.3 Self-weight

The self-weight of floor elements may vary from less than 100 kilograms for joists in beam-block floors, for example, to several tonnes for double-tee elements, for example. The choice of the most appropriate floor may therefore depend on the size of the project and the available lifting capacity on the market. Table 3.1 gives typical self-weight data.

3.2.4.4 Acoustic insulation

The acoustic property is an important criterion in the choice of floor type, especially in residential buildings. The airborne insulation capacity of floors depends on the mass per square metre and the details at the flanking edges of the floor area, where solid units may be required locally. Concrete floors can easily accommodate the required performances. The situation is different for contact noise transmission, where additional measures usually have to be taken for floating floor decks, for example.

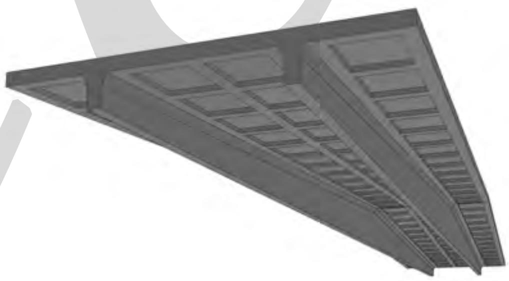
3.2.4.5 Fire resistance

Precast floors in reinforced and prestressed concrete normally assume a fire resistance of 60 to 120 minutes and more. Fire resistance of up to 60 minutes is met by all types of floors, without any special measures required. Fire resistances of 90 minutes and more are obtained by increasing the concrete cover on the reinforcement.

3.2.5 Precast roof solutions

Concrete roof elements are mainly used for industrial and commercial buildings, sport halls etc. There are different types of elements such as slender ribbed units, embossed slabs, Y or V shaped slabs, vaulted roof slabs, single or double wing-elements, through units, etc. but also hollow core units.

Prestressed ribbed roof unit

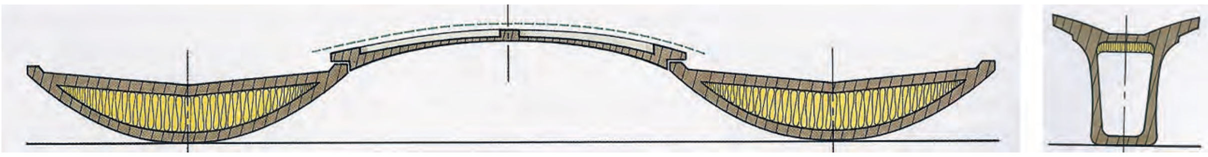


Prestressed saddle-roof unit

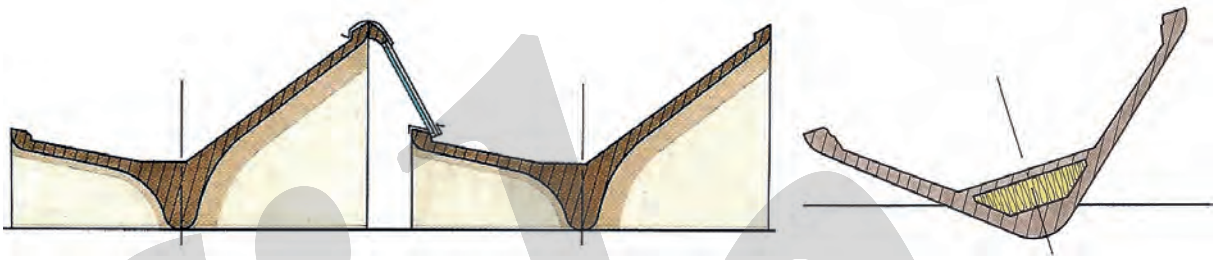
Fig. 3.8: Light, ribbed, prestressed-concrete roof elements

The following pictures show the typical shape of the 'alari' (wings) roofing elements used in Italy, with their variable cross-section [31]. They can either form a continuous roofing slab when laid side by side, or roofs with skylights when they are alternated with lighter elements. In the first case, 'open' sections are possible; in the latter, torsional resistance is required and sections that are partly 'closed' are recommended.

Simple wing with closed section



Box wing with closed section, alternated with vaulted, embossed concrete units or transparent synthetic elements



Wing with closed section, alternated with skylights

Fig. 3.9: Examples of precast roof units and structures

3.2.6 Concrete façades

Concrete used as a façade dates back to the 1920s, to the start of Modernism and the advent of great architects like Le Corbusier, Walter Gropius and Alvar Aalto, among others. At that time concrete was a new material but was already widely used for load-bearing structures and civil engineering works. A remarkable example of what concrete can do for the shape and look of a façade is Le Corbusier’s Notre-Dame-du-Haut Chapel in Ronchamp, France.

Fig. 3.10: Le Corbusier’s chapel in Ronchamp, France

During the second half of the 20th century, the precasting industry developed concrete mixes, moulds and surface finishing techniques that make the production of high-quality façade elements in a large variety of shapes, textures and colours possible. The technique is called 'architectural concrete' to indicate that the material and the means of production and application contribute to the architectural and aesthetic aspects of the project.

Apartment with balcony units, Belgium

Apartment in Helsinki, Finland

One Coleman Street office building, London

Office building in Zaventem, Belgium

Renovated façade of the TsUM shopping centre in Moscow. The precast façade units were manufactured in Belgium and transported over a distance of more than 3500 km.

Reconstructed stone columns and beams at the Médiathèque, Boulogne Billancourt, France

Roof panels of the Paddington Station concourse, London

Fig. 3.11: Examples of façades in architectural concrete

Precast façades are suited to any type of building. They can be designed as load-bearing or simply cladding units. Load-bearing façades perform both a structural and a decorative function.

Architectural concrete façades are often used on skeletal structures. Although the internal structure is often composed of columns and beams, the modern trend is to build offices without internal columns. Hollow-core floor or double-tee units span from one façade to the other over a length of 16 to 20 metres; the span of shallow double-tee elements can reach up to 30 metres for roofs. Non-load-bearing façade panels perform a decorative and enclosing function only. They are fixed to the building structure, which can be either in precast concrete, cast in-situ concrete or steel.

The choice of precast façades in architectural concrete is generally governed by the following factors:

- Architectural appearance

An essential property of concrete is that it can be cast into nearly any possible shape and given a wide range of colours and surface textures. The latest development sees self-cleaning concrete surfaces protect themselves against the impacts of weather, in particular, by repelling rainwater. Consequently, precast façades in architectural concrete are very well suited to buildings that have a pronounced architectural look and need to project a prestigious image for their companies or owners.

- Structural function

Load-bearing façades have a dual decorative and structural function. They support vertical loads from overhead floors and the structure. These elements can also contribute to the horizontal stability of the building. The dimensions of the units are open to choice and can be adapted to the internal modulation of the building structure.

- Thermal insulation

Façades in architectural concrete are often executed as sandwich elements. The thermal insulation is situated between two concrete leaves, and is typically made of mineral wool, fibreglass board or polyurethane foam. The façade has low thermal transmittance, typically U-value = 0.2 $W/m^{2o}C$ for 230 millimetres of thickness, resulting in cost savings for the air conditioning.

Precast concrete sandwich panels offer excellent fabric energy storage (FES), characterized by a slow response to dynamic temperature fluctuations over a period of about six hours. An indicator of FES is the so-called Admittance 'A-value', which is the ability of a panel to exchange heat with the environment when subjected to cyclic variation, typically over 24 hours. The maximum possible A-value is about 8.3 $W/m^{2o}C$ for good FES systems with natural ventilation. Precast concrete sandwich panels have A-values of around 5.5 $W/m^{2o}C$, much greater than a brick and block cavity wall or insulated steel or timber framed structures of equivalent thickness [38].

- Acoustic insulation

Concrete façades dispose of good acoustic insulation because of the concrete mass. For sandwich façades, the insulation capacity is even greater due to the layered structure.

Table 3.2: Sound insulation of some wall structures against traffic noise [17]

Inner concrete layer	Thermal insulation	Outer layer (concrete, plaster)	R_w (dB)	$R_{w,Ctr}$ –against traffic noise (dB)
100 mm 150 mm 200 mm	nil	nil	50 57 61	46 52 57
80 mm 150 mm	240 mm mineral wool	70 mm	54 60	50 56
150 mm	240 mm mineral wool	25 mm	58	53
150 mm	240 mm EPS	10 mm 25 mm	52 54	44 45
Window MSE, 3 glass	frame 170 mm frame 210 mm		46 47	40 42

- Early start of finishing work

Load-bearing façades, which often have windows and other services fitted at the factory, facilitate a weatherproof envelope to the interior of the building during the construction stage, allowing weather sensitive trades to proceed without delays.

- Economy

Precast cladding panels can form an integrated part of the framework of a building. The system with load-bearing façades constitutes an economical solution since it dispenses with the need for edge columns and beams to support the floors.

- Durability

Precast cladding panels are characterized by their high durability due to careful execution and continuous quality control, dimensional accuracy and the consistency of concrete mixes and admixtures. The level of maintenance is therefore the lowest achievable in the concrete industry.

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: housing, offices, commerce, industry, and so forth. In the following, guidelines are given concerning the criteria used in the choice of the most appropriate system for each type of building.

3.3.1 Residential buildings

Precast housing and apartment buildings are usually designed as wall-frame structures. A number of walls are load-bearing; the others have only a separating function. The system is widely used in Europe. The façades are executed as sandwich panels, with a load-bearing internal leaf, an insulation layer of 50 to 200 millimetres in thickness, and a non-bearing external leaf in architectural concrete or brickwork.

The advantages of the system are speed of construction, good acoustic insulation and fire resistance, and smooth ready to paint interior surface finishing. The inconveniences are less flexibility in layout and adaptability of the structure.

The wall structure can be designed either as a cross-wall system or an envelope system. In the first case, the load bearing precast walls are mainly provided in the direction perpendicular to the front façade, and the exterior cladding can be executed in precast concrete or in traditional brick masonry or any other façade material.

In the second case, the precast walls constitute the entire envelope of the building, namely, the walls between apartments or between the front and back façades. They span the floors over the full width of the house or apartment, with spans currently reaching 11 metres. The partition walls are in traditional materials, such as plaster or light masonry blocks.

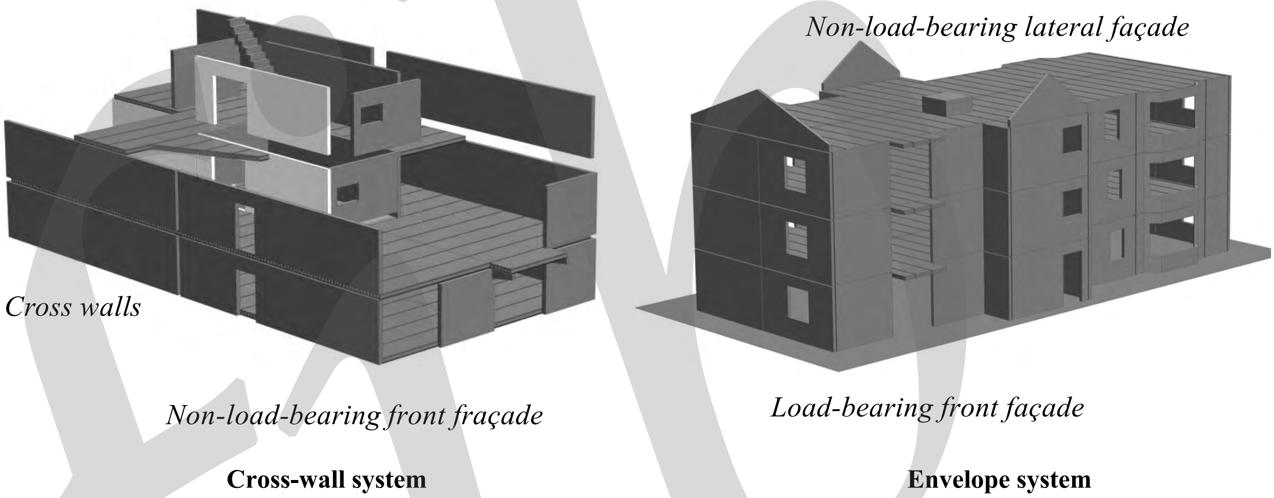


Fig. 3.12: Apartment buildings with wall frame structures

Figure 3.13 shows examples of the exterior and interior of precast wall frame housing, in which the wall panels are connected to stabilizing corner columns (with profiled façades) through a gasketed horizontal shear key, which allows building movement and guarantees water tightness. Continuous vertical reinforcement in the columns provides robustness to the building in case of accidental actions.

Fig. 3.13: Precast concrete in housing using precast concrete wall panels and corner columns, shown in relief on the right in Malaysia

The floors normally span in the longer direction. For integrally precast wall systems, the span may be in different directions but the ideal solution is to have all the floors in a parallel direction.

The following types of floors are used.

In housing, the governing criteria are small to moderate spans (4 to 11 metres), light imposed loading (± 2 kN/m²), a flat, smooth soffit either directly by the floor elements or by plastering, and fire resistance of about 1 hour. Visible longitudinal joints at the soffit of precast floors are not always accepted. However there are techniques to fill the joints and to get a smooth overall finishing. Other criteria in the choice of the type of flooring are the series of houses within the same contract, which may vary from one to more than 100 houses, the available lifting capacity on the job, the presence of large openings in the floors, building tradition, and so forth.

The simplest flooring solution is the beam-and-block floor, comprising prestressed beams (or joists) and infill blocks between. The units are light and easy to erect, but the soffit is irregular, rough and requires a plasterboard ceiling or plastering. Propping during the construction phase depends on the type of joists. Any type of floor layout can be met and modulation is not really needed, although always desirable. An inconvenience of the system is the large amount of manual work on the building site, which does not fit well into modern industrialization policy, namely, to perform as much work as possible in the precasting plant and as little as possible on site.

Small hollow-core floor elements, typically 600 millimetres wide and 120 to 150 millimetres deep, in reinforced or prestressed concrete, are still widely used in Europe. The solution is already more industrialized than the beam-and-block floor and can still be erected with moderate lifting equipment. For single projects this equipment is often the self-loading equipment of the truck. The layout of the floor should preferably be rectangular and the soffit needs plastering. Props are not needed during construction.

Large floor plates in reinforced concrete are only used for important series of housing construction because of the lifting capacity needed. The plates need temporary propping for the casting of the in-situ concrete layer. The soffit is smooth, and the floor layout does not absolutely need to be rectangular. Openings for conduits, stairs, and so forth, can be planned at any place.

Prestressed hollow-core floor elements of 1.20 metres in width are currently used for housing in strong, industrialized countries with an important tradition in prefabrication. The advantages lie in the fast and dry erection and the important span capacity. In North European countries, the presence of a longitudinal joint at the soffit does not constitute any problem. The surface is often finished with a granular paint.

In apartment buildings the contracts are usually large enough to merit the installation of high crane capacity and the type of floor chosen will normally be larger and heavier than for single-family houses. The load level is moderate. The span/depth slenderness of the floor construction, the use of a flat soffit and the speed of construction all influence the selection of flooring type. The most appropriate floor systems will be prestressed hollow-core slabs and composite floor-plate floors.

3.3.2 Offices and administrative buildings

Modern office buildings normally require a high degree of flexibility and adaptability. The interior space should, therefore, be free. Office buildings are usually conceived as skeletal frame systems with stabilizing cores. The façades can be executed in any material. Precast architectural concrete façades may be either load-bearing or not. If they are load-bearing, the most classical solution is the sandwich façade; if not, single-skin units are used.

The present trend in office buildings is to create large open spaces with floor spans of up to 16 to 18 metres. 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. For larger buildings, the same system is completed by one or more rows of internal columns and beams. The cores are executed with precast walls and support floor slabs on one side and stairs and landings, or elevators, on the other.

Prestressed hollow-core slabs are far the most suited floor type because of the large span capacity and slender floor thickness. Today it is common practice to use hollow-core units of 400 millimetres in depth for a span of 17 metres, for an imposed dead and a live load of 5 kN/m². A 500-millemetre-deep unit allows for a span of 21 metres for the same load; however, this type of unit is not widely available yet. The reduced constructional depth is indeed an important parameter for office buildings, especially in urban areas.

Fig. 3.14: Modern office building with hollow core floors spanning from one façade to the other

For smaller spans between 5 and 8 metres in length, composite floor plates (often called composite plank) of a total of 150 to 200 millimetres in depth are used. However, they need to be propped during the constructional phase.

3.3.3 Hotels and hospitals

The projects are generally large and favour solid, precast wall frames for their sound insulation and speed of construction. The floor spans are rather large, with imposed loads in the order of 5 kN/m². Hollow-core units, often with a topping, and composite-plank floors are most commonly used. The façades are analogous with office buildings.

3.3.4 Educational buildings

Educational buildings are characterized by moderate to large span widths, from approximately 8 to 12 metres for schools to more than 24 metres for auditoria at universities, with imposed loads in the range of 3 to 5 kN/m². School buildings are conceived either in skeletal-frame or in wall-frame systems and auditoria mostly in skeletal frame. The façades are often characterized by large window openings. They can be load-bearing or not.

Floors are in prestressed hollow core or double tee with a structural topping. Floors for auditoria may be designed as grading floors. An example of an appropriate solution in precast concrete is the use of step beams.

Fig. 3.15: School in Cowes, Isle of Wight, UK

3.3.5 Industrial buildings and warehouses

Industrial buildings normally require large spans and simple roofs and façades. The buildings are normally designed with portal-frame systems. The stability is provided by moment-restraining columns built into the foundations. Intermediate floors may be installed in the whole building or in parts of it. The floor spans vary from 8 to 15 m and more and the loading from about 5 kN/m² to 15 or even 20 kN/m². Prestressed double-tee floor units are often the only solution when large spans and heavy loading are required. In other cases, prestressed hollow-core elements are used.

Roofs can be in concrete, cellular concrete or light materials such as corrugated cement plates or steel decks. The choice is usually governed by the climatic condition. In cold regions, ribbed concrete slabs are dominant because of the large snow load and durability requirements. In hot countries on the other hand, a concrete roof is a good choice because of its thermal properties.

The façades are created with decorative concrete panels, steel or synthetic cladding, and sometimes in brick masonry. Horizontal concrete panels are fixed to the portal columns, vertical panels to an edge beam at the top of the columns.

Fig. 3.16: Scheme of an industrial building with portal frames and ribbed concrete roof units

An alternative solution to the portal-frame system consists of using saddle-roof elements supported on load-bearing façades or skeletal frames. The solution offers large internal open spaces, with free spans of up to 32 metres in length, and a variable length modulated on 2.4 metres. The internal height can vary up to 8 metres. Intermediate floors may be installed over a part or the whole surface. The saddle double-t-roof slabs in prestressed concrete are characterized by their light weight and large span length.

Fig. 3.17: Large hall with saddle-roof units

3.3.6 Commercial buildings

Commercial buildings generally require large column-free areas. They are usually designed with precast portal or skeletal-frame systems. Besides shopping complexes within cities, there are often smaller shops at the ground level of apartment and/or office buildings. These buildings have to perform several functions, namely, car parking in the basement, shopping on the ground floor and offices or apartments in the upper levels. The structural concept is usually a combination of a skeletal structure at the car park and the shopping levels, and a wall frame or skeletal structure in the upper stories.

3.3.7 Car parks

The basic requirements for modern car parks are large open spaces with a minimum of internal columns, reduced constructional depth, aesthetic outlook, and so forth. They are usually designed with skeletal-frame systems, in combination with staircases and lift shafts. Current grid dimensions are 15.0 to 16.4 metres for the floors and 7.2 to 9.6 metres for the span between columns.

Fig. 3.18: Precast skeletal frame and external cladding at Orlando Regional Medical Center car park in 2010, Florida, USA

There are different design concepts for the internal organization of circulation and parking space:

- Flat-deck car park
- Split-level car park
- Car park with sloping decks
- Combinations of the above types

3.3.7.1 Flat-deck car park

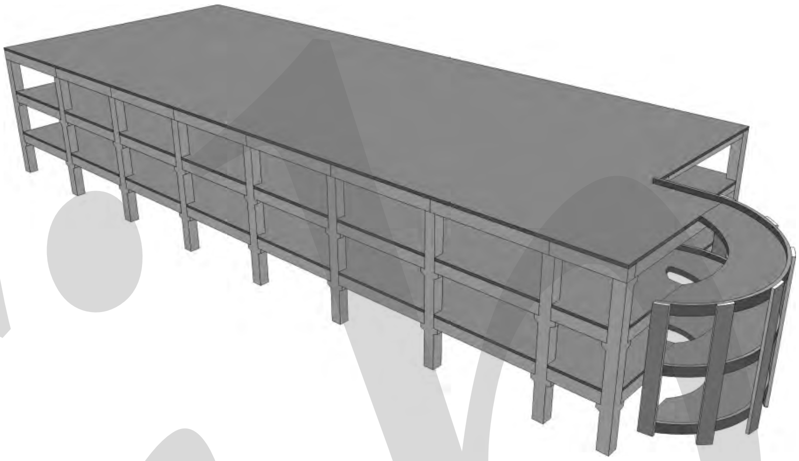


Fig. 3.19: Flat-deck car park with carousel ramp

Flat-deck car parks normally have two or more parallel horizontal parking decks and a structural single or multi-bay layout. The access and exits ramps are usually placed outside the parking decks. There are several solutions, namely, straight or carousel ramps, single or double traffic ramps, external or internal ramps, and so forth. The floor span is usually 15.0 to 16.4 metres (2 x 5.00 metres for the stalls and 4.40 to 6.40 metres for the aisle). They are made with hollow-core or composite prestressed double-tee floors.

3.3.7.2 Split-level car park

Fig. 3.20: Split-level car park

Split-level car parks are typically two-bay structures, where each bay level is shifted over half a storey height with respect to the adjacent bay. The purpose is to use half-storey ramps from deck to deck, thus eliminating the need for separate, expensive external ramps. The floor span is the same as in flat-deck car parks. The precast structure is simple and needs only double floor beams and corbel haunches at the central frame. The up circuit can be made to pass as many of the parking stalls as possible so that the remainder can be passed on the down circuit, thus allowing rapid exit.

Figure 3.20 shows a frame solution with the possibility for upward extension of the car park since top-floor beams are supported on column corbels. When there is no need for future extension, the floor beams at roof level are directly supported on top of the column (Fig. 3.19). The edge beams can be executed as a simple L-beam or as spandrel beams.

3.3.7.3 Car park with sloping decks

In a continuous-parking-ramp layout, the parking decks slope and parking occurs in the normal way on either side of the aisle. Circulation may be both up and down on the parking ramps, or up only with a clear-way down ramp. The floor surface is optimally used and driving comfort is good because of the absence of steep ramps. The façade is characterized by sloping edges.

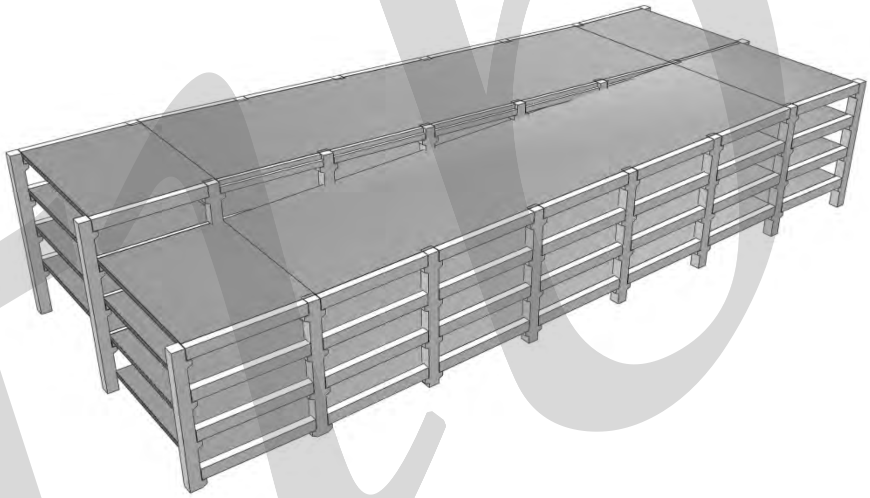


Fig. 3.21: Car park with sloping decks

The overall stability of car-park structures is generally assured by cantilevering columns in combination with lift shafts and stairwells, with floor and roof elements connected to form a diaphragm. The floor systems can have a structural topping or not. When parking structures are built partially or totally below ground level, it may be possible to use the floor system as a diaphragm to brace the retaining walls. (fib Task Group 6.1: Design recommendations for precast prestressed hollow-core floors is preparing a publication on this topic.)

The façades can be in any material with, for example, precast architectural spandrel elements.

3.3.8 Sports facilities

There are different types of sport complexes, each with their specific requirements. The execution is usually in precast concrete because of the short building delay. The following precast concrete solutions are used:

- Large halls are designed as portal-frame structures. The maximum span of the hall ranges from about 40 to 50 metres.

- Arenas and grandstands are usually made with a precast skeletal system, in combination with load-bearing walls. Floors in the circulation areas are either in prestressed hollow-core elements or in double-tee units, with spans of 6 to 10 metres. As shown in Figure 3.22, reinforced-concrete raker beams are stepped to support reinforced-concrete terraces, the design of which is often controlled more by their dynamic response to human excitation (for example, natural frequency and peak acceleration) than by structural strength, and may, therefore, appear larger in cross section than would be envisaged.

Fig. 3.22: Precast raker (inclined) beams support precast terraces at Ghelamco Arena, Ghent Belgium

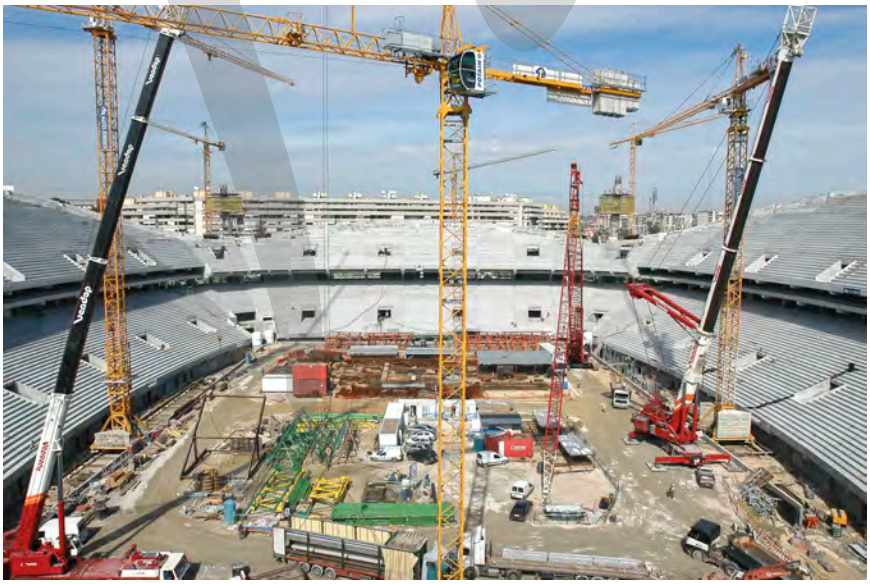


Fig. 3.23: José Alvalade Stadium, Lisbon

- The cantilevering roof above the grandstand can be executed using prestressed beams fixed to the columns with special bolts (Dywidag prestressing bolts or similar), as shown in Figures 3.24 and 3.25. The distance between the frames is 6 to 12 metres. The distance between the beams may be up to 12 metres. A lightweight roof structure spans between the roof beams, often using semi-translucent panels that have a curved, vaulted shape to reduce weight and aid water run-off. The gradin slabs are supported on inclined, prestressed raker beams and are often purpose-designed or in hollow-core units.

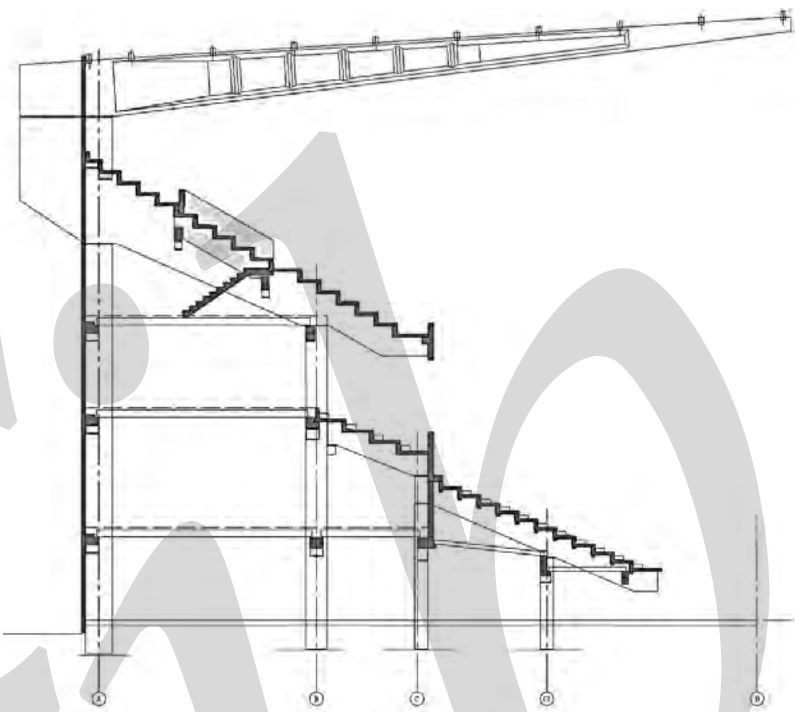


Fig. 3.24: Cross section of grandstand frame in precast concrete

Fig. 3.25: Precast grandstand in Lier, Belgium

Some ice-skating rinks have foundation slabs of hollow-core elements on foundation beams.

4 Structural stability

4.1 Introduction

In the previous chapters attention has been drawn to the importance of a correct design philosophy for precast structures, which is different from the design philosophy for cast in-situ structures. This distinction lies principally in the requirements for structural stability and for connections. Chapters 4 and 5 deal with these subjects.

Cast in-situ concrete structures behave like three-dimensional frameworks. The continuity of displacements and the equilibrium of bending and torsional moments, and shear and axial forces are achieved by reinforcing the joints so that they have equal strength to the members. However, in precast concrete, a three-dimensional framework is seldom applied because it is difficult to achieve moment-fixed connections between linear members. The stability of precast structures are guaranteed when the right, easily achieved systems are used on site: the restraint of columns in foundations, the in-plane stiffness of shear walls, diagonal bracing, floor and roof diaphragms, and combinations of the above. In-situ construction allows for the greater redistribution of forces; detailing is very important in precast construction.

Structural stability is a crucial issue in precast concrete design because it involves the design of (i) the framework in total, including the floor slab acting as a horizontal plate or ‘diaphragm’, (ii) the precast concrete elements and (iii) the connections between them.

There are two design stages to consider:

- Temporary stability during construction

This has certain implications for design, for example, the axial load capacity of temporary column splices must be greater than (at least) the self-weight of the upper column before the in-situ infill grout has hardened, which renders the splice permanent.

- Permanent stability

This includes horizontal diaphragm action in the precast floor slab and the transfer of horizontal loading from the floor slab into the vertical bracing elements and, thus, into the foundations.

The main concepts for structural stability are:

- Unbraced (or sway) structures, where stability is provided by the cantilever action of the columns in the skeletal structure, possibly in combination with diagonal bracing or, sometimes, 2-D frame action.

- Braced structures, where resistance against horizontal actions is provided by shear walls, lift shafts and central cores. The other parts of the precast structure are supported in the horizontal direction by the stabilizing components. The total concept for horizontal stability ensures that horizontal load acting at any location and level is transferred to the stabilizing components and further to the foundations.

When a stabilizing component such as a wall, a frame or a floor is composed of several precast elements, the interaction between them must be secured by the appropriate design of the intermediate connections. The concept of stabilizing components will be described in more detail in the following sections.

4.2 Unbraced precast structures

4.2.1 Cantilever action of columns

A column that is fixed to the foundations by a moment-resisting connection will act as a cantilever when exposed to horizontal actions. The 'cantilever action' of fixed-end columns can be used for the stabilization of low-rise precast structures. Cantilever action can also be obtained in staircase shafts and wall-frame structures.

The restraint of columns in the foundation is an easy solution for stabilizing buildings but the maximum height of the structure, without additional restraint, is limited to about 10 metres. This is because of the limitations on column size and allowable deflections. Second-order deflections (e_2) in columns pinned jointed to beams must be considered over the full height, as shown in Figure 4.3. A total height of 10 metres will attract e_2 = 400 mm per 300 mm deep column, resulting in a prohibitively large bending moment in both the column and foundation, which may be in reclaimed ground. The columns are normally continuous for the full height of the structure.

Horizontal forces parallel to the beams can be distributed by the roof beams so that columns in the same frame interact in bending (H1 in Figure 4.1). Horizontal forces in the transverse direction (H_2 in Figure 4.1) are resisted in the first place by the façade columns. However, for economic reasons it is often advisable to let the internal columns participate. This can be done in two ways, either through the diaphragm action of the roof or intermediate floors or with the help of diagonal horizontal bracing.

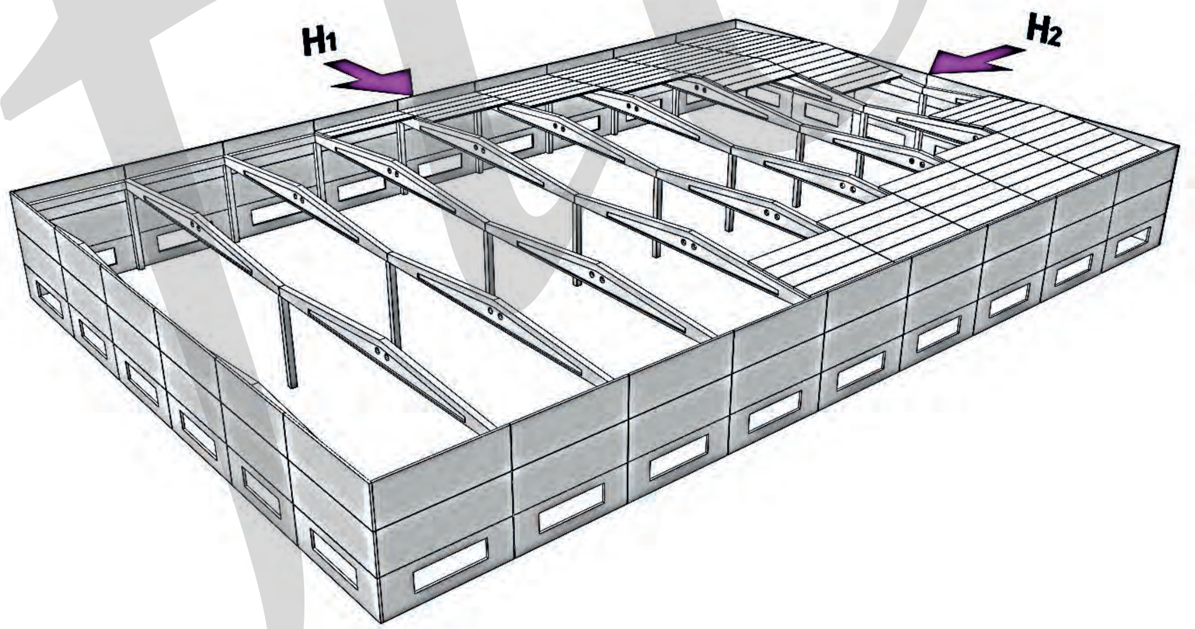


Fig. 4.1: For horizontal stability, interaction between the columns can be achieved by diaphragm action of the precast roof

Diaphragm action by the roof structure is only achievable with concrete or cellular-concrete roof elements. The connections between the elements and the tying systems are designed to resist all in-plane forces. In this way, the total horizontal force acting on the building is distributed over all the columns according to their stiffness.

For light roof structures where diaphragm action cannot be achieved by the roof structure itself, the distribution of horizontal forces on the gable walls, over the external and internal columns, can be secured by diagonal bracing between the beams of the external bays, with the help of steel rods or angles (Fig. 4.2).

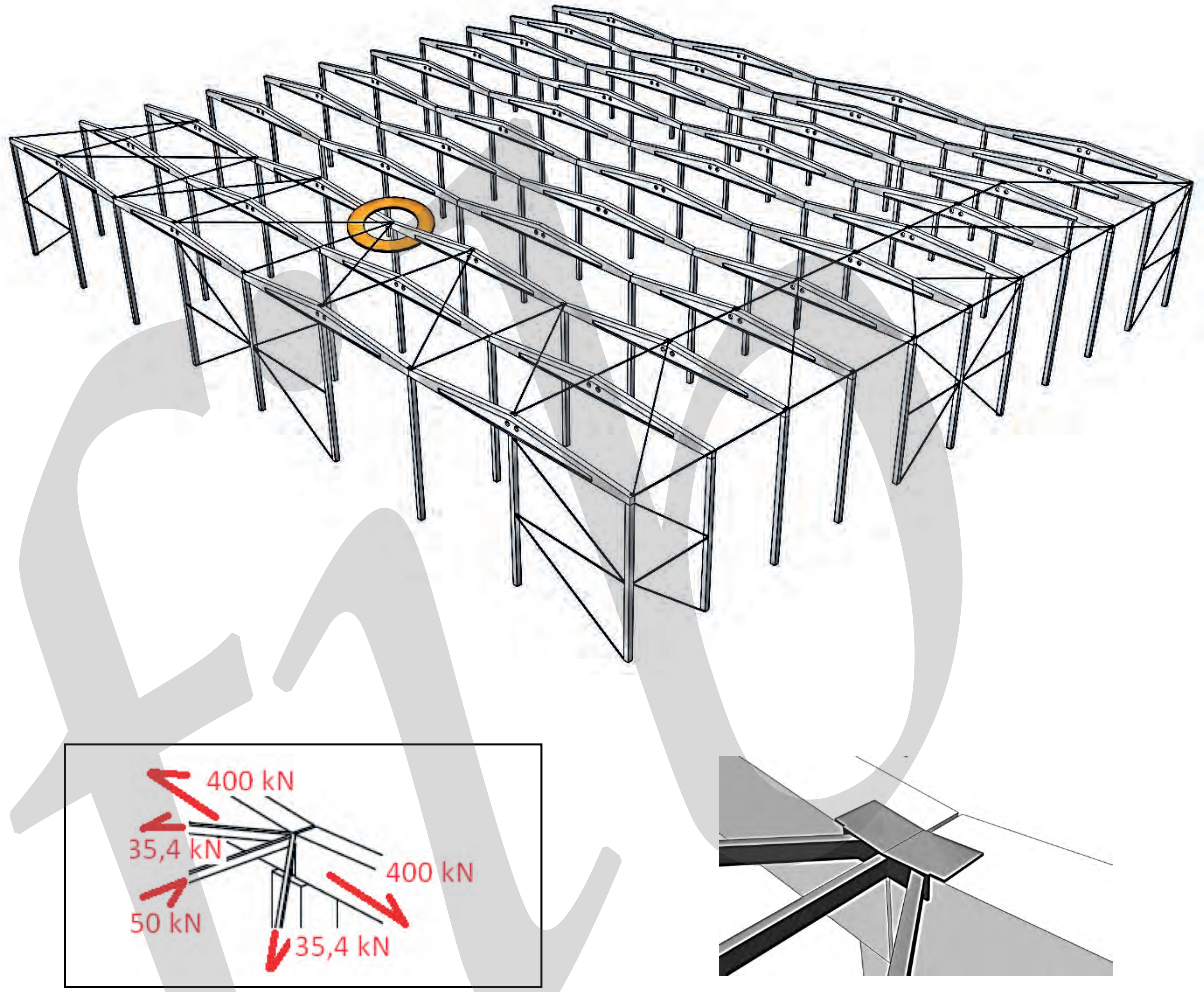


Fig. 4.2: Example of diagonal bracing of roof and / or façade structures in a portal frame hall, and corresponding connection forces and detailing in the bracing profiles with the roof girders

The forces to be taken up in the connections of the diagonal bracing to the roof beams can be very important. Figure 4.2 gives a calculated example of forces acting within the bracing profiles on the roof girder, for a portal structure of 24 metres in span and 6 metres in height. Care should be taken not to introduce moment continuity at the top of the roof beams with the connection details. There might be some risk of concrete splitting off in the roof girders at the execution of the rather heavy welds.

The beam-column connection is usually pinned and acts as a hinge. Figure 4.3 schematically shows the approximate deflected shapes of a three-storey continuous column structure with pinned connections between the beams and the columns. In reality, pinned connections between beams and columns are not completely hinged but semi-rigid because of the limited rotation capacity in the ultimate limit state. However, the design methods are not at present sufficiently understood in the post-elastic regime to be considered for the design of the structure.

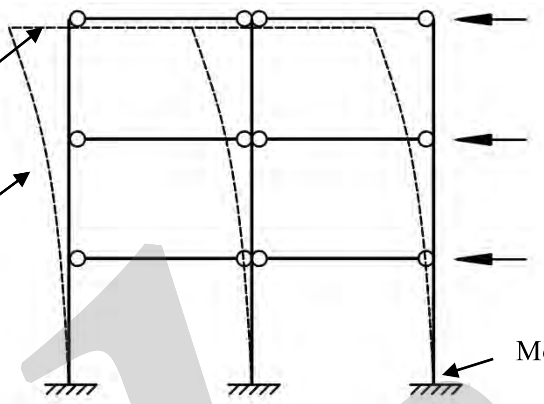


Fig. 4.3: Schematic deflections in an unbraced (sway) frame using cantilever action in columns

When the first and second-order deflections in columns are too large, they can be limited by diagonal bracing between the gable columns in some bays, as shown in Figures 4.2 and 4.4. The horizontal wind force acting on the gable wall must be transmitted either through the diaphragm action of the roof or by edge beams between the portal frames at roof level

Combinations of the two methods may be used in the same structure in different directions, as shown in Figure 4.4, or may be partially braced / unbraced in the same direction.

Fig. 4.4: Precast car park using three different bracing systems: deep columns (known as wind posts, left), stair core and diagonal strut and tie.

4.2.2 Frame action

When the restraint of the columns into the foundations does not provide the required stiffness to the structure, for example, in the case of very slender portal or skeleton frames or excessive horizontal actions, such as in an earthquake, additional horizontal stiffness may be obtained by rigid connections between beams and columns. Normally the system is only used in a two-directional way. These connections need not be installed systematically at any beam-column intersection but may be provided at well-chosen places. Although the frame action in itself is able to provide the complete stability of a frame structure, in prefabrication the system is seldom or never used alone but typically in combination with cantilevering columns restrained into the foundations or braced structures.

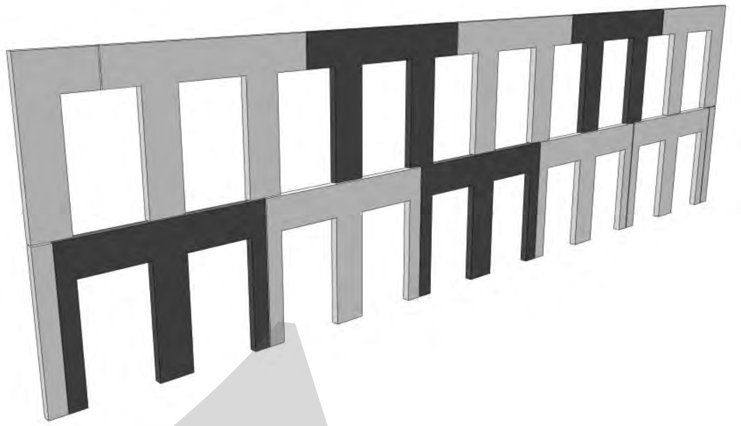


Fig. 4.5: Frame system with Pi or M-shaped wall frames

Fig. 4.6: Building structure with Pi-elements and HC floors in Brussels, Belgium

Special systems now exist to provide partial stiffness and semi-rigid moment connections between columns and beams, which can be used to aid the stability of very slender cantilevering columns (see Section 5.4.4.3). Another solution is to provide steel cross-diagonal bracing between a limited number of bays in the façade frame (Fig. 4.4).

4.3 Braced precast structures

4.3.1 Principle

In buildings more than three stories (10 metres) high the horizontal sway deflections may become excessive and additional bracing must be used. Stability walls, cores or other forms of bracing are commonly used. The usual practice is to provide building stability by using lift shafts, stairwells or internal shear walls, and to connect the rest of the structure via the floor diaphragm to them. Such a structure is braced (Fig. 4.7). The stabilizing elements are so massive that the bending stiffness of the frame elements and connections is not so important. 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 selected members allows for smaller columns and simpler connections. Furthermore, the columns have horizontal support at every floor level, which also assists with the slenderness of the columns.

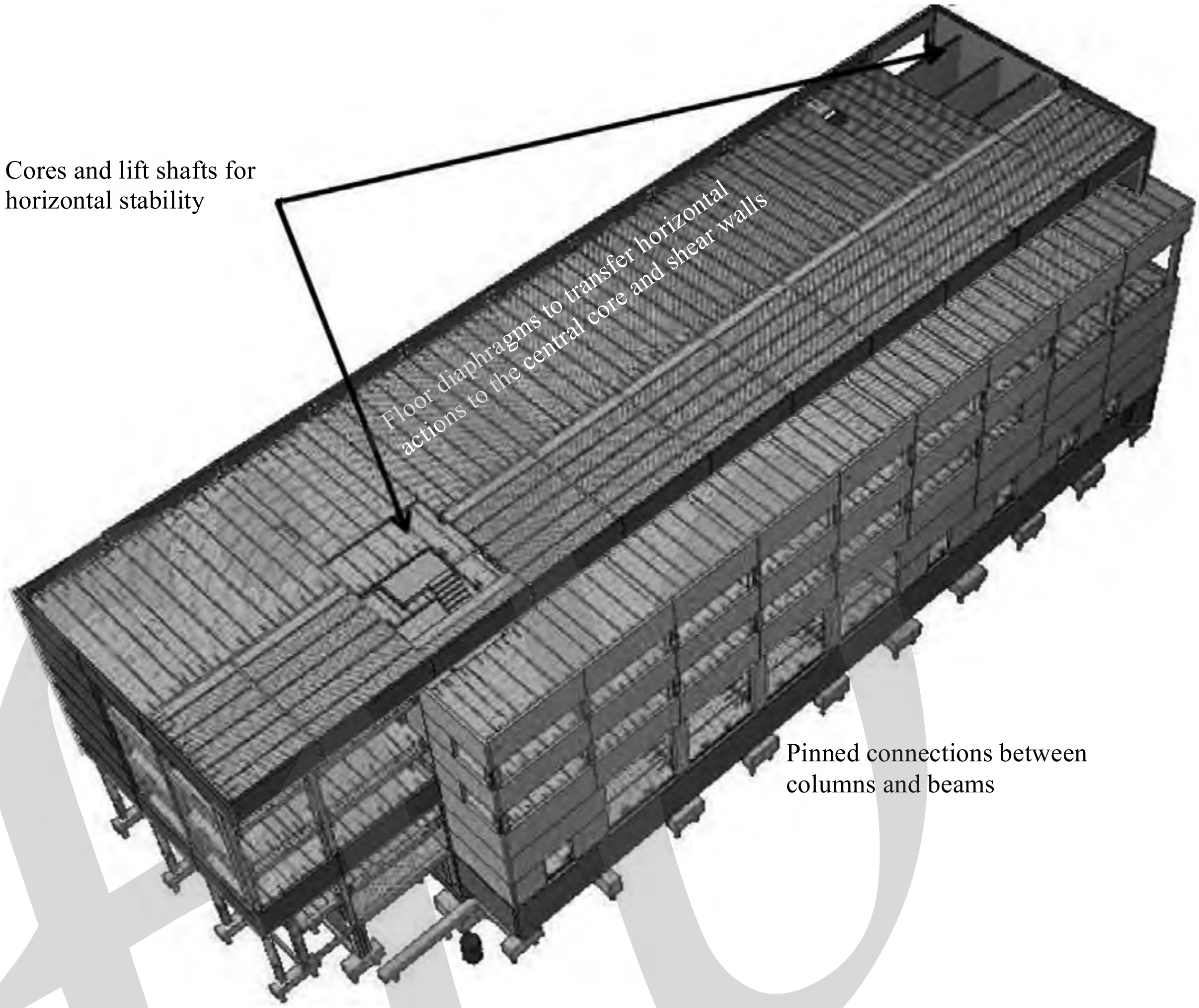


Fig. 4.7: Principle of a braced skeletal structure

4.3.2 Central cores and lift shafts

Braced systems are the most effective solution for multi-storey skeletal structures because the stair and lift shafts are already present for functional reasons, so that the additional cost of utilizing them as stabilizing members is negligible. Two-dimensional walls are easy to manufacture and erect, although three-dimensional cores provide greater immediate stability.

Central cores can be cast in-situ or precast. The most common precast solution is to produce the core out of four or more precast wall elements (Fig. 4.8) connected to each other with vertical joints able to resist shear forces. The subject is dealt with in Chapter 5, Section 5.4.3 ‘Connections transferring shear forces’.

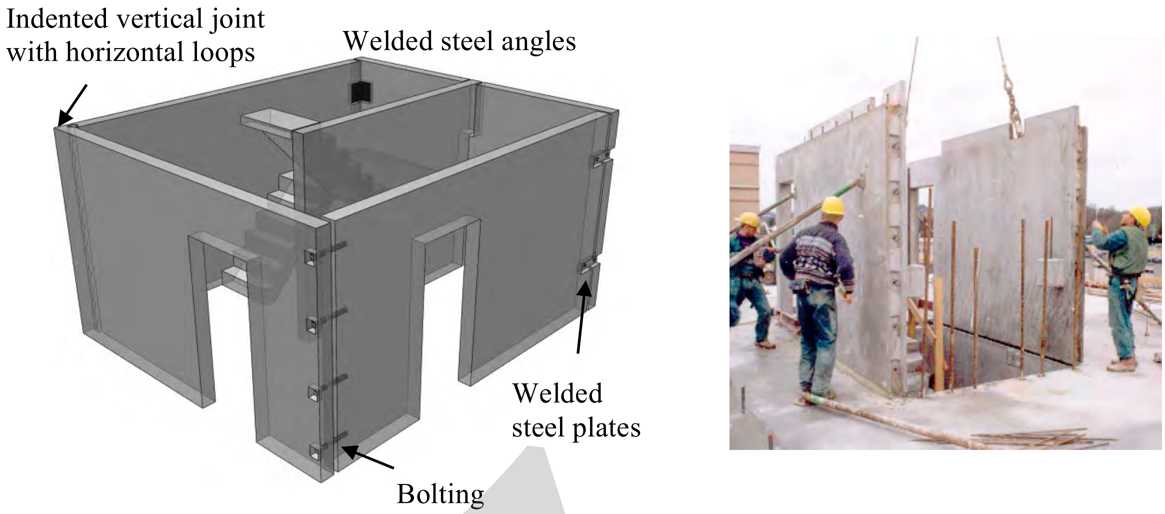


Fig. 4.8: Layout of a precast core
Left: Variant horizontal connections
Right: Vertical connections with lapping vertical tie bars in grout tubes

4.3.3 Shear-wall action

Concrete walls have a large in-plane stiffness. For this reason they are commonly used both in precast and cast in-situ concrete buildings to stabilize the structure against horizontal actions.

Fig. 4.9: Precast concrete shear walls used to brace a 45-storey wall-frame structure in Den Hague, The Netherlands. Storey-high shear-wall panels are connected in such a way that the whole building can function as a cantilevering tube.

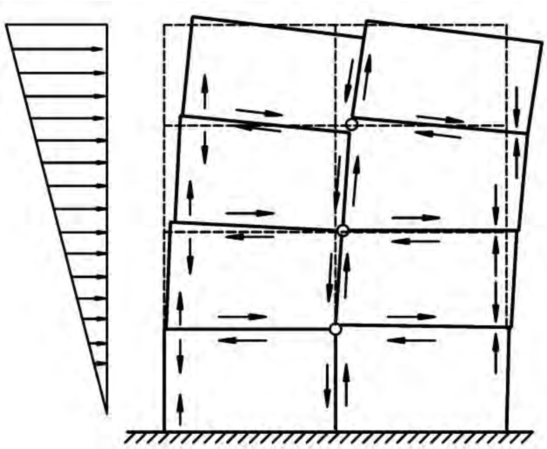


Fig. 4.10: In-plane action of precast walls

The interaction between the wall units is secured by connections and tying systems that transfer the necessary shear, tensile and compressive forces. If necessary, tensile reinforcement is used to anchor the units to the foundation and to provide continuity between successive storey-height units, if insufficient vertical loading is available on the elements.

When walls have large openings, for example for doors, the part of the wall above the door opening could possibly contribute to panel stiffness. If it cannot, only the part of the wall beyond the door opening should be considered.

Shear walls are also often used to complement the horizontal stiffening action of cores, for example, at both ends of a long and narrow building with a central core, or where cores are placed in an eccentric position (Fig. 4.11).

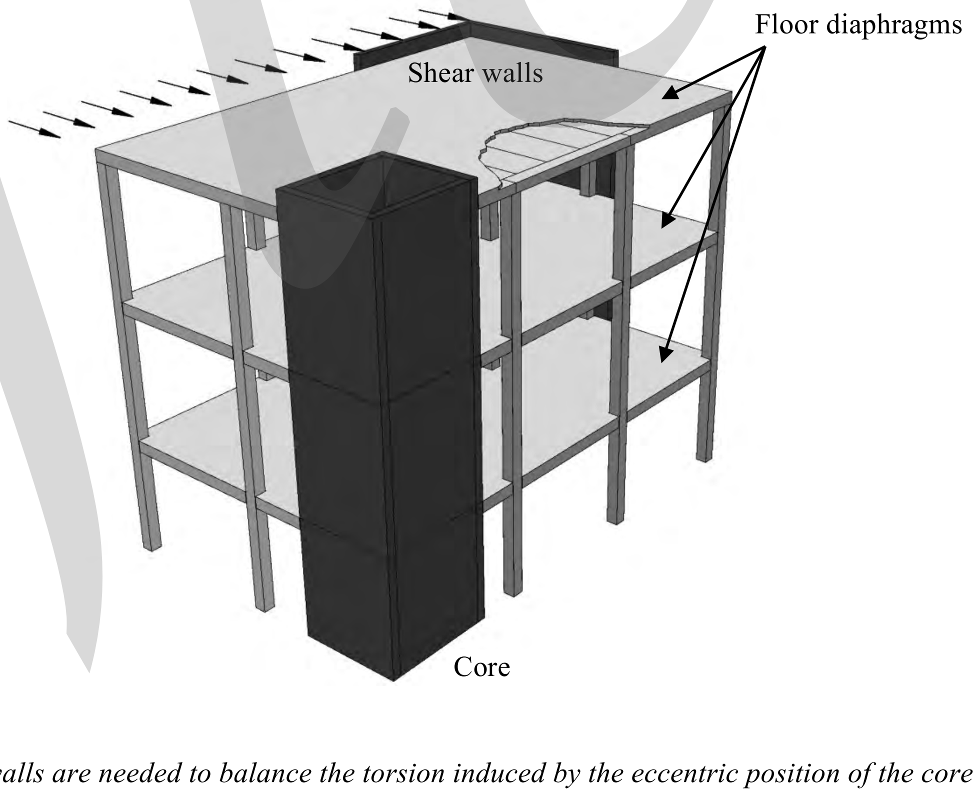


Fig. 4.11: Shear walls are needed to balance the torsion induced by the eccentric position of the core

The distribution of horizontal loading between shear walls and/or cores depend on a number of factors, namely:

- Stiffness of the stabilizing components
- In-plane deflection response of the stabilizing components
- This is predominantly a flexural deflection in cantilever walls, a shear deflection in infill walls and a truss deflection in steel cross bracing.
- Position of the stabilizing components
- Ideally, the structure should be balanced by positioning the stabilizing components according to their stiffness in order to avoid torsional effects.
- Expansion joints in the floor diaphragms (see Section 4.5)

Finally, when locating the stability elements, due consideration has to be taken of dimensional changes. Care should be taken that these deformations can take place without (or almost without) serious cracks occurring.

4.3.4 Infill walls

An alternative type of shear-wall action is created with the infill wall. The infill wall is locked between columns and beams, and when subjected to horizontal forces through the floor plate, develops a diagonal compressive strut, in equilibrium with reactions in the beams and columns. The moments in the columns are zero but the axial forces have to be considered, particularly in light gravity-loaded cases where walls are positioned between 'gable' columns and uplift may occur. Large compressive forces may result in the columns adjacent to infill walls requiring a greater section (or reinforcement) than the remainder. Columns adjacent to cantilever shear walls are assumed not to carry additional axial forces due to overturning moments, despite the presence of a continuous vertical shear key between the column and wall.

Infill walls may be used to brace skeletal structures up to 12 to 15 storeys in height, after which the tie-down force on the windward side of the wall often becomes excessive. Tension piles will then become necessary. The way to avoid this is (i) to ensure that the columns have sufficient gravity dead load to overcome the uplift, or (ii) pairs of walls are placed in line such that the compression from one wall opposes the tension in the other, or (iii) a balanced foundation is used.

4.4 Floor diaphragm action

4.4.1 General

In precast buildings, horizontal loads from wind or other actions are usually transmitted to the vertical stabilizing elements by the roofs and floors acting as plates. Floor diaphragm action means the transfer of horizontal loading across a building.

Floor systems play a key role in the lateral resistance of structures. Their principal functions are:

- to transfer lateral loads at each level to the lateral-resisting system (such as walls and frames)
- to join individual load-resisting elements into a single lateral load-resisting system

4.4.2 Basic requirements

The floor diaphragm must act in all directions but is normally resolved into orthogonal directions, as shown in Figure 4.12.

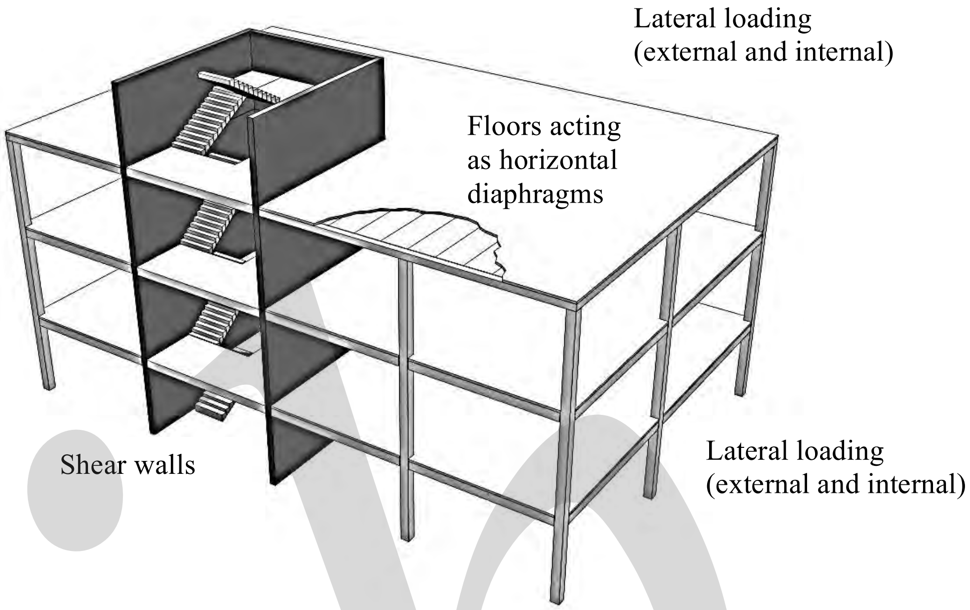


Fig. 4.12: Floor diaphragm action in a pin-jointed, braced skeletal structure

Generally, the behaviour of precast floor systems under vertical and horizontal actions depends on the expected actions it is subjected to. Each system has characteristics and detailing that depend on the construction method, the floor-unit type, the floor-system geometry, its rigidity, and so forth.

When designing structural systems under the assumption that floors and roofs develop full in-plane diaphragm action, that is to say, have infinite stiffness, special care should be taken in cases where:

- large openings are located in the vicinity of the vertical bracings, which may affect the ability of the diaphragm to transfer horizontal forces
- considerable (for instance, greater than 50 per cent) changes in effective diaphragm stiffness appear from one storey to the next, which may lead to undesirable diaphragm discontinuities
- elongated in-plan buildings have the ratio L_{max} / L_{min} ≤ 4,0, which could lead to the in-plane deformability of the diaphragm

In all cases care should be given to the connections between the floor elements (floor-to-floor, floor-to-beam, beam-to-column) in their function of transferring lateral actions.

One way to ensure diaphragm action of floors and roofs is to provide an in-situ reinforced-concrete layer (topping) of adequate thickness (≥50mm) on top of the precast slab elements of floors and roofs (Fig. 4.13).

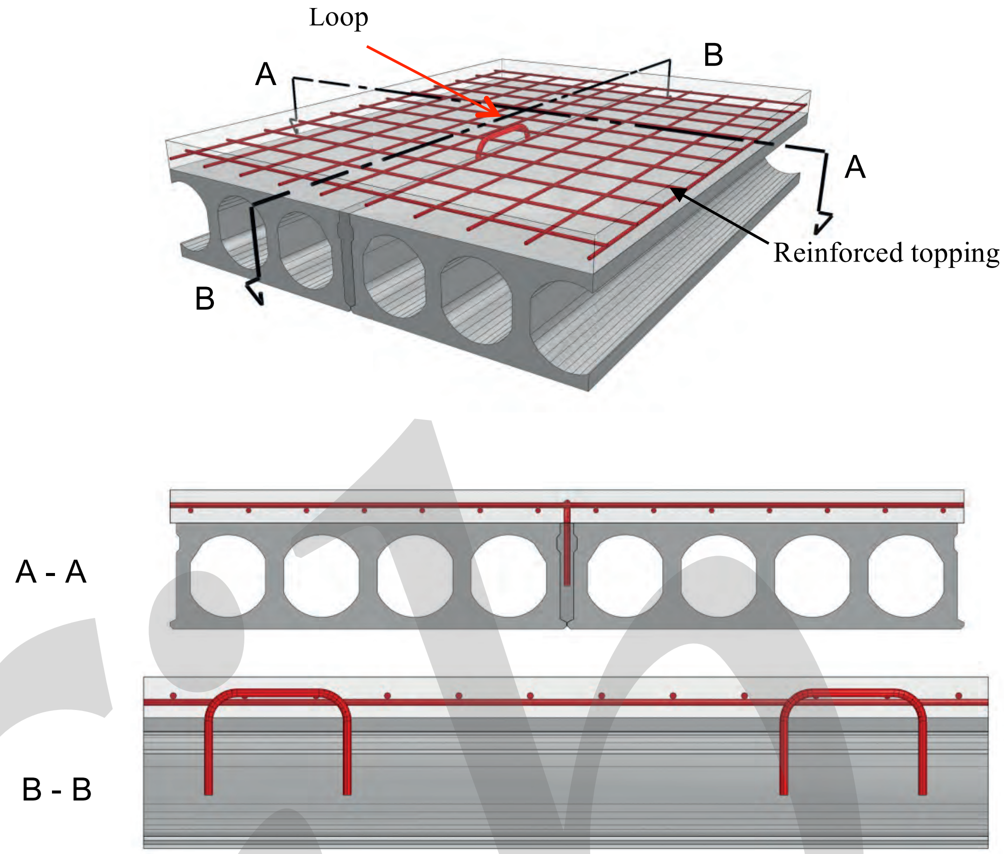


Fig. 4.13: Typical details for a topped hollow-core floor to ensure full transfer of longitudinal shear in the interface of precast units and structural topping. Generally, loops in longitudinal joints between floor units may be omitted.

Floors with toppings are generally designed on the basis that the precast units provide restraint against compressive forces and prevent buckling of the relatively thin in-situ topping or its reinforcement. The shear across joints is assumed to be carried entirely by the topping. To produce continuity, reinforcement in structural toppings must extend into the shear walls or other bracings. The shear capacity of the in-situ toppings is unlikely to be the governing factor in the structural design of the building.

When the diaphragm action is carried out without structural topping, the longitudinal joints between the precast units are critical for the shear force transfer. Shear forces are transferred between adjacent units by shear friction, aggregate interlock and dowel action at the ends. To resist these forces, the units have to be tied together so that shear forces can be transferred across the joints even when they are cracked (the interlocking effect). Not only the shear resistance but also the shear stiffness is relevant as well. The most critical sections are the joints between the floor and the shear walls because the shear forces are at their maximum there. To improve the capacity of the joints, their surface may be given a special profile (roughened, keyed, indented or waved). Figure 4.14 shows a waved profile for hollow-core slabs, which is particularly suited to withstand large horizontal shear [40].

Figure 4.14: Longitudinal joints of hollow-core slabs with waved shear key. The behaviour of the slab-to-beam joints is also relevant. Special attention should be given to the anchorage of the slabs' longitudinal reinforcement into the beams.

The ties of the overall stabilizing system (see Section 4.6.2) act as diaphragm reinforcement. However, they must be dimensioned for this purpose, as the minimum required by the standards may not be sufficient.

In some cases, for instance in longitudinal joints of double-tee slabs, with insufficient depth for developing shear friction resistance, dry connections by means of, for example, steel plates using welding and/or bolding can be provided between the individual horizontal precast members (Fig. 4.15) and between slabs and the lateral resisting elements of the structural system, especially when these elements, for example, shear walls, are concentrated in discrete positions. In Figure 4.15, such a connection is shown indicatively and is mostly used for double-tee floor units. (Such connections are not used in hollow-core floors.)

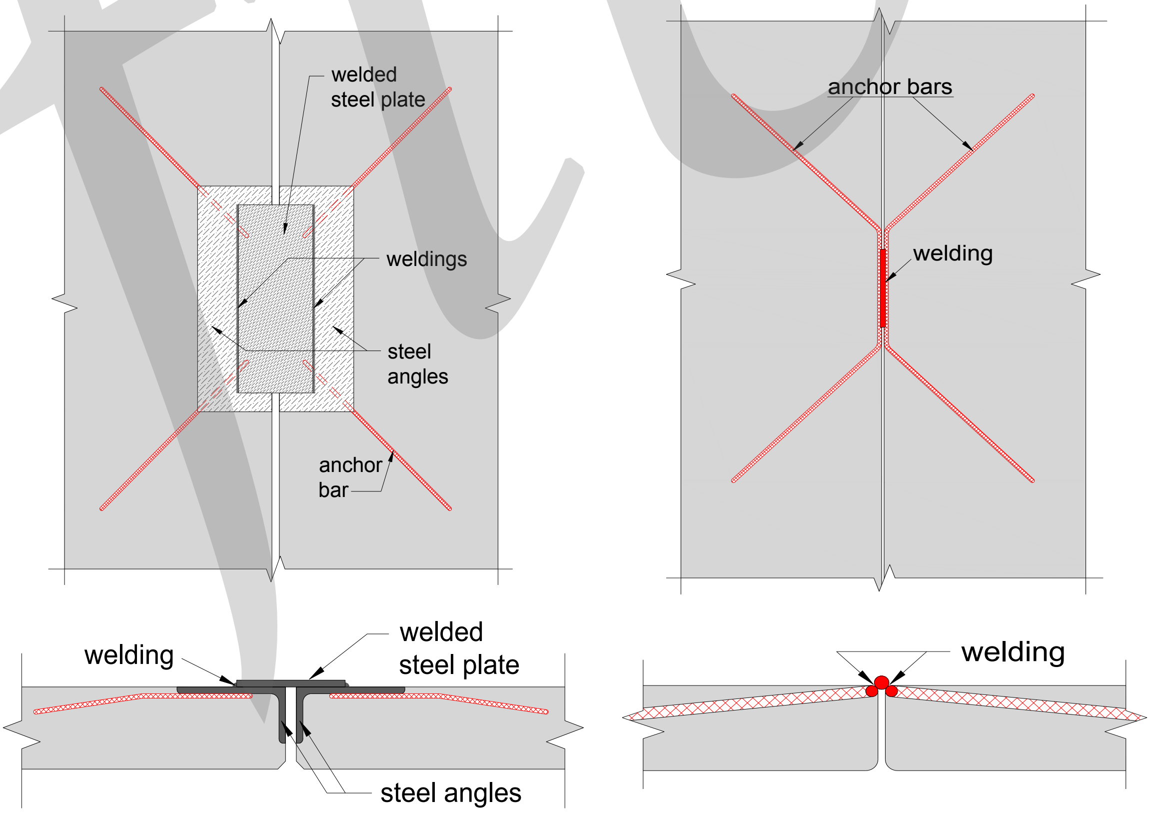


Fig. 4.15: Typical flange weld connections between double-tee floor units

4.4.3 Modelling and detailing

Various structural models have been proposed for the shear transfer mechanism, which relies on the development of clamping forces generated in the tie bars placed along the perimeter and within the diaphragm. In Figures 4.16 and 4.17 the response of floors composed of precast slabs under in-plane horizontal actions are presented schematically.

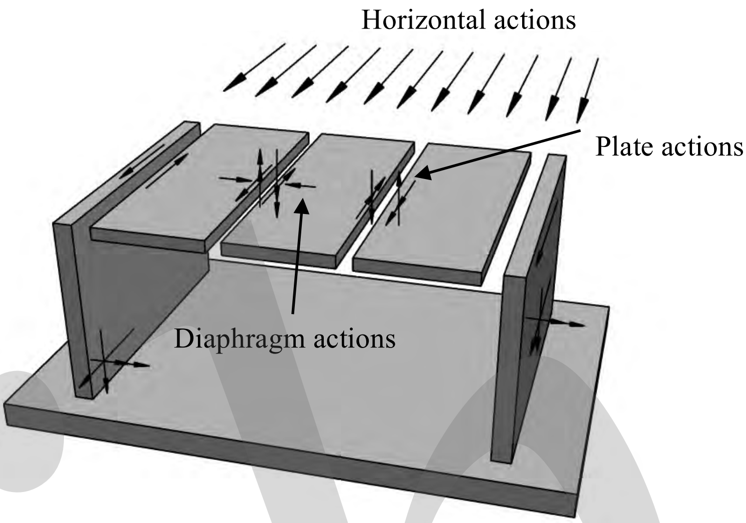


Fig. 4.16: Schematic presentation of the response of a simple floor, composed of precast slabs, under horizontal actions on one side

The sound modelling of the diaphragm behaviour of precast floors and roofs is mainly a matter of the experience and skill of the designer since proper modelling depends on a lot of parameters, such as the geometry of the building, the arrangement of the horizontal and vertical members (columns and/or walls) of the structural system, the existence or not of openings inside the area of the diaphragm, and so forth.

Depending on the aspect ratio and other specific features of the diaphragms, two in-plane force analysis methods are principally used:

- for rather simple, regular floor plans, the horizontal-beam analogy is usually adopted
- for more complicated cases, for example, when the presence of large openings may interfere, the strut-and-tie model is recommended. This method is considered a more comprehensive method to the horizontal-plate girder approach.

The latter method is considered to be a rational analysis for the design and detailing of the internal force path for irregular diaphragm cases and is recommended for use based on research and by several designers and building codes. This method may also be used for any reinforced or prestressed-concrete members as a tool to indentify and describe the flow of forces in a cracked concrete continuum, and as a quantitative tool for performing a complete design. Nevertheless, when using this method particular attention should be given to regions where wide cracks may disrupt the potential path of compression strut force.

Loads as well as multiple supports can be in any position. In fact, this is the case with actual diaphragms: wind loads act partly on upwind façades and partly on downwind; bracing structures are placed in various locations; seismic forces are scattered across the floors while some actions arise from the bracings themselves due to actions applied on other floors; forces from restrained displacements at connections with all vertical elements.

Figure 4.17 shows a deep-beam scheme of a simple diaphragm with two lateral bracings (shear walls), where six forces summarize the horizontal actions and consequent struts and ties are sketched. This example highlights the need for the continuity of the peripheral ties at corners.

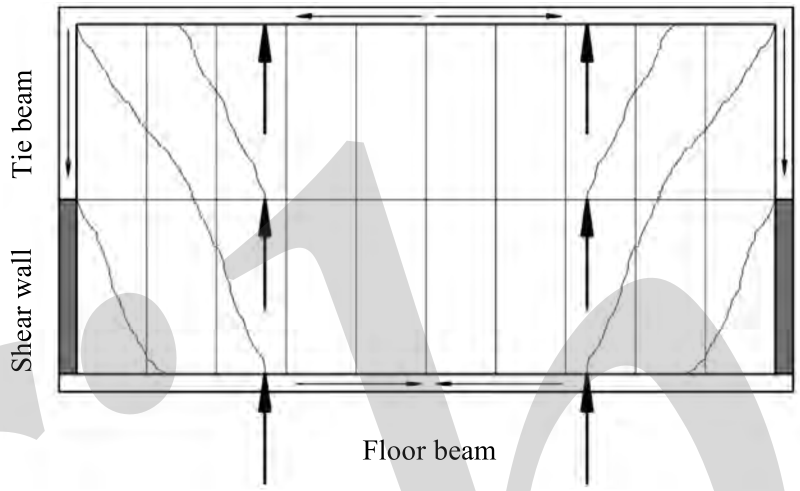


Fig. 4.17: Scheme of a strut-and-tie model of a simple diaphragm

Not only overall shear and bending appear on the diaphragm / deep beam but also compression or tension, as sketched in fig 4.18, representing a seismic situation, where the mass **M** generates the inertial force **F**, which provokes pure tension in the diaphragm's rightmost part, pulling outwards.

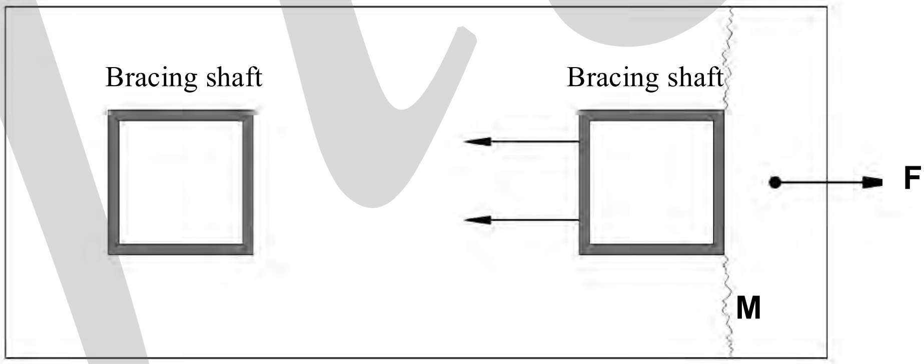


Fig 4.18: Diaphragm scheme with parts possibly subjected to tensile forces

Sufficient ductility in the topping mesh must be provided over the joints between the hollow core units in high-seismic regions.

Particular spots are subject to stress concentrations and require proper detailing, as at the connection of diaphragms to bracings or near re-entrant corners or holes, where peripheral ties may need to be extended inwards, as sketched in figure 4.19.

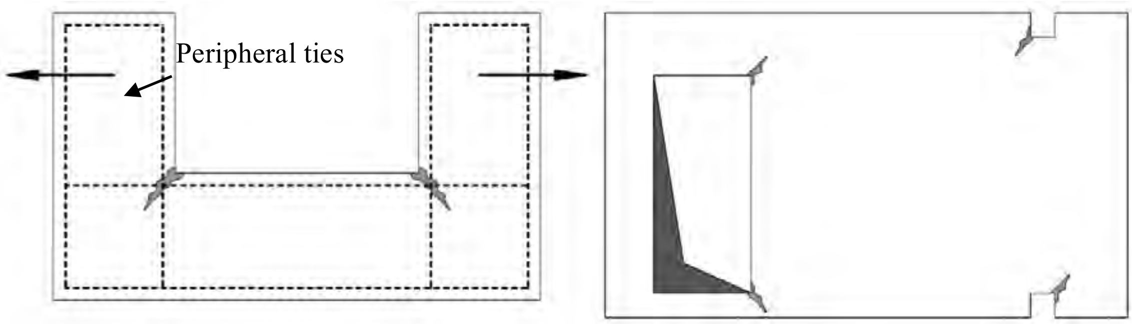


Fig. 4.19: Re-entrant corners and holes in a diaphragm require special detailing of the tie layout

Detailing must also be such to ensure adequate reinforcement anchorage after yield reversals.

4.4.4 Interaction of frame systems with precast floor systems in seismic regions

When dealing with the behaviour of frame systems under seismic situations, attention should be paid to the interaction of diaphragms with the bearing structural system, in particular for the following:

- Strength enhancement of beams

Beams designed to sustain inelastic deformation in the event of a major earthquake, may increase their strength due to the interaction with the floor (Figure 4.21). The enhancement of the strength of the beams on the frame system may modify the hierarchy of resistances assumed in design

- Incompatible displacement of frame and floor

Large beam deformation can induce additional stresses at floor supports (Fig. 4.20), to be accounted for in the detailing.

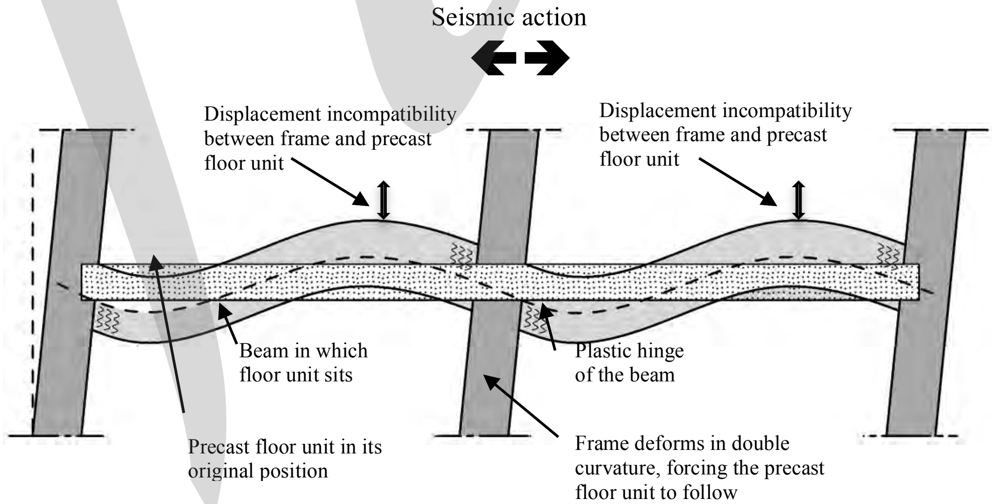


Fig. 4.20: Vertical relative displacement between floor and frame asking for compatibility

- Possible loss of support of precast floor units
- The loss of support of precast floor units (for example, HCS) can occur due to the plastic deformations of the frame caused by a high seismic event. Adequate tying is necessary.

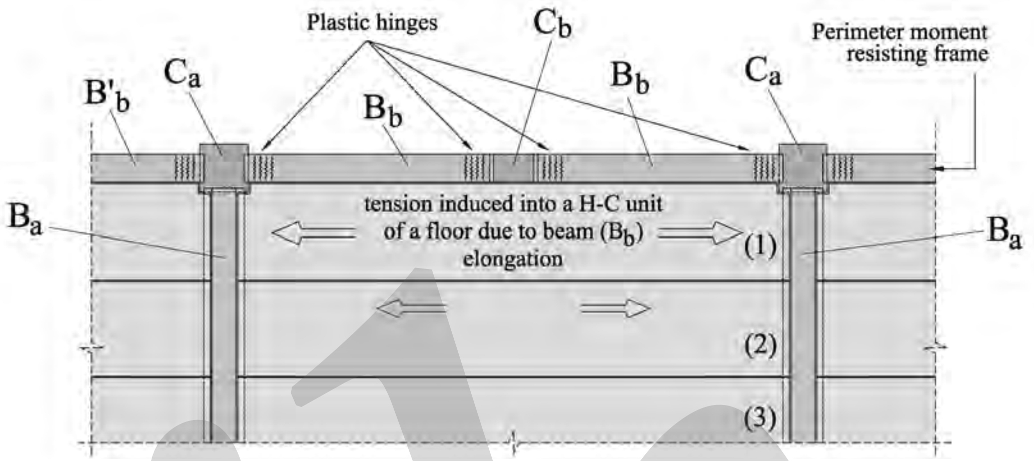


Fig. 4.21 Precast hollow-core floor units adjacent to beams where a plastic hinge develops

4.5 Expansion joints

In long building structures or parts of buildings where restraint could occur due to thermal movement, creep and shrinkage, construction measures should be taken to limit the risk of cracking. Possible solutions include expansion joints at regular intervals, sliding supports or pendulum columns.

In precast concrete structures, an important part of creep and shrinkage has taken place before erection and assembly of the structure, and the remaining movements are essentially due to thermal actions. Their influence will obviously depend on the size and shape of the structure.

For simple structures, long-term deformations due to creep and shrinkage are commonly calculated as the temperature changes of the structure. For this purpose, the modulus of deformation can be taken as the modulus for long-term effects.

It is also possible to calculate for creep and shrinkage deformations by taking the moment-curvature relationships for the different elements of the structure into account, mostly for the columns. This will be a more complicated analysis but will lead to reduced moments in the columns and tensile forces in the floors.

The following examples illustrate two extreme cases of possible restrainment due to the above-mentioned effects:

- a multi-storey skeletal car-park structure, with short rigid continuous columns between parking levels

- a portal frame hall with rather slender high columns, typically for a ware house

Car park

Warehouse

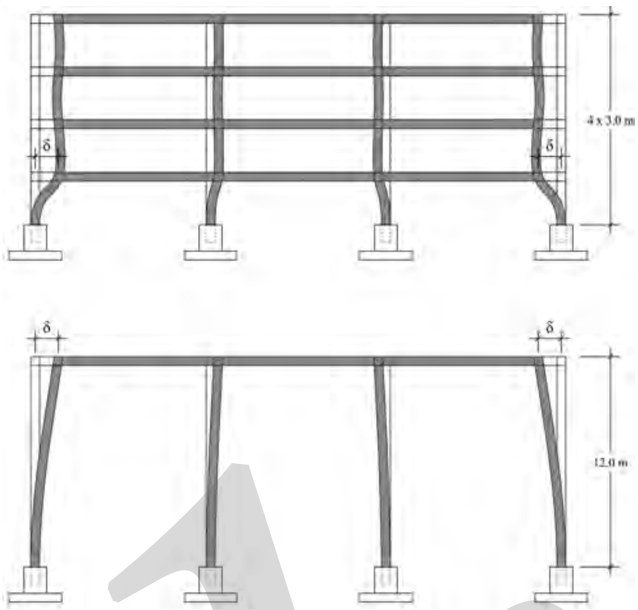
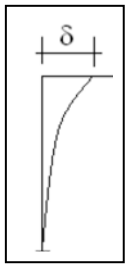


Fig. 4.22: Schematic deflections of a car park and a warehouse structure of 100 m x 150 m in surface

A simple calculation of the acting thermal force gives the following results:



Consider $\Delta T = 25°C \longrightarrow \delta = \Delta T \text{ x } \alpha \text{ x } L/2$

$\delta == 25 \text{ x } 1.0 \text{ x} 10^{-5} \text{ x } 150000/2 = 18.75 \text{ mm}$

The force needed to realise this deformation is: $F = \delta \text{ x } 3 \text{ x } EI/h^3$

The corresponding bending moment in the column is:

$$M = F \text{ x } h$$
$$= \delta \text{ x } 3 \text{ x } EI/h^2$$

which, for a column cross-section of 490 x490 mm² and E = 20.000 N/mm², gives:

- M = 600 kNm and F = 200 kN for the car-park structure
- M = 37.5 kNm and F = 3.1 kN for the warehouse

The comparison shows that the warehouse could be constructed without expansion joints whereas the distance between expansion joints should be carefully studied for the car-park structure.

For the thermal action: $L = (h^2 \text{ x } 2 \text{ x } M) / (3 \text{ x } \Delta T \text{ x } \alpha \text{ x } E \text{ x } I)$

4.6 Structural integrity

The most essential design requirement of precast structures is the creation of a coherent entity from individual precast components. Some elements or parts of the structure have only a load-bearing or separating function, others also perform a horizontal stabilizing function.

The principal means to obtaining structural integrity in precast structures is through tying systems in transverse, longitudinal and vertical directions so that all individual elements interconnect effectively, the structure is stable and redundant load paths are provided. Figures 4.23 and 4.24 give an overview of the types of ties and their positions in precast structures.

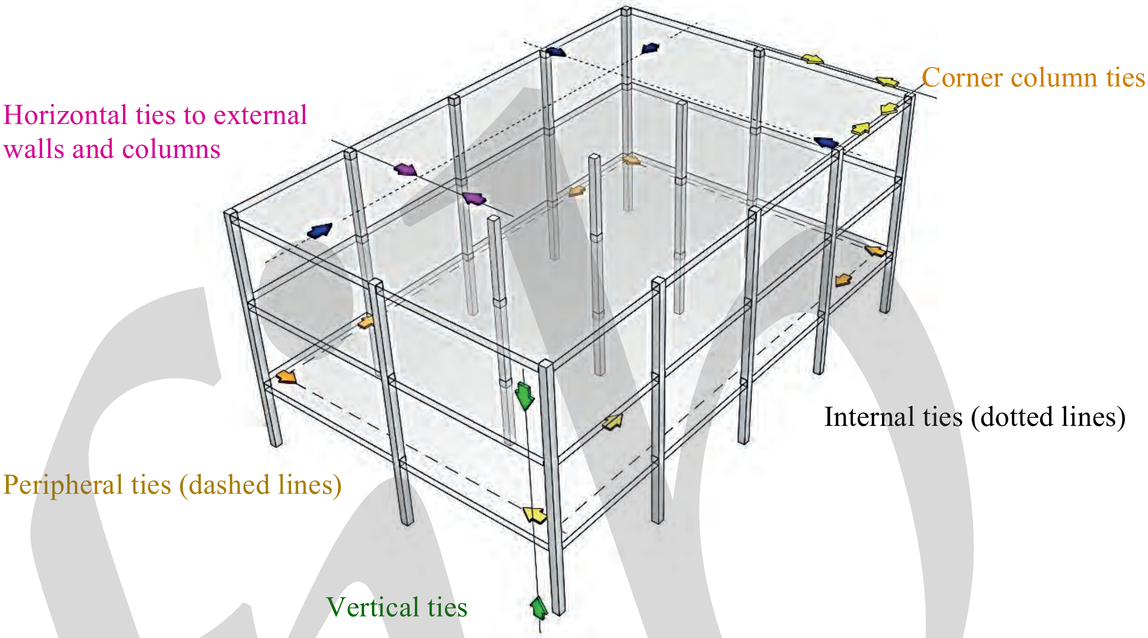


Fig. 4.23: Types of ties in skeletal frames

4.6.1 Tie systems

Ties are continuous tensile elements consisting of reinforcement bars or tendons, placed in cast in-situ infill strips, grouted sleeves or joints between precast elements, in longitudinal, transversal and vertical directions. Their role is not only to transfer normal forces, originating from wind and other loading, between units, but also to give additional strength and safety to the structure to withstand, to a certain extent, loading conditions termed as abnormal loads: settlements, explosions, vehicle or aircraft collisions, tornados, explosive bombings or other accidental actions.

Precast structures are more susceptible to the effect of abnormal loads than some traditional forms of construction because of the presence of joints between the structural components. However, experience has shown that this is perfectly possible to cope with by effectively tying the various components of the structure together.

The infill concrete serves to transfer the tensile and shear forces from the elements to the tie reinforcement, and to prevent it from corroding. The tensile chord can be either in normal reinforcing steel or prestressing tendons. In any case, the tying system should be continuous. This can be obtained by lapping reinforcement or by using threaded couplers, cast-in sockets or other anchored fixings.

4.6.2 Types of ties and resistance

Structures that are not designed to withstand accidental actions should have a suitable tying system to ensure the sufficient integrity and redundancy of the structure. According to Eurocode 2, Part 1-1 [2], the following ties should be used to satisfy this requirement:

- peripheral ties
- internal ties
- horizontal column or wall ties
- vertical ties

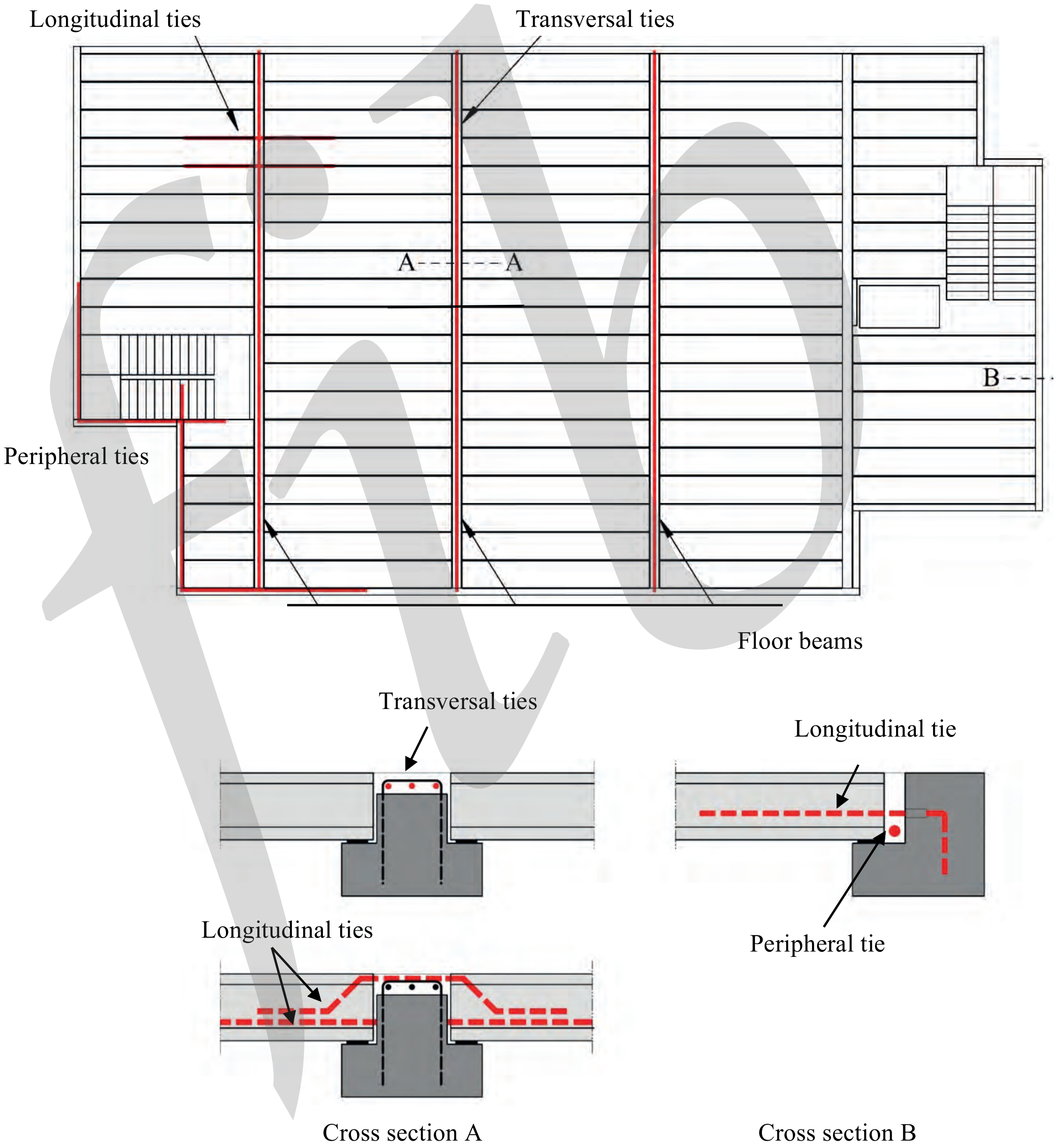


Fig. 4.24: Location of horizontal floor ties. The drawing on left under shows two types of longitudinal ties over the support, either through a hole in the beam, or over the top of the beam into the floor.

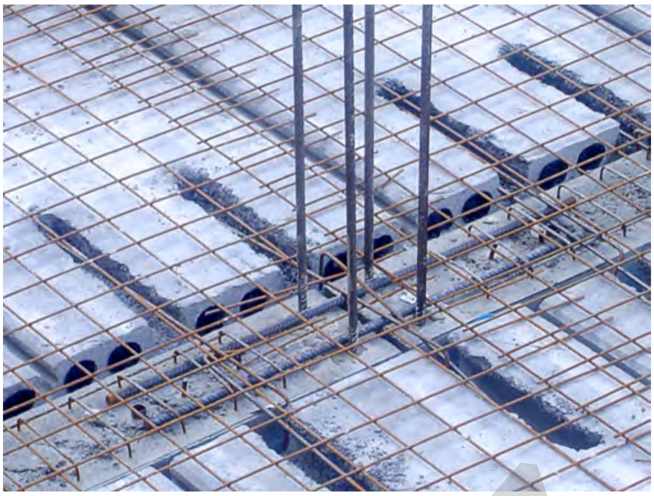


Fig.4.25: Application example of horizontal and vertical ties

4.6.2.1 Peripheral ties

These ties are placed around the entire precast floor, usually in peripheral tie beams or within a distance of maximum 1.2 metres from the edge. Peripheral ties are made continuous around external corners by concreting them into the in-situ edge joints, or by lapping the tie reinforcement with the longitudinal reinforcement in the precast component. At the inner corners of the perimeter of structures with internal edges, the tie reinforcement should be anchored inwards on both sides.

The peripheral ties are calculated as the tensile chord of the floor diaphragm. According to Eurocode 2 Part 1-1 [2] , the peripheral tie should be capable of resisting a tensile force:

$$F_{tie,per} = \ell_1 \cdot 10 \text{ kN/m, but not less than } 70 \text{ kN}$$

where ℓ_1 : length of end floor span

4.6.2.2 Internal ties

These ties are placed at each floor and roof level in two directions, that is to say, parallel and perpendicular to the span of the floor units. The first are longitudinal ties, the second, transversal ties. 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, floor beams, walls or other appropriate positions. In floors without topping where ties cannot be distributed across the span direction, the transverse ties may be grouped along the beam lines.

- Longitudinal ties

They ensure the equilibrium of the horizontal forces acting on internal and façades due to wind, accidental loading, wall inclination, and so forth. They also connect the floors to the supporting structure.

- Transversal ties

They ensure the necessary transversal coherence of the structure. In the case of accidental damage to a supporting wall, they must bridge the damaged area by catenary action to prevent the progressive collapse of the structure. The transversal ties must also take up the horizontal tensile component of the forces acting in the vertical joints between transversal walls. The following satisfactory values are recommended in Eurocode 2 [2] for the tensile capacity of longitudinal ties (design values):

F_{tie} = 20 kN/m x s

where s = spacing of the ties (in m)

The tensile capacity of transversal ties along the beam lines should be at least:

$F_{tie} = [(\ell_1 + \ell_2) / 2]$ x20 kN/m, but not less than 70 kN

where ℓ_1 , ℓ_2 are the span lengths (in m) of the floor on either side of the beam

4.6.2.3 Horizontal ties to columns and walls

Façade columns and walls should be tied horizontally to the structure at each floor and roof level. Corner columns should be tied in two directions. Reinforcement provided for the peripheral tie may be used as the horizontal tie in this case.

Eurocode 2 Part 1-1 [2] recommends the following minimum value for the tie force:

F_{tie} = 20 kN/m for walls

F_{tie} = 150 kN for columns

4.6.2.4 Vertical ties

Vertical ties should take up the bending moments in superposed wall structures and ensure a second load path in case of local damage due to accidental actions. The following requirements are given by Eurocode 2, Part 1-1:

- In panel buildings of 5 storeys or more, vertical ties should be provided in columns and/or walls to limit the damage from collapse of a floor in the case of the accidental loss of a column or wall below. These ties should form part of a bridging system to span over the damaged area.
- Normally, continuous vertical ties should be provided from the lowest to the highest level. The tie should be capable of resisting a tensile force equal to the maximum design ultimate dead and imposed load received by the column or wall from any one storey or the roof.

4.6.2.5 Ties in structural toppings

Ties may also be provided entirely within an in-situ concrete topping. Structural toppings are not always necessary to achieve adequate interaction between the floor units. The trend is to avoid on-site casting work as much as possible and to perform a maximum of the work at the precast plant. Toppings are only necessary where composite action with the floor units is required, or where there are heavy concentrated loads such as those from storage racking and heavy machinery, or moving loads such as those from forklift trucks and in seismic areas. Structural toppings should always be reinforced with a light fabric.

4.7 Design with regard to accidental actions

4.7.1 Design strategies

A structure is normally designed to respond properly, without damage, under normal load conditions, but local and/or global damages cannot be avoided under the effect of an unexpected, but moderate degree of accidental overload. Nevertheless, 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.

The design and calculation of a structure for mitigating the risk of progressive collapse after severe initial local damage requires a different way of thinking than traditional building structural design approaches. The reason lies in the large variation and magnitude of actions and the possible reaction of the building structure. Therefore the guidelines should focus primarily on a design philosophy rather than giving a well-defined recipe based on exact design procedures and calculation formulas.

4.7.2 Design methods

There are three design alternatives to reduce the risk of progressive collapse after severe initial damage. The alternatives are:

- Indirect design approach
- Alternative-load-path approach
- Specific load approach

The last two are also defined as direct design approaches.

4.7.2.1 Indirect design approach

With the indirect design approach, also called the tie-force method, resistance to progressive collapse is considered implicitly through the 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 structural system of ties, a precast structure will achieve an increased ability to prevent the spread of local damage, facilitating alternative load paths and increase robustness after a moderate degree of accidental actions, for example, due to misuse, settlements or execution faults. Design guidelines are given in Section 4.7.3

4.7.2.2 Alternative-load-path method

The alternative-load-path approach presumes that a critical element is removed from the structure due to an accidental loading and that the structure is required to redistribute the gravity loads to the remaining undamaged structural elements. The connections and tie-reinforcement should be designed to resist these resulting actions. Design guidelines are given in Section 4.7.4

4.7.2.3 Specific load resistance method

The method of specific load resistance requires all critical gravity load-bearing members to be designed and detailed to be resistant to a postulated abnormal loading. The other parts of the structure should be designed according to the alternate-load-path method. Design guidelines are given in Section 4.7.5.

The selection of the appropriate method is specified in Eurocode 1, part 1-7 [1] or other national standards and depends principally on the risk. The risk is defined as "a measure of the combination (usually the product) of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequences of the occurrence" in Section 1.5.14. More information is also available on this topic in *fib Bulletin 63: Design of precast concrete structures against accidental actions* [14].

4.7.3 Tie-force method

4.7.3.1 Introduction

Section 4.6 gives guidelines for the structural integrity of precast structures under normal load conditions. This is achieved through tying systems in the transverse, longitudinal and vertical directions that interconnect all individual elements. Sections 4.7.3.2 and 4.7.3.3 give guidelines for the design of tie connections for accidental actions. They are, in principle, also valid for cast in-situ structures and are supposed to resist progressive collapse after a severe local failure. The required tie forces are specified in Eurocode 1 part 1-7 [1] and are higher than those given in Section 4.6 for normal load conditions.

4.7.3.2 Skeletal structures

- Internal ties

$T_i = 0.8(g_k + \psi q_k).s.\ell$ or 75 kN, whichever is the greater (1)

where

s is the spacing of the ties

ℓ is the span of the tie (in the not damaged structure)

ψ is the relevant factor in the expression for combination of action effects for the accidental design situation.

- Peripheral ties

$T_p = 0.4(g_k + \psi q_k).s.\ \ell$ or 75 kN, whichever is the greater (2)

- Vertical ties
  - Each column or wall should be tied continuously from the foundations to the roof level.
  - The columns and walls should be capable of resisting an accidental design tensile force equal to the largest design vertical permanent and variable-load reaction applied to the column from any one storey. Such accidental design loading should not be assumed to act simultaneously with permanent and variable actions that may be acting on the structure.

4.7.3.3 Load-bearing wall structures

- Internal ties

T_i = the greater of $T_i = F_t$ or (3)

$$T_i = \frac{F_t(g_k + \psi q_k)}{7.5} \cdot \frac{z}{5}$$

where

F_t is 60 kN/m or 20 + 4 n_s kN/m, whichever is less

n_s is the number of stories

z is the lesser of 5 times the clear storey height H, or the greatest distance in metres in the direction of the tie, between the centres of the columns or walls spanned by a single slab or a system of beams and slabs.

H is the storey height in metres

- Peripheral ties

$T_p = F_t$

- Vertical ties

Each wall should be tied continuously from the foundations to the roof level

The vertical ties may be considered effective if:

- The clear height of the wall, H, measured in metres between the faces of floors or roof does not exceed 20t, where t is the thickness of the wall in metres
- If they are designed to sustain the following vertical tie force T:

  $T = \frac{34A}{8000} \cdot \left(\frac{H}{t}\right)^2$ N or 100 kN/m wall whichever is the least. (4)

  where

  A is the cross-sectional area in mm² of the wall measured on plan, excluding the non-loadbearing leaf of the cavity wall.

  H is the storey height in metres

  t is the wall thickness in metres
- The vertical ties are grouped at 5-metre-maximum centres along the wall and occur no more than 2.5 metres from an unrestrained end of the wall.

4.7.4 Alternative-load-path method

The alternative-load-path method presumes that local damage occurs in the structure as a result of an accidental action. The building is analysed assuming various alternative locations of local damage to ensure that the acting loads in specific load cases can be redistributed. The structure is required to redistribute all relevant loads in the design, with regard to progressive collapse, to the remaining undamaged structural elements. The connections and tie reinforcement should be designed to resist the resulting actions. The success of this approach depends on the assumptions of local damage.

The alternative-load-path method implies that:

- the local damage must be bridged by an alternative load-bearing system. The transition to this system is associated with dynamic effects that should be considered.
- the entire structure must be shown to be stable even with local damage under the relevant load combination.

An advantage of this method is that it is simple and can be applied at the very beginning of the structural design process. A disadvantage is possible over-mitigation, which could lead to unnecessary additional costs.

The alternative-load-path method implies that a critical element, such as a column or load-bearing wall, is notionally removed 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 path analysis is performed floor by floor for the whole structure. Large displacements are allowed so that catenary action can be considered (Fig. 4.26).

In the 'notional member removal' approach, which mainly focuses on the sudden loss of a critical member, the remaining structure is checked to make sure that it would be capable of redistributing the applied static and dynamic loads and that the extent of any local collapse would not exceed allowable limits.

The mechanisms explained below can be used to provide for an alternative load path in multi-storey, precast-concrete structures.

4.7.4.1 Skeletal structures

Figure 4.26 illustrates possibilities to transfer the loads to the remaining structure:

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

- Catenary action needs anchorage
  At penultimate columns one bay from an edge, 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 lower floor, but not necessarily by the upper floors, due to the damage caused by the column removal.

- Cantilever action of the surrounding structure, for example in case of failure of a corner column
  The horizontal tie reinforcement on top of the floor beam can function as cantilever reinforcement. To this effect the tie-reinforcement should be duly connected to the beam, for example inside projecting stirrups at the top of the units.

- Suspension of the elements to the intact upper structure above the damaged area
  This is achieved by vertical ties from foundation to roof level in all columns and walls.

- Membrane action of floors and roofs

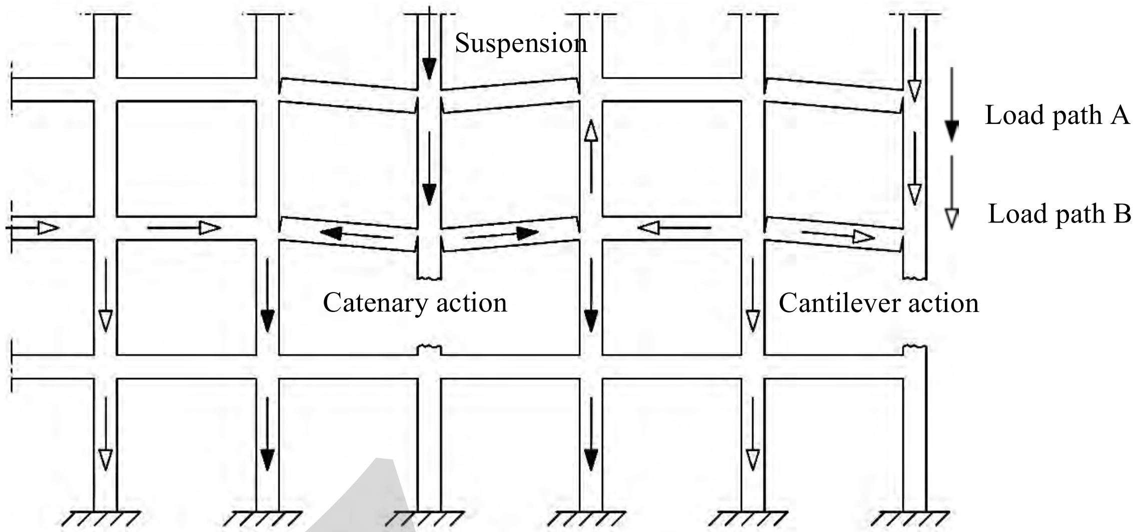


Fig. 4.26: Alternative mechanisms for alternative load path in skeletal structures [41]

4.7.4.2 Wall frame structures

The following mechanisms are generally available in wall-framed structures to provide an alternative load path in case of the notional removal of a load-bearing panel.

- Cantilever action of the wall assembly
- Beam and arch action of the wall panels
- Vertical suspension of the walls
- Catenary and/or membrane action of successive spans of the floor planks above the damaged wall

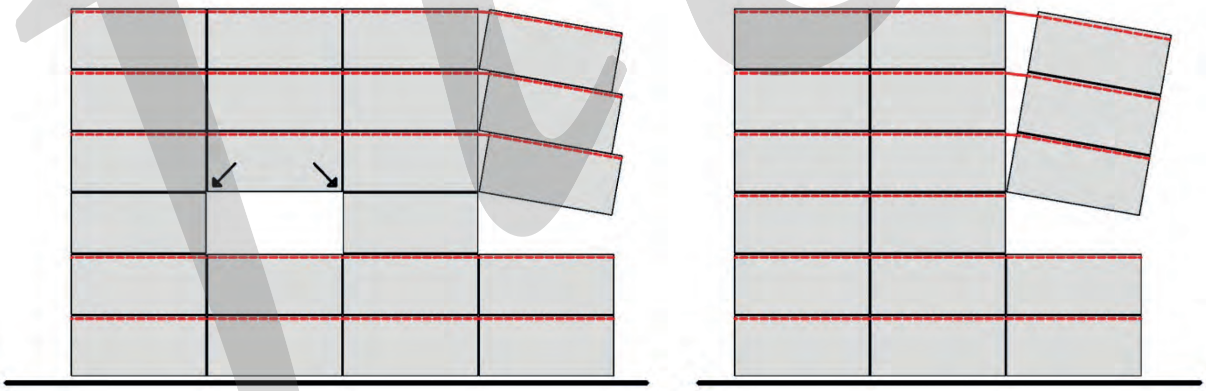


Fig. 4.27: Mechanisms for alternative load path in wall frame structures. Left: an individual cantilever. Right: a composed multi-storey cantilever.

4.7.5 Specific-load-resistance method

Key elements should be capable of sustaining an accidental design action of A_d 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 commonly recommended value for A_d for building structures is 34 kN/m² (design ultimate load). Many standards, however, do not define values for accidental loads, but leave them to the designer.

The difficulty with strengthening key elements is that it must be done with a specific threat in mind. The value of 34 kN/m² is meant for domestic gas explosions. A better strategy to cope with the problem of key element design is to modify the structural arrangement of the building in such a way that key elements do not appear anymore.

Detailed design guidelines are available in the *fib* Guide to good practice "Structural integrity of precast concrete structures under accidental actions" [14].

4.7.6 Comparison with seismic design

Designing a building for earthquake action could improve the resistance against accidental actions. This is due to several reasons, such as:

- Individual members (mostly vertical) are designed with confinement such as to increase not only their compressive strength but also their ultimate compressive strain in the concrete.
- In addition, horizontal members are required to have minimum continuous reinforcement in their compression areas, which is about ¼ of the main reinforcement.
- Floors and roofs are usually designed to provide diaphragm action and to have ductile connections to beams, columns and walls.
- All vertical and horizontal members are tied together both vertically and horizontally with particular attention and requirements to ensure the ductile behaviour of their connections.

Although there is some overlap between the disciplines, mostly in the area of progressive collapse prevention, earthquake resistant buildings are unlikely to meet the direct effects of an air-blast loading acting on the exterior skin of a building. The reasons for differences are as follows:

- Explosion loads act directly on the exterior envelope whereas earthquakes load buildings at the base of the building. Consequently the focus is on out-of-plane response for explosions and in-plane response for seismic loads.
- Explosions are characterized by a single high-pressure, highly localized action acting over milliseconds compared to the seismic action, which acts in an oscillating manner over many seconds on the total superstructure due to its mass response, which in turn depends on the accelerations of the soil and the specific features of the structural system (Fig. 4.28b).
- Explosion actions generally cause localized damage in the initial phase whereas seismic loads usually cause global damage (Fig. 4.28c).

- Mass generally helps resist explosion loads, but worsens earthquake response.

The scheme of Figure 4.28 illustrates the relation between action and response to action, both for explosion and earthquake actions.

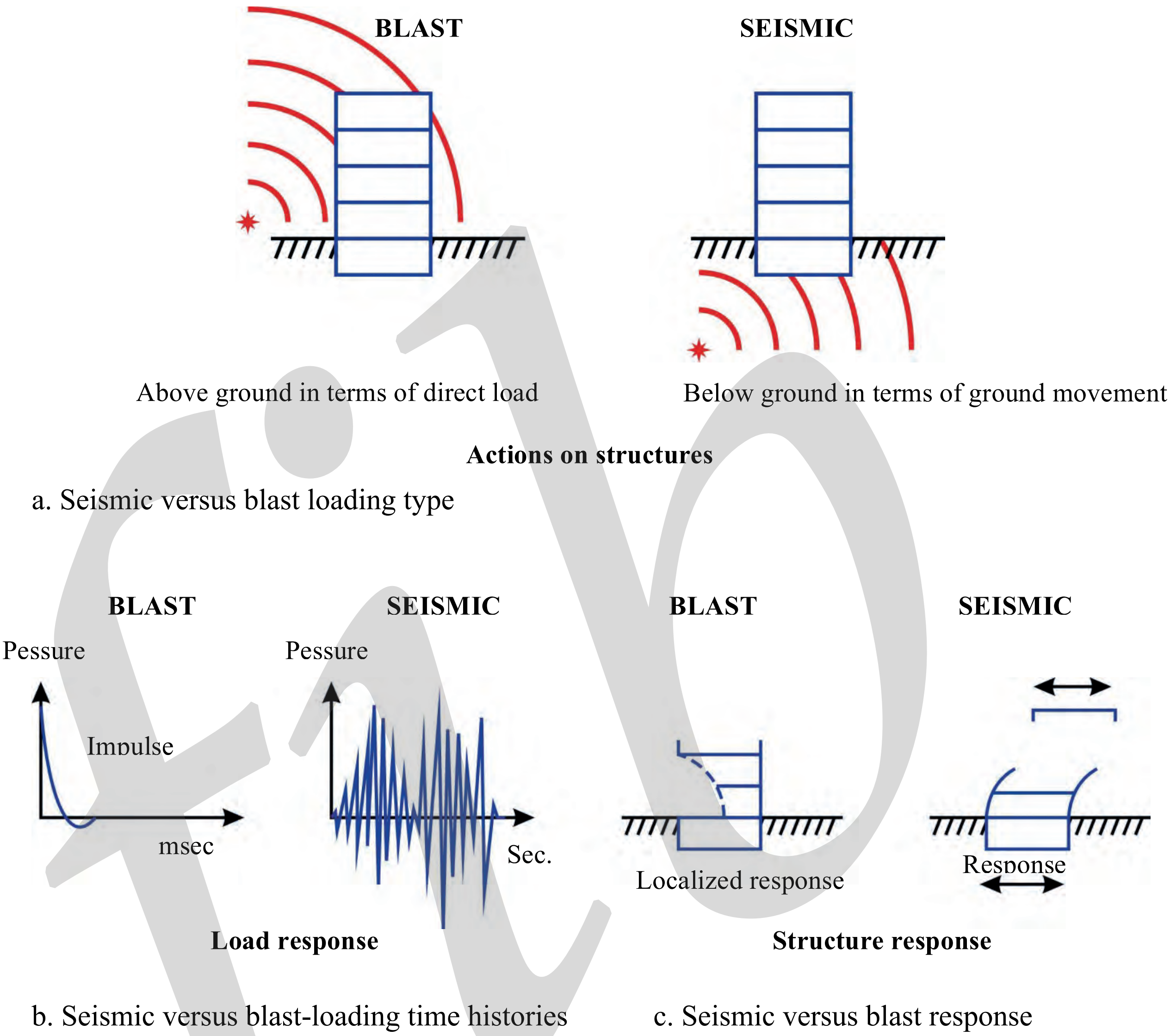


Fig. 4.28: Comparison between the actions of an earthquake and an explosion [14]

5 Structural connections

5.1 General

Connections are among the most essential parts in prefabrication. Their role is to provide a coherent and robust structure out of individual elements that is able to take up all acting forces, including indirect actions resulting from shrinkage, creep, thermal movements, fire, and so forth. The designer should be aware of the flow of forces through the structure under vertical and horizontal loads and understand how connections are interacting within the overall system.

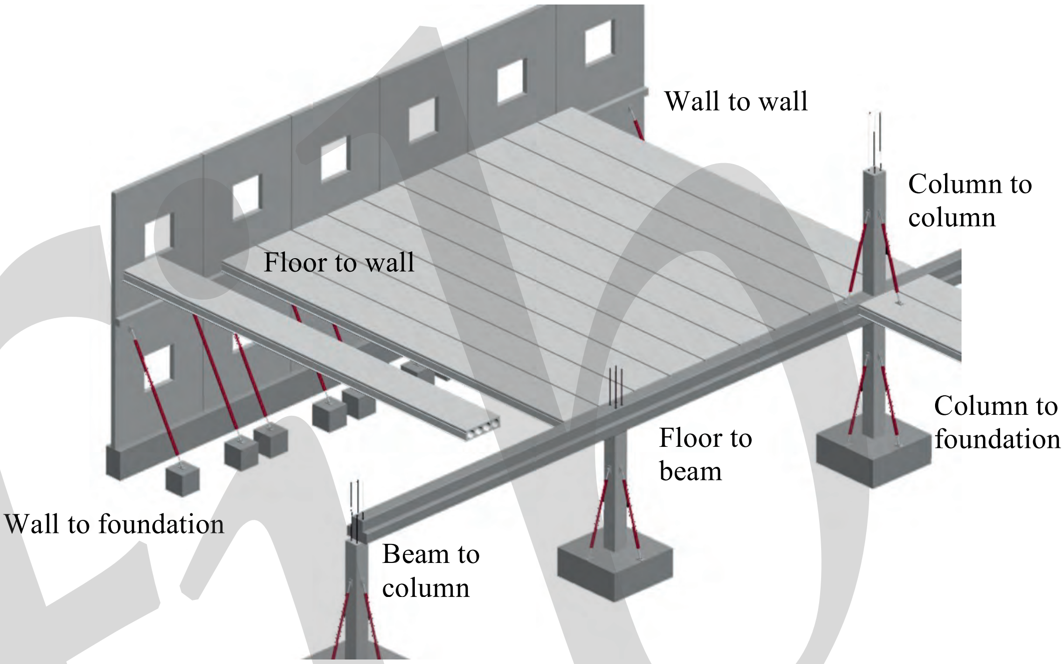


Fig. 5.1: Connections in precast structures

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:

- Connecting units to the bearing structure
- Securing the intended overall behaviour of precast subsystems, such as the diaphragm action of floors and the shear-wall action of walls
- Transferring forces from their point of application to the stabilizing structure

Other aspects to do with the function and appearance of connections may result in specific design and execution requirements, for instance, as regards durability and visual appearance. The detailing of connections should also fulfil requirements with respect to the manufacture, transport and erection of the precast units.

The design of connections is not only a question of choosing appropriate connecting devices. The connection must be regarded as a whole to include joints, joint fill, joint faces and the end zones of the precast units. The end zones provide for the force transfer from the connecting devices into the units and must be detailed and reinforced in regard to the introduction of forces and possible deformations.

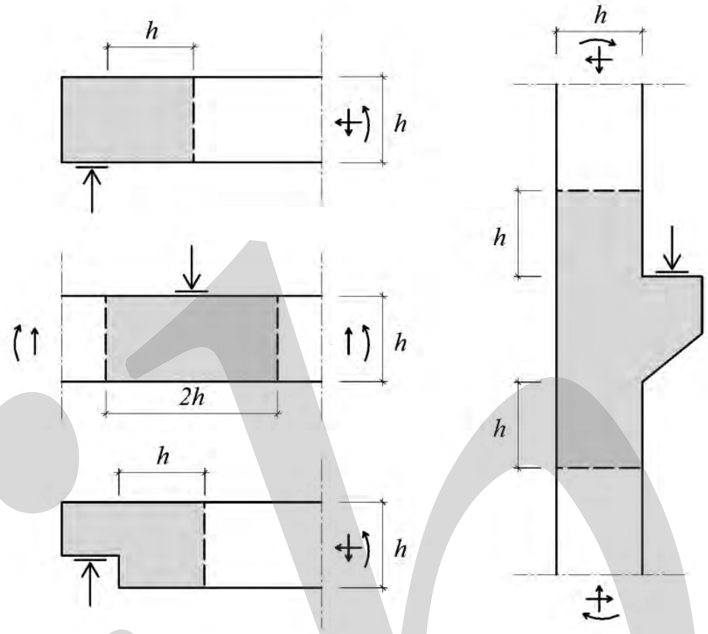


Fig. 5.2 Transition zones for connections

Examples of standardized types of structural connections are usually listed in design handbooks or in the catalogues of precast-element producers. However, the design of structural connections is not just a question of selecting an appropriate solution from listed, standard solutions. The *fib* Commission on Prefabrication published a design guide on structural connections in *fib* Bulletin 43 [12]

In this chapter the basic principles and design criteria are given, enabling the designer to understand the design philosophy of connections in precast structures in general. Practical examples of good connections are also given in Chapters 6 to 9.

5.2 Basic design criteria

The design of structural connections in precast buildings must consider a variety of criteria related to the structural behaviour, including dimensional tolerances, fire resistance, manufacture, handling and erection. The most essential design criteria are described hereafter.

5.2.1 Structural behaviour

5.2.1.1 Strength

A connection should be designed to resist the forces to which they will be subjected during their lifetime. Some of these forces are apparent, caused by dead and live gravity loads, wind, earthquake, and soil or water pressure. Others are caused by the restraint of volume changes in the members or additional forces that might appear due to the unintended inclination of load-bearing columns and walls and unintended eccentricities. In seismic regions the connections must be designed to cope with these actions.

In the design of connections the possibility of accidental actions should also be considered. Forces can be introduced in the connections as a direct effect of accidental loads, such as explosions and collisions. However, in cases where accidental actions cause severe damage on the building structure a need will arise for force redistribution and the formation of alternative load-bearing paths that can bridge the damaged area. The connections, as essential parts of the structural system, should facilitate such transformations. In designing for such situations, it is not only the force transfer capacity that is of interest but also qualities such as deformability and ductility or even the full load-displacement relationship of connections.

5.2.1.2 Influence of volume changes

The combined shortening effects of creep, shrinkage and temperature changes can cause tensile stresses in precast concrete components and their connections. There are, principally, two ways to take care of the need for volume changes, either by allowing the displacements to occur at the connections or by giving the connections the necessary restraint to prevent them. In the latter case the connection must be designed for quite considerable restraint forces. In practice it is possible to choose solutions in between. If some relative displacement is possible, for instance, due to the elastic deformations of structural members or connection details, the restraint stresses will be relieved. Partial freedom for movement will have the same effect. In this context it is not only the force transfer capacity of the connections that should be considered, but also the full load-displacement relationship and the deformability.

5.2.1.3 Deformations

The appearance of deformations in a structural system occurs because of service load, concrete creep and shrinkage, temperature variations, support settlements, and so forth. This is very important at connections where various structural elements meet and may be restrained by each other. If the need for movement is not taken into consideration the risk of damage to the connection zones will arise. Such damage can be especially dangerous when it appears in support regions (Fig. 5.3).

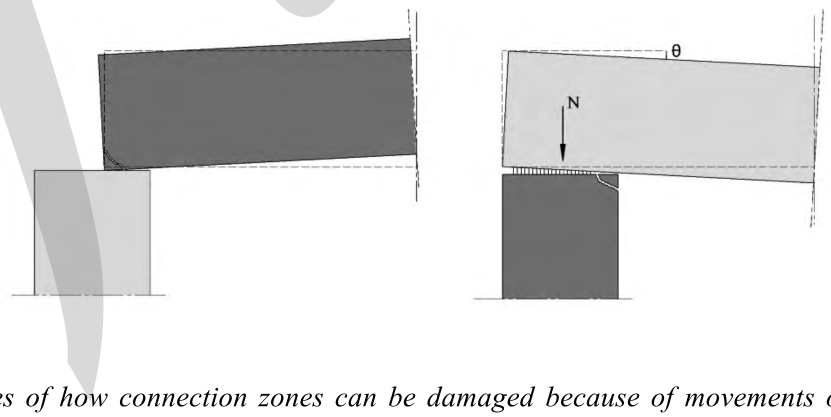


Fig. 5.3: Examples of how connection zones can be damaged because of movements of the precast elements

The need for movement can be considered in two essentially different ways.

One extreme is to fully satisfy the need for movement by detailing the connection so that the corresponding movement can fully take place without restraint. Freedom for movements can be achieved by providing sliding bearings, hinge details, and so forth. Force transferring details can be provided by gaps and slots, among others, so that force is not transferred before the need for movements becomes greater than expected, for instance, in the case of accidental actions and other extreme load cases.

The other extreme alternative is to fully prevent movements between adjacent precast elements. In this case the connection and the elements must be designed to resist the corresponding restraint forces that will develop. These restraint forces can be considerable and, in practice, it is not possible to fully prevent movements from occurring.

5.2.1.4 Ductility

It is always advisable to design and detail the connections to avoid brittle failures in case the connection becomes overloaded by, for instance, underestimated forces. Ductile behaviour of the connections is desirable. Ductility is the ability to undergo plastic deformations without a substantial loss of force-transferring capacity. Ductility is often quantified by the ductility factor, which relates the ultimate deformation to the deformation at the end of the elastic range.

Ductility should not be mixed up with deformability and not only be associated with the transfer of bending moments. In case of overloading, a ductile connection will reach yielding and start to deform plastically. The plastic displacement will give the necessary relief of the restraint force and a new state of equilibrium will be reached. Large displacements will be the result, but the force transfer ability is maintained and brittle failures and damages of connection zones are prevented. These large deformations provide warnings that something has gone wrong.

To secure the ductile behaviour of connections, the principle of balanced design for ductility could be applied. The aim of a balanced design for ductility is to ensure that the full deformation capacity of the ductile links can be mobilized. Brittle failures in the other elements should be prevented before the full plastic deformation is obtained in the ductile ones. Hence, the other links should be designed to resist not only the yield capacity but also the ultimate capacity of the ductile ones with a sufficient margin. In this respect, an unexpectedly high value of the ultimate strength of the ductile component is unfavourable and should better be considered in the design by the introduction of characteristic high values.

Figure 5.4 illustrates the principle for a balanced design on a more complex tie connection. The ribbed anchorage bars are identified as the ductile elements and presumed to play the most important role in plastic displacement. All the other components, namely, the anchorage of the embedded bars, the steel angles, the transverse steel rod and the welds should be designed to resist an unexpectedly high value of the ultimate capacity of the anchor bars.

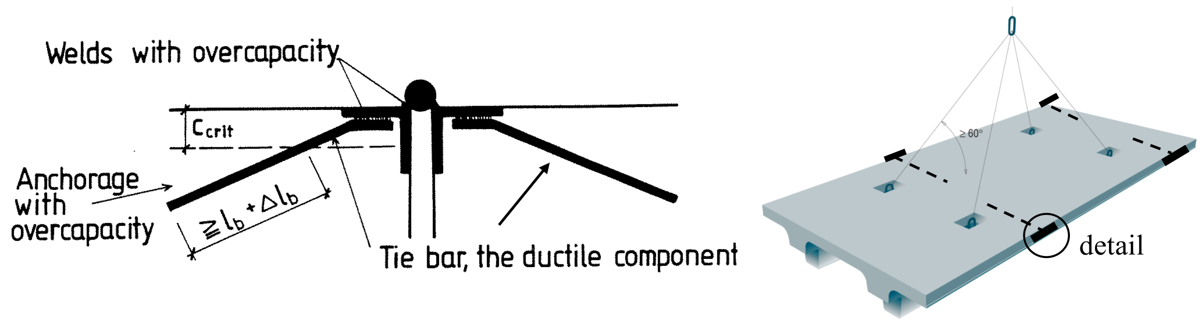


Fig. 5.4: Balanced design for ductile behaviour of a tie connection. The anchor bars are the ductile components. The other components, mainly the weldings, should be designed to resist a high value of the ultimate capacity of the anchorage bars. The bar is anchored in a safe region to prevent splitting failure and the bar is bent into the safe zone, where it is properly anchored by sufficient anchorage length

In order to avoid brittle failures initiated by splitting cracks, sufficient transverse confinement should be provided by, for instance, transverse reinforcement or a concrete cover exceeding critical value. For straight bars, the anchorage length should be sufficient to prevent brittle pull-out failure. In this respect the possible yield penetration in the plastic stage should be considered.

When connection details are placed near free edges of the element, the concrete cover of the anchorage bars may be less than the critical value related to brittle splitting failure. In such cases the bar should be bent into a zone with sufficient concrete cover and anchored by sufficient anchorage length within the safe zone. Alternatively, transverse reinforcement can be provided in the anchorage zone.

When tie bars are anchored in narrow joints or recesses, additional precautions are needed to ensure efficient anchorage. This is especially true when there is a risk of improper encasing of the bar or incomplete grout filling or concrete in the joint or recess.

5.2.1.5 Durability

When it comes to durability, the risk of the corrosion of the reinforcing steel and the cracking and spalling of the concrete must be weighed and attention must be paid to the real environment. Steel exposed to aggressive environments should be covered in permanent protection. This can be achieved by applying a layer of epoxy, rust proof paint or bitumen, or by casting-in with concrete or mortar. In many cases the connections cannot be inspected or maintained after the building has been completed. In such cases the connection, which will receive no maintenance, should have a life expectancy that exceeds that of the structure. If the maintenance of the exposed steel is not possible, the use of stainless steel is recommended. In case of dissimilar metals are used, galvanic corrosion could become a risk. Galvanic corrosion occurs when metals of different nobility come into electrical contact and are bridged by an electrolyte such as water.

5.2.2 Dimensional tolerances

Dimensional tolerances and variations are inevitable in the construction of a building and during the manufacture of the precast units. They must be taken into consideration in the design of the connections or serious problems may occur during the erection of the structure. A typical example concerns the support length of a precast unit. Both the length of the supported element and the position of the supporting structure may deviate from the original design values. These deviations will normally be concentrated at the connections. In this example the dimensional tolerances should be taken up by the support length and the bearing pads.

Another important principle related to dimensional tolerances is that all fixings, of whatever type, should allow for three-way adjustment to enable the units to be adjusted and levelled.

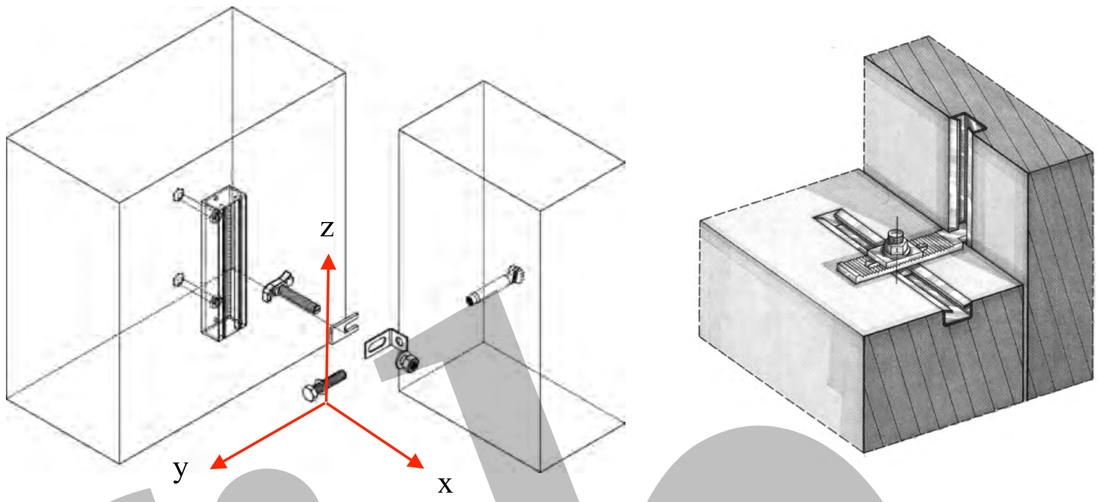


Fig. 5.5: Connections should allow for three-way adjustment

In the example of Figure 5.5 the adjustment in the z-direction is carried out with a vertical anchor rail, in the x-direction by the shims in between the steel angle and the anchor rail, and in the y-direction by the elongated hole in the steel angle.

5.2.3 Fire resistance

There are mainly two aspects that should be taken into consideration in the design of connections for possible fire exposure. One is the effect of fire on the force transferring capacity and the other the separating function of the connections. When connection details are directly exposed to fire, the force-transferring capacity may be reduced as a result of high temperature. Therefore, connection details, which are vital parts of the structural system, should be protected to the same degree as other structural members. Protection can be obtained by using cast-in-situ concrete or fire-insulating materials. However, steel details partly embedded in concrete will experience a lower rise in temperature than non-embedded steel because of the thermal conductivity of the surrounding concrete.

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. If the slabs or beams rest on elastomeric pads or other combustible materials, protection of the pads is not generally needed because their deterioration will not cause collapse.

In the case of fire, walls and floors have an important separating function in regard to thermal insulation and fire penetration. The connections at the joints in walls 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 11.

5.3 Basic force transfer mechanisms

Structural connections are usually composed of a number of components that ensure the transfer of forces through the whole connection: joint fill, tie bars and other coupling devices, anchor bars and the joint zones of the considered prefabricated concrete elements. The transfer of forces from one component to another one, or within a connection as a whole, is based on a number of principles, which are explained in this section.

5.3.1 Encasing

A connection can be created by sliding one component into another and filling the remaining space with grout, fine concrete or even glue. The latter solution is not commonly used in precast concrete. A classic example of encasing is the connection between a precast column and a pocket foundation. Another example is the insertion of a steel billet into a column used for a hidden corbel. Here the space between the opening and the steel billet is normally filled with epoxy glue.

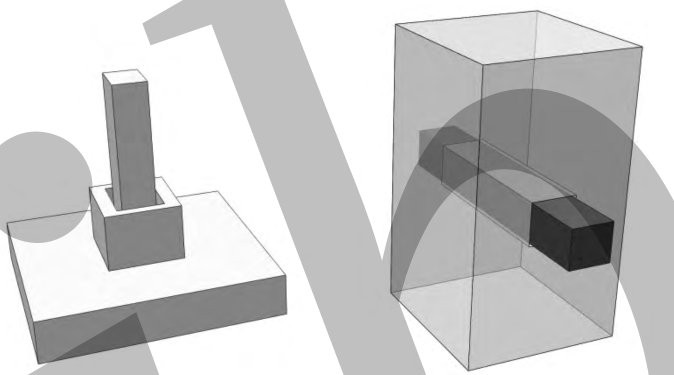


Fig. 5.6: Connections created by encasing

5.3.2 Anchorage of reinforcing bars

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

When anchor bars are anchored by bond, ordinary ribbed or indented reinforcement bars are normally used. It is also possible to use threaded bars. When it comes to bond properties, threaded bars are similar to indented reinforcement bars. In anchorage by bond, tangential tensile stresses appear in the concrete around the bar. By providing sufficient concrete cover and anchorage length, the anchorage capacity will exceed the tensile capacity of the bar.

When this is not possible because of limitations in geometry, the tensile capacity of the tie connection is determined by the anchorage capacity itself. The anchorage can be lost by splitting failure in the concrete cover (Fig. 5.7a) or by bond failure (Fig. 5.7b).

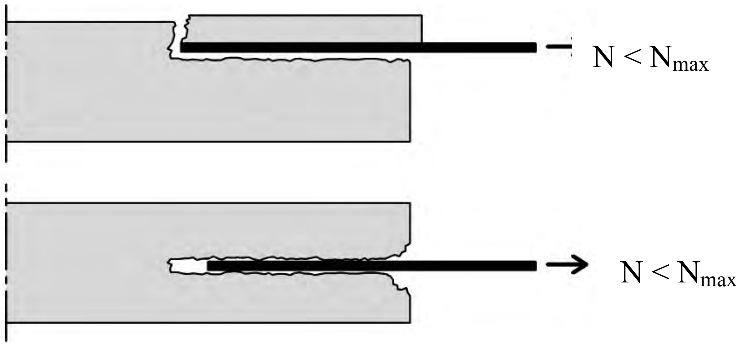


Fig. 5.7: Anchorage failures of ribbed anchor bar with a. splitting failure and b. pull-out failure

End anchors can be anchor heads, anchor bends, or anchor hooks (Fig. 5.8).

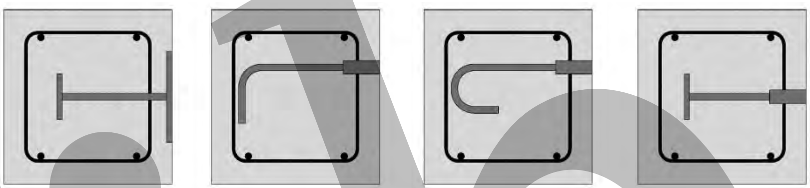


Fig. 5.8: Various types of end anchors

Bolts and studs are provided with anchor heads. Anchor bends and hooks can be used to improve the anchorage of ribbed bars when the space for anchorage is limited. Smooth reinforcement bars must always be anchored with hooks. At the anchor a concentrated force is introduced into the concrete element. This may result in splitting cracks and a brittle anchorage failure.

Two or more adjacent steel bars can be coupled longitudinally by casting them into a concrete prism enclosed by stirrups. The transmission of forces from one bar to the other is ensured when the lap length is sufficient and the clear distance between the bars is limited.

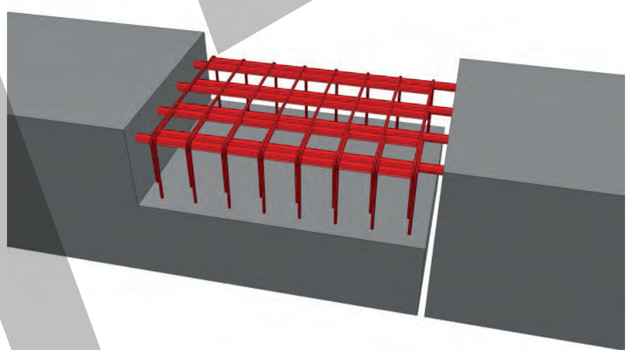


Fig. 5.9: Principle of lapping

Anchorage via the bonding or lapping of reinforcement bars is often used to connect precast members. The precast units have projecting bars, which are embedded in cast-in-situ concrete after erection.

When the necessary anchorage length is not available, the force transfer between reinforcing bars can be created by inserting a transversal bar into two hairpin bars. The force transfer is then based on a combination of lapping and dowel action.

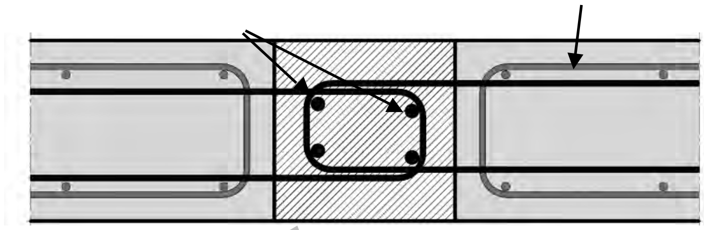


Fig. 5.10: Transversal bar to connect hairpin reinforcements

5.3.3 Dowel action

The transfer of horizontal actions from one element to another is often brought about in precast structures by means of dowel action. This connection principle is illustrated in Figure 5.11.

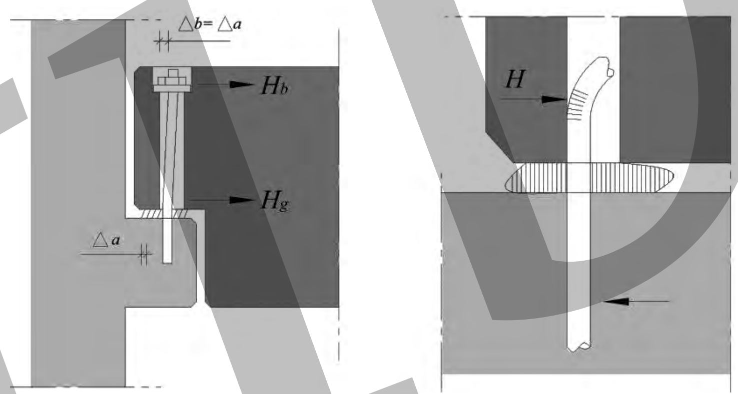


Fig. 5.11: Principle of force transfer by dowel action

The dowel action of partly embedded steel bars is a basic mechanism in the transfer of shear force. The simplest case is when a bar embedded at one end is loaded with a shear force acting along the joint face or at some distance from the joint face (Fig. 5.12). When this load case is considered, in the theory of elasticity, as a beam on an elastic foundation, the concrete stresses in a plane through the dowel pin vary along said dowel pin (Fig. 5.12). As a result, high bearing stresses will occur under the dowel pin near the joint face. The dowel pin will be subjected to shear force, with signs of changes along its length, and a bending moment with a maximum value developing some distance below the joint face.

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 placed in a weak element or in little concrete cover will more naturally lead to the splitting of the element. However, when the bar is placed in well confined concrete (large concrete covers) and the splitting effects are controlled by properly designed splitting reinforcement, the dowel pin will normally fail in the bending by formation of a plastic hinge in the steel bar at some distance below the joint face.

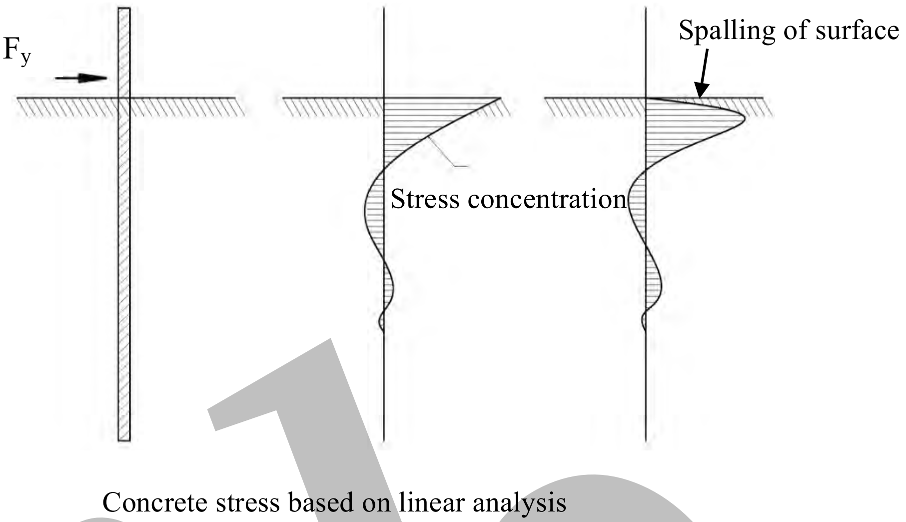


Fig. 5.12: One-sided dowel pin used for transfer of shear force and bearing stresses in the surrounding concrete in the plane through the dowel pin

The connection zone must be designed and detailed so that this concentrated reaction is safely spread and transferred into the element. The concentrated reaction tends to split the element but the splitting can be controlled by reinforcement designed to establish an equilibrium system in cracked reinforced concrete. The strut-and-tie method can be used in such designs (Fig. 5.13).

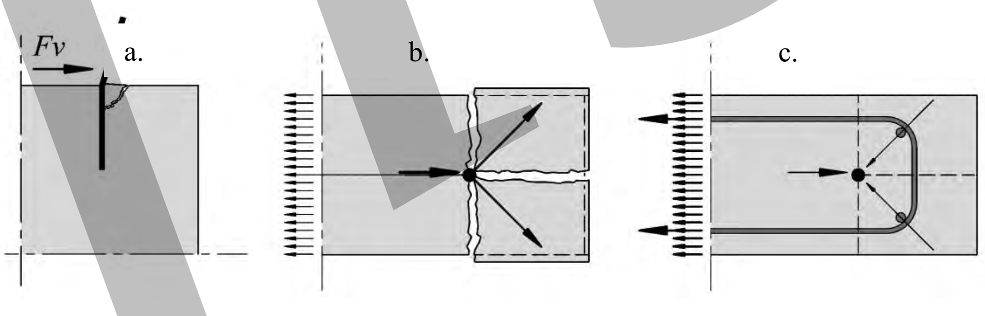


Fig. 5.13: Spitting effects around dowel pin loaded in shear with a. basic load case, b. potential planes of cracking and c. strut-and-tie model for design of splitting reinforcement

5.3.4 Bond

Connection alone by adhesion and bond between, for example, precast concrete and cast-in-situ concrete is only acceptable for small interface stresses, for example, composite action between toppings and precast floors. The factors that affect bond and shear transfer at the interface surface are: surface roughness, surface strength and cleanliness.

Test data shows that the treatment of the precast surface is at least as important as the degree of roughness [18]. Factors such as the cleanliness, compaction, curing and wetting of the surface have a major influence on the shear strength of the interface. Indeed, with the optimum combination of these, and with a good mix design and workmanship for the in-situ concrete, it is possible to develop an interface strength with a flat surface obtained by, for example, extrusion, slip form or natural compaction, which is equal to, or even greater than that obtained with a rough surface, where less attention has been paid to surface cleanness, excessive surface water, and so forth.

5.3.5 Friction

In a joint interface with some roughness, shear forces are mainly transferred by friction. However, compressive stresses are needed at the joint interface to create the friction resistance (Fig. 5.14a). A permanent compressive force can be obtained by gravity load that is transferred across the joint, or by prestressing. For many applications it is not possible to obtain a compressive force in this way. However, it is possible to induce compressive forces by reinforcement bars, which are placed across the joint and strained when the connection is loaded in shear (Fig. 5.14b). 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. The induced compressive force makes shear transfer by friction possible, the so-called shear-friction effect (Fig. 5.14c). The shear resistance increases with increased amount of transverse reinforcement, and increased frictional coefficient.

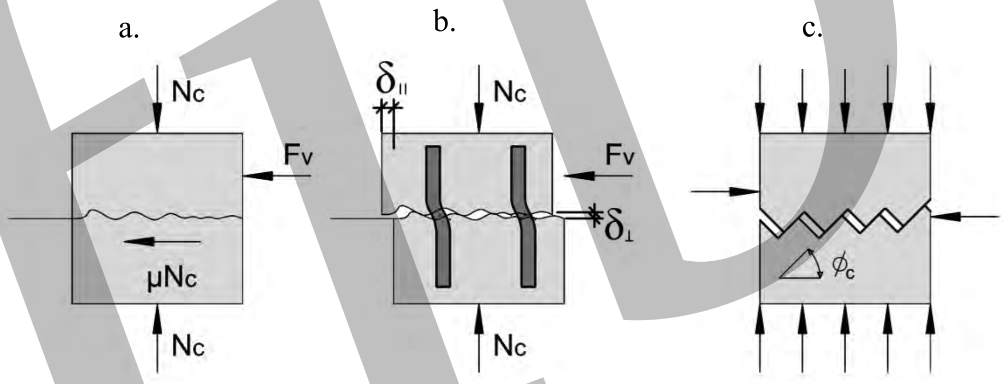


Fig. 5.14: Transfer of shear forces by friction

The principle of transfer of forces by bond and friction is applied in grouted longitudinal joints between floor and wall elements.

5.3.6 Shear interlock

Shear forces can be transferred through indented joint faces (Fig 5.15 a). The keys work as mechanical locks preventing any significant slip along the joint. Prerequisite to the correct functioning of the system is that the elements are prevented from moving apart under shear loading. This is usually done by means of reinforcing ties at the top and bottom of the joint (Fig 5.15.b). Another solution is to place transverse steel loops along the length of the joint (Fig.5.15.c).

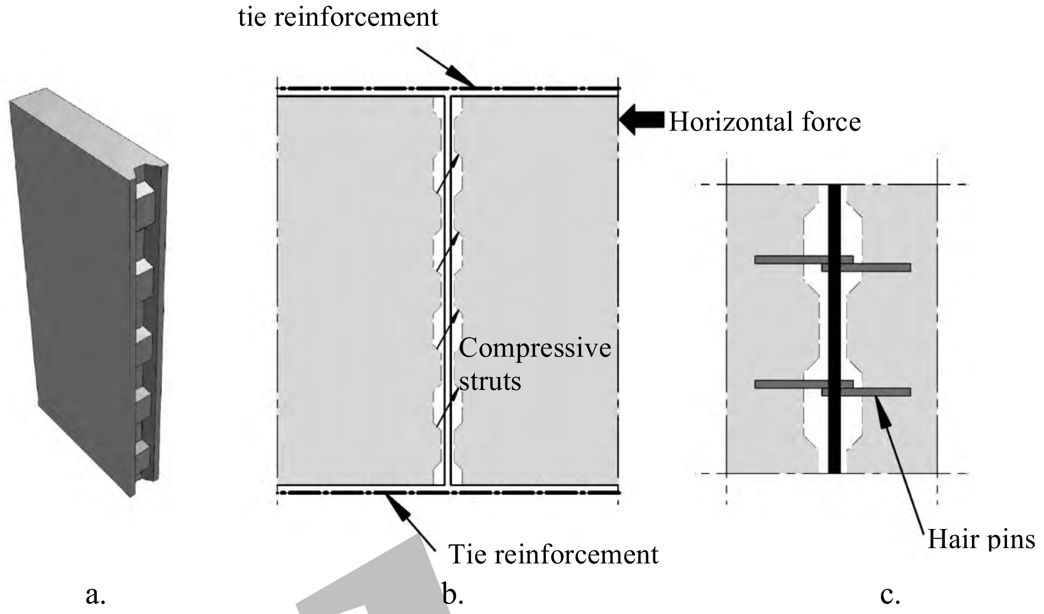


Fig. 5.15: Force transfer through indented joint faces

5.3.7 Staggered joints

Shear forces can also be transferred by staggered vertical joints in walls (Fig. 5.16). The shear force is transferred from one component to the other by compression in the horizontal part of the joint. The horizontal force component is taken up by the tie reinforcement above the wall panels.

Fig. 5.16: Transfer of shear forces by staggered joints in wall panels

5.3.8 Bolting

Bolting is used extensively to transfer tensile and shear forces. Anchorages, such as bolts, threaded sockets, rails or captive nuts attached to the rear of plates are anchored in the precast units (Fig. 5.17). Tolerances are provided using over-sized holes in the connecting member.

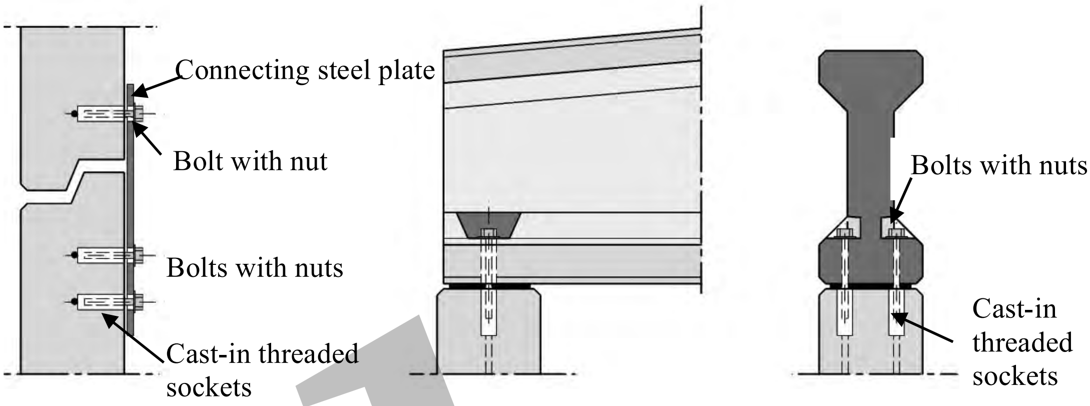
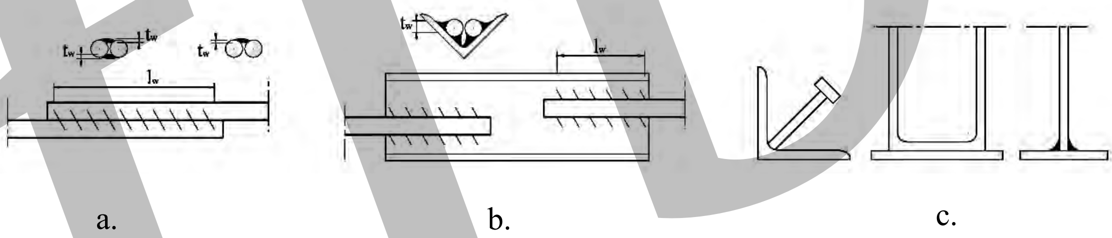


Figure 5.17: Examples of bolted connections

5.3.9 Welding

Welding can be used to directly connect protruding details, for example, reinforcement bars that overlap (Fig.5.18a).



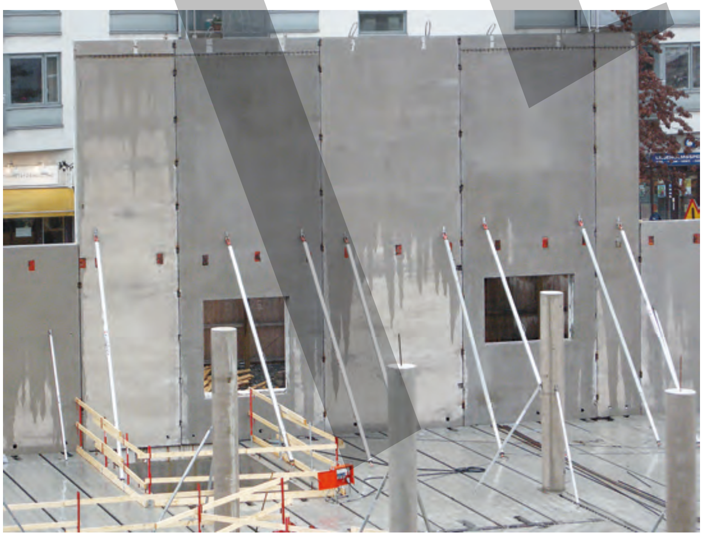


Fig. 5.18: Types of welded connections

An alternative is to use an intermediate steel piece that is used as a link between the concrete units. The intermediate piece can be welded to protruding details (Fig. 5.18b) or to anchor plates or angles embedded in the element surface (Fig. 5.18c). The anchor plates are fixed to the concrete elements by welded bars anchored in the units by bond or end anchors arranged according to the principles detailed previously.

5.3.10 Post-tensioning

Post-tensioning is used in segmental construction and in the walls of tall buildings. Cable ducts are installed in the units and after erection, the prestressing cables are placed in the ducts and post-tensioned and grout filled. The joints between the units are able to resist tension and shear forces.

5.4 Types of structural connections

Structural connections can be categorized by the types of action that are to be resisted:

- Compression
- Tension
- Shear
- Flexure
- Torsion

For many structural connections, behaviour is dominated by one of the actions mentioned above. However, the connections should very often be capable of transferring a combination of these basic actions. For example, besides support reaction, a connection between floor beams may need to transfer bending moments and shear.

5.4.1 Connections transferring compressive forces

Every precast concrete element has to be supported at one or several locations in order to transfer its own weight down to the foundations. These forces will, normally, be compressive forces. In most cases, dead loads from other elements and live loads will increase these compressive forces. Compression forces between adjacent elements can be transferred via direct contact, via joint mortar or similar padding, or via bearing elements.

5.4.1.1 Principles for transfer of compressive forces

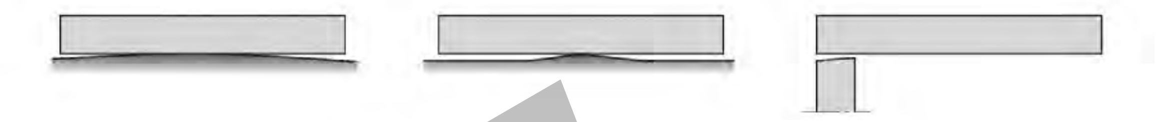
A connection designed to resist compressive forces must include the design and detailing of the splitting reinforcement (local reinforcement) in the adjacent components due to the force paths from the joint to the main member reinforcement. The main factors to consider include:

- Stress concentrations due to uneven contact faces
- Lateral tension stresses in the transition zones due to concentrated loading
- Lateral expansion through different materials
- Possible effects from rotation of supported members

- Stress distribution in contact faces

It is important to consider the risk of uneven contact faces. They may cause stress concentrations at the effective contact areas, eccentric application of forces and torsional effects (see Fig. 5.19). Direct contact between the elements with no intermediate bearing material can only be used where great accuracy in manufacture is obtained, and where the bearing stresses are small ($\leq 0.3\ f_{cd}$ according to Eurocode 2, Part1-1 [2]).

Mortar or fine concrete is used to even out irregularities between the joint faces. It is often used in joints between load-bearing elements, such as columns and walls, sometimes between floors and supporting beams, but seldom under beams. Normal joint thickness ranges between 10 to 30 millimetres for mortar and 30 to 50 millimetres for fine concrete.



a. Uneven contact surfaces b. Local stress concentrations c. Unparalleled contact faces

Fig.5.19: Examples of uneven contact faces at bearings

Soft bearing materials like neoprene bearing pads will also even out irregularities and distribute the stresses over the contact area. They are often used for supports under beams and floors. The thickness varies between 2 and 20 millimetres or even more. The larger thicknesses are used to allow displacements and rotations in order to reduce force built up at the connections. Above a certain thickness and loading, the bearings can be made from neoprene laminates with intermediate steel plate reinforcement.

- Lateral tension stresses in the transition zones

When concrete members are subjected to concentrated loadings, transverse tensile stresses will occur in the contact zone. The tensile forces transverse to the axial loads are often referred to as 'splitting' forces. The Model Code, CEB-FIP (1992), refers to 'splitting' forces close to the load, and 'bursting' forces in the transition region.

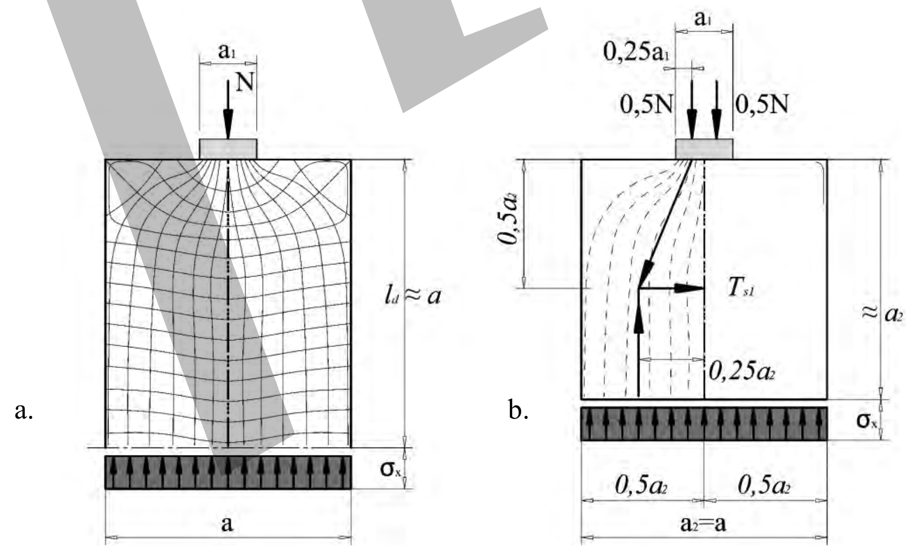


Fig. 5.20: Stress field under one centric load

An eccentric load will increase the tensile forces close to the load, and reduce the tensile forces in the transition zone (Fig. 5.21). One simplified approach to estimate these forces may be to treat them separately (Fig. 5.22). See *fib* Bulletin 43 for more details [12].

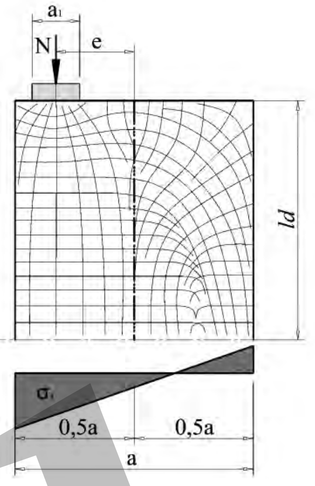


Fig.5.21: Stress field under one eccentric load

Lateral dilatation of locally compressed concrete is hindered by the surrounding mass of the non-loaded concrete and by the presence of surrounding stirrups or helical reinforcement, which provides lateral confinement to the loaded strut.

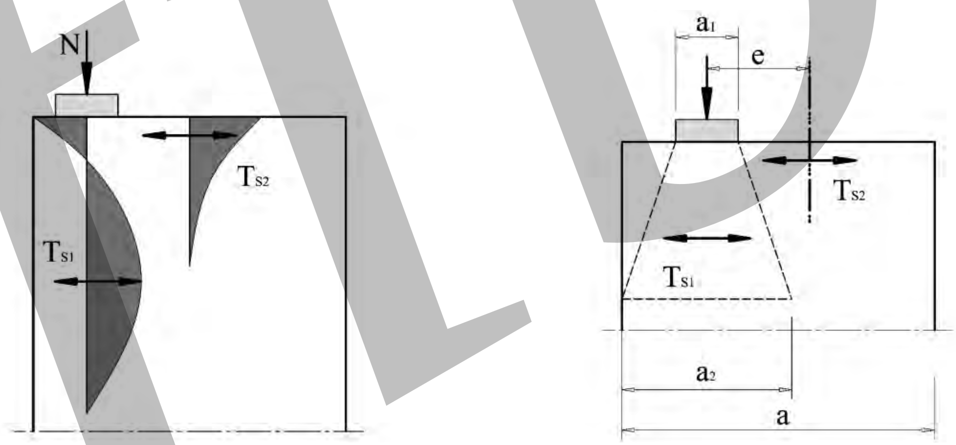


Fig. 5.22: Simplified approach to consider eccentric load, a. transverse stresses and resultants, b. notation

The concrete compressive failure mode will practically always be due to the secondary tensile stresses. It is therefore emphasized that the lateral tensile (splitting) forces in the concrete in the transition zone normally are of much greater importance than the local concrete compressive stresses directly under the load bearings.

- Lateral expansion through different materials

The deformation of the supporting material can also lead to significant tensile stresses in the adjacent elements.

- Steel joint

- Steel has a lower value for ν/E (lateral strain) than concrete and will therefore impose lateral compressive stresses σ_y on the concrete, (Fig. 5.23). The steel plate can be seen as transverse reinforcement in the connection. These compressive stresses will increase the concrete bearing capacity. However, the effects are small, and are normally neglected for both concrete and steel in connection design.

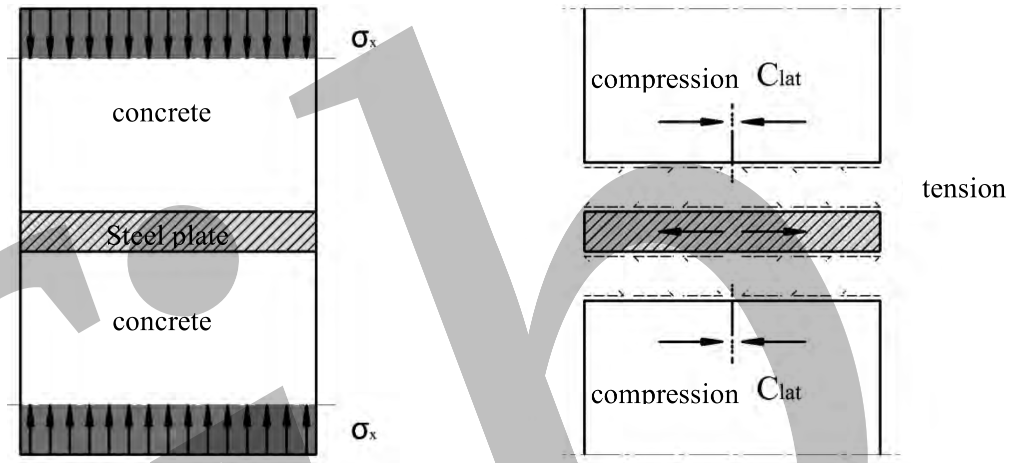


Fig. 5.23: Compression through concrete and steel plate

- Mortar joint

  The mortar will normally be of poorer quality than the concrete elements. The mortar, thus having a higher lateral strain ν/E than the concrete elements, will cause lateral tensile stresses in the elements close to the joint, and cause lateral compressive stresses in the mortar (Fig. 5.24). The tension effect on the concrete element is normally very small compared to other effects, and is often neglected in connection design. The compression effect on the mortar is of great importance and will normally increase the bearing capacity σ_x to the level of the concrete element. However, account should be taken of an ineffective zone at the edges of the mortar joint, because the edges of the mortar bed tend to spall off. The ineffective zone could be taken equal to about half the mortar joint thickness. However, when the joint is confined by surrounding concrete, for example by a structural topping, the mortar joint behaves as a hard pack and would achieve compressive stresses greater than f_{cu}.

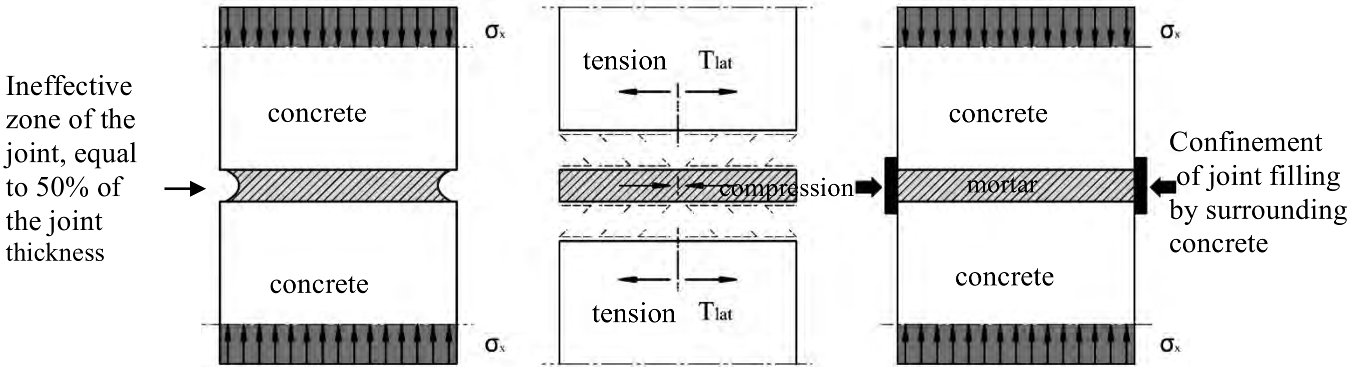


Fig. 5.24: Compression through concrete and mortar joint

Good quality mortar or fine concrete joints between concrete columns and walls are usually considered as hard pack. The load bearing capacity should, in principle, be governed by the resistance of the adjacent elements and not by the capacity of the joint. According to the German DIN 1045 Standard [6], this condition is fulfilled when the following requirements are met:

$r_c \geq 0.5$

$r_{th} \leq 0.7$

where

r_c = the ratio between the compressive strength of the mortar used and the lowest compressive strength of the adjacent precast concrete parts.

r_{th} = the ratio between the joint thickness and the joint width. It is generally accepted that the condition is always fulfilled for joints, which are enclosed by structural concrete, for example a structural topping.

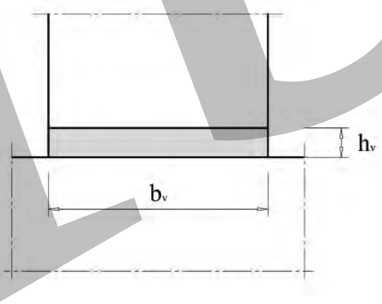


Fig. 5.25: Dimensions of mortar and concrete joints

- Joint with soft materials

Soft materials, like plain elastomeric bearing pads, have much larger values for ν/*E* than concrete. The effect will be the same as for mortar, but with much higher values (Fig. 5.26). The lateral strain is often so large that the pad will expand along the concrete surface. The friction is a function of the surface roughness and the type of pad.

The tension effect on the concrete element is often of such a degree that it should be included in the design of the splitting reinforcement. The compression effect on the bearing pad is essential for the bearing pad behaviour and is always included in the design.

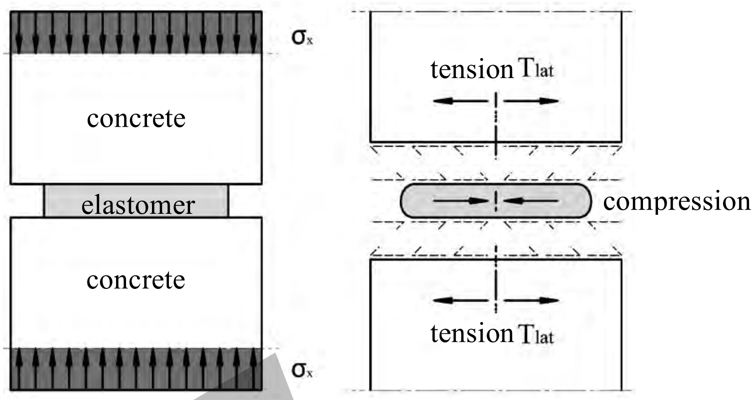


Fig. 5.26: Compression through concrete and plain bearing pad

- Possible effects from rotation of supported members

The pads should be placed at some distance from the support edge, as load transfer at the edge might result in damages. The pad should make allowance for beam deflection so that direct contact between the beam and the support edge is avoided. Hard bearing materials, such as steel plates, are used where large forces are to be transmitted, or for welded connections between the supported units.

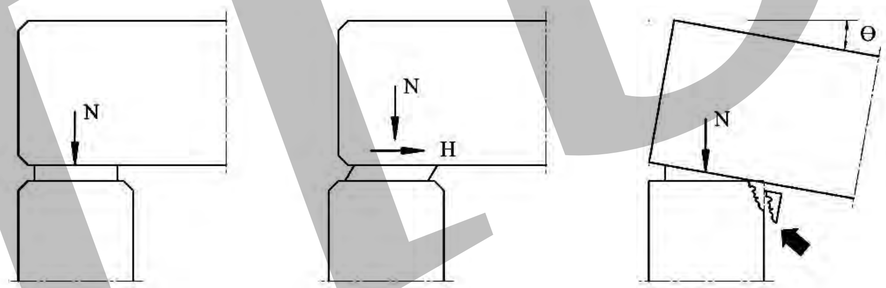


Fig.5.27: Loading conditions on elastomeric pads

5.4.1.2 Bearings

Bearings are a substantial part of the structural functioning of connections. Their role is to ensure a correct transfer of loads and other actions from the supported element to the supporting one. In this section an overview is given of the existing bearing types. Detailed information on the dimensioning of supports is given in Chapter 10 and in *fib* Bulletin 43: Structural connections for precast concrete buildings [12].

<u>Bearing types</u>

The following types of bearings are used (Fig. 5.28):

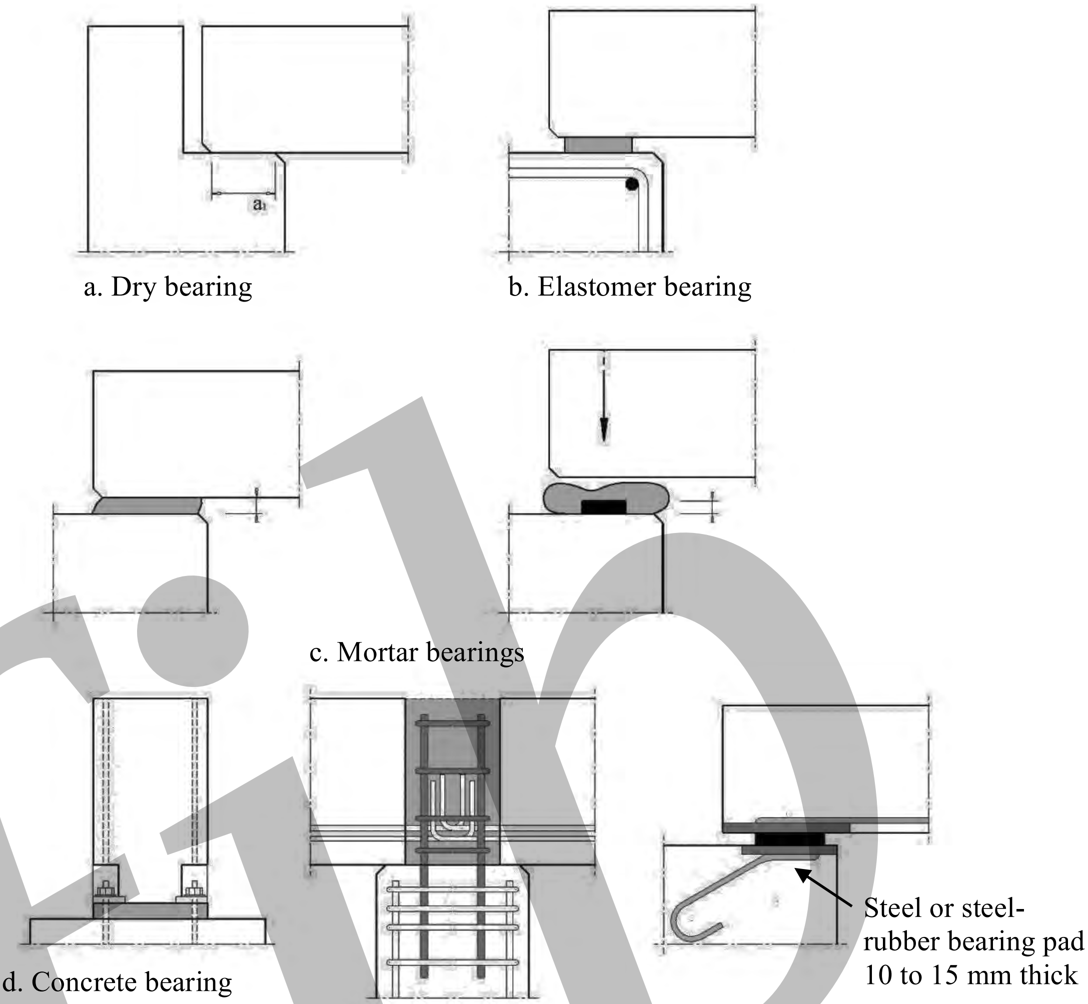


Fig. 5.28: Types of bearings

- Dry bearing of precast-to-precast or precast-to-in-situ concrete
- Elastomeric or soft bearing using neoprene rubber or similar bearing pads
- Thin mortar bearing where components are positioned onto a prepared semi-wet sand/ cement grout or located on thin (3 to 10 millimetre thick) shims and the resulting small gap is filled using semi-dry sand/cement grout
- Hard concrete bearings
- Indirect supports where the temporary bearing is small and reinforced in-situ concrete is used to complete the connection
- Steel bearing using steel plates or structural steel sections

a. Dry bearings

Compressive connections without joint material can only be used for short slabs with small support loads, negligible rotation and horizontal forces (Fig. 5.29).

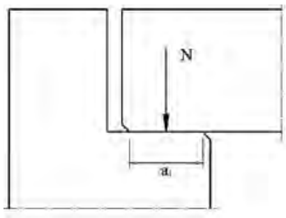


Fig. 5.29: Connection with concrete against concrete

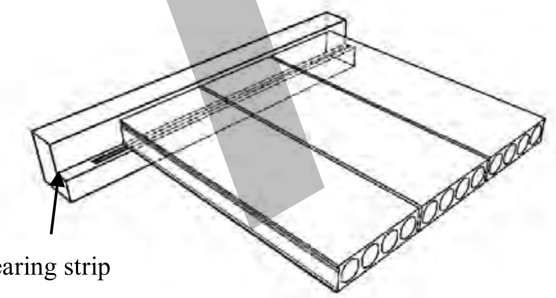
The effective support length a_1 must be adequate, taking into account production and erection tolerances (see Chapter 10, Section 10.2.2). The bearing stress $\sigma_m = N/(a_1 \cdot l)$ should be limited to approximately 0,2 - 0,3 N/mm^2. The corner of the supporting member should be chamfered.

b. Elastomer bearings

Many types of bearing strips are used for slabs. Most types are composed of various rubber profiles but strips of high-density plastics are also extensively used. Hollow core slabs and solid slabs normally have a support load varying between 10 to 100 kN/m. The compressive stress in the bearing strips in the serviceability limit state normally varies between 0,5 to 3,0 N/mm^2 (Fig. 5.30).

Synthetic rubber bearings are usually called elastomeric bearing pads. Neoprene and chloroprene are synthetic rubbers with special resistance to ozone, chemicals, heat and cold. The rubber hardness is normally classified by Shore A. Normally, a 50 to 70 Shore A will be used.

Rubber pads are normally designed in the serviceability limit state because of the very large deformations they may undergo at ultimate load. Rubber is practically an incompressible material (Poisson's ratio is 0,5) and will therefore show large lateral expansion (bulging) when subjected to compression. The lateral expansion can be restrained in two ways, namely, by friction in the loaded contact area and by vulcanized reinforcement. The effect of reinforcement varies from negligible (one layer of fibre) to very large (several layers of steel). The latter is mainly used in bridge construction.



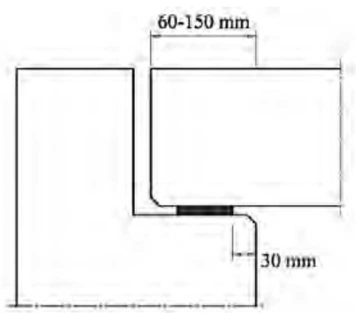


Fig. 5.30: Support connection of hollow core slab

c. and
d. Mortar and concrete bearings

Joints filled with mortar, grout and concrete are typical in wall and column connections. The thickness of mortar bearings ranges from 10 to maximum 30 millimetres, and for concrete bearings from 30 to 50 millimetres. Horizontal movement is not allowed and only a very small rotation is permitted. The basic behaviour of this type of joint is explained in Section 5.4.1.1. The mortar or concrete can be placed either immediately before the erection of the wall units, or after erection.

e. Indirect support or extended bearings

The term indirect support is used for bearings that are constructed in two steps. First the beams are placed on provisional supports and penetrate over a short distance of about 30 millimetres in the joint. Then the projecting reinforcement from the beam ends is completed with additional stirrups and a mould is placed on the joint edge. In a final step, the joint between the beams is filled with in-situ concrete. In this way, a partial support continuity is created.

Fig. 5.31: Example of an extended bearing

The system has advantages and disadvantages:

- The connection is without corbels
- The width of the column can be smaller than the normal support length of the two beams plus the joint width
- Dimensional tolerances on the beam length are easily absorbed
- The solution is complex
  There is a need for provisional support and side moulding. It is difficult to add additional reinforcement, to compact the in-situ concrete and to remove the temporary support.
- The solution is time consuming and not really industrialized (semi-prefabrication)

Extended bearings are often used to restore the monolithic characteristic of traditional cast-in-situ construction and, as such, are not in line with the basic design philosophy of precast construction.

f. Steel bearings

Steel bearings are used for very high support stresses, for example, in bridge construction, or when the support length is too small for normal bearings. Another application is for precast units connected by welding (Fig. 5.54).

5.4.2 Connections transferring tensile forces

Tensile forces are normally transferred between concrete elements by various types of steel connections: lapping of projecting reinforcement, dowel action, bolting, welding, mechanical connectors, and so forth. The tensile force capacity of the connection can be determined by the strength and cross-section of the steel details and /or by their anchorage capacity. The latter can be obtained by bond action along deformed bars or by means of various types of end anchors.

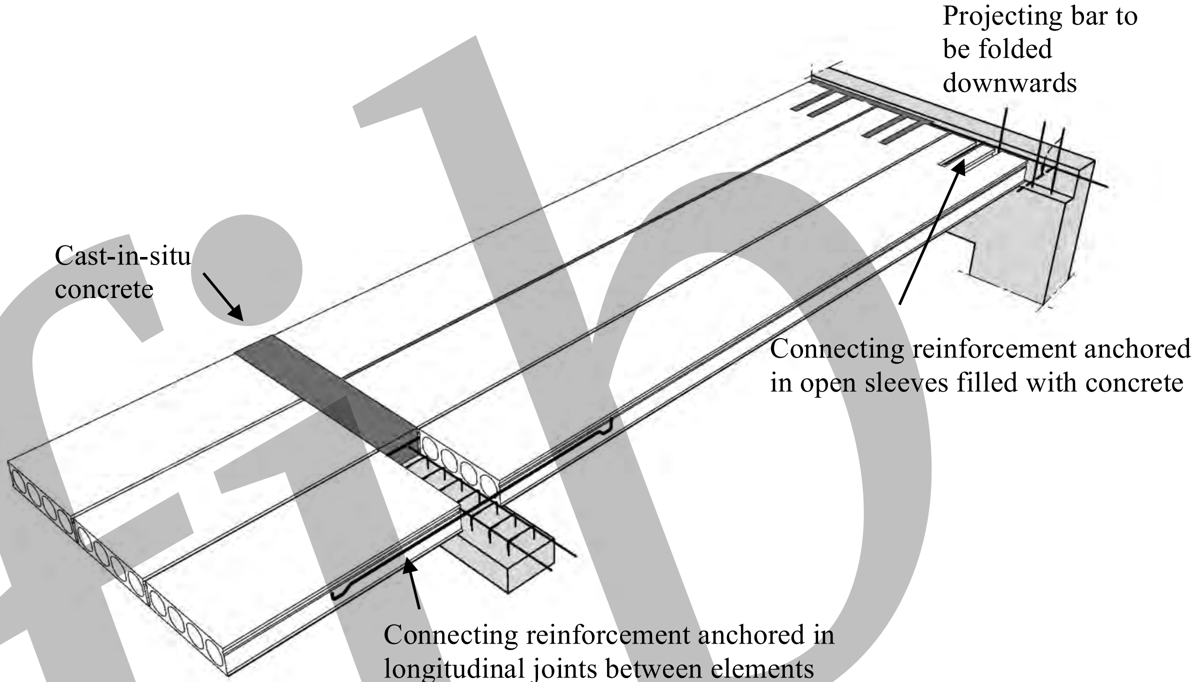


Fig. 5.32: Tensile connections with projecting and other reinforcement

The tensile force capacity of the connection can be determined by the strength and cross-section of the steel details and/or by their anchorage capacity. The latter can be obtained by bond action along deformed bars or by means of various types of end anchors.

Anchorage by lapping is often used to connect precast members. The precast units have projecting bars to be embedded in cast-in-situ concrete after erection (Fig. 5.32). When the length of the straight bar is too short for complete anchorage, end anchors can be used in the form of anchor heads, bends, hooks and similar. The force transfer is created through lapping with the reinforcement in the units, sometimes in combination with dowel action, or other means.

Loop connections, as exemplified in Figure 5.33, can be used to transfer tensile force, bending moment and shear force. They are used between solid slabs where continuity is demanded. However, production is more difficult due to projecting bars.

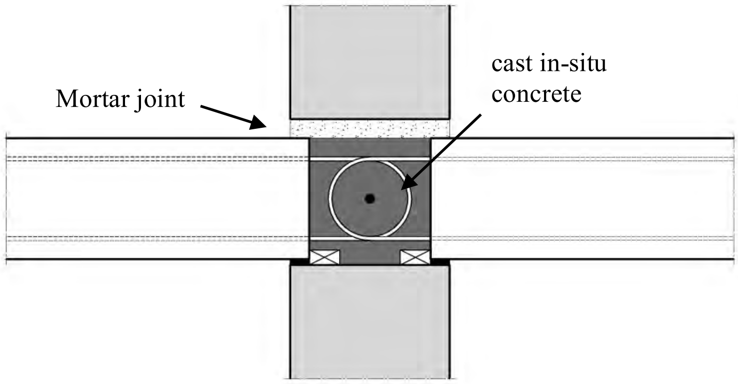


Fig.5.33: Tensile joint by lapping reinforcement combined with dowel action

The tensile force from one element to the other is transferred by inclined compressive struts between overlapping loops (Fig. 5.34).

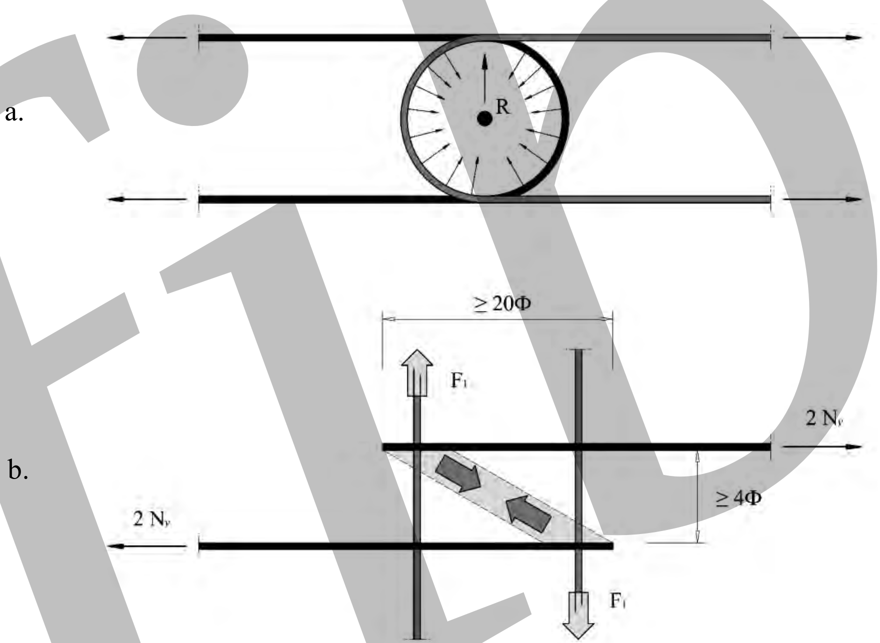


Fig. 5.34: Transfer of forces in loop connection; a. radial stresses against the bend, b. inclined compressive strut between overlapping loops

The inclined strut originates from radial stresses acting against the bend of the loop. The spread of stresses from the loop to the strut causes local splitting stresses due to high bearing stresses inside the loop. Furthermore, the inclination of the strut also creates a transverse tensile force that needs to be balanced. As a consequence of this model the transverse reinforcement needs to be distributed between the two ends of the overlap and be placed inside the loops.

The transfer of tensile forces between adjacent units may also be accomplished by using steel bars grouted in cast-in tubes lapped with the main reinforcement in the column. A typical example is the connection between a column and the foundation, or between two superposed columns (Fig. 5.35).

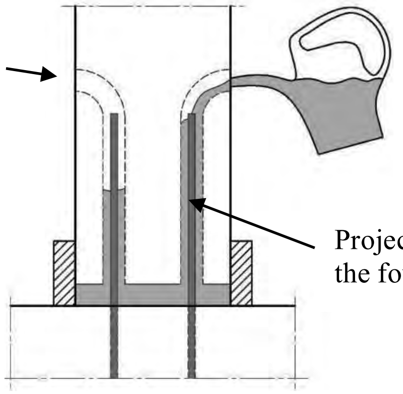


Fig. 5.35: Anchorage of projecting bars in grouted tubes

Figure 5.36 shows a façade panel fixed to a floor by bolting or welding. Tolerances are provided using anchor rails, elongated holes in the steel angles and shims between the connecting members. In the case of a welded connection, a steel angle is welded to an anchor plate embedded in the façade panel and a steel angle embedded in the corner of the floor. It is preferable to leave a small gap around the anchor plate to avoid local spalling of the concrete through thermal dilatation of the plate during the welding.

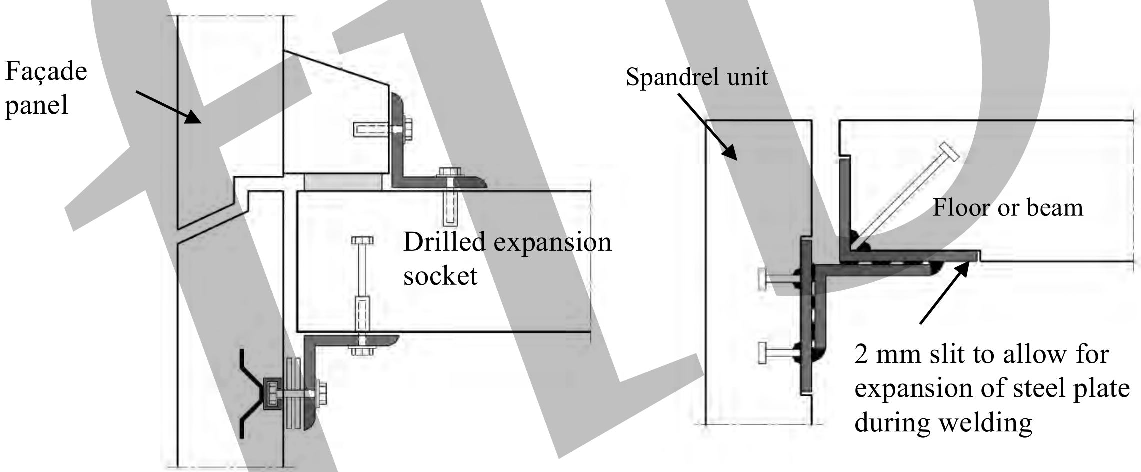


Fig. 5.36
Left: Tensile joint through bolting
Right: Tensile joint through welding

The tensile capacity of the welded connection can depend on the capacity (strength and dimension) of tie bars, connection details and welds, among others, but also on the anchorage of the steel details in the concrete elements. Anchorage can be obtained by bond along ribbed bars or by various types of end anchors. Examples of how embedded weld plates can be anchored by ribbed anchor bars, or smooth bars with end anchors are shown in Figure 5.37.

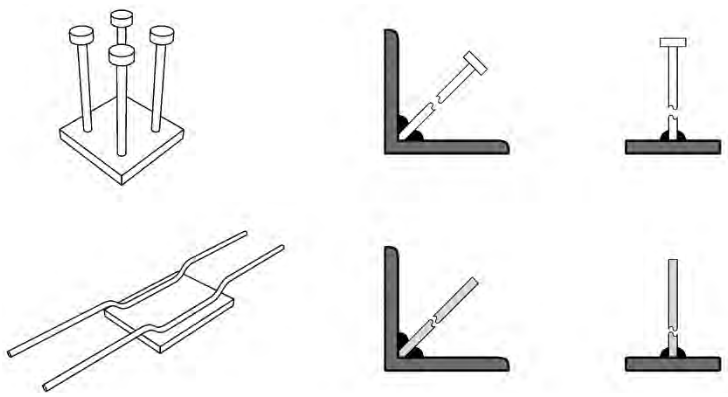


Fig. 5.37: Anchorage of weld plates by ribbed anchor bars or smooth bars with end anchors

Precast façade units are often fixed to the structure by means of suspension fixings. They are intended to transfer the weight of the panel back to the structure and to resist the positive and negative wind loads.

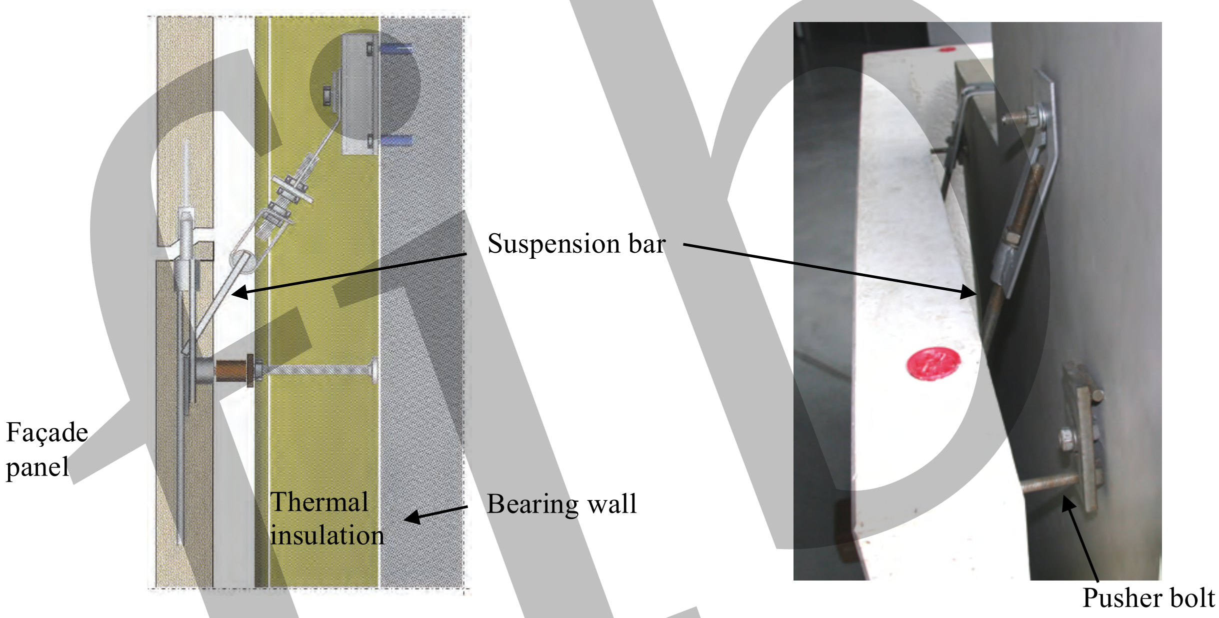


Figure 5.38: Suspended panel fixing

5.4.3 Connections transferring shear forces

Shear forces between adjacent concrete elements can be transferred through bond, the friction in joint interfaces, interlocking by shear keys, the dowel action of transverse steel bars or rods, or by mechanical shear devices. Examples of shear joints are given in Figures 5.39 to 5.41.

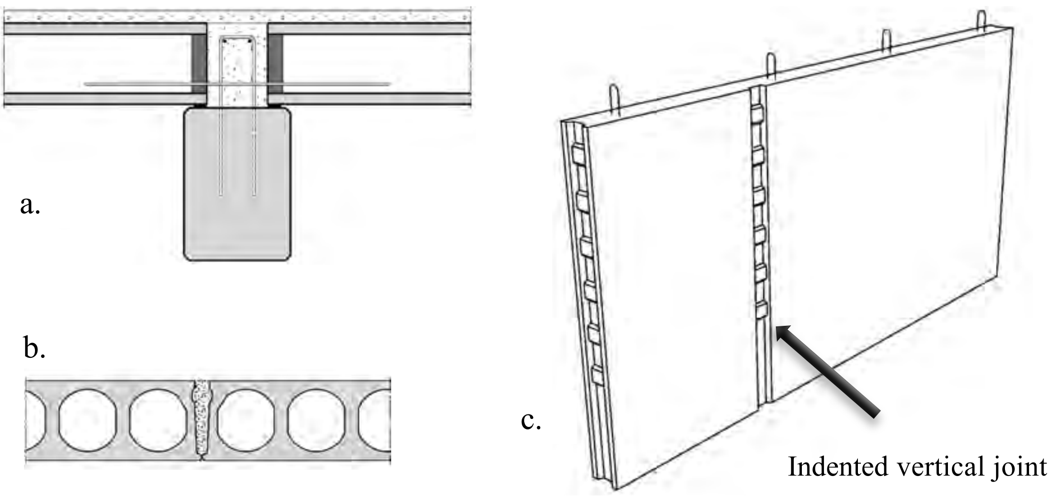


Fig. 5.39: Shear joints: a. horizontal shear transfer between floor and supporting beam through friction and stirrups; b. vertical shear transfer between floor units through the profile of the joint; c. shear transferred through indented vertical joints between wall panels

5.4.3.1 Transfer of shear forces through friction

When a joint face has a certain roughness, shear forces can be transmitted by friction even if the joint is cracked. One condition, however, is that compressive stresses act across the joint, as shown in Figure 5.14a.

In some joints, for instance, horizontal joints in precast walls, the weight of the wall above the joint undergoes permanent compressive stresses in the joint. It is also possible to generate compressive forces across the joint by means of the pull-out resistance of transverse reinforcement bars, bolts, and so forth, that are strained when the connection is loaded in shear along the joint section. Because of the roughness of the joint faces, the joint will separate a little when shear slip develops along the joint. This separation results in tensile stresses in the transverse bars and the resulting tensile force must be balanced by a compressive force of the same magnitude acting across the joint. This effect of the transverse bars means that the adjacent elements are clamped together when shear slip develops along the joint. This self-generated compressive force makes the shear transfer possible, as shown principally in Figures 5.14b and 5.40. The shear force capacity along the joint increases with increased amount of transverse reinforcement and with increased frictional coefficient. The shear capacity can also increase by treatment of the joint faces in order to improve the roughness.

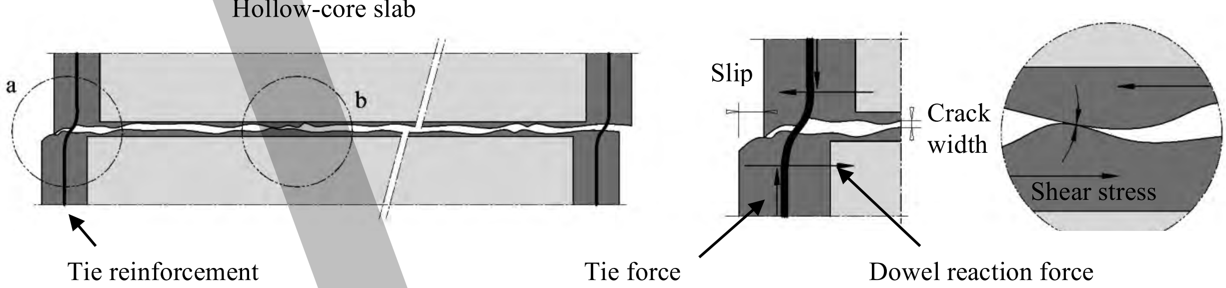


Fig. 5.40: The transfer of shear forces by friction, is only allowable when the contact faces are rough and subjected to compression, a. through exterior compression on the joint faces, b. through indirect compression generated by reinforcement through the contact faces

Shear forces along an uncracked joint can be transferred by the adhesive bond between joint grout and the adjacent concrete elements. The adhesive bond, however, depends to a large extent on the workmanship and cleanness of the joint faces during grouting. If the joint faces are dirty from sand, cement or oil wastes, the adhesive bond can be reduced or entirely lost. There is also a risk that even well executed joints crack because of restraint actions in the structure. This means that, in practice, it is not possible to rely on adhesive bond for shear transfer; the joints must be assumed to be cracked and the shear transfer must be secured by shear friction, shear-keys or mechanical devices. However, in so-called ‘low shear’ applications, where the shear stresses are very small - for example, in structural floor toppings - it may be possible to take account of the adhesive bond at the joint interfaces.

5.4.3.2 Transfer of shear forces through indented joints

Shear resistance can also be created by joint concrete or joint grout cast in-situ between toothed joint faces. A toothed joint face is shown in Figure 5.42. When this type of connection is loaded in shear along the joint, the shear resistance depends on the strength of the shear keys, on condition that transverse reinforcement or other tie arrangements are provided. The shear keys work as mechanical locks preventing any significant slip along the joint. To function in the intended way, the shear keys must fulfil certain minimum requirements concerning tooth length, tooth depth and tooth inclination. Figure 5.41 gives the minimum requirements imposed by Eurocode 2 Part 1-1 [2].

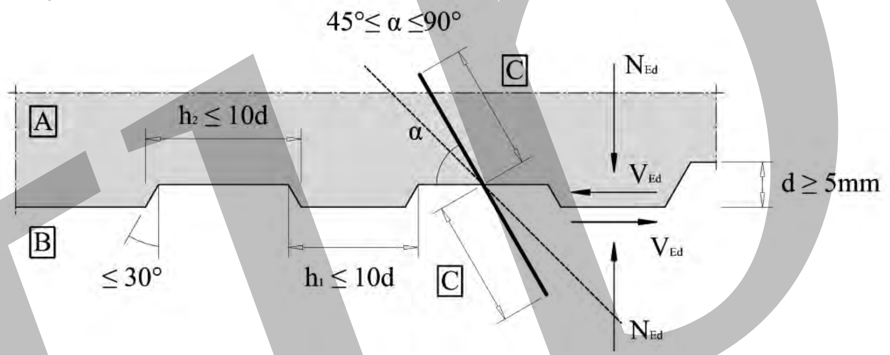


Fig.5.41: Dimensional requirements for indented construction joints

The joint should also be protected from uncontrolled joint separation by transverse reinforcement or other transverse tie arrangements. Transverse steel can be concentrated at the ends of the joint or be distributed along the joint (Fig. 5.43).

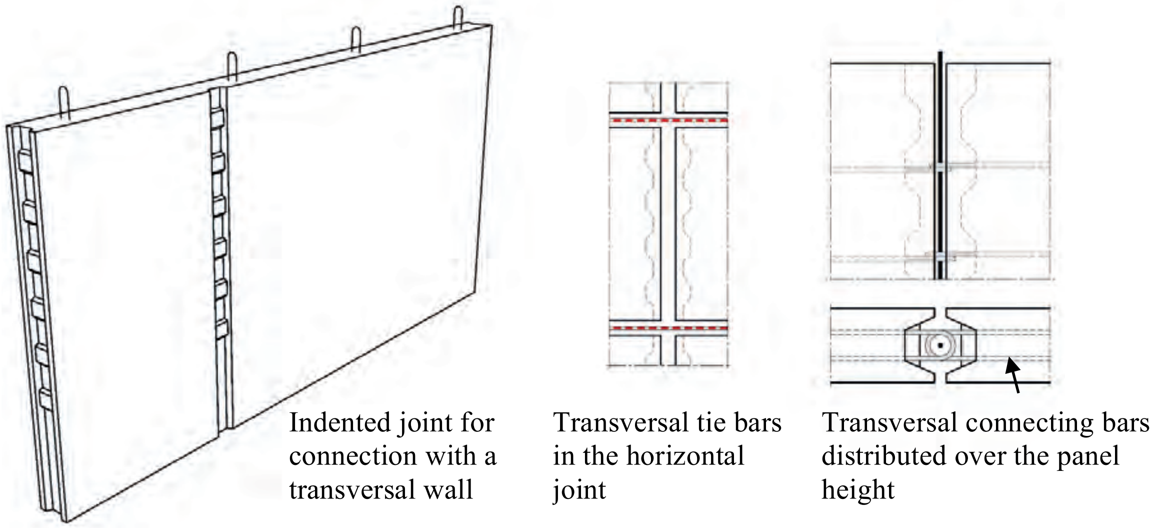


Fig. 5.42: Vertical joints between wall elements; a. indented wall joints, b. connecting reinforcement concentrated in the horizontal joints, c. connecting reinforcement distributed over the panel height

A connection with indented joint faces has very stiff behaviour until the shear-key effect is destroyed by cracking or local crushing at the heaviest loaded contact areas. Various failure modes are possible, of which the most common types are shown in Figure 5.42. When the shear-key effect decreases due to the degradation of the shear keys, the behaviour changes to a frictional phase associated with significant shear slip along the cracked section.

The normal and shear forces in connections of large panels have to be transferred between precast panels and the cast-in-situ concrete of the joint. This involves the transfer of forces across the interface of concrete of different ages. Several basic force-transfer mechanisms involving both the concrete and reinforcing steel can generally be identified.

The adhesion that exists in the interface will initially transfer shear between the two concrete surfaces until it is 'broken' and a crack forms along the interface. Once slippage commences along the crack at the interface, shear is transferred across the crack by a number of other mechanisms.

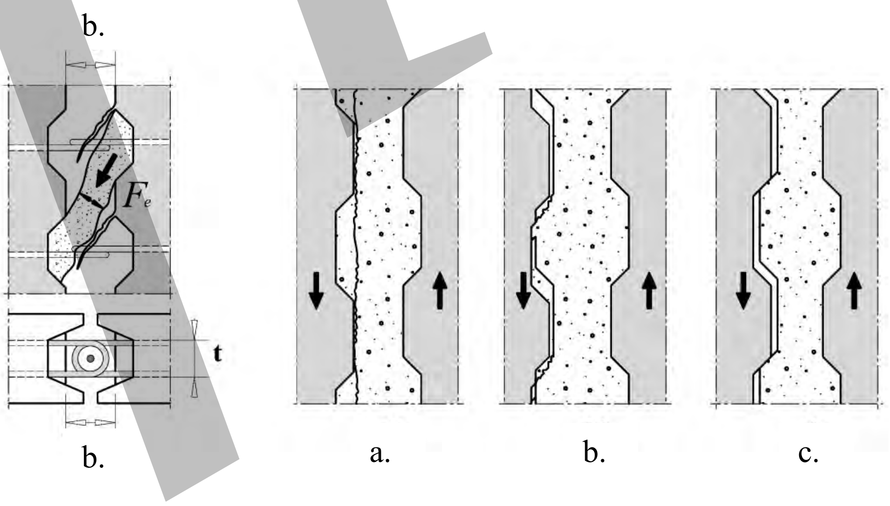


Fig. 5.43: Model for the transfer of shear forces and typical failure modes in indented joints; a. cracking of the infill concrete along the indentations, b. crushing of the compression struts, c. opening of the joint.

The simplified model of shear transfer of joints with shear keys (Fig. 5.43) may serve as an example of how the mechanisms of shear transfer (which contribute to the strength under monotonic loading) may be taken into account. Direct diagonal compression transfer between keys, friction and dowel action are mobilized due to shear displacements at the interface between the prefabricated concrete panel and the in-situ joint concrete. The shear keys work as mechanical locks preventing any significant slip along the joint. The horizontal component of the inclined compressive force must be balanced by transverse tensile forces. For this purpose, transverse reinforcement must be provided, connected be means of loops or welding, well anchored in the body of the panel. This transverse steel may be concentrated at the ends of the wall element or be distributed along its height. The latter solution is created by using projecting loops from the edges of the vertical joint. After erection, a vertical bar is placed through the loops. Another possibility is to connect the wall units by the welding of steel plates anchored in the panels along the joint.

The maximum capacity is governed by the failure of the shear-keys. Significant shear slip occurs when the shear-key effect is reduced by shear cracks along the joint or by local cracking or crushing of the shear-key corners. In the residual stage, after the destruction of the shear-keys, the shear connection can still transfer considerable shear stresses by shear friction, if the connection is adequately tied together by transverse tie bars. This means that after the initial stiff behaviour, the shear connection will behave more or less like connections without shear-keys.

5.4.3.3 Transfer of shear forces through dowel bars

Precast beams are usually designed to be simply supported. The connection between the beams and the support only needs to transfer tensile and compressive forces due to wind loading, thermal movements, and so forth. Pinned connections are very suitable as they are simple to produce and easy to connect. Horizontal forces are transferred by the dowel action of the steel bar when the vertical slot in the beam is grouted. The bar may be anchored in the slot by the grout filling alone (Fig. 5.44) or with bolts and steel plates (Fig. 5.45). Some movement can be obtained when the slot is filled with a soft material like polyurethane foam or bitumen (see also Section 6.6.2).

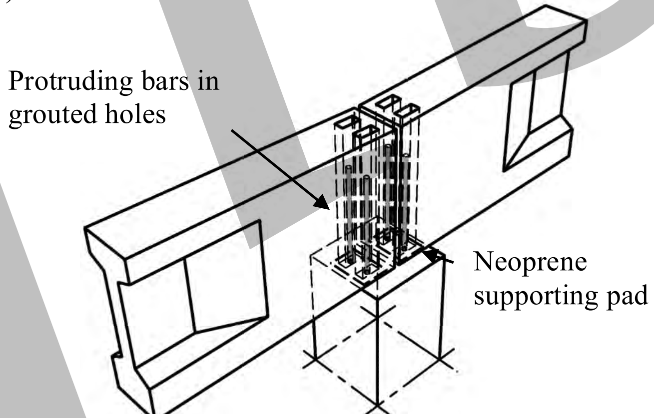


Figure 5.44: Illustration of a pinned connection

The end block of I-shaped beams may be omitted to simplify the moulds. The anchorage of the prestressing force and the shear capacity is then ensured by a thicker web. In this case, the dowel bars in the pinned connections are created by bolting.

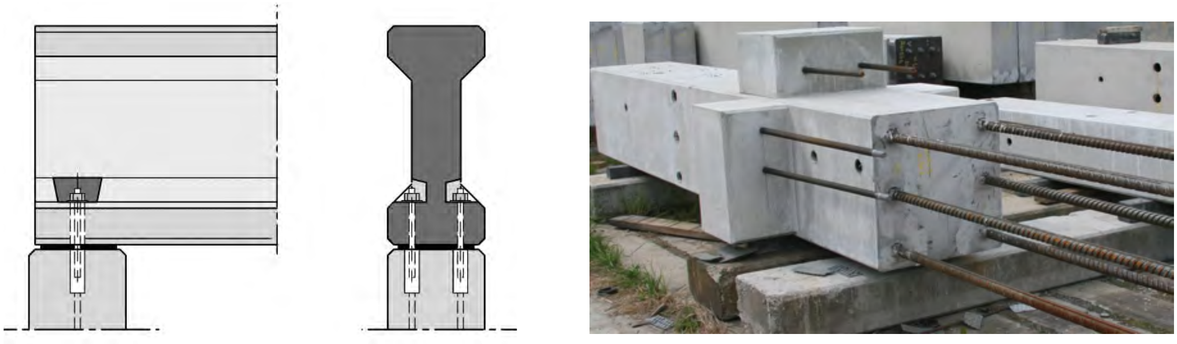


Fig. 5.45: Pinned connection of floor or roof beam and column detail

5.4.4 Connections transferring bending moments

5.4.4.1 General

Moment-resisting connections between precast components may be used to resist horizontal wind and seismic actions. Connections are placed in critical regions and the approach is to use strong connections that possess stiffness, strength and ductility approaching that of cast-in-situ construction. Moment-resisting connections are often used for columns with foundations but also in other situations, for example, for very slender and high precast columns in portal sway frames, beam-floor connections in skeletal structures and, in exceptional cases, floor continuity over the supporting beams or walls.

Moment-resisting connections are used mainly to:

- stabilize and increase the stiffness of portal and skeletal frames
- reduce the depth of flexural frame members
- distribute second order moments in beams and slabs, and thus reduce column moments
- improve the resistance to progressive collapse

The following sections give guidance on the design and construction of moment-resisting connections. The main design criteria are strength, stiffness, ductility and ease of construction.

A connection can be designed to transfer bending moments by means of a force couple of tensile and compressive forces. Reinforcement bars, bolts or other steel components with tensile capacity are arranged and anchored in such a way that an internal force couple can be developed when the connection is subjected to imposed rotation. On the opposite side of the connection, the transfer of the corresponding compressive force through the joint must be ensured.

In all cases, the effects of joint movement must be considered, including thermal expansion/contraction, creep, shrinkage, imposed elastic deformation, and so forth. The forces that are resisting bending moments should also be capable of being generated in precast components in combination with other forces, such as end shear.

Moment-resisting connections may be formed at the following locations:

- Column-foundation and column-column connections
- Beam-column connections
- Floor-beam and floor-wall connections

Beam-to-beam moment-resisting connections are not commonly used for the prevention of torsion in the receiving beam when the beams are perpendicular. This is not the case when the beams are in line with each other. Edge beam-to-floor slab moment-resisting connections are dealt with in Section 5.4.5.

5.4.4.2 Column to foundation

In portal-frame and low-rise skeletal structures, the horizontal stability of the frame is obtained by the cantilever action of the columns into the foundations.

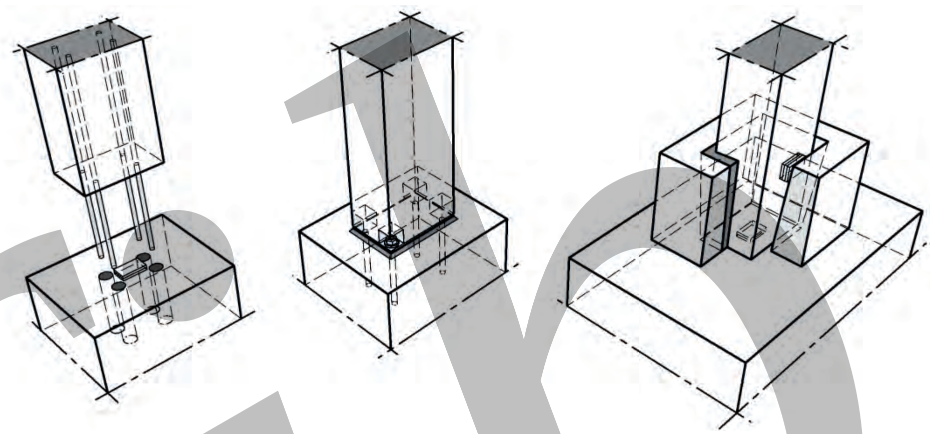


Fig. 5.46: Moment-resisting connections between precast columns and foundations

There are several ways to obtain the restrainment of the precast columns into the foundations (Fig. 5.46).

- Reinforcement bars projecting from the foundations, anchored in grouted holes in the columns
- Bolted connection with projecting bolts from the foundations fixed to steel angles (shoes) anchored in the columns
- Pocket foundations, where the columns are clamped into a concrete foundation pocket with fine cast-in-situ concrete placed in the annulus between the column and inside face of the pocket

Methods a. and b. are most commonly used. The shoe connection is advantageous because the column may be stabilized and plumbed vertically by adjusting the level of the nuts to the holding-down bolts. This is particularly important when working in soft ground conditions, where temporary propping may not provide adequate stability alone.

Projecting bars in grout tubes

There are two options: projecting bars from the foundation (Fig. 5.35 and 5.47) or projecting bars from the column (Fig. 5.48). Full compression or tension anchorage lengths are provided in both the precast column and in-situ foundation.

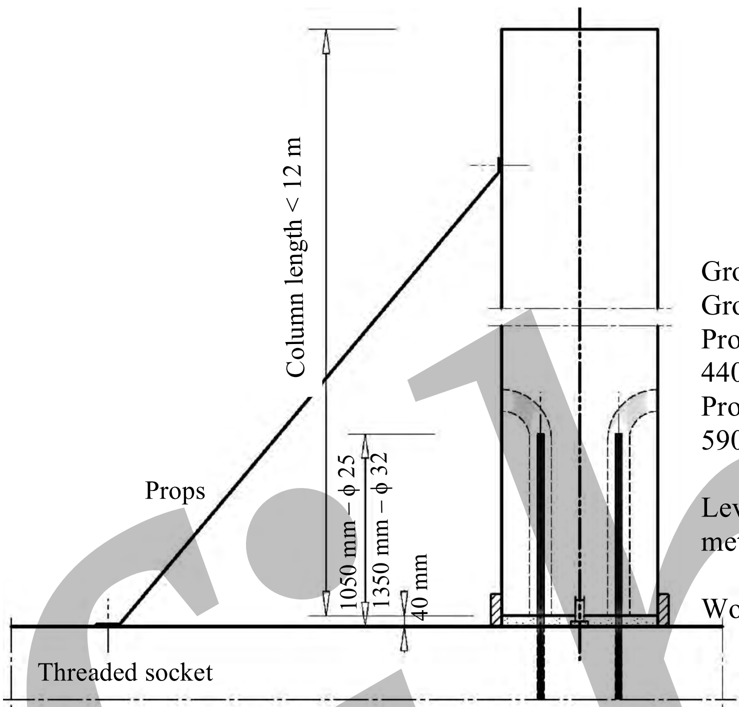


Fig. 5.47: Column base connection with grouted sleeves in the column and projecting reinforcement from the foundation

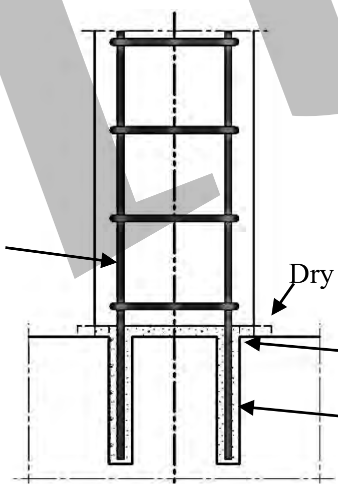


Fig. 5.48: Column base connection with grouted holes in the foundation and projecting reinforcement from the column

In design the connection may be envisaged as monolithic, provided the bedding joint and grout sleeves are completely filled. The minimum internal diameter of the grout tube is 60 millimetres when using a 25-millimetre-diameter deformed bar. Nominal cover to the tube and the minimum distance between tubes should be at least 75 millimetres.

Steel shoe splice

Prefabricated steel shoes (Fig. 5.49) are used where it may be necessary to generate bending moment and tensile forces in splices. The so-called 'column shoe' may be used both at column splices and at foundation connections. In all, four or more shoes are used, one at each corner of a rectangular column and also sometimes in the middle of the edges, depending on the necessary moment capacity. Modified versions of the standard shoe are possible for non-rectangular columns. Positioning errors of up to 10 millimetres are possible by the use of cleverly designed eccentric hole plate washers.

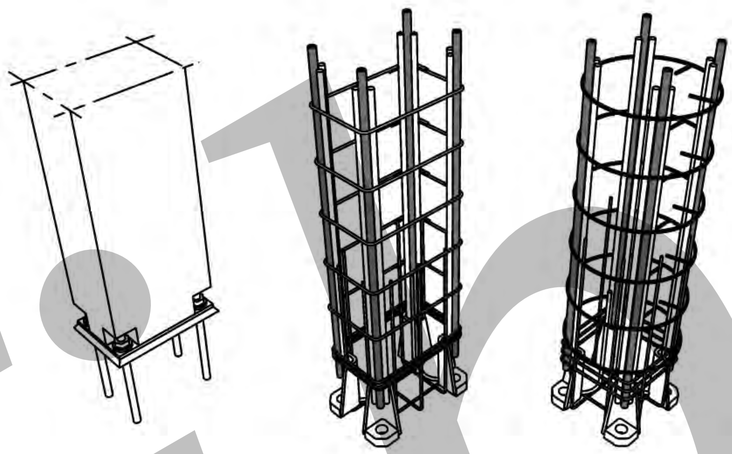


Fig. 5.49: Column-to-foundation connections with splice shoes

Each shoe consists of a thick steel plate, typically 12 to 40 millimetres thick and 100 to 150 millimetres square, joined to a thin plate metal shroud forming an approximately 80-millimetre open box cube, and 3 rebars in a triangular formation. The bars, which are typically 16 to 40 millimetres in diameter, provide the bond force to the concrete column. The base plate has a punched hole at its centre, which is there to receive the threaded bars from the adjoining column (similar to the welded plate splice detail). The column shoe may be recessed in the column if the splice connection is exposed; otherwise, the edge of the base plate is made flush with the column.

Columns in pockets

This solution is restricted to situations where fairly large concrete pad footings can easily be established at small foundation depths. Either precast or in-situ concrete pad footings can be used. The in-situ concrete foundation is cast using a tapered box shutter to form the pocket. The gap between the pocket and the column should be at least 75 mm at the top of the pocket. There are two alternatives, either pockets with smooth surfaces or with keyed surfaces.

- Pockets with smooth surfaces (Fig. 5.50)
  - The forces and the moment may be assumed to be transferred from column to foundation by compressive forces F_1, F_2 and F_3 through the concrete filling

and corresponding friction forces as shown in figure 5.50. This model requires ℓ ≥ 1.2 a, where *a* is the largest dimension of the column cross-section.

- The coefficient of friction should not be taken greater than μ = 0,3.
- Special attention should be paid to:
  - detailing of reinforcement for F_1 in top of pocket walls;
  - transfer of F_1 along the lateral walls to the footing
  - anchorage of main reinforcement in the column and pocket walls
  - shear resistance of column within the pocket
  - punching resistance of the footing slab under the column force, the calculation for which may take into account the in-situ structural concrete placed under the precast element.

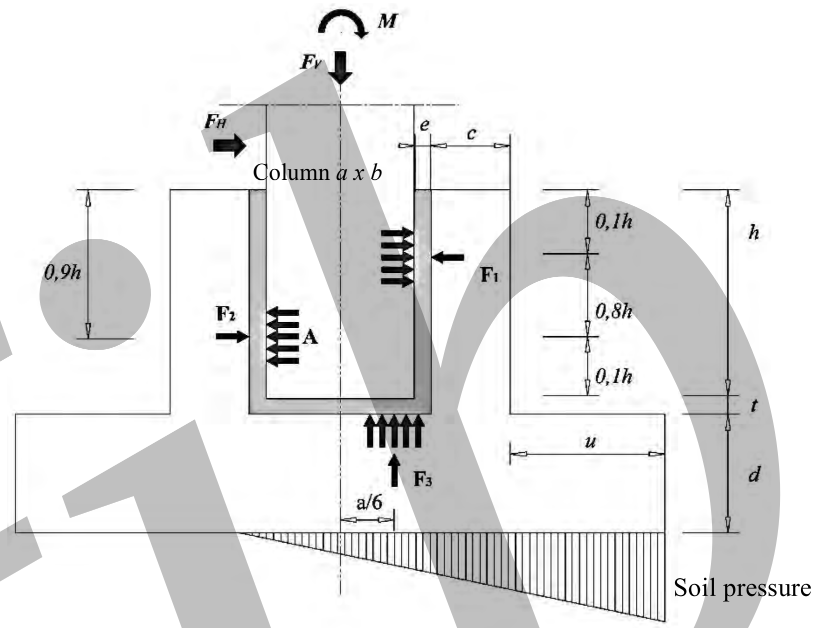


Fig. 5.50: Pocket foundation with smooth surfaces

- Pockets with keyed surfaces (Fig. 5.51)
  - Pockets expressly wrought with indentations or keys may be considered to act monolithically with the column.
  - Where vertical tension due to moment transfer occurs, careful detailing of the overlap reinforcement of the column and the foundation is needed, allowing for the separation of the lapped bars. The lap length should be increased by at least the horizontal distance between the bars in the column and in the foundation (Fig. 5.51). Adequate horizontal reinforcement for the lapped splice should be provided.
  - The punching shear design should be as for monolithic column/foundation connections, provided the shear transfer between the column and footing is verified. Otherwise the punching shear design should be as for pockets with smooth surfaces.

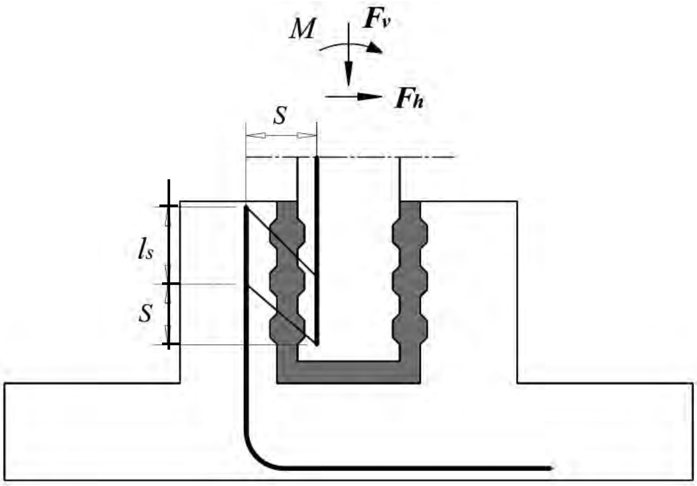


Fig. 5.51: Pocket with keyed-joint surface

If overturning moments are present, half of the skin friction is conservatively ignored due to possible cracking in all of the faces of the precast/in-situ boundary. Ultimate load design takes into consideration the vertical load transfer by end bearing based on the strength of the gross cross-sectional area of the reinforced column and equal area of non-shrinkable sand/cement grout. The design strength of the infill grout is usually $f_{cd,}$ = 40 N/mm^2 and the specification is the same as for the grout used in splices.

5.4.4.3 Beam to column

There is a very wide range of beam-column connections varying in complexity, cost and structural behaviour. See Figure 5.52 for a major subdivision, in that either:

- The vertical member is *continuous* and horizontal components are framed into it. This is termed a 'beam-end' connection.
- The vertical member is *discontinuous* (only in construction terms) and the horizontal components are either structurally continuous or separate across the junction. This is termed a 'column-head' connection.

These two cases will be dealt with separately because of the differences in structural behaviour.

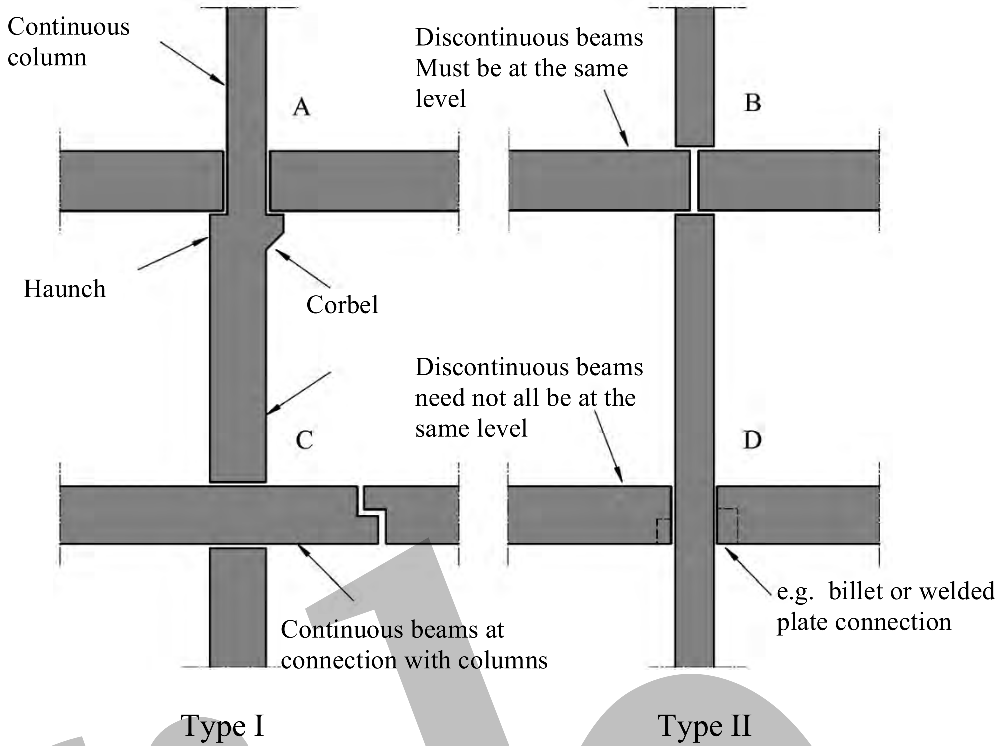
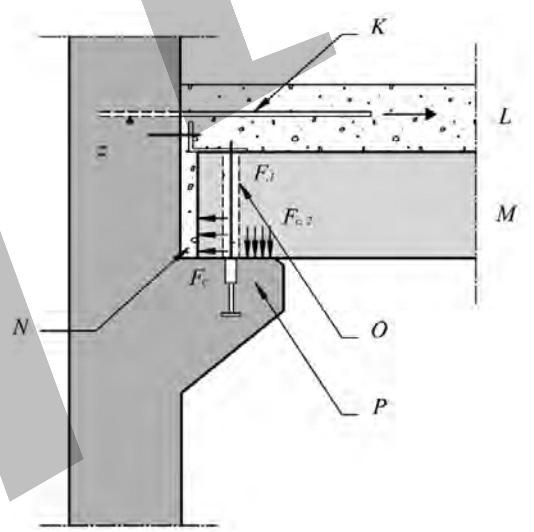


Fig. 5.52: Types of beam-column connections. A: beam-end-to-column corbel; B: beam-end-to-column head; C: continuous beam-to-column head; D: beam-end hidden connection to continuous column;

- Type A: Beam end on column corbel or haunch

Partial moment fixity between columns and beams could be achieved through a force couple with the column at the beam end. The gap between the column and the beam, which should be at least 50 millimetres to ensure good compaction, is site grouted, enabling full compressive strength to develop. The tensile forces at the top of the beam are transferred to the column by tie bars anchored to the column by means of sockets or with alternative solutions.



K: tie bar
L: cast-in-situ tie beam
M: precast beam
N: in-situ infill grout
O: threaded dowel or projecting bar grouted in hole
P: corbel

Fig. 5.53: Structural mechanism for the beam-end connection with the column. The solution presented is complex and therefore not recommended.

The following figures show alternative solutions in the form of bolts cast into sockets (Fig. 5.54a) and welded plates (Fig. 5.54b).

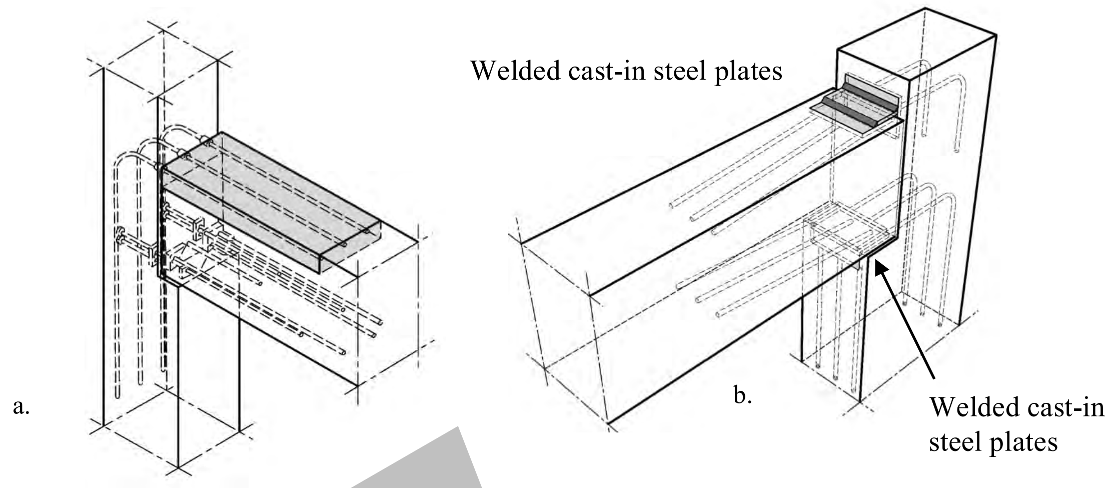


Fig. 5.54: Beam-column fixity through cast-in sockets or welded plates

- Type B: Beam-to-column head

This type of connection is mainly used to avoid column corbels, mostly for aesthetic reasons. In Figure 5.55a, the force couple is provided through tensile reinforcement placed and anchored in the in-situ infill concrete between the floor slabs on top of the floor beam, and the compressive force thought the joint fill at the bottom of the beam. Moment continuity exists only for imposed loads after the in-situ infill has maturated to full strength. The solution presented in figure 5.55b gives only a restricted moment continuity, which would normally not be taken into account in the design of the beams.

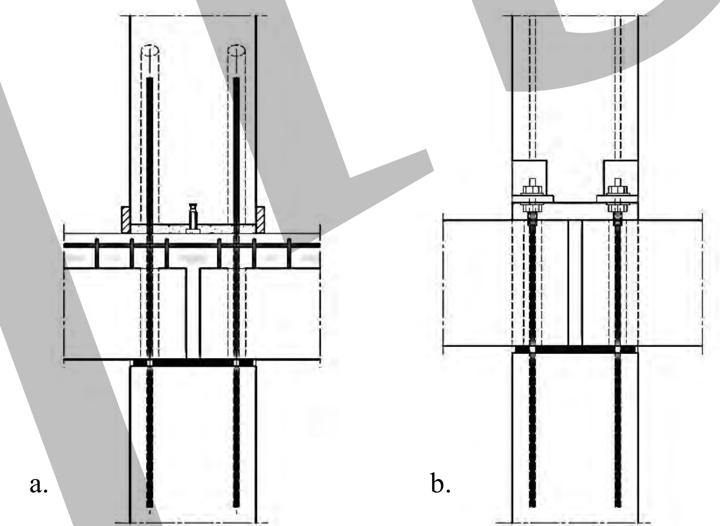


Fig. 5.55: Examples of partial beam continuity at column intersections

- Type C: Continuous beam to column head

This connection may be used in skeletal frames where continuous (or cantilevered) beams are required, as shown in Figure 5.56. The column head connection may be designed with, or without contributions from floor slabs and tie steels.

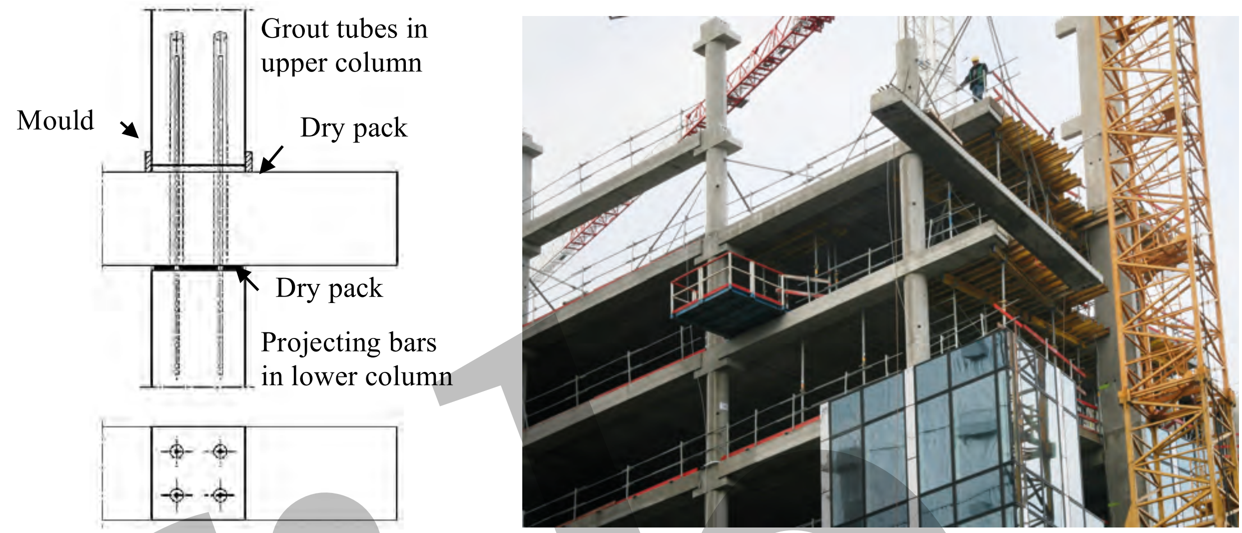
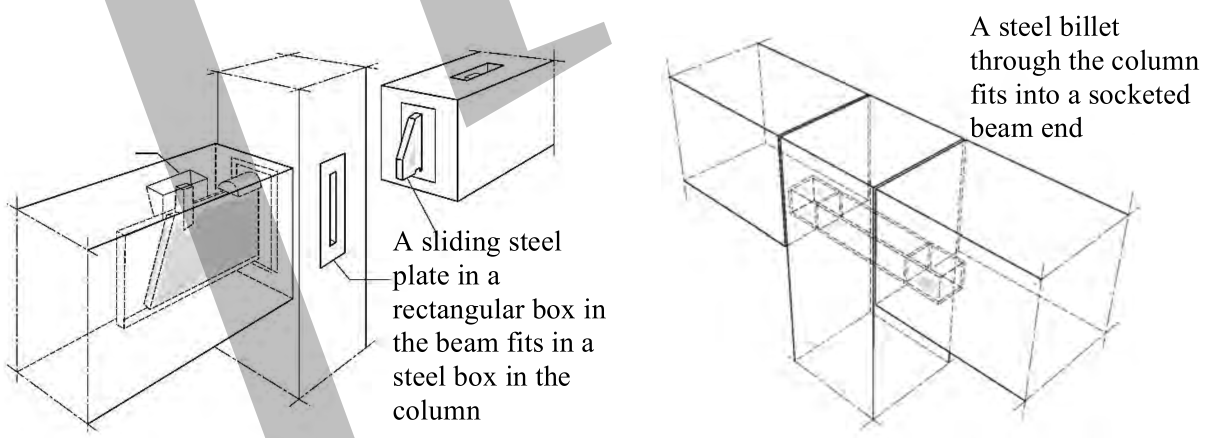


Fig. 5.56: Continuous cantilever beams at column head

- Type D: Hidden beam end to column

There is a recent trend to use hidden steel corbels in beam-to-column connections. The advantage of the solution is that the intersection between beam and column is neat, without the need for an underlying corbel. The connection is also attractive from an aesthetic point of view. Various solutions exist on the market. Detailed information on the design is given in Section 10.3.4 of this bulletin, *fib* Bulletin 43 [12] and *Multi-storey precast concrete framed structures* by K.S. Elliott [41].



BSF hidden steel corbel

Steel insert in column and recessed beam end

Fig. 5.57: Examples of hidden corbels

5.4.4.4 Floor to beam

Precast hollow-core floors are normally designed and constructed as simply supported and one-way spanning. Top reinforcement is sometimes provided to guard against flexural cracking during to handling, and to cater for shrinkage and thermal effects, and so forth, but is otherwise not provided for negative moment continuity at supports, unless the units are cantilevering. The latter could be obtained by placing site top reinforcement across the support and filling the gaps and joints at the ends of the units with structural cast-in-situ concrete (Fig. 5.58) or by top reinforcement in a structural topping (Fig. 5.59).

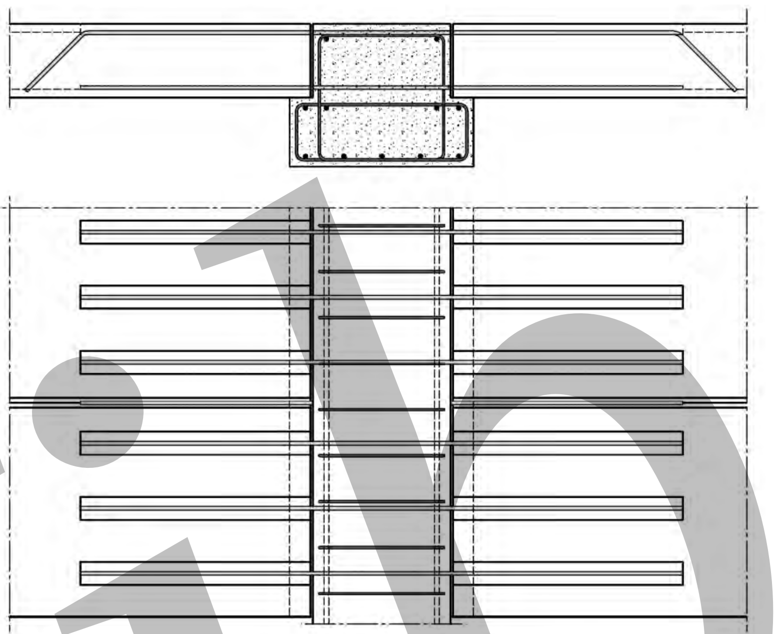


Fig. 5.58: Partially restrained connection

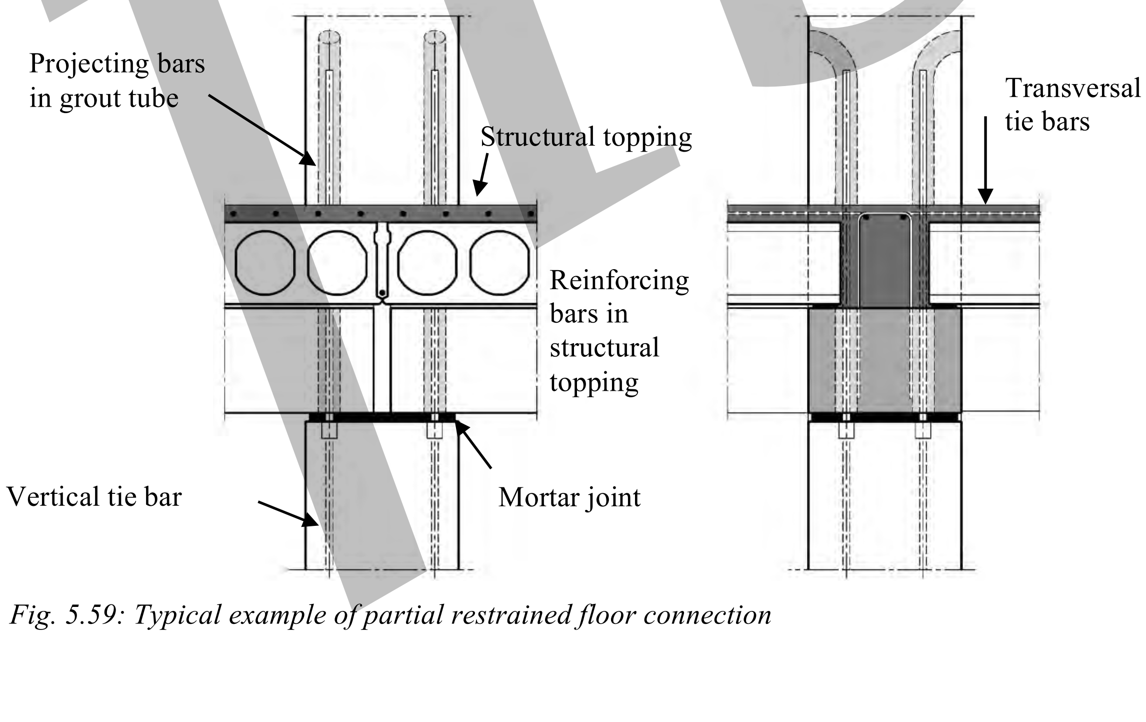


Fig. 5.59: Typical example of partial restrained floor connection

During the design it must be understood that the floor connection is only complete once it is installed and any in-situ concrete that is installed and matured. As such, this type of connection can only provide continuity for loads imposed after erection. Cases with partial continuity will require certain consideration in design since standard methods are not applicable.

5.4.4.5 Floor to wall

Floor-to-wall connections can be achieved using direct support on the wall without corbels (Fig. 5.60).

Fig. 5.60: Typical floor-wall connection without wall corbels

A certain moment capacity can be obtained as a secondary effect. Typical examples are connections in floor slabs. The floor elements are designed to be simply supported but, due to requirements of minimum tensile capacity within floors and between floors and their supports, a certain number of tie bars are required. Together with the jointing with grouts and cast-in-situ concrete, the tie connections will attain a certain bending moment capacity that was not required in the design. The moment capacity depends on the position of the tie bars in relation to the applied moment. This secondary moment capacity can be favourable in the case of slender floors for limiting deformation as well as in accidental situations, since it provides ductility and facilitates force redistribution.

5.4.5 Connections transferring torsion

Torsional moments appear often in precast floor beams that are loaded only on one side, for example, at the edge of a floor. The resulting torque at the beam end should be resisted by the connections. This action also needs to be considered during the construction phase.

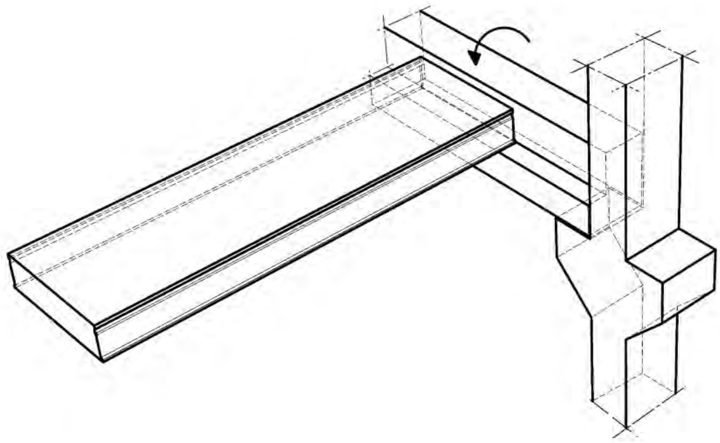


Fig. 5.61: Torsional loading in an edge floor

There are two principle approaches to appraising eccentric loading on beams. In both cases, the aim is to avoid complex behaviour by applying simple support conditions, either at the beam-floor connection or at the beam supports.

- The floor is simply supported on the beam (Fig. 5.62a). The torsion that results from the eccentric loading must be resisted by the beam and the resulting torsional moment must be carried at the beam support. In this case, no special reinforcement is needed in the floor-beam connection for the eccentric loading to be taken up. However, the beam and its connection to the column must be designed to accommodate torsion.

- The floor is firmly connected to the beam and the beam is considered an integrated part of the floor, which means that the floor span increases (Fig. 5.62b). The beam-floor connection is designed for the eccentric loading. In this case the support of the beam should not be able to resist torsion but be free to rotate around its centroid axis.

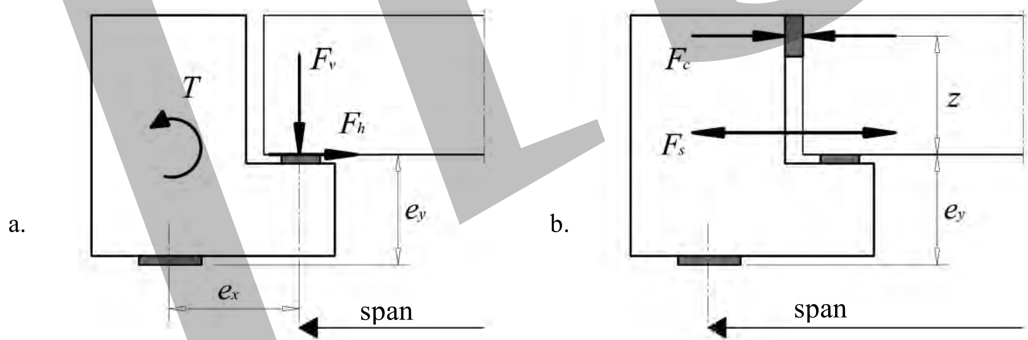


Fig. 5.62: Principle ways to consider eccentric loading on beams, a. the floor is simply supported on the beam, b. the floor is firmly connected to the beam, which is free to rotate at its supports.

A typical example of a beam-floor connection designed according to design approach A is shown in Figure 5.63. The connection is able to transfer a tensile force from the floor to the beam to fulfil demands on structural integrity. When the floor is loaded the floor elements rotate, but this rotation is not transferred to the beam. However, since the beam is connected for tension transfer, in-plane deflection of the beam is prevented and it cannot deform fully freely.

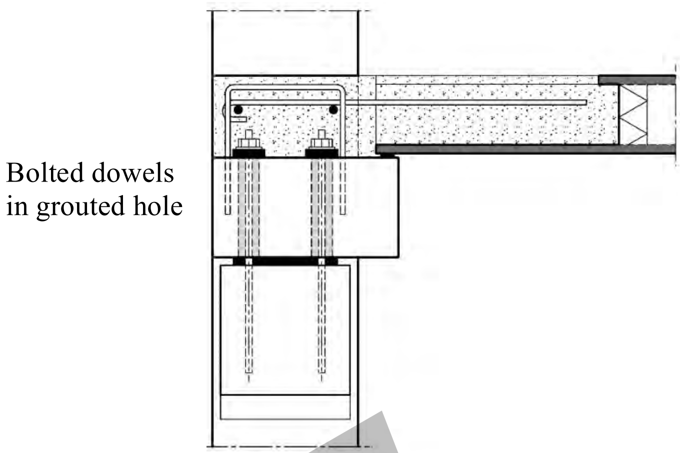


Fig. 5.63: Connection between hollow core floor element and edge beam where the torsion is taken up by the support connection.

Typical examples of connections based on design approach b are given in Figure 5.64. The intention is that when the connection is completed, the floor and the beam interact compositely.

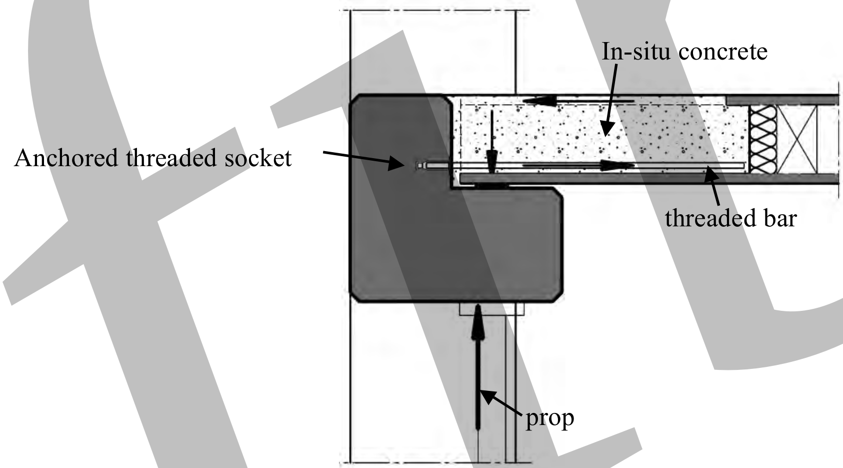


Fig. 5.64: Connection between floor slab and ledge beam providing torsional restraint in a hollow-core floor.

In this case the floor-beam connection is designed and detailed to establish a force couple that counteracts the action from the eccentric vertical load from the floor. The connection is provided in the bottom part with devices that are able to transfer the tensile force in the force couple. These force-transferring devices could be weld plates, anchor bars or loops from reinforcing bars that are anchored by grouting in recesses and cores. The compressive force transfer can be achieved with steel plates, inserts or wedges placed in the joint between the floor element and the web of the beam or a joint filled with joint concrete or grout. The tensile force capacity provided between the floor and the support beam should also take into account diaphragm action in the floor and possible restraint forces due to shrinkage, temperature effects, and so forth. The common design approach is to calculate the horizontal force couple so that it counteracts the moment from the vertical load relative to the shear centre of the beam.

5.5 Other design criteria

One of the most important principles in the design of connections is to keep them simple. Maximum economy of precast concrete construction is achieved when connection details are kept as simple as possible and consistent with adequate performance and ease of erection. Furthermore, complex connections are more difficult to design, to make and to control, and will often result in poor fitting in the field. This can contribute to slow erection and less satisfactory performance.

5.5.1 Manufacture

Below is a list of items to take into account during the design to improve fabrication simplicity. In many cases, some of these items must be compromised to enable the connection to serve its intended function.

- Avoiding congestion

The area of the member in which the connection is made frequently requires large amounts of additional reinforcing steel, embedded plates, inserts, block-outs, and so forth. It is very important to design the connections in such a way that sufficient room is left for concrete to be placed correctly between the different details. If congestion is suspected, it is helpful to draw large-scale details of the area in question.

- Avoiding penetration of forms

Projections that require cutting through the forms are difficult and costly to place. Where possible, these projections should be limited to the top of the member as cast, or detailed as threaded inserts to allow a threaded bar or bolt to achieve the required projection.

- Minimizing embedded items

Items that are embedded in the member, such as inserts and plates, require plant labour for precise location and secure attachment. Therefore, these items should be kept to a minimum. This especially applies to items embedded in the top surface. However, if the same steel plate is placed at the bottom or on the side of the form, it can be placed with great accuracy.

- Using standard items

Wherever possible hardware items, such as inserts and steel shapes, should be standard items that are readily available, preferably from more than one supplier. Custom fabricated or very specialized proprietary items add cost and may cause delays. It also simplifies fabrication if similar items on a product or project are standardized as to size and shape. There is also less risk of error.

- Using repetitious details

It is very desirable to repeat details as much as possible. Similar details should be identical, even if this leads to slight overdesign. Once workmen are familiar with a detail it is easier for them to repeat it than to learn a new one. Repetition also requires fewer form set-ups and improves scheduling.

- Allowing alternates

Very often, a precast concrete manufacturer will prefer certain details to others. The producer should be allowed to use alternative methods or materials, provided the design requirements are met. Allowing alternative solutions will often result in a more economical and better performing connection.

- Using ' fool-proof ' details

A general rule is that the connection device should be made as 'fool proof' as possible. It should be easy to place in the mould, to orient correctly and to adjust within the necessary tolerances with minimum effort and with no room for mistakes.

5.5.2 Storage and transport

Due consideration has to be given to the fact that the shape and dimensions of the chosen connection details and protruding bars can cause problems during the transport and storage of the elements. Reinforcement bars or other protruding items can be troublesome during handling and storage. They may also prevent efficient loading on trucks. Protecting bars can sometimes be replaced by threaded inserts and loose threaded rods or bars that can be screwed into the inserts at the building site.

5.5.3 Erection

Much of the advantage of precast concrete construction is the ability to erect the structure quickly. To achieve this and to keep costs within reasonable limits, field connections should be kept simple. However, in order to fulfil the design requirements it is sometimes necessary to compromise fabrication and erection simplicity.

Connections should be designed so that the unit can be lifted, installed and unhooked in the shortest possible time. When fastening is needed before unhooking to achieve temporary or final stability, the operation should be as fast and simple as possible and not sensitive to weather conditions. Connections that require the element to be moved horizontally in the final position or the units to be hoisted at a skew angle should be avoided.

For fast erection, connections that are adjustable to allow for dimensional deviations are required. Not only the tolerances of the precast elements should be considered but also the possibility of the incorrect placing of the elements. This can occur because of deviations in the cast-in-situ foundations or supporting structure. It is advisable to allow for large tolerances for such parts of the construction. For example, clearance gaps between a cast-in-situ structure and cladding panels are normally not visible in the finished building and, thus, should be made as large as practical considerations demand.

Connections should be accessible during installing for the placement and fastening of bolts and nuts or for carrying out welding, for example, and to inspect and check the quality afterwards. In the design for force transfer capacity it is also important to consider so-called transient situations. During handling, storing, transport and erection, connection details can be exposed to special loading cases. Temporary supports, eccentric loading during erection, wind loads before the building is completed, lifting and temporary stabilizing are examples of transient situations.

6 Portal-frame and skeletal structures

6.1 Introduction

This chapter focuses on portal-frame and skeletal structures, both of which combine long span beams (or rafters) and columns with, in the case of skeletal frames, shallow prestressed concrete floor slabs and, occasionally, precast staircases and lift shafts. Precast concrete walls and cores may be used to stabilize skeletal structures rather than just as load bearing elements, and portal frames may include wall panels in the façade, but both structures are essentially open frameworks, giving the architect total freedom in terms of wide spaces, tall ceilings, shallow floor depths, and high bearing capacity, together with accurate, high-quality and fire-resistant finishes. Although both types are beam-and-column structures, their purpose is quite different, with single-storey portal frames being used mainly for warehouses, storage, industrial manufacturing, farm buildings, sports halls, among others, while multi-storey skeletal structures are used for offices, commercial and retail structures, parking garages, stadia, and so forth. There is a broad division between plain (grey) concrete skeletal structures, for example, in car parks, and exposed architectural concrete for offices and commercial buildings.

Portal frames are designed and constructed as shown in Figure 6.1 The columns in the raftered portal frame in Figure 6.1 have rigid and moment-resisting foundations, such that the frame sways in a vertical cantilever action from the foundation to the eaves, where pinned jointed rafters are simply supported over a span of typically 15 to 40 metres, and even 50 metres. The rafter, often in high-strength prestressed concrete, may contain circular voids to reduce weight and allow services to pass, and has a roof pitch of 4 to 10 degrees over a span of up to 50 metres. Precast concrete or steel purlins are simply supported between the rafters.

Fig. 6.1: Portal frame consisting of rigidly founded columns and simply supported rafters

Skeletal structures are the most architecturally and structurally demanding because in both disciplines designers feel that they have free rein to exploit the structural system by creating large uninterrupted spans while reducing structural depths and the extent of the bracing elements. The skeletal structure is distinguished from other types because imposed gravity loads are carried to the foundations by beams and columns, and horizontal loads by columns, walls or cores.

A feature of the skeletal structures is that the volume of structural concrete is in the order of 3 to 4 per cent of the volume of the building, which is much lighter than traditional cast-in-situ frames thanks largely to specifying prestressed floors (with an approximate 30 to 50 per cent void ratio), long-span prestressed and composite beams and slender columns. For buildings up to 3 to 4 levels high, the horizontal stability is usually provided by columns restrained into the foundations and acting as vertical cantilevers.

For higher buildings, the skeletal frame is designed as a braced structure using slender columns with long-span prestressed concrete floors acting as a horizontal plate between them. The horizontal stability is ensured by one or more cores or a number of strategically positioned shear walls, all of which are shown in the classical skeletal structures in Figure 6.2.

Fig. 6.2: Precast concrete skeletal frame, designed as a braced structure using precast shear walls with long-span prestressed concrete floors acting as a horizontal plate between them

While precast concrete shear walls or cores provide a structurally efficient solution for horizontal stability, these are not always used in countries where the effects of seismic forces are concentrated at these points. Designers may therefore use the moment capacity of the columns to resist horizontal forces and drift, namely, the notional tilt of the building, which produces about a 1.5 per cent horizontal component of the self-weight of the structure.

Precast skeletal structures consist of rectangular or circular columns and rectangular or inverted-tee shape beams, assembled and connected to form a robust structure by the use of small but ductile strips of cast-in-situ infill between the precast elements (tie beams), or a structural screed able to support and transfer vertical and horizontal actions from the floors and façades to the foundations. Skeletal structures are most commonly used for low to medium-rise buildings, about 90 per cent of which are between 3 and 10 storeys high, where the concrete compressive strength is about 50 N/mm^2. However, recent developments in high-strength concrete up to 100 N/mm^2 have enabled precast columns to compete, and beat, equivalent structural steel Universal columns, when the combined advantages of strength, section size, self-finish and fire resistance are all taken into account. An example of this is shown in Figures 1.19 and 6.18, where tower buildings of 36 storeys have utilized high-strength concrete in columns as small as 600 millimetres in diameter, shallow prestressed concrete beams, and lightweight hollow-core floor slabs. A key element to the success of these buildings was the speed of construction: 2 storeys per 8 working days.

The main difference between cast-in-situ and precast frames and skeletal structures lies on one hand in the general design philosophy and connections between elements, and on the other in the possibilities for larger spans and smaller cross-sections of columns and beams. The general considerations in design, as opposed to the detailed design of elements, include the selection of the frame, the optimum use of elements, provisions for services, special features such as cantilevers and other items requiring specification, such as appearance and finish, and other performance requirements.

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, mezzanine floors, and major subdivisions of the building. The choice of external elements is governed by the façade; 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.

6.2 Types of precast framed structures

Precast framed structures are comprised of 'linear' elements - columns, beams and slabs - as opposed to 2-D wall and load-bearing façade elements. There are a number of different solutions for linear precast concrete structures. They generally vary according to the height and the use of the building, as follows.

6.2.1 Portal frames

The basic elements of a single portal frame consist of two columns and a roof beam. Figure 6.3 shows these basic elements. The columns are rigidly connected into the foundations and function as moment-resisting cantilevers, the columns having a square or rectangular cross section typically 400 to 600 millimetres deep by 250 to 400 millimetres wide. Slenderness about the minor axis is a major problem for economy, according to the Eurocode 2 - Part 1.1 [2], because of the reduced likelihood that an unbraced column becomes slender. The beam, rafter or truss is usually simply supported on the columns with pinned connections. The total skeleton of the building is composed of a series of basic portal frames with a distance of approximately 5 to 12 metres between each.

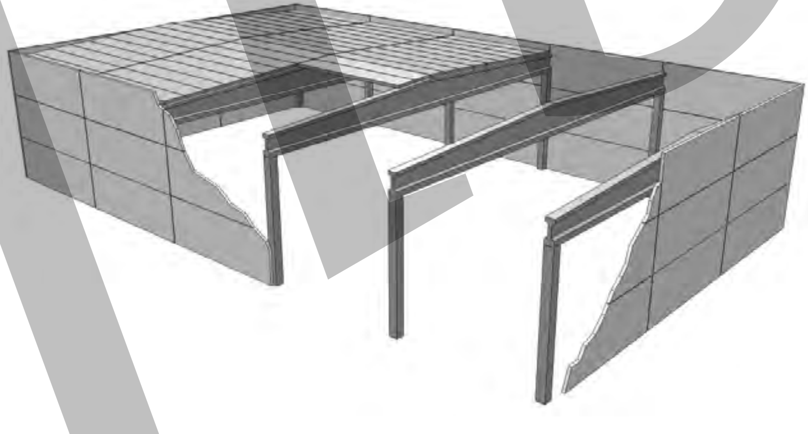


Fig 6.3: Portal-frame schematic arrangement

There exist several variant solutions in regard to types of beams, use of primary and secondary roof beams, roof elements, shed roofs, and so forth. The modern tendency is to use very large spans of up to 50 metres for the main roof beams (prismatic section), rafters (I section with sloping top flange) or girders (trusses). The distance between the portal frames is governed by the span of the roof and façade construction - normally 5 to 6 metres for steel deck, 6 to 9 metres for hollow-core roof slabs and 9 to 12 metres for lightweight ribbed roof units.

Fig. 6.4: Roof structure with rafters for the long span and T-shaped rectangular beams for the smaller spans, Salamanca, Spain

Figure 6.5 shows the transport and Figure 6.6 the erection of the 52-metre-long roof rafters. Two trucks were needed to limit tensile stresses in the beams during transport through the hilly region. The front truck is pulling the beam whilst the back truck is pushing. Two cranes were used for construction.

Fig. 6.5: Transport of 52m-long roof rafters in hilly region in Spain

Fig. 6.6: Erection of 52m-long roof rafters with two mobile cranes for a project in Salamanca, Spain

The roof can also be executed with straight I-profile beams (Fig. 6.7). In both cases, the same solutions for the roof elements are used. In portal frames with straight beams, the roof slope for evacuation of rainwater is obtained by alternating the height of the supporting beam rows. At the façade, the roof slabs are supported on beams or on load bearing walls.

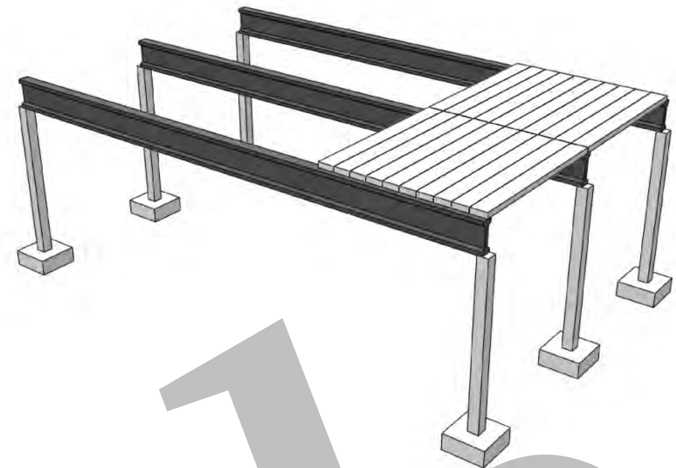


Fig. 6.7: Portal frame with straight roof beams

When the distance between portal frames is larger than the span of the roof deck elements with, for example, cellular concrete slabs of 100 millimetres in thickness, corrugated cement plates or steel deck, secondary roof beams, also called purlins, are used (Fig. 6.8). The length of these beams normally lies between 8 and 12 metres.

Fig. 6.8: Portal structure with secondary roof beams supporting roof panels

The recommended span lengths and heights for the types described above, and defined in Figure 6.9, are listed in the following table.

Table 6.1: Guidance for span lengths and beam heights of classical precast roof beams

	Minimum (m)	Optimum (m)	Maximum (m)
Main roof beam (B)	12	15 – 30	50
Purlins (C_1)	4	6 – 9	12
Primary roof beam span (C_2)	12	12 – 18	24
Column height (H_1)	4	12	20

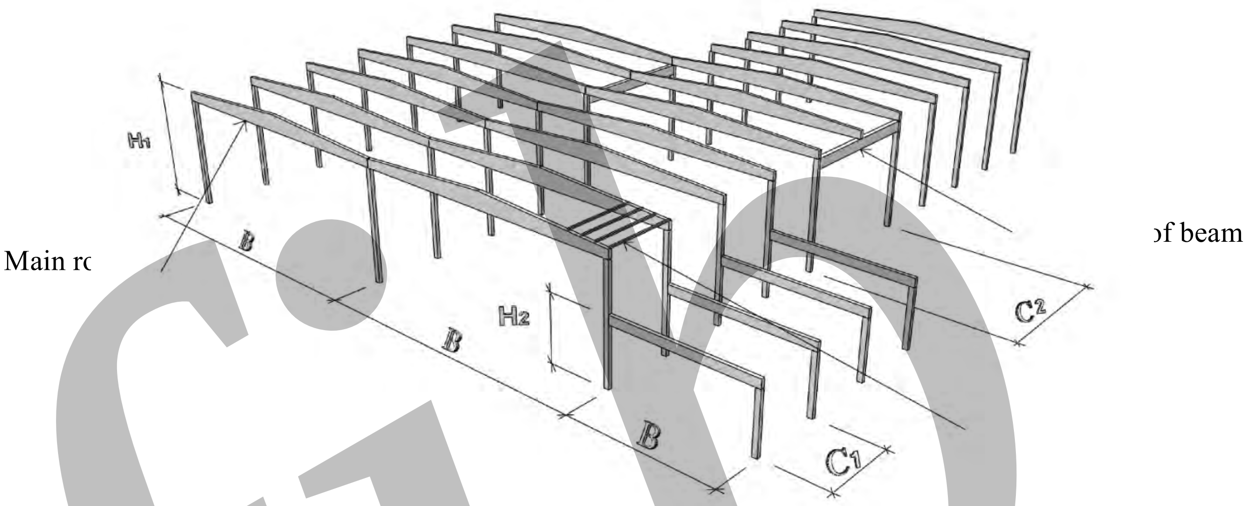


Fig. 6.9: Guidelines on span lengths

6.2.2 Portal frames with intermediate floors

In buildings constructed as single-storey structures, it is possible to insert intermediate or mezzanine floors (Fig. 6.10) in some parts of the building or over the whole surface. This is commonly achieved by adding a separate beam/column assembly. The loads on the floors are generally much larger than on the roof; consequently, the spans will normally be shorter. Span A in Figure 6.10 will normally be between 6 and 18 metres long, depending on the live loads and the type of floor slab selected. A good module for span B, with prestressed hollow-core roof slabs or ribbed elements, is 7.20 to 9.60 metres. Low-rise portal-frame structures are normally stabilized through the cantilever action of the columns. The precast columns are fixed into the foundations with moment-resisting connections.

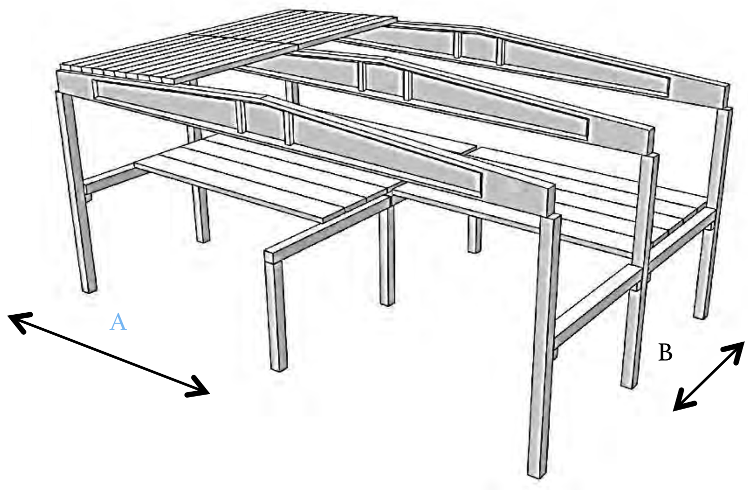


Fig. 6.10: Portal frame with intermediate floor

6.2.3 Skeletal structures

A skeletal structure is, as the name suggests, a framework in which the beams, columns and floor slabs make up the skeleton of the structure such that the load path from gravity dead and live loads passes through the slabs and beams, as simply supported elements, to the columns. From this simple framework of linear elements, often based on a modular grid of 600 or 1200 millimetres, skeletal structures offer the client, architect and building services engineers a huge choice in the building layout, architectural style and features, and provision for in-built services, for buildings from 2 to 40 storeys high.

Fig. 6.11: Schematic illustration of a skeletal structure

The internal and external layout, grid-lines, floor levels and structural depth zones, among other elements, may vary in most buildings. The interior is governed by the spatial requirements for function, and by the location, size and orientation of lift shafts, stairs, mezzanine floors, major partitions, and so forth, while for the exterior the precast structure can be designed for a very wide range of architectural features. The precast designer will therefore provide an external structure that is, in the main, totally different to the internal arrangement.

The first decisions are, therefore, the:

- selection of the structure
- functional requirements of the building
- framing plan by making optimum use of precast elements
- bracing methods, whether by walls/cores, or column action

The possibilities for the external structure are:

- non-structural precast concrete cladding (Fig. 6.12), in-situ masonry, or glass-curtain walling supported on a plain concrete beam-column structure (Fig. 6.13).
- a structural envelope, comprising either precast concrete spandrel beams (Fig. 6.14) or a structural load-bearing façade (Fig. 6.15).

Fig. 6.12: Non-load-bearing architectural precast concrete façade

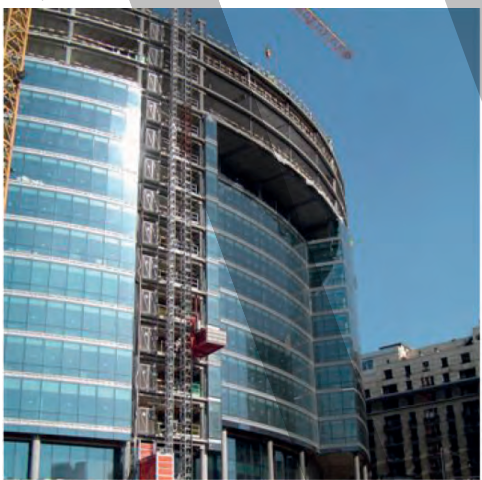


Fig. 6.13: Precast skeletal structure clad with glass curtain walling on curved elevation beams (See also Fig. 6.22)

Fig. 6.14: Skeletal framework with load-bearing spandrel beams

Fig. 6.15: Total precast solution with load-bearing façade

For the internal frame, which is not necessarily established on the same dimensional grid as the exterior, the following solutions exist:

- a totally precast internal frame comprising a structurally stable beam-column frame, floors, walls, staircases and lift shaft walls (Fig. 6.16)
- a combination of a skeletal frame and load bearing walls (see Chapter 7)

6.2.3.1 Structural systems

Precast skeletal structures are composed of columns supporting floor and roof beams (Fig. 6.16). The columns have a height of one or more storeys, typically up to about 4, but 6 is possible if transporting 20-to-25-metre-long pieces is practical and ground conditions are good for propping in the temporary condition. For buildings with 4 or more storeys, the horizontal stability cannot be achieved by the cantilever action of the columns alone, and the structure is completed with central cores and/or shear walls.

Fig. 6.16: Precast concrete skeletal structure

Unbraced frames do not have shear walls or cores and, therefore, the building layout is completely free. However, braced frames will require stabilizing walls/cores at frequent intervals and in both orthogonal directions in the building plan, typically 30 to 50 metres 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.

Floor beams are either rectangular, L-shaped or inverted-tee, as shown in Figures 6.34, 6.35 and 6.37. Internal beams are typically symmetrical in cross section, and most commonly prestressed inverted-tee, if the structural depth zone is minimized, or reinforced rectangular beams for short spans or connecting beams around stairwells and lift shafts.

Prestressed hollow core slabs for spans of 4 to 20 metres are by far the most common types of floors in this type of structure but ribbed floors (double-tee for spans of 8 to 25 metres) and, occasionally, composite plank floor for spans of 4 to 6 metres are also used.

6.2.3.2 Low-rise and medium-rise buildings

For buildings up to 3 storeys high, horizontal stability may be provided by the cantilever action of the columns. They are normally continuous for the full height of the structure. However, for multi-storey skeletal structures, braced systems are the most effective solution irrespective of the number of storeys. The horizontal stiffness is provided by shear walls or cores around staircases, elevator shafts or service shafts, as shown in Figures 6.17 and 6.18. In this way, connection details and the design and construction of foundations is greatly simplified. Central cores can be cast in-situ or precast.

Fig. 6.17: Precast concrete skeletal frame braced against horizontal forces by in-situ core

The recommended span lengths and column heights are listed in Table 6.2. Over the last few decades the maximum available span length of prestressed hollow-core elements has increased constantly to cope with the demand for large open spaces, especially in administrative buildings. A span of 16 metres is achievable for a live load of 5 kN/m² using a hollow core unit of 400 millimetres in depth. In some countries the concept of spanning from one façade to the other without intermediate support is already applied in office buildings.

Table 6.2: Guidance for span lengths and column lengths

	Minimum	Optimum	Maximum
Length floor beams (m)	5	6 - 10	15
Span floor slabs (m)	6	7 - 16	18 - 20
Column height (m)	3 - 4	6 - 12	20 - 25

6.2.3.3 High-rise buildings

In Western Europe over the past years a true breakthrough took place in the construction of high-rise buildings in precast concrete, with heights reaching up to 140 metres and 45 storeys (Fig. 6.18 and 6.20). The buildings are characterized by an extensive central core, cast in-situ with climbing-mould technique, and surrounded with a complete precast structure comprising load-bearing columns, prestressed floor beams and prestressed floors.

Fig. 6.18: Tower building of 36 floors in Brussels during erection.

A typical trend in modern office buildings is the application of circular columns, with most projects insisting on the same constant slender cross-section over the complete height of the tower buildings as the columns remain visible from the outside through the glass façade. The maximum column size is often imposed by the architect, in many cases only 500 or 600 metres. It can take a lot of imagination and design ingenuity from the precaster to ensure that the heavy loading at ground level is accommodated. A possible solution is to incorporate a heavy steel profile in the concrete column or incorporate heavy steel collars in the top and bottom of the columns. The maximum construction depth of the floors, including the floor beams, is also often imposed by the architect. Compared to classical in-situ construction, it is possible to add an additional storey within the maximum building height imposed by the town planning directives in Brussels.

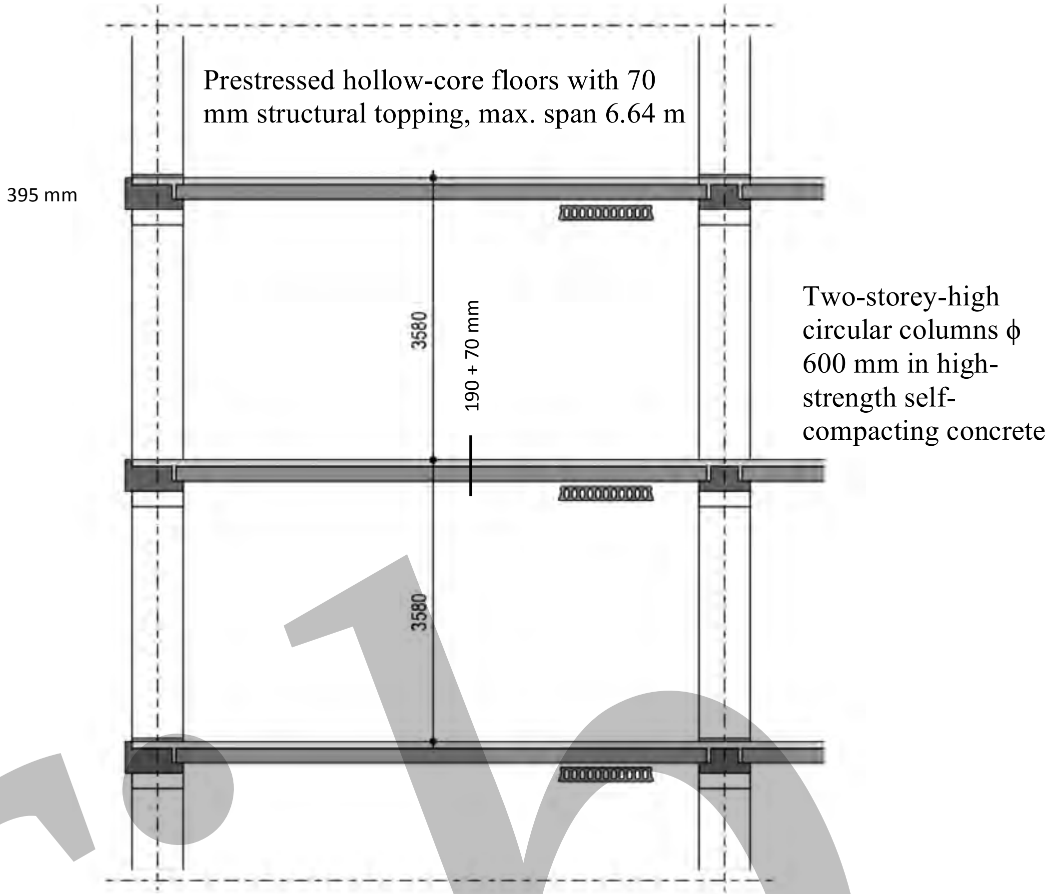


Fig. 6.19: Typical vertical cross-section

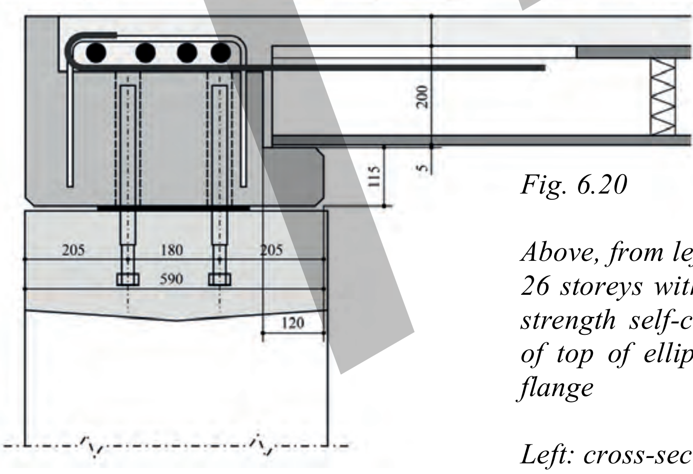


Fig. 6.20

Above, from left to right: Precast skeletal structure of 26 storeys with circular columns ϕ 500 mm in high-strength self-compacting concrete; structural layout of top of ellipse floor; floor beam with curved top flange

Left: cross-section of floor beam

6.2.3.4 Cantilevering beams

Large cantilevering floors are designed and constructed with cantilevering beams. The beams are usually simply supported on two columns (Fig. 6.21). They can be designed as simply reinforced or as prestressed beams. In the latter case, the prestressing tendons are both at the bottom and top section of the beam. To avoid the influence of the negative bending effect of the bottom prestressing force at the cantilever, the tendons are locally debonded by, for example, locating plastic tubes over the strands.

Fig. 6.21: Skeletal frame with cantilevering beams

6.3 Layout and modulation

The planned use of the buildings will, in most cases, determine the span lengths and the direction of the floor slabs and beams and, thereby, the selection of their type. In office buildings where there are open plan areas or offices either side of a central corridor the span of the floor elements will often be perpendicular to the main façade (Fig. 6.22).

Fig. 6.22: Floor slabs spanning parallel to the façade

Precast concrete floor slab are one-way spanning and are, therefore, supported only on the two end beams. The floor is tied to edge and internal beams using reinforcement in the top opened cores or the mesh in a structural topping (Fig. 6.23). Therefore, it is not required to specify beams spanning parallel to the floor units, unless there is a good reason to do so, for example, in the case of a change of direction of the floor span.

Fig. 6.23: Prestressed hollow-core floor units prepared for the additional structural topping, giving a total depth of 260 mm or a span/depth ratio equal to 35

In industrial buildings, roof beams will normally span in the direction of the smallest side of a rectangular floor layout. The floor units will span in the same direction. The reason lies in the repetition of the elements, the possibility of using the edge floor beams to support the façade cladding, the erection sequence of the units, and so forth. For square floor layouts, the bays will be chosen based on the use of the building.

Although parts of the precast industry still labour under the misconceptions of modular precast concrete buildings, the recent projects shown in Figures 1.3, 1.4 and 6.20 demonstrate the versatility of standardized prefabrication. Theses frames originate from the same range of elements produced by a single precaster. However, many texts, old and new, still state that the design of a precast concrete structure should be based on a modular grid, preferably with a basic module of 0.6 metres. There is a clear distinction between 'modular coordination' and 'standardisation'. Modulation is an important economic factor in the design and construction of precast buildings, both for the structural parts and the finishing. The use of modular planning is not supposed to be a limitation on the freedom of planning as it is only a tool to assist in developing economical schemes and to simplify connections and detailing. As far as skeletal frames are concerned, one need go no further than the standardization of families of precast concrete elements to obtain the optimum solution for any building.

When planning a building, it is advisable, but not necessary, to modulate dimensions to suit the element widths. There are certain guidelines for the proportioning of a building in plan that can be useful for the simplification of construction; for example, the standard width of the precast floor units can be modulated at 1200 millimetres. In a simple structure all the floor elements should preferably span in the same direction, which simplifies the layout and, in the case of prestressed elements, limits the number of camber clashes within a bay.

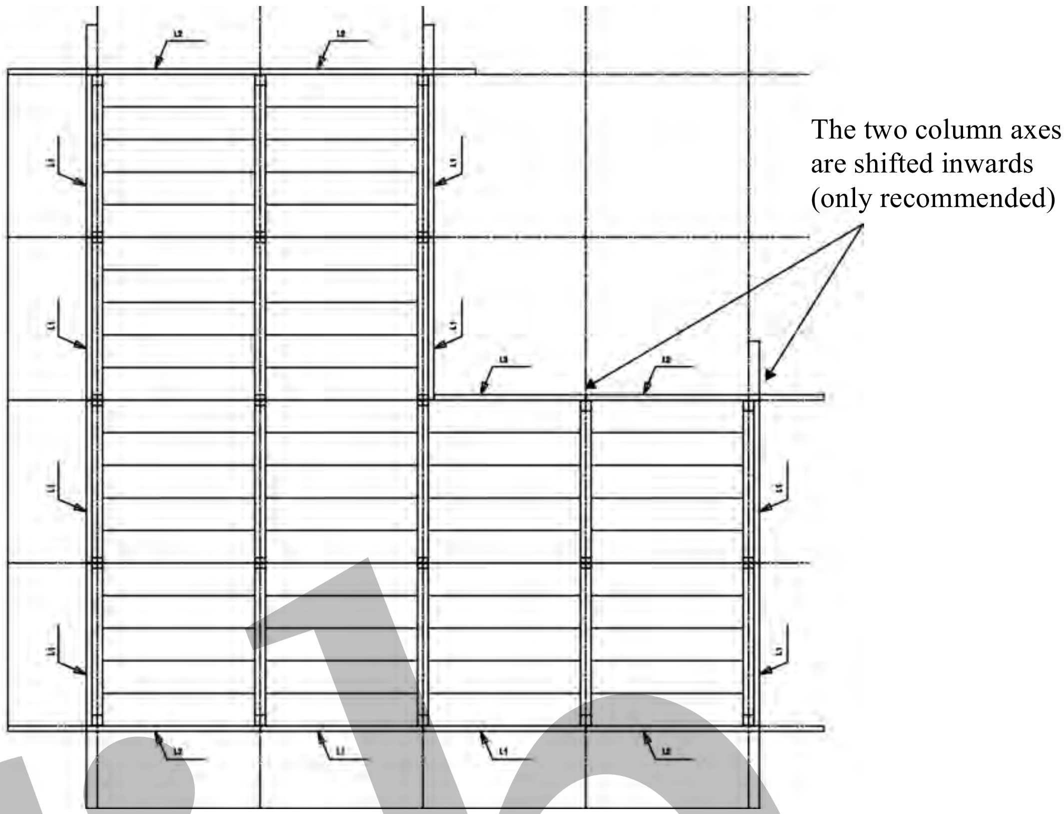


Fig. 6.24: Example of modulation of a floor plan. The two exterior columns at the re-entrant part of the building are shifted inward over a distance of half the column widt, to avoid casting of an in-situ strip in the gap between the hollow-core floors and the wall elements (see Chapter 8, Fig. 8.26b). For possible extension of the building at a later stage, it is preferable to put those columns on the grid axes.

6.4 Structural stability

The different solutions to stabilizing a precast concrete frame and skeletal structures are:

- Cantilevering columns fixed to the foundation (see Section 4.2.1 and Fig. 6.25)
- Bracing achieved by central cores (Fig. 6.26), shear walls or truss action (see also Section 4.3)

Combinations of the two methods may be used in the same structure in different directions or may be partially braced/unbraced in the same direction.

The restrainment of columns into the foundation is an easy solution to stabilize buildings but the maximum height of the structure without additional restraint is limited to about 10 metres. The reasons are the limitations on the column size and the allowable deflections. Second-order deflections e_2 in columns pinned jointed to beams must be considered over the full height, as already discussed in Section 4.2.1.

Fig. 6.25: Precast concrete skeletal frame for car park, designed using the columns acting as vertical cantilevers to resist horizontal forces and drift

Structures stabilized with frame action alone are normally not used because of the concentration of forces in the connection and the difficulty of execution on site. There now exist special systems to achieve moment-fixed connections between columns and beams. They can be used to assist the stability of very slender cantilevering columns.

Braced systems are the most effective solution for multi-storey skeletal structures because the stair and elevator shafts are already present for functional reasons, so the additional cost of utilizing them as stabilizing members is negligible. The concentration of all horizontal actions to some selected members allows for smaller columns and simpler connections. Furthermore, the columns will, in effect, have a horizontal support at each floor level, which also assists with column slenderness

Fig. 6.26: Precast skeletal structure braced by a cast-in-situ central core

The skeletal structure must be looked upon as a whole in its three-dimensional form. Normally it consists of two perpendicular principal directions. The system may be different in the two directions. The system may also vary along the height of the building, for example, with shear walls in the lower storeys and beam-column systems at the higher levels.

6.5 Elements

6.5.1 General

Precast frame and skeletal structures are generally composed of standard elements. They are conceived by the precaster in different shapes and sizes, for example, rectangular beams, I-shaped beams, hollow core and double-tee floor slabs. Each manufacturer gives the dimensions and performances in product catalogues. When planning a project, the designer needs to choose the most suitable elements for his/her project or discuss the possible options with the precaster. In the following general sections information is given about the existing products.

6.5.2 Columns

Precast columns are manufactured in a variety of sizes, shapes and lengths. The concrete surface is smooth and the edges of rectangular columns are chamfered. Columns generally require a minimum cross-sectional dimension of 300 millimetres, not only for reasons of manipulation but also to accommodate the column-beam connections and tolerances. The 400-millimetre dimension normally provides a two-hour fire rating, making it suitable for a wide range of buildings. The strength of concrete used in structural columns is typically C50/60 but C80/95 is also possible.

An example of standard nominal dimensions for rectangular and circular sections is given in Table 6.3, where recommended sizes are shaded (cross-sectional dimensions in millimetres):

Table 6.3: Standard column dimensions

b / h	300	400	500	600	800
300					
400					
500					
600					
Circular					

Columns with a maximum length of 20 to 24 metres can be manufactured and erected in one piece, namely, without splicing, although normal practice is also to work with single-storey columns. Columns may be continuous to the full height of the building or may be stepped back at an intermediate level to satisfy architectural requirements. As with any form of construction, it is desirable to keep columns in vertical alignment and it is preferable to terminate columns at positions where the floor or roof construction can span over the columns. Reasonable changes in the dimensions or shapes of column cross-sections can be produced, either in a single precast unit or by splicing different sections together.

At floor levels, columns may have corbels or structural inserts to provide support for the beams (Fig. 6.27). The positions of the inserts or corbels may be varied to provide connections at different levels on each face of the column, but it is preferable and more economical to keep these variations to a minimum.

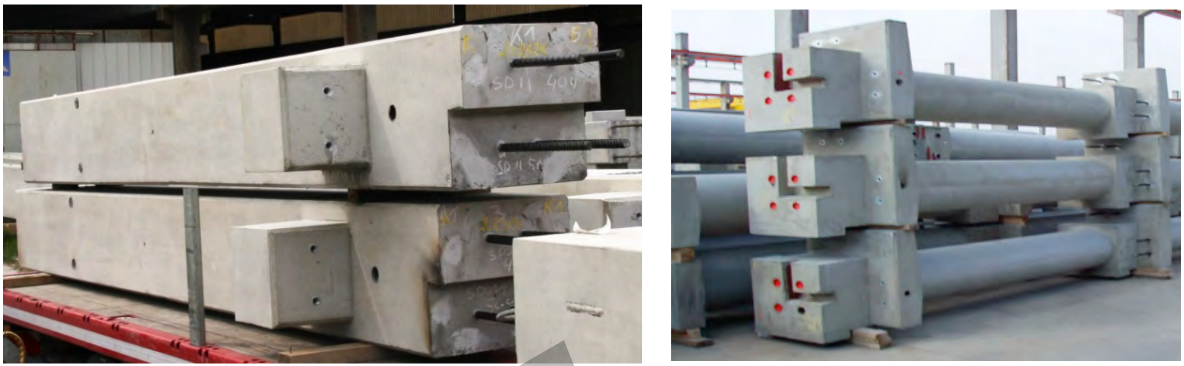


Fig. 6.27: Precast columns with rectangular and circular cross section. The sleeves in the top of the circular columns are used for the continuity of the peripheral tie bars around the corners of the building. The vertical tie bars in the corbels are fixed in anchored sockets.

6.5.3 Beams

The most typical types of precast beams for portal frames and skeletal structures are given in this section; of course, any geometric shape is possible. For each type, a wide range of standardized cross-section sizes is available in the product catalogues of the manufacturers.

- Roof beams

Beams in variable heights are commonly used for industrial buildings where long spans are required. The I-shaped cross section is typical for the prestressed beams shown in Figure 6.28. Examples of classic sizes are given in Tables 6.4 and 6.5.

Table 6.4: Normal sizes of roof rafters and beams of variable height

Width (mm)	Height (mm)	Web thickness (mm)	Span (m)
250 - 300	800 - 1200	80 - 120	10 - 23
300 - 400	1200 - 2000	80 - 140	15 - 30
400 - 500	1300 - 2500	100 - 160	20 - 40
600 - 800 (*)	1700 - 3000	100 - 120 (**)	40 - 50

(*) top flange up to 1000; (**) sometimes with vertical ribs (Figure 6.30)

Fig. 6.28

Left: Roof rafters of variable height

Right: Roof or floor beams of constant height. Roof beams for industrial buildings usually have an I-section with a flat soffit and sloping top, increasing in depth according to the inclination, which is typically between 4° and 10°. Due to the increasing depth, the cross-section properties increase more rapidly than the bending moments and so the rafter is usually critical at about 30% of the span.

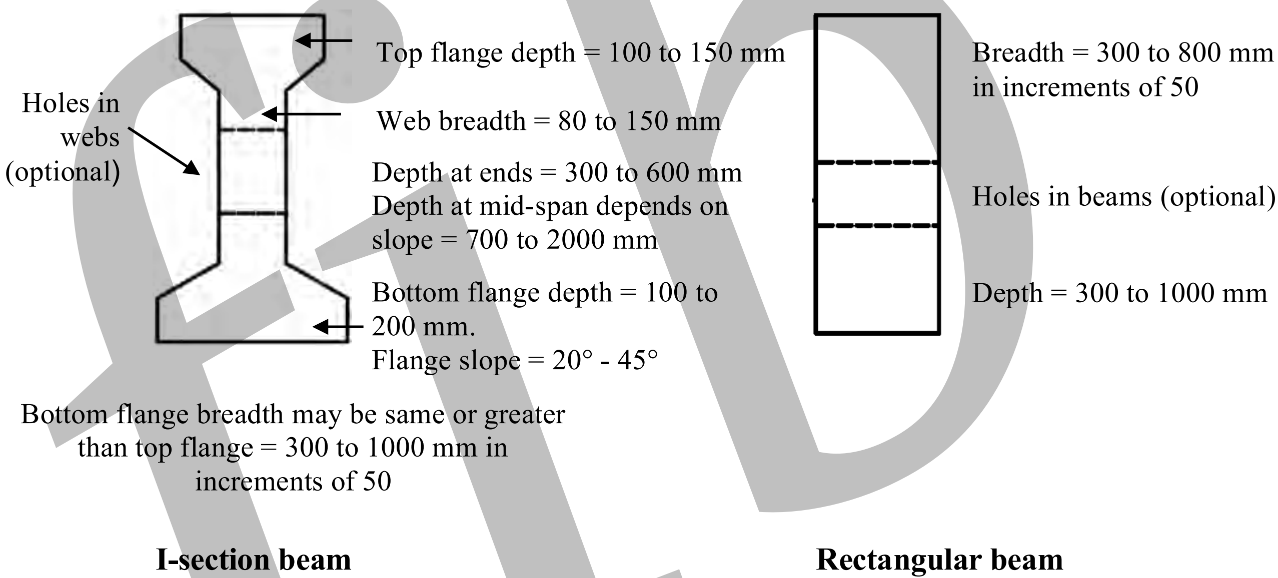
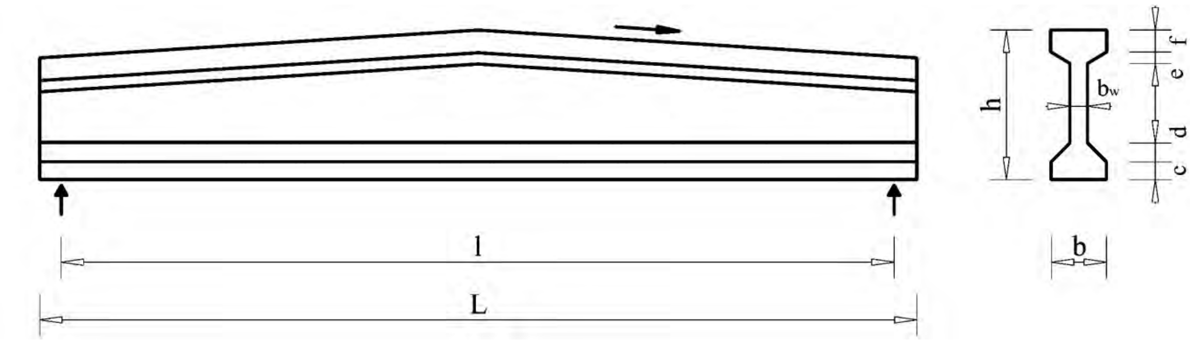


Fig. 6.29: Cross section of rafters and beams used in portal frames

Table 6.5 gives an example of existing dimensions for sloping rafters. The rafters are precast using grade C50/60 concrete and, typically, are prestressed.

Table 6.5: Standardized dimensions of prestressed concrete rafters



Profile	**h**	**b**	**c**	**d**	**e**	**f**	**b_w**	**L_{min}**	**L_{max}**
SI 900/500	900	500	150	190	95	150	120	6000	12000
SI 1050/500	1050	500	150	190	95	150	120	6000	12000
SI 1200/500	1200	500	150	190	95	150	120	8000	16000
SI 1350/500	1350	500	150	190	95	150	120	10000	20000
SI 1500/500	1500	500	150	190	95	150	120	12000	25000
SI 1650/500	1650	500	150	190	95	150	120	14000	28000
SI 1800/500	1800	500	150	190	95	150	120	15000	30000
SI 1950/500	1950	500	150	190	95	150	120	16000	32000

Figure 6.30 provides an indication of imposed load versus span, corresponding to the maximum depth information of Table 6.5. They are used for the preliminary design stage. The calculation is according to Eurocode 2 – Part 1-1 [2].

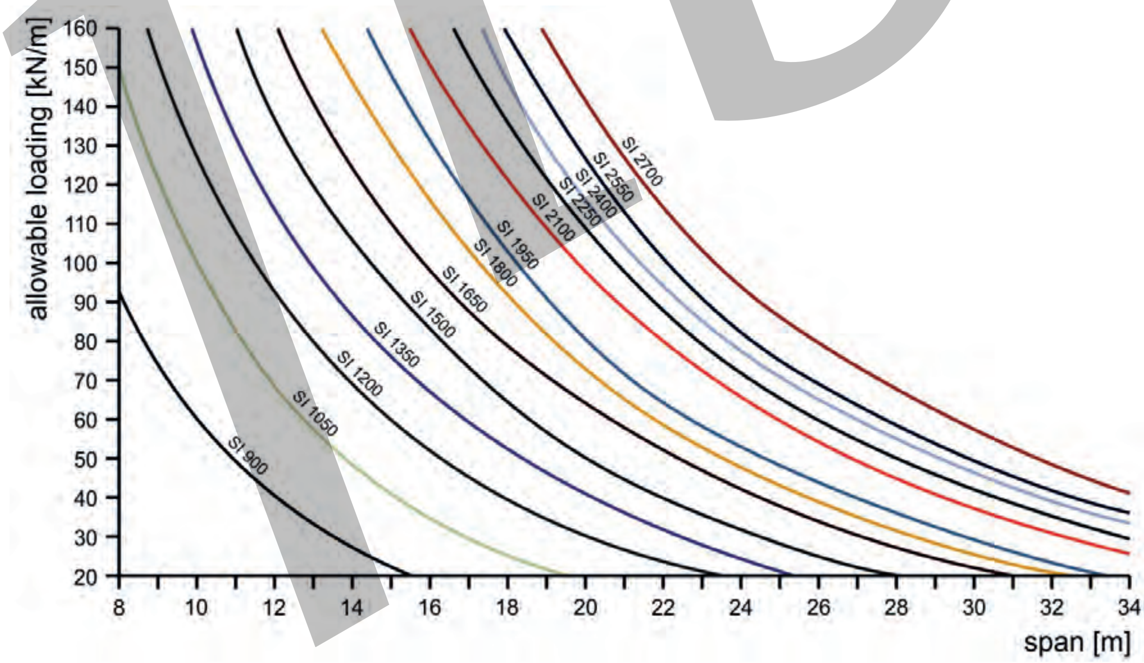


Fig. 6.30: Example of imposed load versus span data for 1/16 sloping I-section rafters, applicable at the preliminary design stage

Other beam cross sections for roof construction are straight I-beams, rectangular beams and 'shed' beams. Straight I-beams are used for both roofs and floors. Normal use of these beams is for long spans and floors with heavy loads. Spans range from 10 to 35 metres.

Beams with a rectangular cross-section are very common. The normal beam width varies from 300 to 600 millimetres and the height from 300 to 800 millimetres. Normal spans are from 6 to 24 metres. Rectangular beams often have half joints to conceal the corresponding rectangular corbels at the supports. They normally do not act compositely with the floor.

A range of Y and U shaped beams, generally called valley beams, are designed with both strength and aesthetics in mind, as is evident in Figure 6.31. Roof construction of this type is popular in some parts of Europe, in particular in Italy.

Fig. 6.31: Example of Y or U roof beams for large open spaces

Trough roofs, designed using the folded plate principle, act both as the structure and roof finish (Fig. 6.32); this type of roof construction is popular in South America. The thickness of the trough walls is 50 to 70 millimetres, and in some cases includes polypropylene and short steel fibres to partially replace rebars in an attempt to control cracking. The self weight varies from 2.0 to 3.5 kN/m^2, as the depth of the trough varies from 300 to 600 millimetres, depending on span, which is possible up to about 20 to 25 metres respectively. The breadth (between crests) is commonly 1.0 to 1.25 metres. Typical span/depth ratio is 40 to 50.

Fig. 6.32: Precast concrete portal with primary I-section beams and trough section roof units

Purlins of greater cross section than normal roof purlins may be used as secondary roof beams. The cross section may be rectangular, I-shaped or trapezoidal (narrowing towards the bottom). Normal span lengths are from 6 to 12 metres and heights are from 250 to 600 millimetres.

Fig. 6.33: Portal structure with roof rafters and purlins

- Floor beams

Floor beams are either rectangular, L-shaped or inverted-tee (Fig. 6.34). Internal beams are typically symmetrical in cross section and most commonly prestressed inverted-tee, if the structural depth zone is minimized, or reinforced rectangular beams for short spans or connecting beams around stairwells and lift shafts.

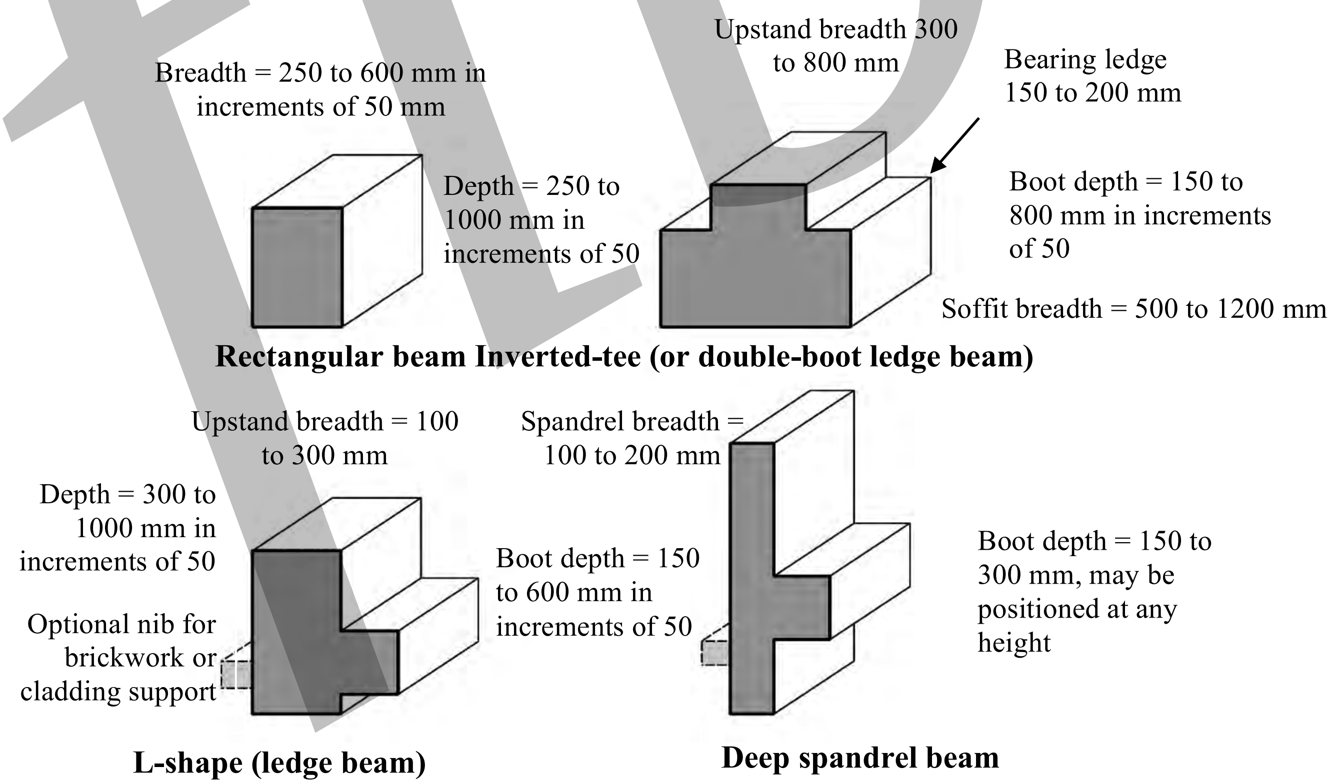


Fig. 6.34: Cross section of beams used in skeletal structures

The most common type of floor beam in precast construction is the ledge beam, with an L-shaped or inverted T-shaped cross section (Fig. 6.35). The beams are in prestressed or reinforced concrete. Changes in floor level may be accommodated with L-beams (or single boot beams) or by building up one side of an inverted tee-beam. Where differences in the floor levels of adjacent spans exceeds about 750 millimetres, a solution is to use two L-beams back to back, separated by a small gap. This is often used at the split-level in car parks but particular consideration needs to be given to transverse ties across the structure. Figure 6.36 gives an example of load versus span capacities of inverted-tee beams.

Fig. 6.35: Prestressed concrete inverted-tee and L-beams

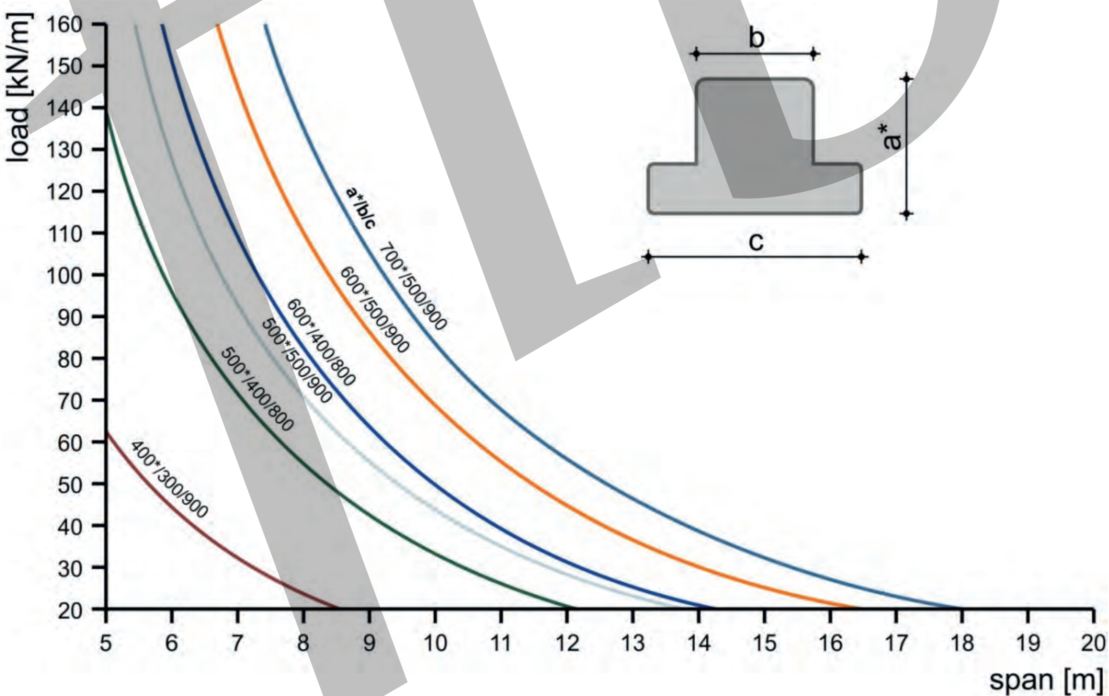


Fig. 6.36: Example of load versus span data for non-composite prestressed concrete inverted-tee beams. The self weight is not included. Concrete is grade C50/60 and the tendons are 12.5-mm-diameter strands with f_{pk} = 1770 or 1860 N/mm^2.

While rectangular beams are the most structurally efficient in terms of load-bearing capacity per unit mass, the extra depth required below the soffit of the slab may be prohibitive. For this reason, inverted-tee beams are commonly used, as shown in Figure 6.34 and 6.35, where the floor slab is recessed into the upper part of the beam, called the upstand, and as such the net depth visible below the beam is reduced.

Precast concrete beams, and more efficiently prestressed beams, may be designed and constructed compositely with the floor slab to enhance the flexural and shear capacity, fire resistance and stiffness. Interface shear reinforcement, in the form of dowel bars or loops, is cast into the beam at the factory (Fig. 6.37).

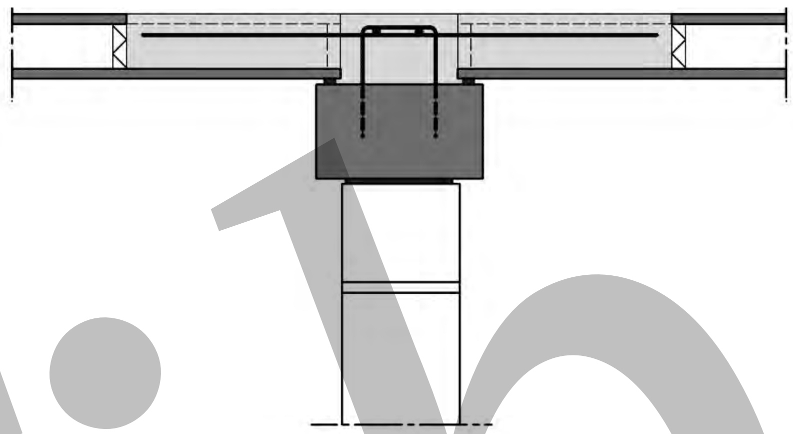


Fig. 6.37: Composite action between the rectangular floor beam and the floors

There are two alternatives: limiting the composite action of the infill concrete to the width of the floor beam or taking into account the collaborating width of the floor slab and the structural topping (Fig. 6.37). The width of the beam is limited only by the need of temporary stability during the construction stage. Design guidelines are given in the FIP Guide to good practice *Composite floor structures* [22] and in *fib* Bulletin 6 *Special design considerations for precast prestressed hollow core floors* [9].

The total depth of the beam can be reduced because the imposed dead and live loads, after the floor slab has been constructed, is resisted by the composite section, which may have section properties 20 to 50 per cent greater than the basic beam, depending on the relative depth of the beam and slab. For example, in the previous exercise, if the 850 by 750 beam is designed compositely with a 250-millimetre-deep hollow-core floor slab, the composite beam depth would be reduced to h = 700 mm, and the depth below the soffit of slab h' = 700 – 250 = 450 mm, which is a saving of 250 millimetres net in depth over the non-composite beam (the number of tendons remains the same).

The main advantage of a composite beam structure is that it permits less structural depth for a given load-bearing capacity. The breadth of the compression flange can be increased to the maximum permitted value, as in monolithic construction. For composite action with hollow-core floors, the collaborating section is through the unfilled hollow core. This comprises only the top and bottom flanges of the slab plus the structural topping screed.

In most cases the effective span ℓ of the beam is calculated from the point of support on the column corbel. However, if the beam support is shifted along the beam to a 'scarf joint', by a distance of approximately 0.15ℓ, for example, 600 to 750 millimetres (Fig. 6.38), the shift causes a negative bending moment at the column, allowing a reduction of the total beam height underneath the floor. The scarf joint may be positioned at one end of the beam or both.

Fig. 6.38: Reinforced concrete inverted-tee beams designed with a scarf joint at approximately 750 mm from the face of the column creates a negative bending moment at the column that will reduce the depth of the beam

Both the classical beam-column connection with corbel supports and the connection with the shifted beam joints present advantages and inconveniences. Solution A in Figure 6.39 gives an excellent force transfer performance, the joints are easily accessible for workers and the large forces in the columns are readily transferred from one column to another. Solution B, with a beam passing over the column and one half beam joint will give better moment distribution over the beams but, most probably, a poorer performance at the column connection. The column connection, which now consists of two mortar joints, will have to overcome the following two handicaps:

- As the beam passes over the column and has a certain negative moment, its top fibres will elongate. This elongation transferred through the mortar joint to the underside of the upper column will negatively influence the bearing capacity of this column, as it will tend to undergo the same deformation and split.

- The joint at the underside of the beam is, in turn, difficult to reach. Placing the mortar is difficult, resulting (most probably) in bad quality and subsequent poor force transfer.

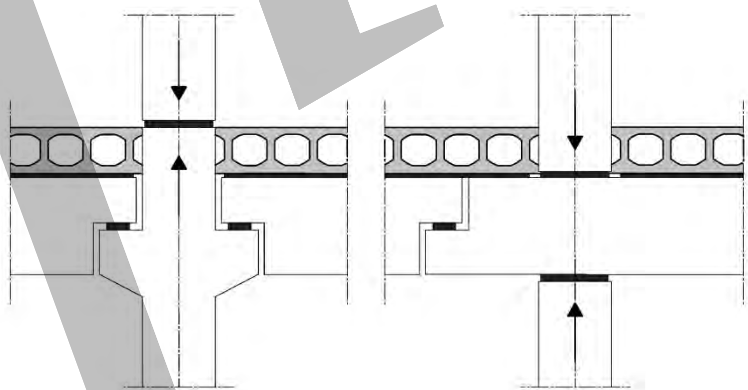


Fig. 6.39: Two alternative solutions for column-beam detailing in precast concrete. Solution A will perform better for load transfer than solution B.

Edge beams or, more precisely, beams with floors supported on only one side, which may not always be at the perimeter but perhaps around staircases, service shafts, and so forth, are typically of L-section (Fig. 6.35, left). 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 which cladding panels, brickwork, and so forth, are attached. As such, edge-beam depths of around 1,200 millimetres are quite common (Fig. 6.40).

Fig. 6.40: Precast reinforced deep-edge beams provide the perimeter enclosure and support for the floor slabs

Floor beams can have either the same width as the column, with the boot projecting inside the column face (Fig. 6.41a), or a smaller breadth, with the boot inside the column width (Fig. 6.41b). In the first case, the floor units pass in front of the columns but, in the second case, it will be necessary to cut notches in the floor units around the columns. The outstanding boot on the beam is generally not continuous past the faces of the columns. The first solution is highly recommended.

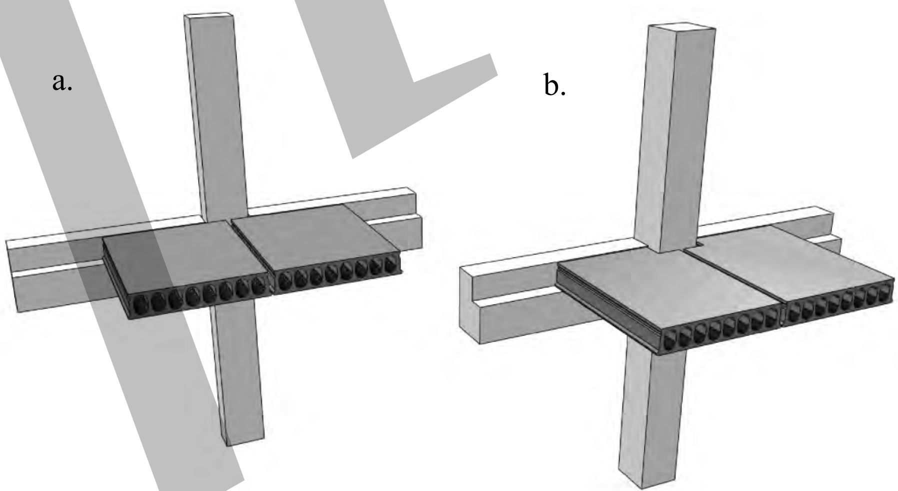


Fig. 6.41: Variant solutions for L-shape edge beams - a. wider beam allows floor slab to be uninterrupted; b. floor slab is notched at the column

6.6 Examples of typical connections

Some examples of typical connections in portal-frame and skeletal structures are given in this section. The idea is certainly not to offer a complete overview of all existing solutions but to make the designer familiar with common types of connections for precast skeletal structures. The principles applied in the majority of solutions are valid both for low-rise and multi-storey buildings. In the full structure they are to be completed with other examples relative to floors, walls and façades. The basic design guidelines for structural connections are given in Chapter 5.

6.6.1 Column-to-foundation

Pocket foundations are used for foundations on good soil. The pocket shall be large enough to enable good concrete filling below and around the column. When the inner surfaces of the pocket is smooth the vertical force is assumed to be transmitted directly under the column base, and so a depth of at least 300 millimetres is required beneath the column (see also Fig. 5.50). If the inner surface is roughened to expose the coarse aggregate or castellated with a shear key (Fig. 5.51), the vertical force and bending moments, provided the foundation is suitable, are transferred successively by shear at the interface. Pocket foundations are either cast in-situ, partially or completely precast (Fig. 6.42). The gap between the pocket and column is filled using structural-grade low-shrink concrete or grout.

Fig. 6.42: Precast pocket foundations are an alternative to site cast pockets, particularly if the foundations cannot be prepared in time for the installation of the column
Left: Totally precast
Right: Precast pocket and cast-in-situ slab

Protruding bars are often used with in-situ slab foundations or foundation piles. The moment-resisting connection is achieved with reinforcement bars protruding from the foundation (Fig. 5.35, 5.47 and 6.43), which is most common, or from the column (Fig. 5.48). They are inserted into ducts to provide lap with the main reinforcement after grouting. The diameter of the ducts should be oversized to allow for placing tolerances (Fig. 5.47).

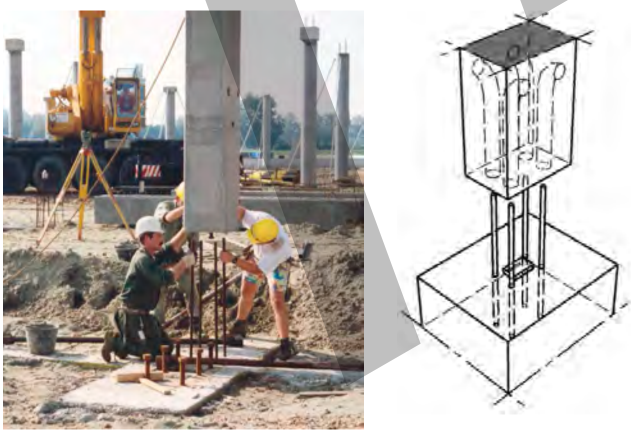


Fig. 6.43: Protruding bars from the foundation positioned in grout ducts in column to enable full moment connection

Bolted steel connections between columns and foundation are used with in-situ foundation slabs or piled foundations. The steel connection is cast into the column and the force transfer is achieved through the lapping of the column main reinforcement bars with steel bars welded to the steel.

There are various alternative solutions:

- Steel column shoes (Fig. 6.44), described in Section 5.4.4.2. The number of shoes in the column depends on the dimensions of the column, moment capacity and the type of column used. This solution is also applicable for circular columns.
- Base plate with starter bars welded to plate. The steel plate is either overhanging beyond the edge of the column, typically 100 millimetres (Fig. 6.45), or may be flush. The disruption to the manufacture of the precast column may be considerable because the plate cannot be contained within the internal confines of the mould.

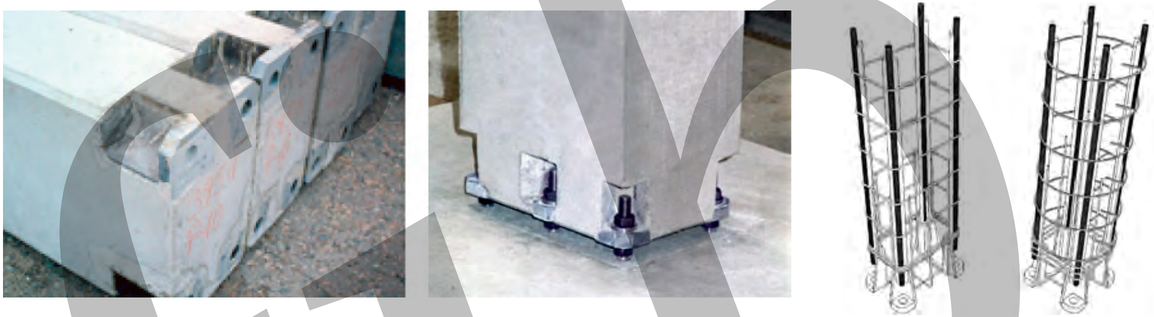


Fig. 6.44: Columns bolted to foundation
Left: Columns with four steel shoes in the stockyard
Centre: Column shoes bolted to foundation by means of threaded splice bars projecting from the foundation, with nuts and counter nuts
Right: Connection of steel shoes to main column reinforcement

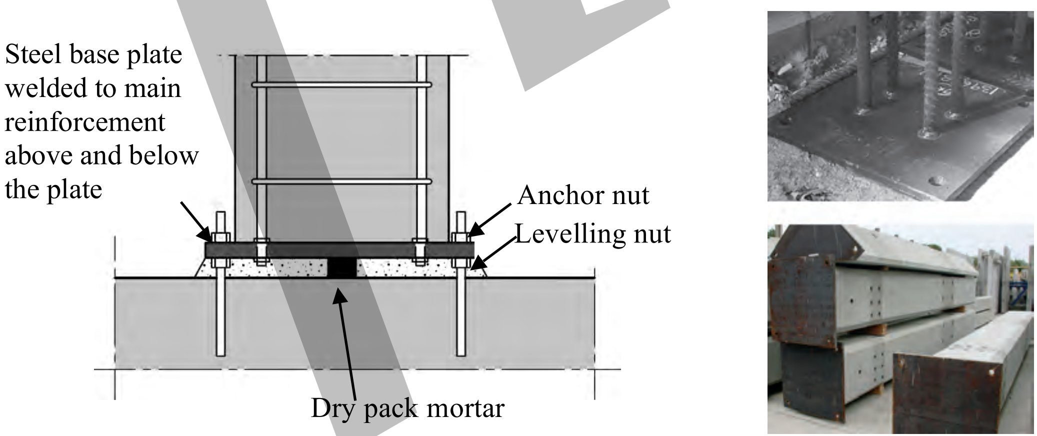


Fig. 6.45: Base plate with starter bars welded to plate
Clockwise from left: Base plate with starter bars welded to plate; base plate with welded starter bars; base plates for rectangular and triangular columns

Although the cost of base plates is clearly greater than the end preparation for pockets, the cost saving is in speed of erection, with the foundation connection immediately stable, thus allowing the floor beam to be positioned almost immediately. The moment-resisting connection in the foundation is achieved with anchor bolts. Holes in the shoes or base plate should always be oversized to allow for dimensional tolerances.

Connections between superposed columns are also known as 'splices'. The splice is usually at the levels of the non-finished precast floor. This allows the joint to be hidden under the column within the structural topping screed. Column splices are, in principle, similar to connections between columns and foundation slabs.

6.6.2 Beam-to-column

6.6.2.1 Pinned connections

Beam-to-column connections form a vital part of precast concrete construction. They have a major influence on the structural behaviour of portal frames but, more importantly, on the stability and execution of skeletal structures, where the full spectrum of connection types, from pinned jointed to fully rigid, are available to the designer. Although the technology exists to design connections within this range, simplicity of execution is far more important than complexity in design, and in this context pin-jointed connections are still the preferred choice.

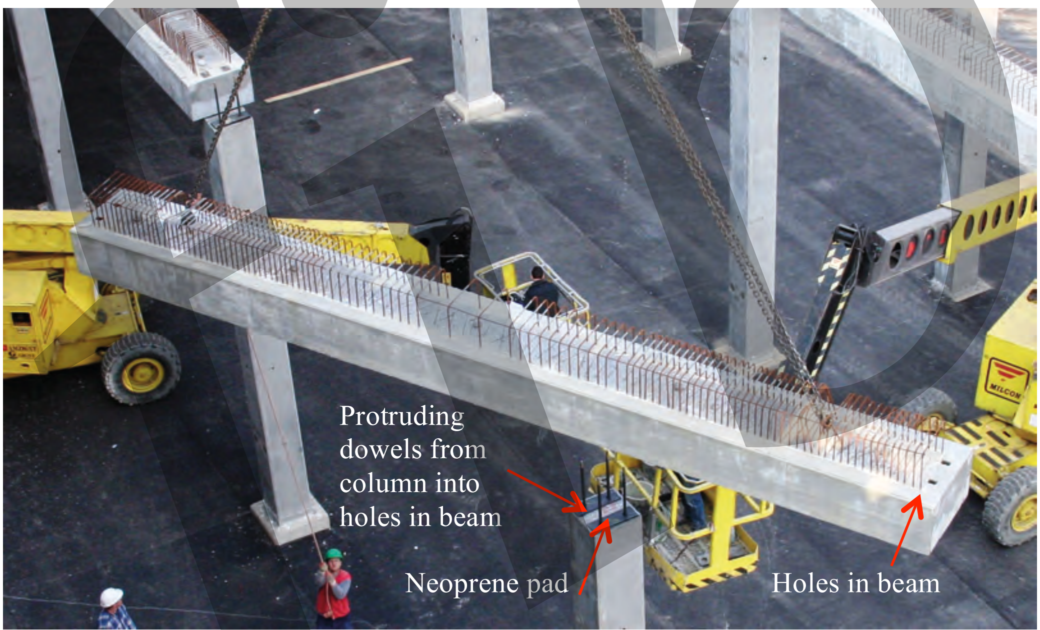


Fig. 6.46: Example of pinned connection between floor beam and column

For a normal pinned connection, dowel bars, which are typically high-tensile rebars 16 to 25 millimetres in diameter, are positioned within the top of columns or corbels to match vertical tubes or slots in the beams. This type of connection is commonly used at the top of columns in portal frames and at the beam-column connections of single-storey columns (Fig. 5.44, 5.45 and 6.46) or beam-corbel connections at multi-storey columns in skeletal frames. Vertical slots, for example, 50 by 80 millimetres in dimension, are provided at the support zone of the beam. Bolts or dowels project from the top of the column or the corbels. After erection the slots are grouted with structural grade grout, with time required for grout hardening. Some allowance for relative movements can be achieved by leaving the slot open or filling it with plastic material, such as bitumen or polyurethane foam. Neoprene rubber or similar bearing pads should be provided at the support of the beam. For edge spans, roof beams can be placed either over the whole top of the column or over half of the top. The latter case may be chosen to allow for the future extension of the building using the same column.

6.6.3 Column-to-column-to-beam

There are two main alternative solutions for column-beam-column connections: with corbels or without corbels (Fig. 6.47). Their specific features are discussed below.

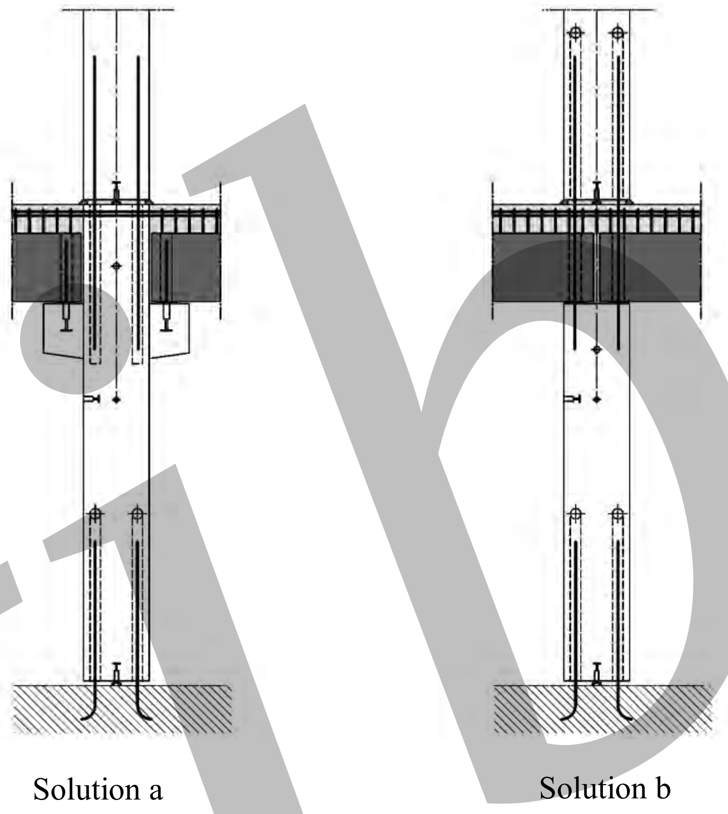


Fig. 6.47: Alternative solutions for column-beam-column connections

6.6.3.1 Solution with corbels

- Floors at the same level on both sides of the beam

The generic types of beams-to-column connections are either at the face of multi-storey columns, or at the head of single-storey columns. Connections at column faces are usually made onto concrete corbels, with dowel bars from the corbel into grouted sleeves at the beam end (Fig. 6.48). Corbel design is dealt with in Section 10.3.

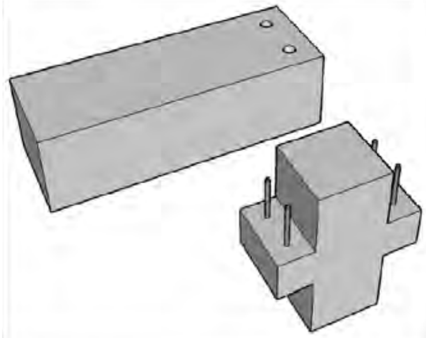
 

Fig. 6.48: Beam-to-column connections using dowels in concrete corbels

The dowel can be anchored by bond or by an end anchor composed of a steel plate, a threaded dowel end and a nut (Fig. 6.49). The latter solution is needed when the length of the dowel bar is short (Fig. 5.45). The grout hole in the beam should be large enough to allow for tolerances in the position of the dowel bars. A practical size for a rectangular hole is 50 by 80 millimetres.

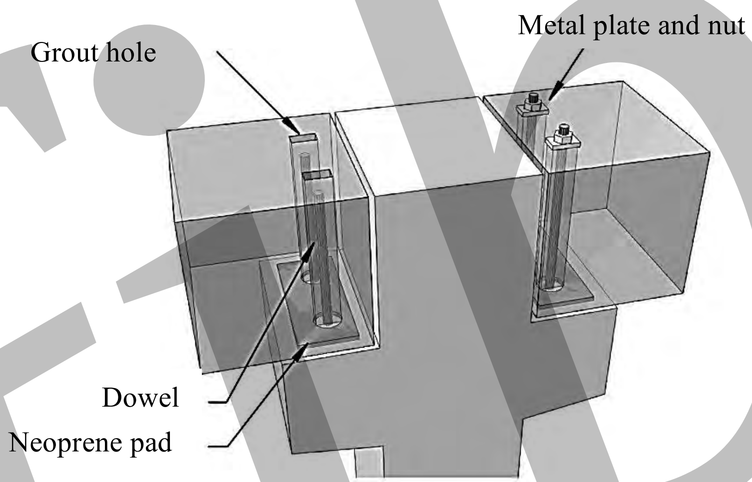


Fig. 6.49: Detailing of pinned connections

The force path through the connection is simple and clear. The beam loading is directly transferred to the column through the corbels and the columns are directly positioned on top of each other. The rotation of the beams because of loading and delayed deformations is free. All parts of the connection are easily accessible for inspection during execution.

Additional erection details applied to high-rise skeletal structures are shown in Figures 6.50 and 6.51.

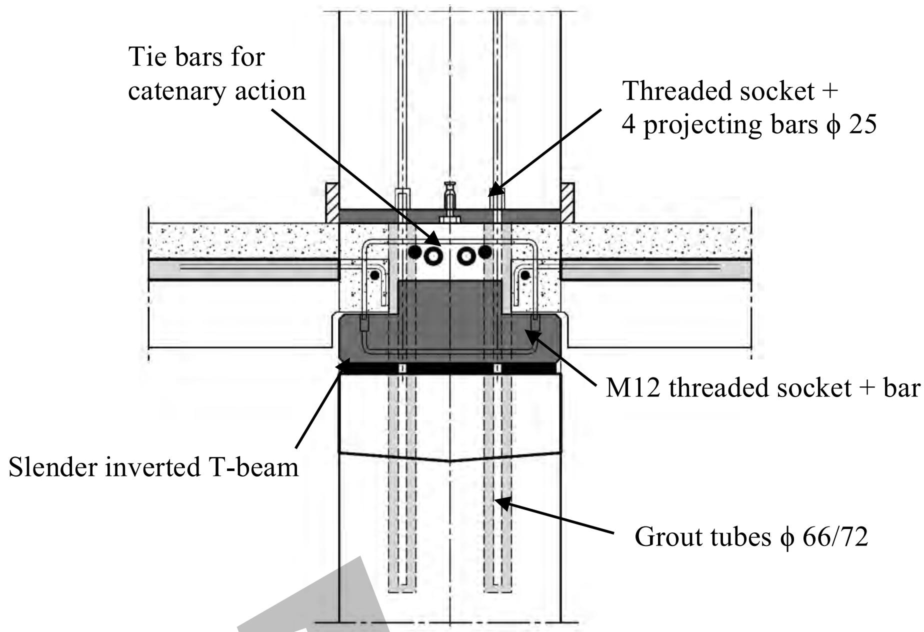


Fig. 6.50: Detailing of a connection between ribbed floors, inverted T-beam and column

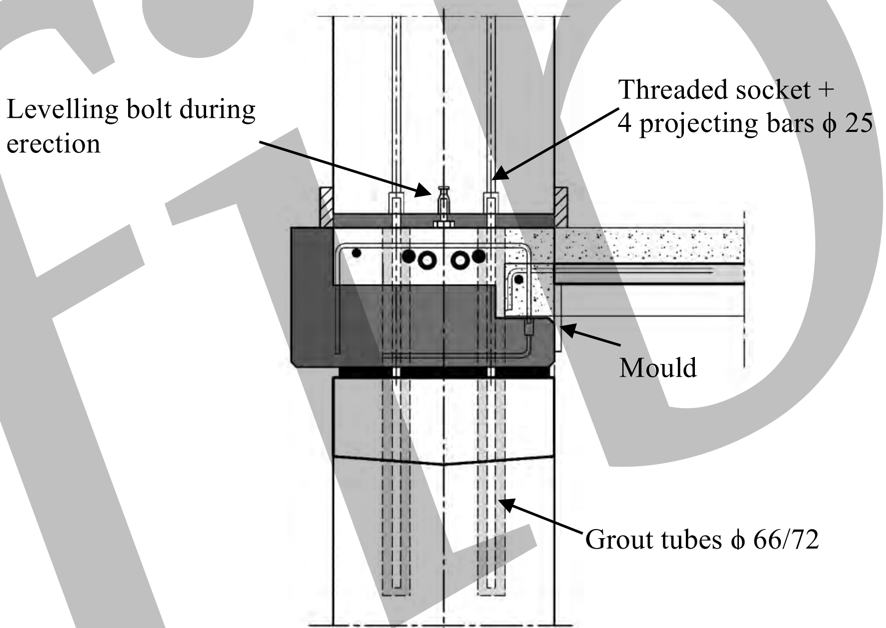
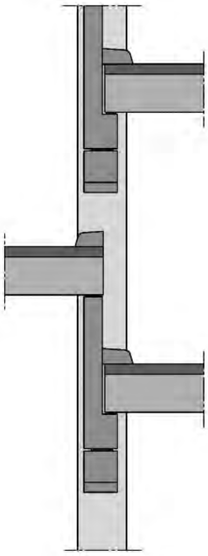


Fig. 6.51: Same as in Fig. 6.50 for an L-shaped edge beam

- Floors at staggered positions on both sides of the beam

- This solution is currently used for split-level car parks. Split-level car parks are typically two-bay structures, where each bay is shifted over half a storey height with respect to the adjacent bay. The floor beam is typically L-shaped and connected to column corbels with pinned connections.



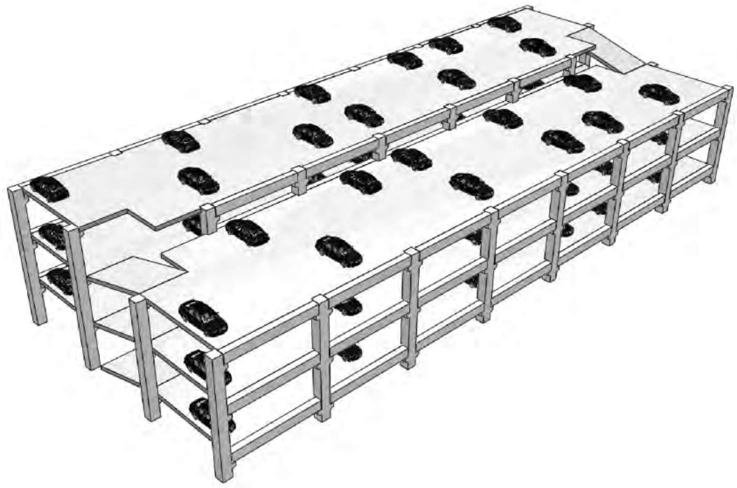


Fig. 6.52: Typical beam solution for split-level car parks

6.6.3.2 Solution without corbels

Solution b of Figure 6.47 is mainly used for architectural reasons, for example, for façade columns with complete open bays to avoid corbels inside the window opening or in cases where the free height of the room is limited. The upper column is directly positioned on the beam ends, with the vertical connection dowels between the columns positioned through the vertical holes in the beams (Fig. 6.53).

This solution has some disadvantages, including:

a. The rotation of the beams at their supports is hindered by the clamping force from the column load on the beam ends. The resulting elongation transferred through the mortar joint to the underside of the upper column might have a negative influence on the bearing capacity of the column as it will tend to undergo the same deformation and may cause splitting in the column.
b. The load transfer from the upper column to the lower column is going through a complex path through two mortar joints and two beam ends. Consequently, the maximum load transfer is more restricted than is solution a.
c. The joint at the underside of the beams is difficult to reach. The placing of mortar may be difficult and there is a risk of bad quality execution.

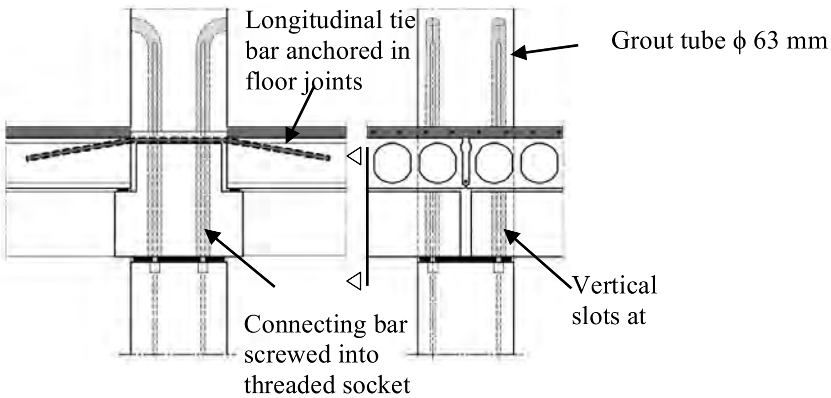


Fig. 6.53: Beam-column connection without corbels

An alternative solution is shown in Figure 6.54. The main difference with the solution from Figure 6.53 lies in the possibility to provide moment continuity for the imposed loading. After the erection of the floors, the transverse tie beam between the slabs is cast, and after hardening, the upper column is erected. The beam acts compositely with the transverse tie beam, as explained in Section 6.5.3 b, and Figure 6.37.

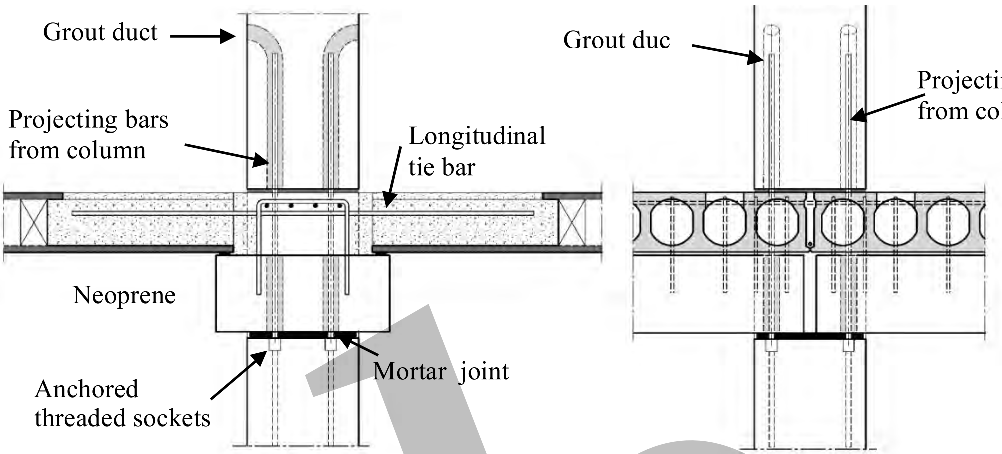


Fig. 6.54: Column-beam connection without corbels and large floor beams

6.6.3.3 Hidden steel connection

In some cases, typically for architectural reasons, the underlying corbel is not acceptable. In these cases a solution with a hidden corbel may be used. This type of connection consists of steel inserts forming a mechanical welded or bolted connection. Various solutions exist in the market. Figure 6.55 shows an example.

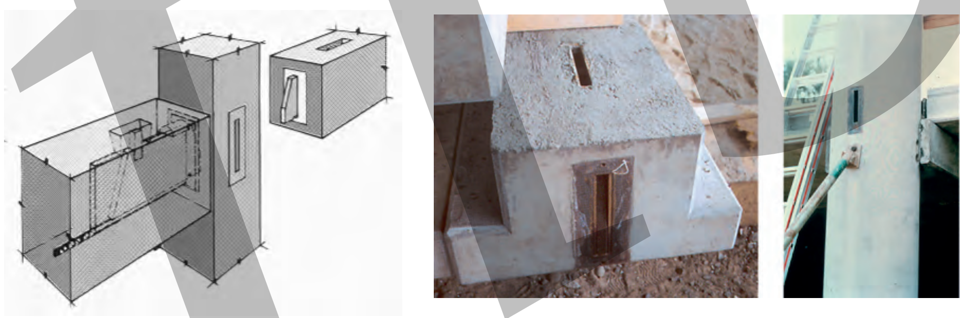


Fig. 6.55: Beam-to-column connection using a sliding plate connector (right)

The exemplified solution consists of a beam unit with a 'sliding knife' in a 'sliding box' and a column unit to receive the knife. Both units are made of structural steel with the units cast into precast concrete beam and column elements (Fig. 6.55, left). The sliding knife has a safety notch to lock the connection. The reinforcement for beam ends and around the column units are designed for various load-carrying capacities. The joints between the column and the beams are normally filled with quickset, non-shrink mortar. Consequently, the connections have fire protection. Production systems have been developed that ensure that the deviations are within the tolerances since allowable tolerances within the system are rather tight.

Precast manufacturers and contractors have extensively tested connections of this type and while design procedures may not always follow normative design rules, they are, nonetheless, of proven strength, stiffness and ductility. The advantage of this solution is that the intersection between the beam and column is neat, without the need for an underlying corbel. The connection is also attractive from an aesthetic point of view.

More information regarding the detailing of the sliding plate connection is given in Section 10.3.4.

6.6.4 Beam-to-beam

Where it is not possible to terminate a beam at a column, connections between secondary beams and primary beams may be made (Fig. 6.56). This connection requires special attention, particularly within the primary beam, where the combined effects of bending, shear, torsion and bearing stresses may cause problems within the shallow depth of the beam. Information on design and detailing is given in Section 10.3.

Fig. 6.56: Beam-to-beam connection using a corbel at mid span of the primary roof beam

7 Wall-frame structures

7.1 General

Precast wall systems may be used for both internal and external walls of low and multi-storey buildings. The wall elements are generally storey height. The thickness is determined by national codes for stability, acoustic insulation and fire resistance. The length varies depending on the project, the equipment at the precasting plant and transportation restrictions.

Precast wall systems are mainly used in housing and apartments but also in hotels, hospitals, office buildings and other similar types of construction. They are also frequently used for central cores, lift shafts and stiffening or infill walls in all types of buildings. Finally, precast walls are commonly used as fire separation walls.

Besides fast and industrialized construction, precast walls offer a smooth and ready-to-paint surface finishing, good acoustic and thermal properties and fire resistance of up to 6 hours.

7.2. Structural systems

Wall frame systems can be divided into three main categories depending on the orientation of the main load-bearing walls in relation to the long axis of the building. They are:

- Integral wall system
- Cross-wall system
- Spine wall system

7.2.1 Integral wall system

In this system the bearing walls run both longitudinally and transversally to the long axis of the building (Fig. 7.1). Some of the walls, depending on the functional needs of the building, may serve as non-bearing partition walls.

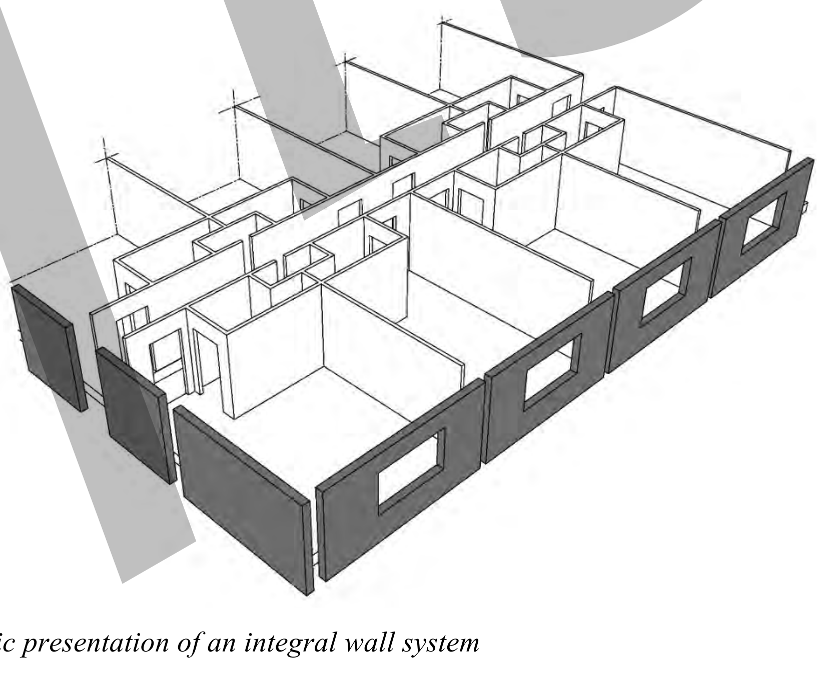


Fig. 7.1: Schematic presentation of an integral wall system

Fig. 7.2: This fully precast residential structure incorporates perimeter precast load-bearing walls, precast stair flights and landings with full-span hollow-core flooring, allowing for long term internal flexibility

Figure 7.3 shows an example of the outline of an apartment building in which all the walls are of precast concrete. Some of them are load-bearing; others perform only a separating function. The façades are usually designed as sandwich elements, whereby the internal leaf can be load-bearing or not. Floors are mostly made up of hollow-core elements or floor plank systems.

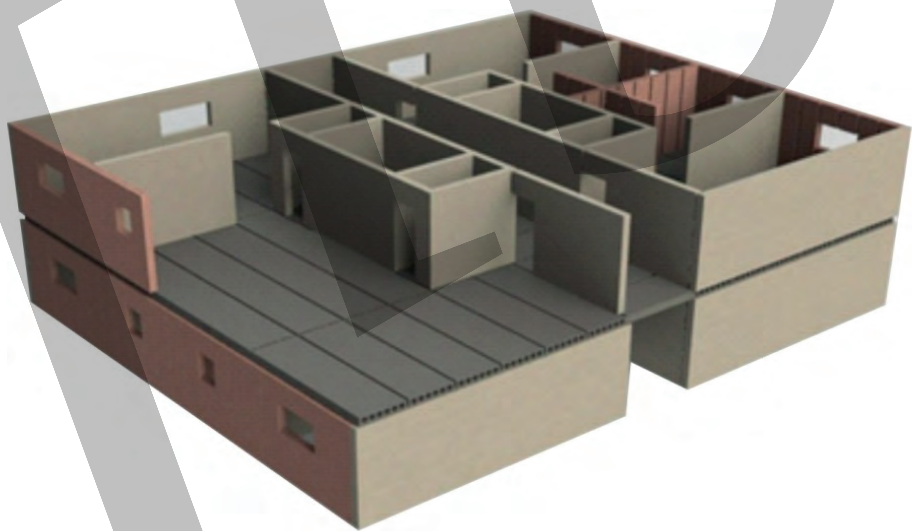


Fig. 7.3: Integral wall system

7.2.2 Cross-wall system

Figure 7.4 schematically illustrates a cross-wall system.

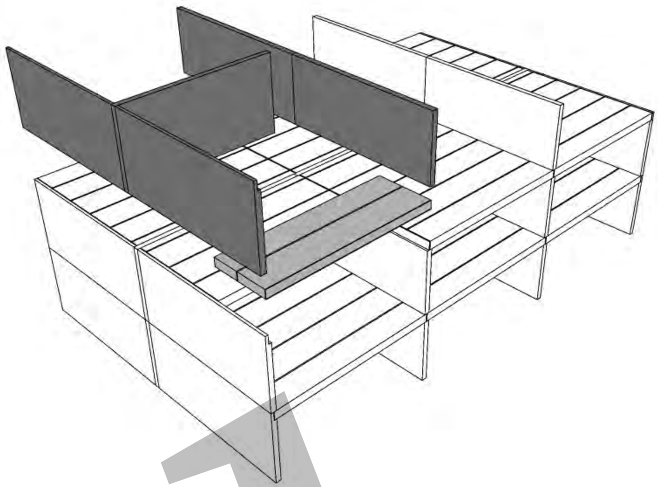


Fig. 7.4: Schematic presentation of a cross-wall system

In the cross-wall system the walls run across the building width and the slabs span between the crosswalls along the longitudinal axis of the building; it is typically used for cellular buildings such as apartments, hotels and prisons. The floors are normally made of prestressed hollow-core slabs, with a span of approximately 9 to 12 metres. The aim of modern design philosophy is to create a large, open space inside the whole apartment. In this way, one obtains not only more flexibility for the internal layout of the floor plan but also the possibility to modify it in the future.

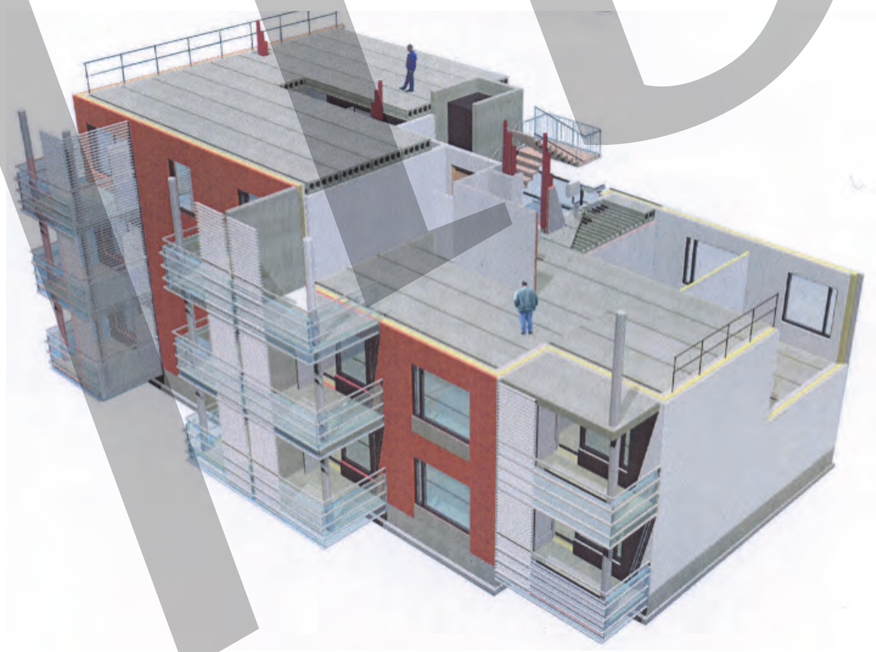


Fig. 7.5: Outline of a building with load-bearing cross walls

7.2.3 Spine-wall system

Figure 7.6 schematically illustrates a spine-wall system

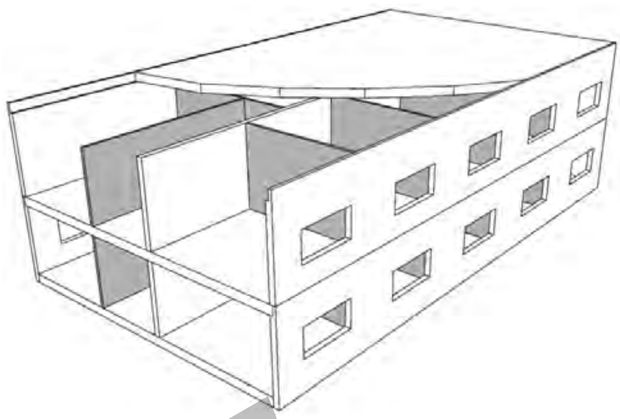


Fig. 7.6: Schematic representation of spine-wall system where dark surfaces indicate non load-bearing panels

In Figure 7.7, the load-bearing walls form the front and back façades of the building and support the floor slabs. The spine-wall system generally comprises loadbearing sandwich panels on the external elevations. When the total width of the building exceeds the span capacity of the floors, internal spine walls are positioned along the longitudinal axis of the building. They often form corridors/bathrooms/stair cores/service risers. The spine-wall system enables both large open-plan areas and smaller rooms subdivided by non-load-bearing partitions.

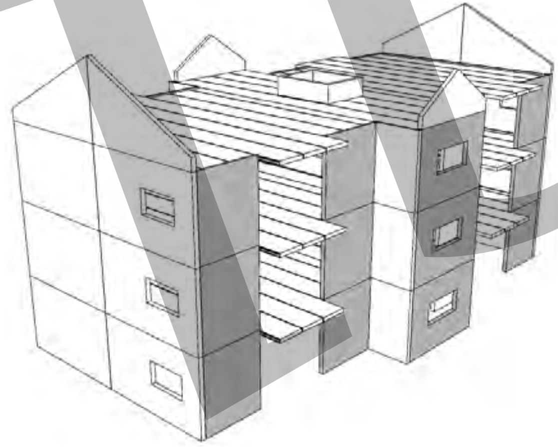


Fig. 7.7: Outline of apartment building with load-bearing façades. Intermediate walls used where the total span between opposite façades becomes too long.

7.2.4 Cores and lift shafts

Load-bearing wall panels are often used to construct elevator and stairwell shafts. The panels are connected after erection to form composite T, L, U or box-shaped sections (Fig. 7.8 and 7.9). 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 options lies mainly in the quality of the surface finishing, the speed of construction and the improved organization of the erection of the fully precast structure.

Fig. 7.8: Precast core with two-storey-high wall panels, connected temporarily during erection by bolting, followed by welding of cast-in anchor plates

Fig. 7.9: Precast core with single-storey panels, connected by a combination of wet-shear joints for a project in the Netherlands

7.3 Modulation

Depending on their profile and length, apartment walls are composed of one or more panels positioned in line with each other. The maximum length of a single panel is governed by the equipment at the plant and the on-site cranage. It is normally between 6 to 9 metres long, but can, exceptionally, reach 14 metres. When the room is larger than these dimensions several panels may be used. Where possible, joints should be located at the connection to perpendicular walls. The minimum panel dimension of 2.40 metres is only dictated by a practical concern for savings and should not be considered as a restriction or limitation.

7.4. Structural stability

The analysis of the stability of precast concrete walls against vertical and horizontal forces comprises:

- The resistance of the panels in the most loaded cross section
- The resistance against buckling
- The resistance of the horizontal and vertical connections

7.4.1 Actions

Wall frame structures are designed for the following actions:

- Vertical actions: self weight and imposed loads
- Horizontal actions caused by wind, seismic actions (if they exist), and the eccentricities and inclination of the vertical structure
- Accidental actions such as fire, explosions, impact, and so forth

7.4.2 Eccentricities

Loading from floors and upper wall elements are transmitted to the lower walls with a certain eccentricity. They introduce bending moments in the walls and tensile forces in the connections to the floor diaphragm. The following initial eccentricities are taken into account in calculating the walls and the connections to the floor:

- Structural eccentricities
  - eccentric location of the floor support on the wall e_{fl}
  - eccentricity of the load from the upper wall unit e_s
  - eccentricity of the self weight of the panel e_G

- Eccentricities due to geometric imperfections
  - inclination of the panels
  - defaults in execution at manufacture and erection e_p and e_m

7.4.2.1 Structural eccentricities

- Eccentricity of the floor support

The total load of a simply supported precast floor is transferred to the wall with an eccentricity e_{fl}. When the floor unit is placed without supporting pad or mortar, the location of $(G + Q)_{floor}$ is at 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. For the location of e_{fl}, possible positioning tolerances are not taken into account.

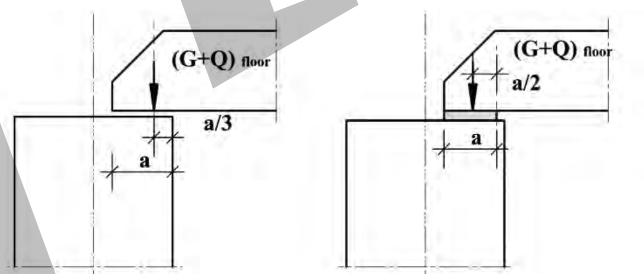


Fig. 7.10: Eccentricity of floor load

For precast floors with restrained supports, the floor load is applied in two steps with two different eccentricities:

- G_{fl} is the part of the load that is transferred to the wall before the hardening of the in-situ concrete. The eccentricity is the same as for simple, supported floors.
- Q_{fl} is the part of the loading transferred after the hardening of the in-situ concrete. The load applies to the centre of the wall.

- Eccentricity of the load from the upper wall unit

The load from the upper wall panel is transferred with an eccentricity e_s between the centre of gravity of the mutual contact zone between the upper and lower panel, and the load application centre at the bottom of the lower wall.

- Eccentricity of the self weight of the panel

The eccentricity e_G of the self weight of the lower panel is also determined with respect to the load application at the bottom of the lower wall.

- Combination of eccentricities between superposed wall panels

Figure 7.11 shows the forces that act on a wall panel and their eccentricities with respect to the hinge at the bottom of the panel.

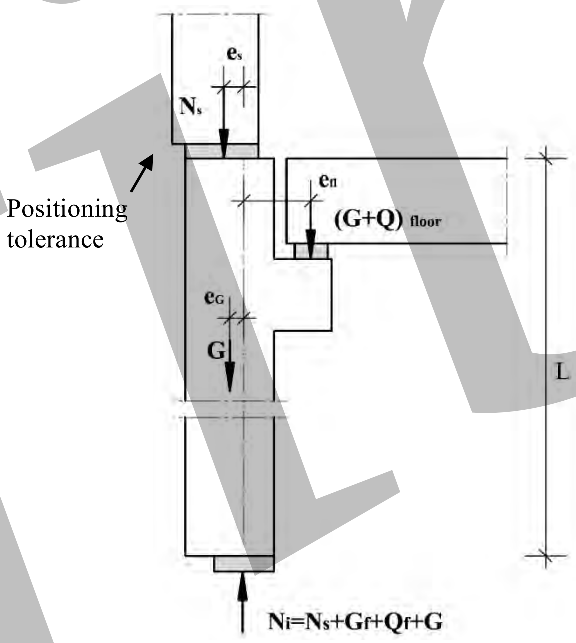


Fig. 7.11: Load eccentricities acting on a wall panel

7.4.2.2 Eccentricities due to geometric imperfections

- Inclination of the elements

The unfavourable effects of possible deviations in the geometry of the structure and the position of the loads shall be taken into account in the analysis of members and structures. Eurocode 2 Part 1-1[2] stipulates in § 5.2 (2) that for members with axial compression and structures with vertical load deviations may be represented by an inclination θ_i. With normal execution tolerances, the following design value for the inclination may be used:

$$\theta_i = \theta_0.\alpha_h.\alpha_m$$

where

θ_0 is the basic value (see National Standard prescriptions; the recommended value is 1/200)

α_h is a reduction factor for the height

$\alpha_h = 2/\sqrt{\ell}$ $\quad 2/3 \leq \alpha_h \leq 1$

α_m is a reduction factor for the number of members

$\alpha_m = \sqrt{0.5(1 + \ell/m)}$

ℓ is the height of the panel

m is the number of members contributing to the total effect.

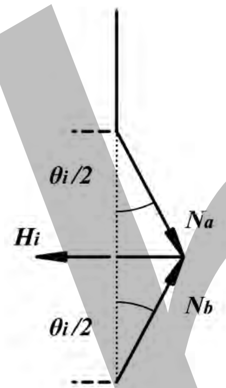


Fig. 7.12: Effect of inclination of wall panels

The effect of the inclination θi (Fig. 7.12) may be represented by transverse forces, to be included in the analysis, together with other actions.

$$H_1 = \theta_i (N_b + N_a) / 2$$

As a simplified alternative the effect of the possible inclination of the wall panels may also be predicted by a notational horizontal force equal to 1 per cent of the vertical load on the considered floor level, with a minimum value equal to 30 kN/m wall (see CEB-FIP Model Code 1978).

- Eccentricities due to variations in execution

The following eccentricities are mentioned in the CEB FIP Model Code 1978 [23]:

a. Variations in flatness

   $e_p = 2\ell/1000$ to $3\ell/1000$ where ℓ is the panel height

b. Tolerances in positioning

   The following values are indicated:
   e_m = 5 mm when the underlying panel is visible during erection
   e_m = 10 mm when it is not visible

7.4.3 Horizontal stability

7.4.3.1 General

The horizontal stability of a structure with precast walls is ensured by means of the cantilever action in walls and cores, and the diaphragm action of the floors. Precast walls are commonly incorporated to function as stiffening walls. The layout of walls and cores shall be such to withstand horizontal actions in any direction with their eccentricities, namely, to be able to offer adequate stiffness and resistance at least in two orthogonal directions and against torsion about a vertical axis (Fig. 7.13).

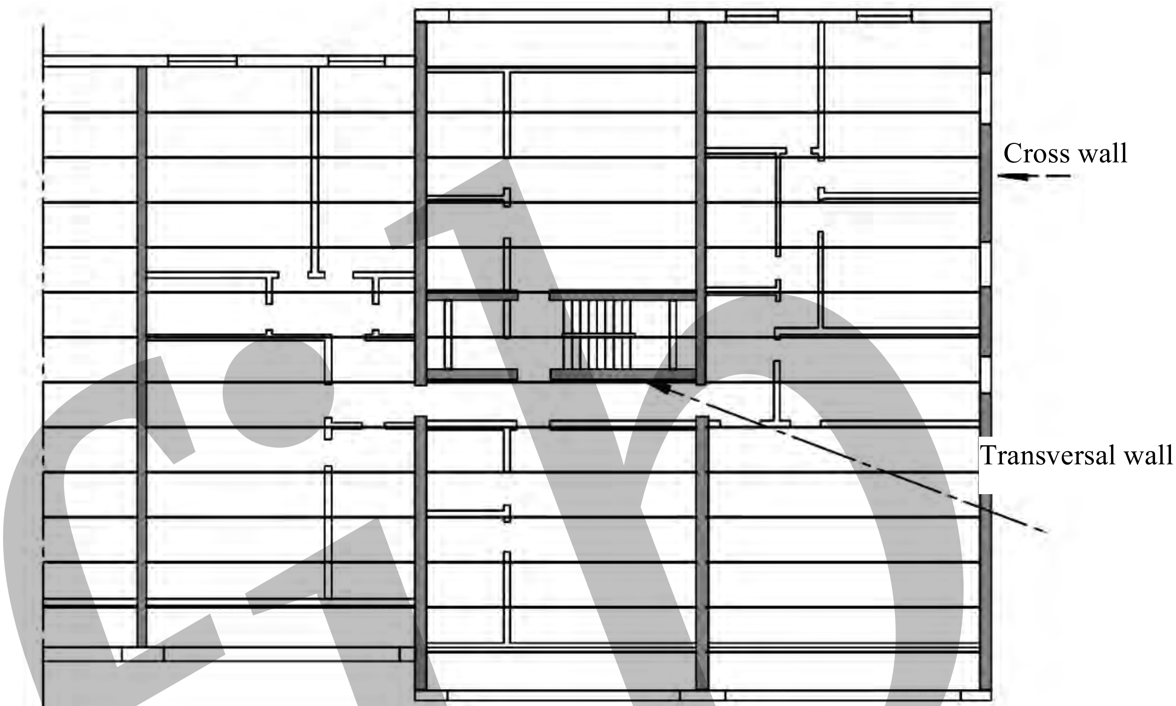


Fig. 7.13: Positioning of stiffening walls

When walls have rather large openings, for example, for doors, the part of the wall above the door opening may possibly contribute to the horizontal resistance. If it cannot, only the part of the wall outside the door opening should be taken into account. Nevertheless, in some cases, particularly in earthquake conditions, lintels above openings may serve as parts of the structural system that contribute to seismic energy dissipation (see Section 7.4.3.5).

The composite action of adjacent wall panels forming L, H or T shapes is possible, provided the vertical joints between the panels can transfer the required shear forces.

7.4.3.2 Shear-wall action

Every panel that is incorporated into a multi-level wall should be connected in such a way that the total wall can function as a single unit in a vertical cantilever subject to shear and bending, in addition to the compression due to the bearing action. Single and composite cantilevers may be distinguished (Fig. 7.14).

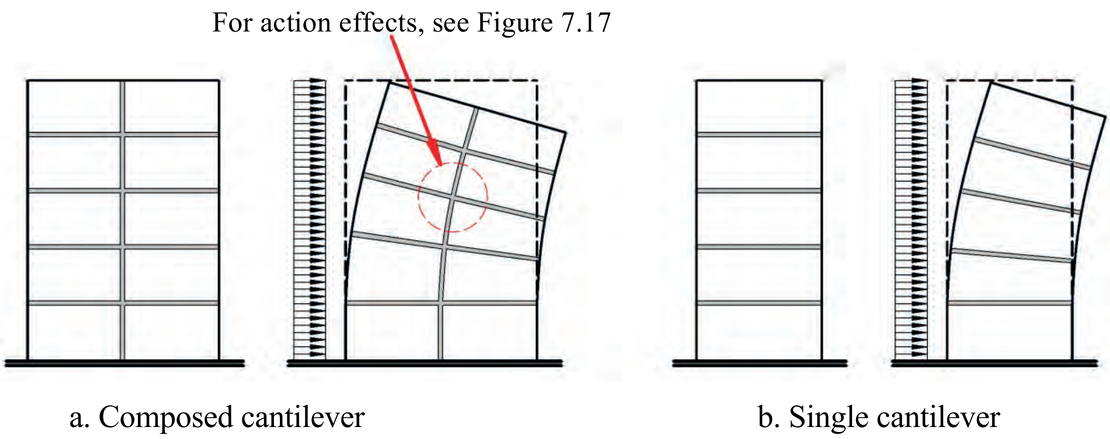


Fig. 7.14: Composite vertical cantilever

The connections between the different wall panels should be able to transfer the internal shear, compression and tensile forces (Fig. 7.15).

7.4.3.3 Structural response of connections between walls (wall-to-wall connection)

- Action effects

Figure 7.15 schematically shows the action effects developed in the vertical and horizontal connections between walls due to the vertical loading, horizontal loading and other types of secondary actions, for example, thermal deformations.

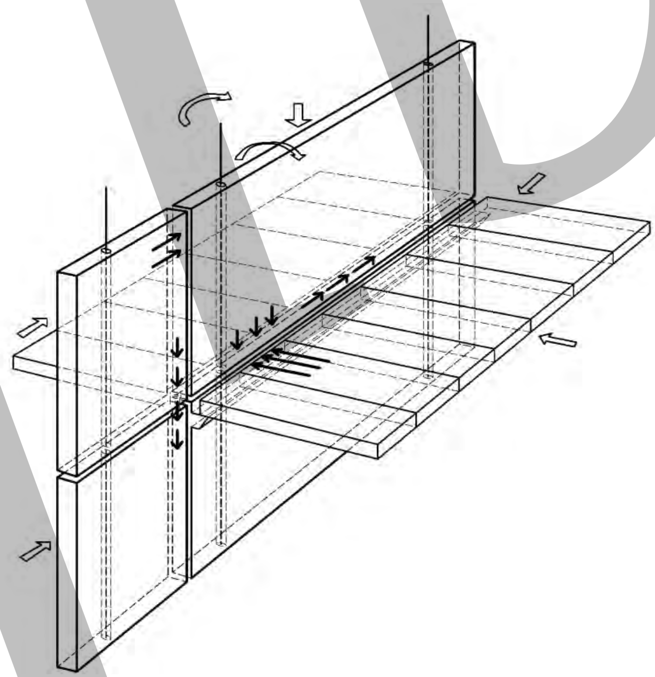


Fig. 7.15: Action effects in the vertical and horizontal connections between wall panels

The diaphragm action of the floors plays an important role in the transfer and distribution of the horizontal forces over the different stabilizing components (see Sections 4.3 and 4.4). The following actions are considered for:

- Horizontal connections:
  a. Compression due to upper walls and support of the slabs
  b. Shear due to lateral loading and diaphragm action of the slabs
  c. Horizontal forces acting in the plane of the slabs
  d. In-plane bending moment due to flexure of the walls in their plane.
  e. Out-of-plane bending moment due to flexure of the slabs.

- Vertical connections:
  a. Shear forces mainly due to the cantilever action of the walls (in their plane) under horizontal loading and due to other secondary actions.
  b. Bending moments due to the total response of the structural system for any kind of actions and deformations.

- Configuration of connections

- Primarily shear connections (vertical joints):

  Generally, for high-rise buildings, vertical shear joints between wall panels are provided with indented interfaces (see also Section 5.4.3.2). The panels can be situated in the same plane, or form an angle, for example, in stabilizing cores. The horizontal tie reinforcement could be concentrated in the horizontal joints between wall panels at each floor level or distributed in the vertical joints over the total panel height. The latter option is mostly needed in seismic situations (Fig. 5.42). After erection, the vertical joints are filled with concrete to enable the transfer of shear forces from one wall panel to the other.

  In Figure 7.16, taken from 'Structural behaviour of RC precast panel's connections' [54], the configuration of keyed shear connections is shown schematically with an indication of the possible evolution of cracks along the joint in the ultimate limit state, for monotonic and cyclic loading due to earthquake.

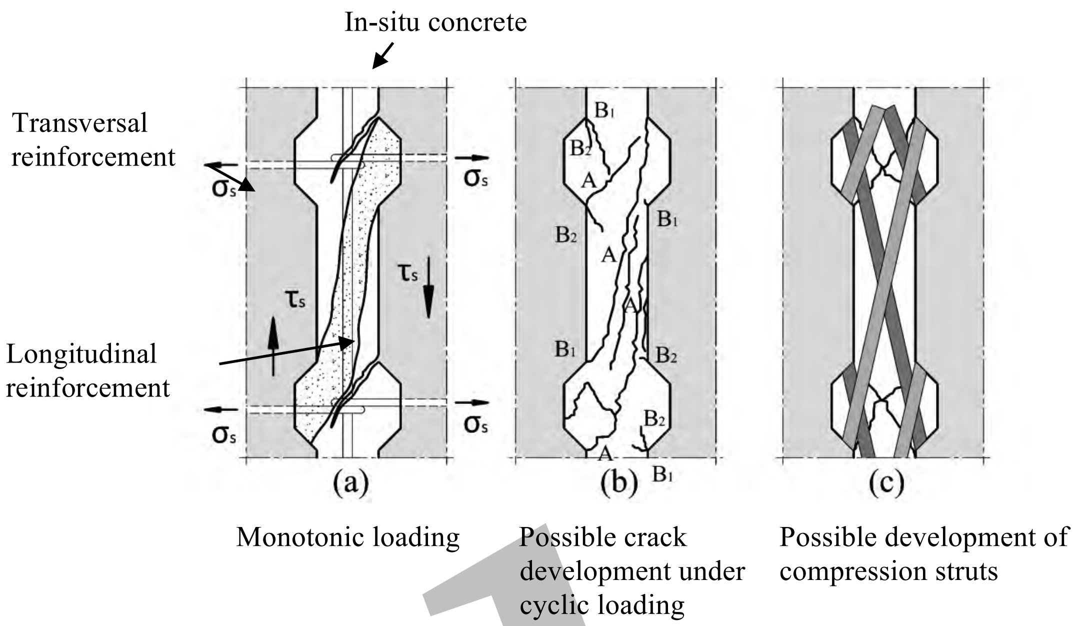


Fig. 7.16: Configuration of shear connections with indication of possible development of cracks for monolithic (a) and cyclic loading (b) and possible development of compression struts (c) under cyclic loading. In all cases the same type of transverse reinforcement is assumed.

Figure 7.17 gives indicative curves of shear stress -shear slip relationship, depending on the configuration of the interface between the precast panels and the in-situ concrete joint.

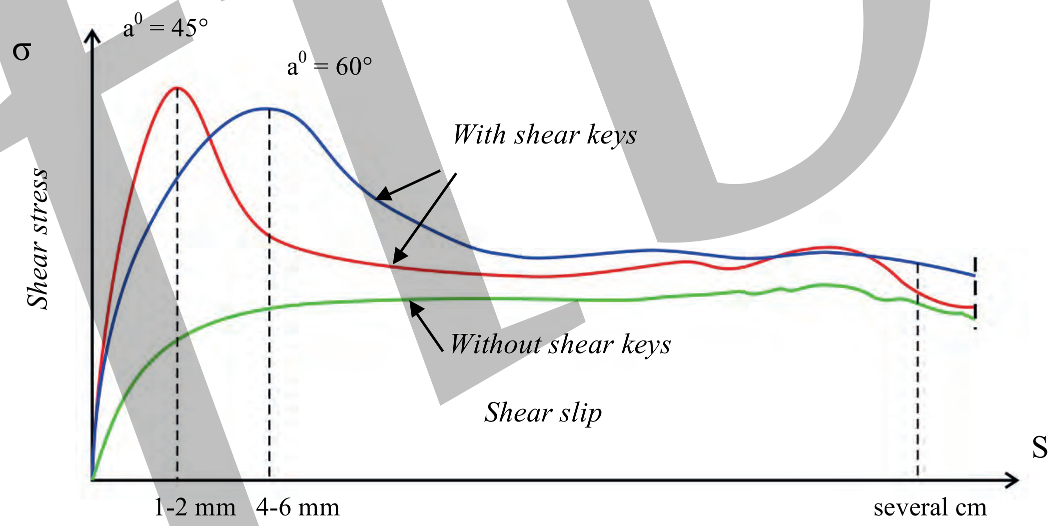


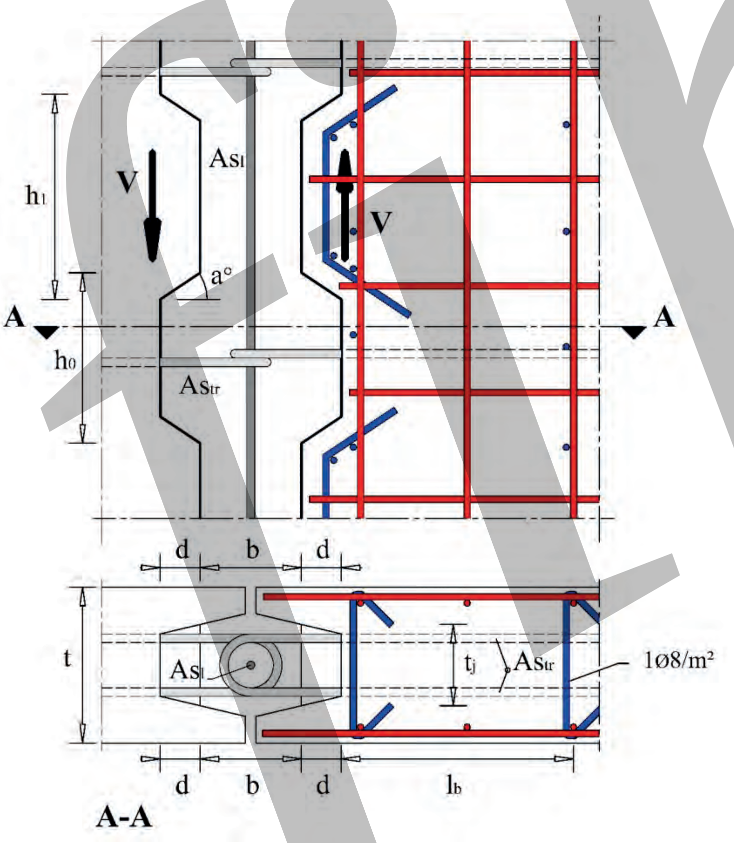
Fig. 7.17: Indicative - informative, shear stress - shear slip relationship (for monotonic loading) in function of the slope of the keys for indented shear joints, and for plain joints (without shear keys) [54]

Depending on the magnitude and type of the shear actions (monotonic or cyclic), the following configuration of the vertical shear joints could be adopted:

1. Plain joints without shear keys are used for simple horizontal wall connections without a stability function and in the absence of seismic actions. Their shear resistance is neglected, namely, they cannot be used to form positive cantilevers. 'Staggered' connections can be used as well (see Section 5.3.7).

2. Indented vertical joints with horizontal tie reinforcement concentrated in the horizontal joints between panels are used for connections with moderate vertical shear loading.

3. Indented vertical joints with distributed horizontal loop reinforcement along the height of the connection (Fig. 7.16 and 7.18). They are used, for example, in vertical joints between the wall panels of stabilizing cores and, generally, for connections between wall panels in seismic areas. The slope α° of the shear keys will provide strength in the shear joints depending on the magnitude of the shear slip during earthquakes. However, we can conclude from Figure 7.17 that the ductility is almost the same and does not depend on the slope of the keys.



l_j = length of the vertical joint between floors
t = thickness of the precast panel
t_j = average thickness of the joint concrete
α^o = slope of the key (45^o to 60^o recommended values)
n = number of keys along l_j
a = thickness of the key inside the panel (≥5mm)
b = width of the joint, b ≥ t
h_o = length of the key inside the precast panel
h_1 = distance between keys
$\frac{n \cdot h_o}{l_j}$ = density of the keys (≤ 0.5)
Astr= cross-sectional area of transverse reinforcement (loops)
As_l = cross-sectional area of longitudinal reinforcement
l_b = required anchorage length of the loops
main reinforcement of the connection and of the precast panels
additional local reinforcement

Fig. 7.18: Typical reinforcement of a vertical shear connection of closed type and typical reinforcement of the precast panels (shown in one panel only)

- Primarily compression connections (horizontal joints)
    - The configuration of the horizontal connections is similar to that of the vertical connections (Fig. 7.18) especially for tall buildings and/or in areas of high seismicity.

      In all cases, a minimum amount of vertical tie reinforcement at horizontal joints between wall elements should be present, which can be placed either in the vertical joints or inside the elements (Fig. 7.28).

      If prestressed is used or when it can be shown by the analysis (under the design combination of actions) that the entire length of the horizontal joint is under compression (while also taking into account the unfavourable effect of the vertical component of the seismic action in case of earthquakes), the interfaces between precast and in-situ concrete (in the joint) may be without shear keys and without main reinforcement across the joint, since compression drastically increases the shear resistance of the joint and reduces its shear sliding.
    - In the absence of seismic actions and when it can be shown that the horizontal joint is under full compression, horizontal joints between precast walls in multi-storey buildings are usually filled with dry-pack mortar after the erection of the wall units. To get good bearing capacity, the thickness of the joint should be 30 to 50 millimetres. An alternative solution is to place the walls in a mortar bed poured between two foam strips prior to the placement of the walls. In this case, the joint thickness may be 10 to 20 millimetres.

Hollow-core floors can be supported on wall corbels (Fig. 7.19) or directly on the walls. Solution 7.19b is only acceptable for buildings of up to about 5 storeys because of the risk of unintended restraint due to the clamping of the floor ends by the wall load. Solution 7.19c with slanted ends is acceptable for higher buildings (see *fib* Recommendations [15] and [19].

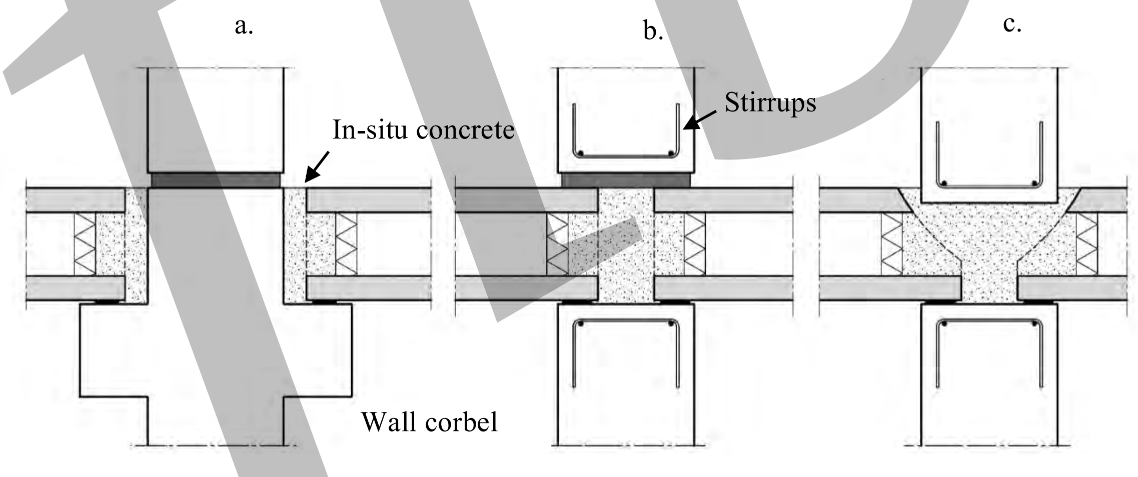


Fig. 7.19: Examples of wall-floor connections with hollow-core elements

An alternative method for horizontal jointing is the 'tongue and groove' joint. The groove forms a well for the grout and the tongue displaces the grout to make the bed. This detail is particularly useful when there are no floors on either side of the wall, for example, in stair cores or lift shafts. This detail is also very useful when cores have to be erected in advance of floors for either programming or access reasons.

In multi-storey wall-frame buildings with hollow-core floor elements, the ends of the slabs, together with the jointing concrete or mortar, transfer the loads from the upper wall element to the lower one.

Tests carried out at the VTT laboratory in Finland [50] recommend the following form to be used for predicting the strength of the joint with straight floor ends for vertical load transfer.

$$N_{Rd} = 0.5 f_{cd} b_j L_j$$

where

f_{cd} is the design strength of the wall concrete or grout, whichever is lower
L_j the length of the joint
$b_j = min\{ b_w, b_{grout}\}$, the smallest width of the joint in the transverse direction (Fig. 7.19b)

The resistance of joints with slanted floor ends (Figure 7.19c) can be calculated using:

$$N_{Rd} = 0.6 f_{cd} b_j L_j$$

The use of these two equations assumes that the upper and lower parts of the wall elements are provided with a horizontal rebar diameter of 16 millimetres in each corner next to the joint and with stirrups 8 millimetres in diameter spaced at a maximum of 200 millimetres.

Another more detailed method for the calculation of horizontal joints between bearing walls and hollow core floors is given in the PCI Manual for the design of Hollow core slabs [24].

In cases where the horizontal connections are partly under compression and partly under tension (under the design combination of loadings), the total tensile force (corresponding to the tension zone of the joint) should be covered by continuous vertical tie reinforcement through the upper and lower panels.

There are numerous ways by which the connections between vertical ties may be achieved. The requirements are mechanical continuity (not produced by mere overlap), interaction with the horizontal ties crossing and/or surrounding the floor (for realizing 'nodes') and ductility.

7.4.3.4 Verification of primarily shear connections with simultaneous action of compression or tension.

Generally, the shear resistance of shear joints between walls (Fig. 7.18) may be described as follows:

$$V_{Rdj} = A + B + C$$

where A, B and C denote the contribution to shear due to :

- the roughness of the interface (A)
- the minimum external normal compression force (if it exists) across the interface (B)
- the transverse reinforcement along the joint (C)

For example, in Eurocode 8 EN1998 – *Design of structures for earthquake resistance* the following evaluation for A, B and C (for reinforcement perpendicular to the length of the joint) is given by:

$A = c \cdot f_{ctd}$	where *c* is a factor depending on the roughness of the interface
$B = \mu \cdot \sigma_n$	where *μ* is factor depending on the roughness of the interface and σ_n is the stress per unit area caused by minimum external permanent force (if it exists), acting simultaneously with the shear force, positive for compression ($\sigma_n < 0.6\ f_{cd}$) and negative for tension. When σ_n is tensile, then *A* should be taken as 0
$C = \rho \cdot f_{sy}$	where *ρ* is the percentage of the reinforcement across the joint

Thus, according to Eurocode 8, the shear resistance of the joints may be verified as follows for monotonic loading:

$$V_{Rdj} = c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{sy} \leq 0.5 \left[0.6\,(1 - \frac{f_{ck}}{250})\right] \cdot f_{cd}$$

The factors c, and μ, range as follows:

c_{mon} = 0.25 to 0.5

μ = 0,50 to 09

Both depend on the roughness of the interface (very smooth, smooth, rough and indented).

In the case of seismic actions, the following should be regarded in relation to the above formula for V_{Rdj} : $c_{cycl} = 0.5\ c_{mon}$, and σ_n, should be calculated taking into account the minimum value of permanent compression acting along the joint, but in this minimum value for the compression stress, the unfavourable action of the vertical component of the seismic loading has also to be considered.

Alternatively, shear between wall panels may also be transferred by welding steel plates to cast-in anchor plates in the wall units (Fig. 5.18; see also Section 5.3.9), or by the use of staggered joints (see Section 5.3.7).

7.4.3.5 Basic concepts for possible mechanisms for energy dissipation in seismic areas

In Figures 7.20, 7.21 and 7.22 examples are given of the possible types of behaviour of large panel systems that contribute to ductility.

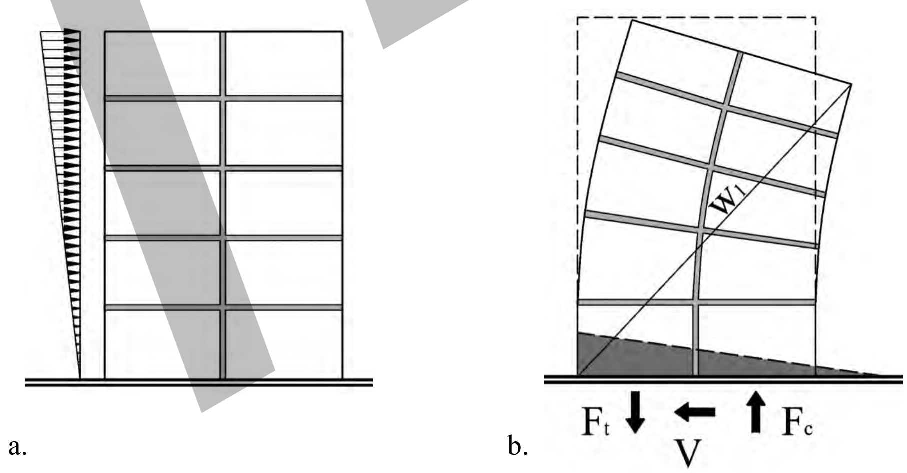


Fig. 7.20: a. Large panel wall; b. Ductile behaviour on the base of the wall

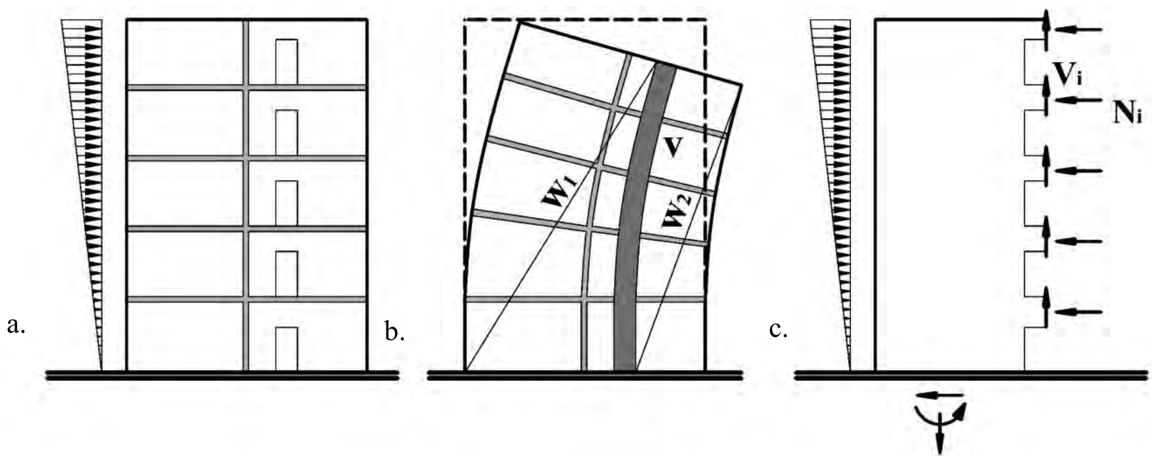


Fig. 7.21: a. Large wall panel with openings; b. Ductile behaviour along the lintels; c. Mid-span actions in the lintels

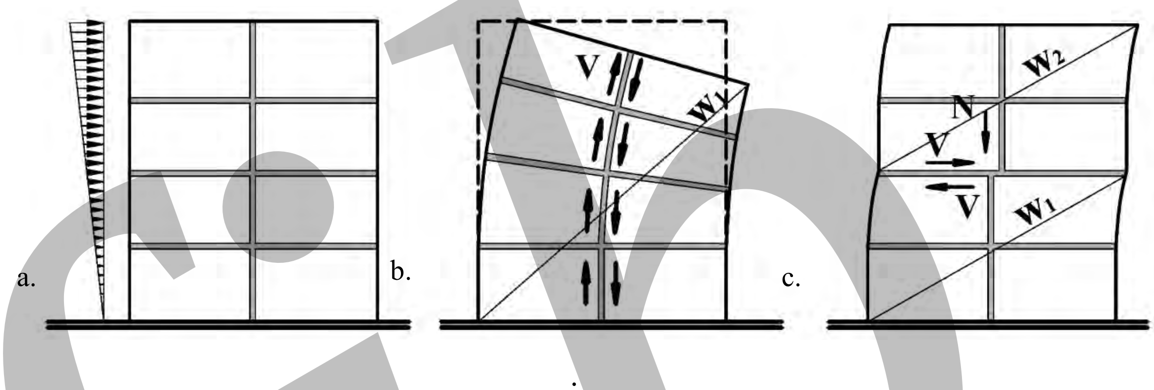


Fig. 7.22: a. Large panel wall; b. Ductile behaviour along the vertical joints; c. Sliding along the horizontal joints.

In the case of Figure 7.20**,** panels and their connections are designed so as to behave monolithically (W_1 in Fig. 7.20b) along their height. This assumption leads to overdesigned vertical and horizontal connections to ensure the monolithic behaviour of the total wall (panels and their connections).

In this respect, the ductile behaviour of the total wall may be assumed at its base in terms of rotation (development of a plastic hinge), as in equivalent monolithic walls. This case is mostly suitable for the use of plain individual precast panels (without openings).

Figure 7.21 refers to a cantilever wall composed of wall panels with one row of door openings. In such cases, the primary energy dissipation may be assumed to occur in the lintels (between the openings, along their height), which constitute the weakest part of the wall. Thus the total wall may be treated as a two-band cantilever wall linked by the lintels (W_1, W_2 in Fig. 7.22b). The same assumption may be made in the case of cantilevers with two or more rows of openings, which may be treated as multiple bands.

In the case of Figure 7.22, the primary energy dissipation mechanism may be assumed to happen along the vertical shear connections**.** Possible sliding along the horizontal connections should be avoided since this may endanger the overall stability. Figure 7.21 shows examples of vertical wall-to-wall connections for low tensile forces. Thus, the principle of weak vertical connections and strong horizontal connections should be followed, and the unfavourable action of the vertical component of the seismic load also taken into account.

It should be noted that in all cases (especially Fig. 7.21 and 7.22), a combination of different energy dissipation mechanisms may be assumed.

7.5 Elements

7.5.1 Solid interior walls

The thickness of massive wall units depends on the requirements of strength and sound insulation. The elements are usually storey height, with a maximum of approximately 4.20 metres and, exceptionally, 4.50 metres. This value is governed by transport conditions. The length of the panels mostly ranges between 2.40 and 14.00 metres.

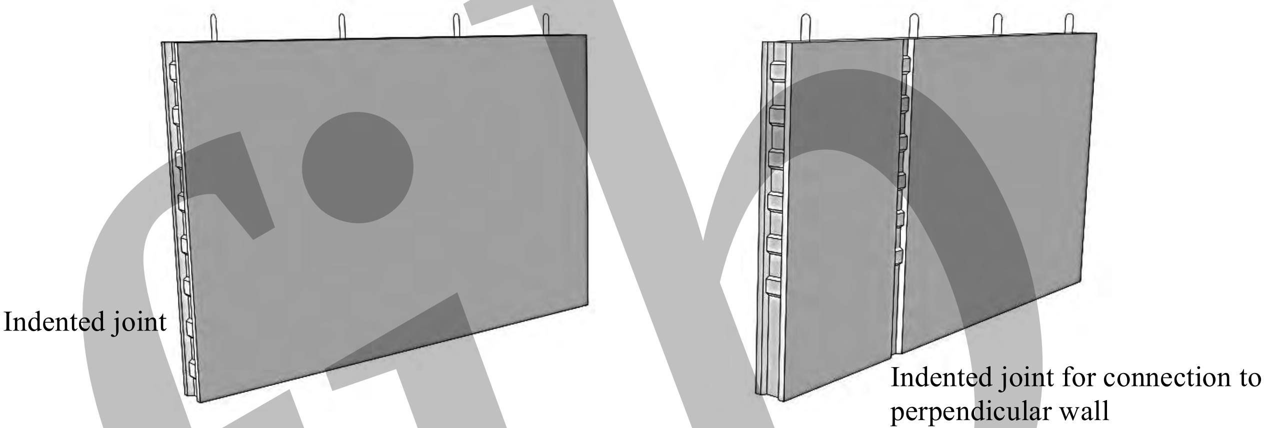


Fig.7.23: Precast wall elements

Precast walls are manufactured on long tables or in battery moulds. Service ducts and inserts for electricity may be incorporated prior to casting. The dimensions for door openings and windows are generally free, although some precasters prefer standard sizes. For reasons of stability at demoulding and handling, the minimum dimensions of lintels and mullions between windows and at panel edges are required. Figure 7.24 gives an example of recommended dimensions.

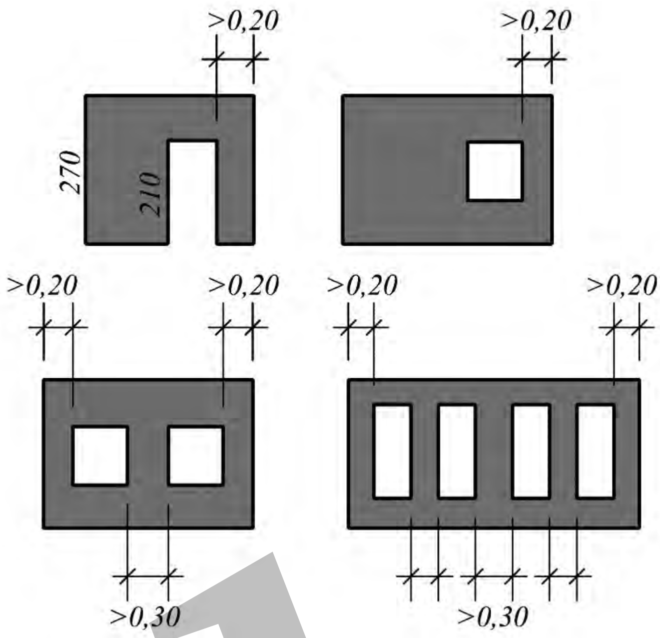


Fig. 7.24: Minimum dimension for lintels and mullions

7.5.2 Twin walls

Twin walls comprise two individual planks assembled during manufacture and connected by means of their projecting triangular lattice reinforcement. Twin walls have a smooth as-cast finish on both sides. After the erection of the walls, the voids between the planks are filled with cast-in-situ concrete. Connecting bars between adjacent walls and between walls and floors are anchored in the cast-in-situ infill concrete.

Fig. 7.25: Application example of twin wall system for a central core

Precast planks lend themselves to automatic manufacture, where robots may be used to measure out, select and assemble the reinforcement. The concreting and lifting operations are carried out using minimal labour.

7.5.3 Retaining walls in underground structures

Precast wall are frequently used as a ground-retaining structure in the basements of office buildings, car parks and similar structures. The horizontal soil and water pressure is taken up by the floor structure. This pressure can be considerable and the walls have to be designed for this action in addition to the vertical loads from the imposed dead weight and live loads.

Fig. 7.26: Application example of two-storey height precast retaining walls

7.5.4 Foundation walls

Precast wall units are frequently used in cold climates for the foundation walls of small buildings. The units are supported on simple cast-in-situ foundation blocks. The solution enables fast construction even during cold weather.

Fig. 7.27: Foundation walls for single-family housing

7.6 Examples of typical connections

Wall connections are classified with respect to location, direction and function, for example, interior or peripheral, horizontal or vertical, and wall-to-wall or wall-to-floor.

7.6.1 Wall-to-wall

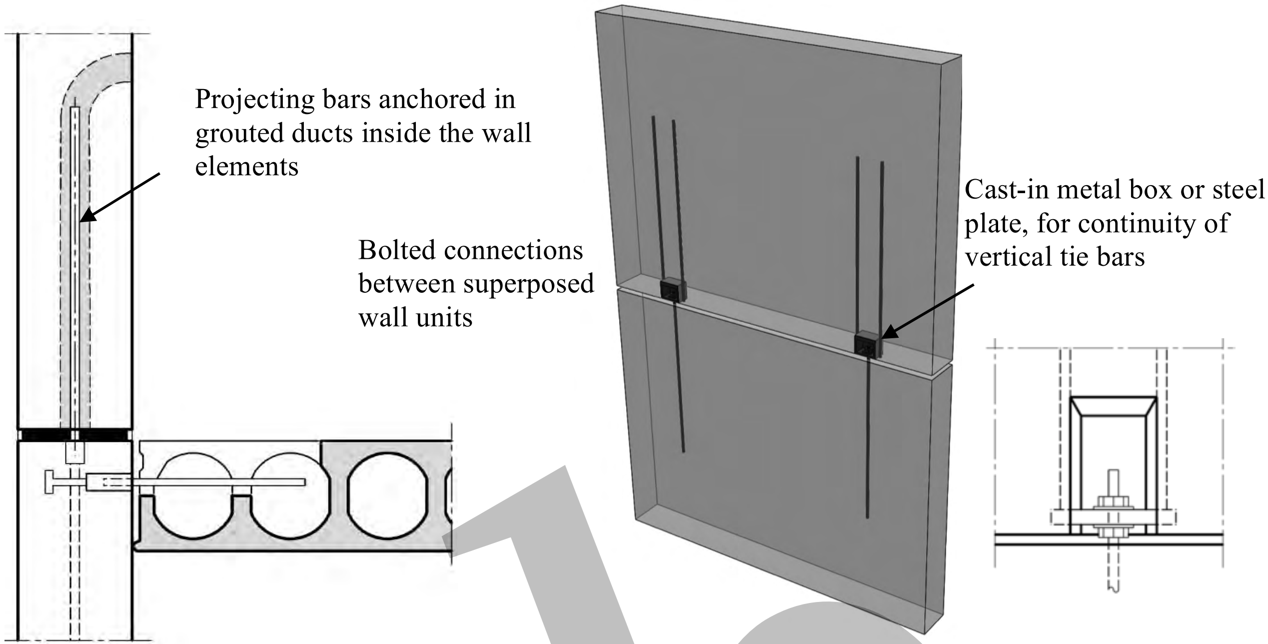


Fig 7.28: Examples of vertical wall-to-wall connections for low tensile forces

7.6.2 Wall-to-floor

Connections between precast walls and floors are among the most studied items in precast construction. The main objective of these tests was to check the structural behaviour of all sorts of connections with differing joint configurations, reinforcement, concrete filling, and so forth. This information is of great importance in the detailed design of wall connections. Extensive literature is available on this subject.

- Floors supported on wall corbels

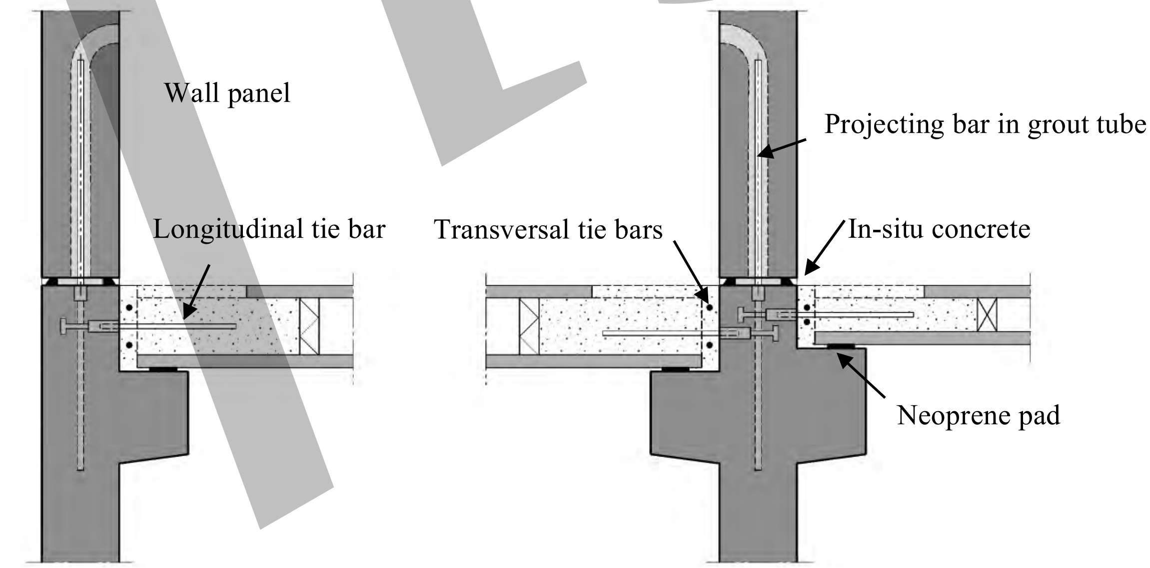


Fig. 7.29: Typical examples of wall connections with hollow-core floor

- Floors supported directly on the wall

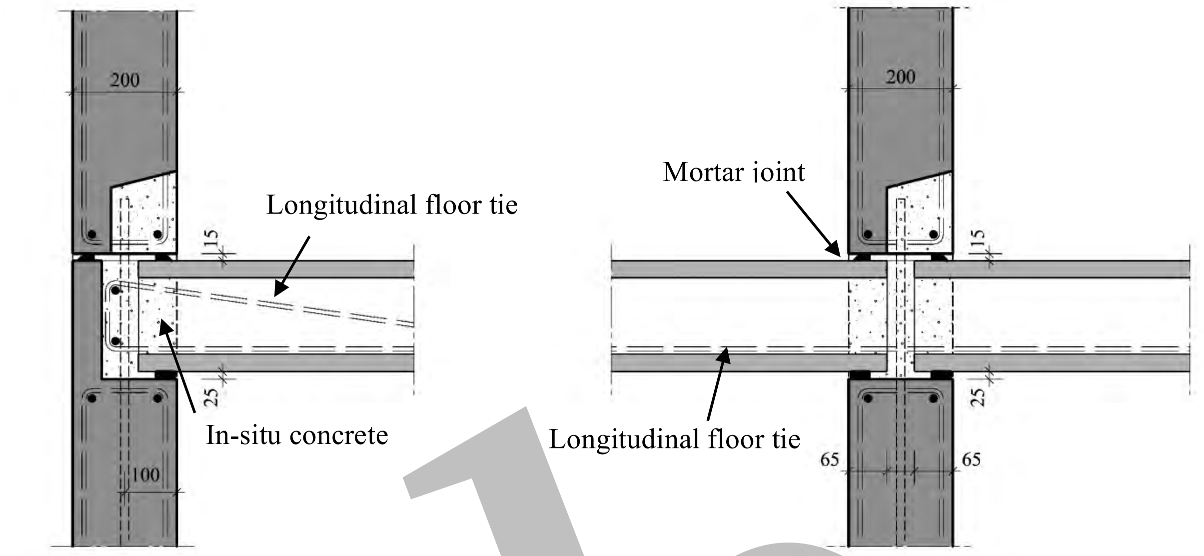


Fig. 7.30: Wall-to-floor connections at intermediate load-bearing walls and hollow-core floor elements for building up to maximum 10 storeys.

For buildings higher than 5 storeys, designing hollow-core floor units with slanted ends is recommended (see Section 8.8.4).

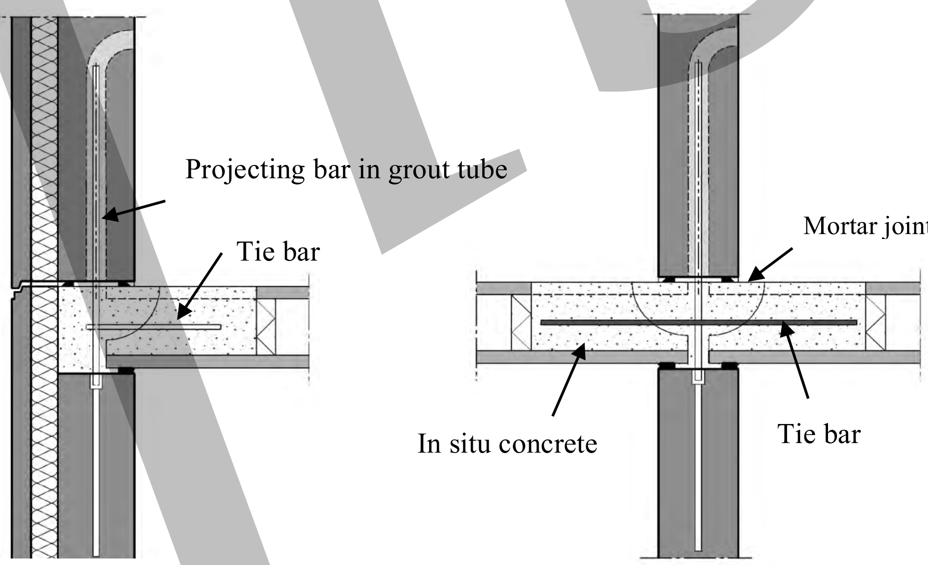


Fig. 7.31: Typical connection between wall and floor for buildings higher than 10 storeys

7.6.3 Wall-to-stair

Figures 7.32 and 7.33 show examples of the arrangements of stair connections with separate flights and landings in a precast shaft. The solution in Figure 7.32 is with corbel supports, while the solution in Figure 7.33 is with hidden connections.

Fig. 7.32: Application example of stair and flight connections with corbels and beams

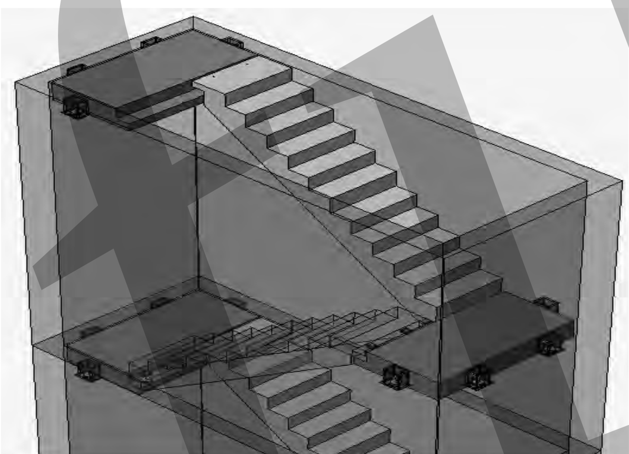


Fig. 7.33: Schematic view of hidden connections for stair landings in lift shaft

7.6.4 Wall-to-beam

In some building projects, skeletal systems are used for the basement and the ground floor and wall-frame structures for the higher levels. Figure 7.34 gives an example of a connection between wall panels and supporting beams. It is important that the panel load is transferred over the whole length of the beam and not only at the ends of the wall panel because of the deflection of the beam. For longer spans, it might be necessary to support the beam on, for example, three supports instead of two.

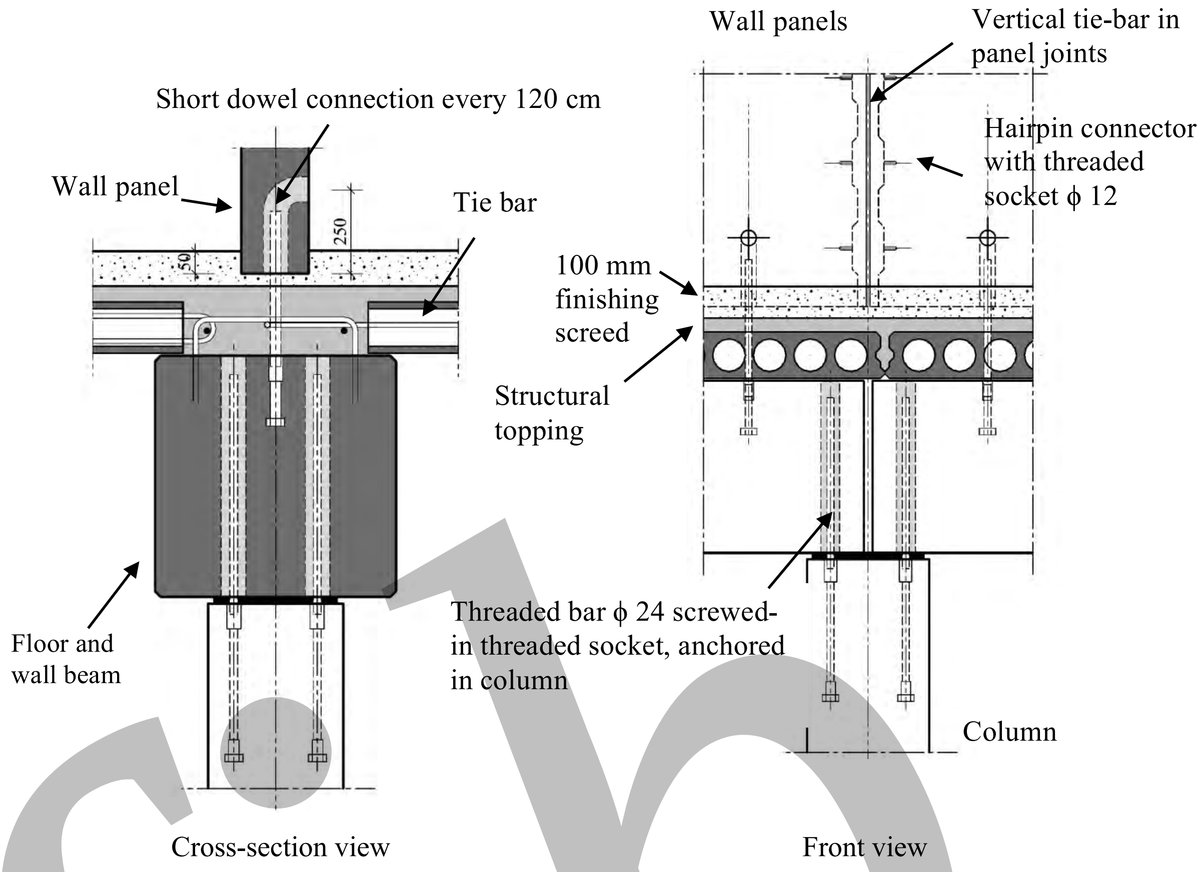


Fig. 7.34: Application example of a connection between a wall panel and a supporting beam

8 Floor and roof structures

8.1 General

Precast floors offer many advantages over cast-in-situ floors, for example, the absence of scaffolding, short construction time, smooth underside, high mechanical performances, durability, long spans, and many others. The market offers a large variety of precast floor systems, of which the five main types are:

a. Hollow-core floors (prestressed a_1 and reinforced a_2)
b. Prestressed ribbed floors
c. Light ribbed roof elements
d. Solid slab floors
e. Composite floor-plate floors
f. Beam and block floors

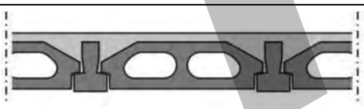
Ref.	Floor and roof type	Max. span (m)	Floor thickness (mm)	Most common unit width (mm)	Unit weight (kN/m²)
Fully precast floors					
a_1		± 20	120 - 500	600 - 1200 - 2400	2.2 - 5.2
a_2		± 7,5	130 - 250	300 - 600	1.9 – 2.9
b		24 –30	200 - 800	2400	2.0 - 5.0
c		12	175 - 355	2400	1.2 - 1.8
d		6	100 - 250	300 - 600	0.7 - 3.0
Semi-precast floors					
e		7 - 10	100 - 400	600 - 2400	2.4 - 4.8
f		6	200 - 220	515 - 635	1.7 - 2.3

Table 8.1: Indication of the size and weight of the main types of precast floors

The main structural function of a floor is to provide the required load bearing capacity. Depending on the use, floors also have to provide stiffness, the transverse load distribution of concentrated loads, the distribution of horizontal actions by diaphragm action, fire resistance or even thermal and acoustic insulation.

8.2 Main types of precast floors

8.2.1 Introduction

Precast floors can also be classified according to their manufacture into fully precast and semi-precast structures. Fully precast floors are composed of units that are entirely cast at the plant. After erection, the units are connected to the structure and the longitudinal joints are grouted. In some cases, a cast-in-situ structural topping is added. Semi-precast floors are composed of a precast part and a cast-in-situ part. Both parts work together at the final stage to achieve composite structural capacity.

8.2.2 Fully precast floors

8.2.2.1 Hollow-core slabs

Prestressed hollow-core units have longitudinal voids or cores of which the main purpose is to optimize the use of raw materials and to reduce the weight of the floor. They are mainly used in buildings with large spans, such as office buildings, hospitals, schools, shopping centres and industrial buildings. Another frequent application is for apartment buildings and houses because of the favourable cost rate and fast erection. They may be either in reinforced or prestressed concrete.

The elements are available in different depths to satisfy the various performance requirements for span and loading. Typical cross sections are shown in Figure 8.1. The percentage void (volume of voids to total volume of solid slab of equal depth) for hollow-core slabs varies from 30 to 50 per cent.

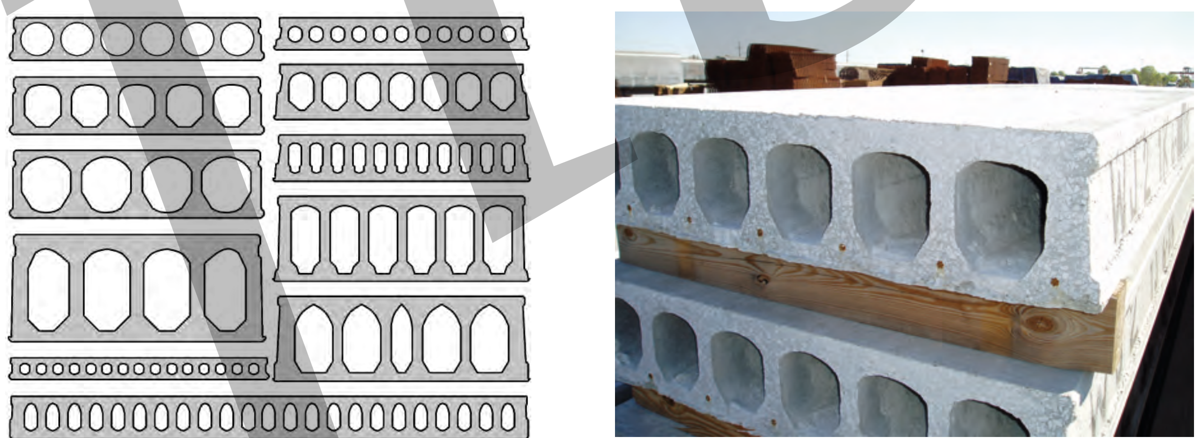


Fig. 8.1: Typical cross-sections of prestressed hollow-core elements

Prestressed hollow-core units are normally 1,200 millimetres wide and up to 20 metres long. The actual unit width is usually 3 to 6 millimetres shorter than the nominal size to allow for constructional tolerances. It prevents the overrunning of the floor layout due to cumulative over-width units. The edges of the units are profiled or keyed to ensure adequate vertical shear transfer across the grouted joint between adjacent units.

Prestressed hollow-core units are manufactured using either long-line extrusion or slip-form processes. The steel or concrete beds are usually 1,200 millimetres wide and 80 to 180 metres long. The degree of prestressing, strand pattern and depth of units are the main design parameters.

After hardening, the elements are cut to the specified length using a large, circular, concrete saw. A rectangular end is standard but skew or cranked ends, which are necessary in a non-rectangular floor plan, may be specified. In order to increase the thermal insulation properties of the hollow core, the elements can also be cast directly against an insulating layer (for example, expanded polystyrene).

Reinforced hollow-core elements are usually 300 to 600 millimetres wide. In some countries they are widely used in domestic buildings. These elements are cast in moulds with specified length. Directly after compaction by means of vibration, the elements are demoulded and stored on pallets in a climate room until they have the required strength for lifting and storage.

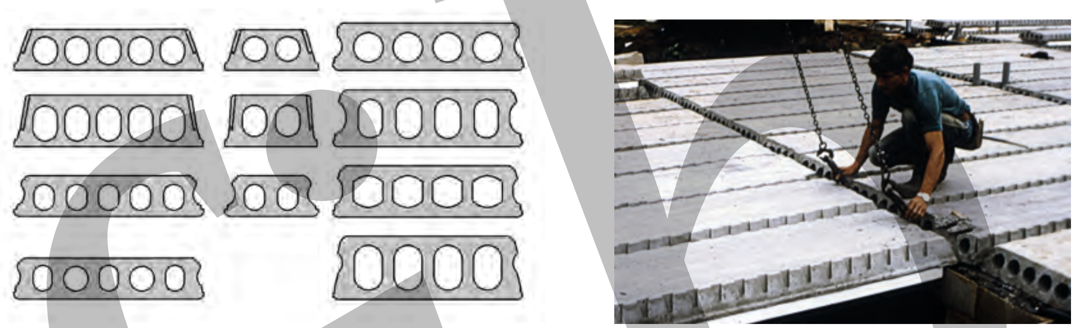


Fig. 8.2: Typical cross-sections of reinforced hollow-core elements

8.2.2.2 Ribbed floors

There are three different types of ribbed precast floor elements on the market:

- Double-tee elements
- Single-tee elements
- U-shaped elements

A typical cross-section of double-tee units is shown in Figure 8.3. Normally, these components are prestressed. The main advantages in using this type of floor units include:

- Load-bearing capacity in combination with large spans
- The ends of the units can be notched to one third of the overall depth to form a halving joint to reduce the overall structural depth
- Double-tee units are manufactured as standard up to 2,400 millimetres wide (2,390 millimetres, to be exact) or 3,000 millimetres wide, thus reducing the number of units to be placed on site

The depth of double-tee units ranges from 150 to 800 millimetres, allowing spans of up to 22 metres. These units have excellent load-bearing capacity, offering a long-span unit able to carry relatively high loads. Where units with a shallow flange depth (40/50 millimetres) are used, an in-situ reinforced concrete structural topping is normally required to ensure both vertical shear transfer between adjacent units and horizontal diaphragm action in the floor.

Fig.8.3: Ribbed floor units

Single-tee elements offer the longest span solution of all ribbed floors but at the expense of greater depth. The slab elements may be up to 3.0 metres across and the shear capacity of the wide ribs can allow higher loads than are possible with double-tee units.

U-shaped slabs can be either upright or inverted. Inverted U-shaped units are generally manufactured for light loading over short to medium spans, typically for roof applications. They are typically produced in 600-millimetre-wide elements that give a rather low self weight.

Upright U elements are ribbed elements with a flat and smooth underside. The overall depth varies between 250 and 650 millimetres, with achievable spans of up to 22 metres. For floors, the U elements are covered with a shuttering slab unit and an in-situ topping cast over the whole surface. They constitute a variant solution to box girders. The overall depth typically lies between 500 and 700 millimetres, with a maximum span of 18 metres.

8.2.2.3 Ribbed floors

Massive slab units can be produced using normal-weight concrete. However they are often made in lightweight or in cellular concrete, to reduce self weight and improve the thermal properties. They are mainly used in housing and sometimes for roofs of industrial and commercial buildings. The main reason for using such slabs is either to improve acoustic insulation or for hygrothermal reasons. Both reinforced and prestressed massive slabs are used. This slab type can also be used for two-way spanning floors, which may assist in reducing the depth of the slab.

8.2.3 Semi-precast floors

8.2.3.1 Composite floor-plate floors

This typical semi-prefabricated floor system consists of precast solid or ribbed-floor planks that are used as permanent formwork for an in-situ concrete topping to achieve a solid composite floor. The precast plank units are typically 0.6 to 2.4 metres wide and 40 to 120 millimetres thick. The total depth of the slabs ranges from 150 to 400 metres. Floor planks are made either in prestressed or reinforced concrete, with lengths produced to suit the floor span.

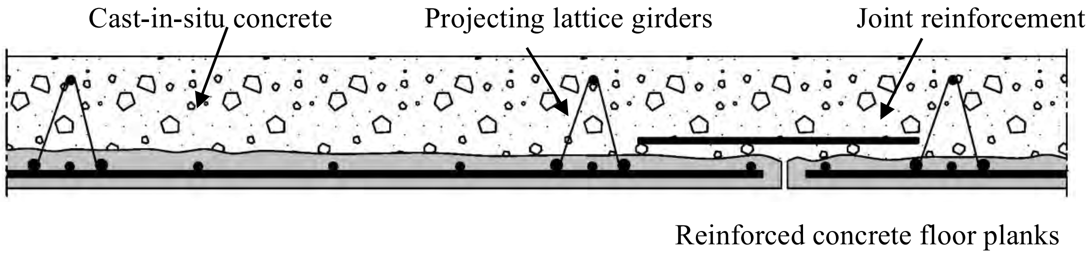


Fig. 8.4: Composite floor plank floor

Prestressed floor planks have a larger rigidity than their reinforced counterparts and usually do not need projecting lattice girders for handling and transport, as may be the case for reinforced planks. The soffit has a smooth finish.

To ensure good interaction between precast plates and in-situ concrete, the planks may be provided with protruding reinforcement or lattice girders. The floor-plates normally need propping during construction, at a spacing of 1.5 to 3.5 metres, depending on the upper flange of the girder.

The essential advantages of this system, compared to traditional cast-in-situ floors, are that, apart from temporary support during the construction phase, no moulds have to be used and the sagging reinforcement is already incorporated in the prefabricated plate. A steel mesh in the in-situ topping acts as hogging reinforcement. The floor slab may therefore be designed as continuous.

8.2.3.2 Beam and block floors

This semi-prefabricated floor system consists of precast load bearing beams, also termed joists, placed parallel to each other at a modular distance of approximately 600 millimetres. The beams are generally prestressed and produced on long beds similarly to hollow core elements. Other types of beams are also available (Fig 8.5). A special type of reinforced joist is the so-called lattice joist, which is made up of a concrete flange and a steel truss.

The opening between the beams is filled with blocks or infill steels. Usually, these infills have no structural function and act as formwork for an in-situ topping. In some cases, however, they contribute to the structural capacity of the whole floor. The blocks are made of concrete, polystyrene, woodchip concrete, clay and sometimes even plastic shells. A structural, reinforced topping is required to provide the structural integrity of the floor.

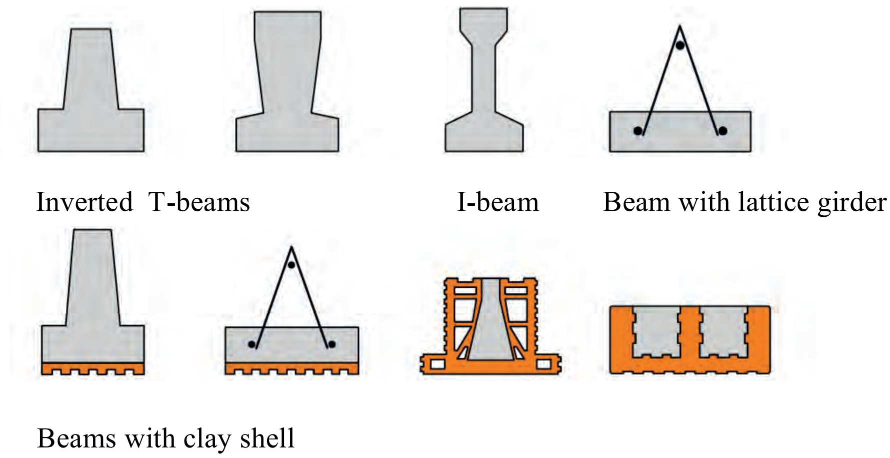


Fig. 8.5: Beams in beam-and-block floors

These floor types are mainly used above crawlspaces and basements. Because of the very low self weight of the components (each element can be carried by one or two people), these floors are very appealing for renovation projects where wooden floors are replaced by beam-and-block floors.

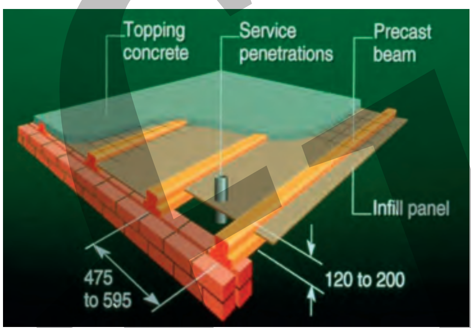


Fig. 8.6: Beam-and-block floors

8.3 Concrete roof elements

Concrete roof elements are mainly used for industrial and commercial buildings, and sport halls, among others. There are different types of elements, such as slender ribbed units, embossed slabs, Y or V-shaped slabs, vaulted-roof slabs, single or double-wing elements, trough units, and so forth, but also hollow-core slabs. The main features of the elements are:

- Reduced self weight due to slender cross-sections
- Large spans
- Smooth, flat or embossed soffit

Typical cross sections of concrete roof elements are shown in Table 3.1 and Figures 3.8, 3.9, 6.31, 6.32 and 8.7. Normally, these components are prestressed.

Fig. 8.7: Ribbed roof elements for an industrial hall

Prestressed ribbed-roof units are available in widths of 1,200 to 2,400 millimetres and depths of 175 to 355 millimetres, for spans ranging from 6 to 12 metres. The number of ribs varies from two to three and sometimes four. The self weight of the units is in the order of 125 to 175 kg/m².

Fig. 8.8: Typical ribbed light roof units

Special roof elements are normally used for spans ranging from 12 to 30 metres. There are many types and shapes of roof elements available, for example, wing elements of different depths, shed elements, and so forth. In some cases, design will be based on testing.

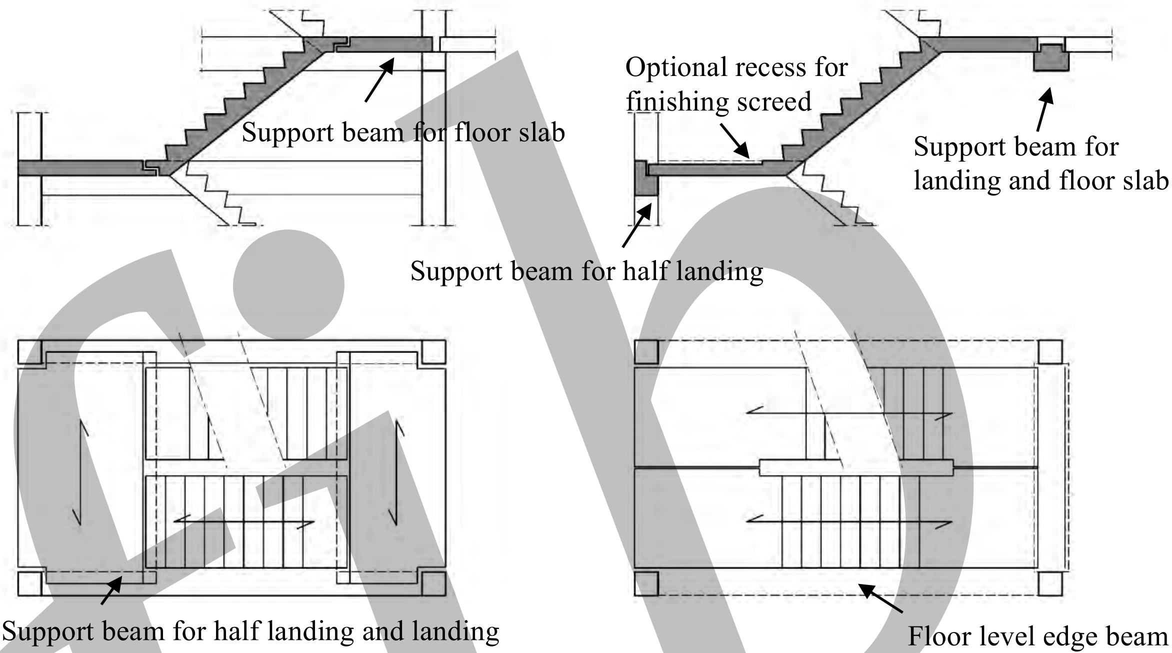
Illustrations of typical cross sections of precast roof units that are often used in Italy are given in Figure 3.8. The typical width of the unit is 2.40 to 2.50 metres with a depth of 800 to 1,100 millimetres. The span ranges from 12 to 30 metres.

In most cases, the roof is designed for all the required functions, such as load-bearing elements, rainwater evacuation, skylights and ventilation.

8.4 Stairs

Precast concrete staircases are very attractive products because of their improved finishing and cost efficiency. Traditionally cast in-situ staircases are very labour intensive, with additional finishing material always needed and the effective total cost often underestimated. Precast concrete stair units are industrialized products with a high degree of finishing, ranging from smooth as-cast to polished concrete. The most common staircases are described in this section.

The first category comprises straight stair units. They are made up of either individual precast flights and landings or combined flight and landing units. In the latter solution there may be differential levels at floors and half-landings, necessitating a finishing screed, levelling infill or other solution.



a. Separate landing, flight and half-landing units

b. Combined landing, flight and half-landing unit

Fig. 8.9: Alternative layouts for two-flight staircases

Figures 8.9b and 8.10 show examples of straight stairs with combined landings. The advantage of the solution lies in the speed of erection, fewer connections and better stability against accidental actions.

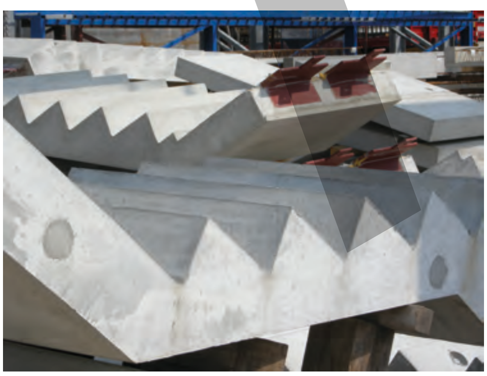


Fig. 8.10: Examples of precast stairs with combined landings

The second category comprises monoblock staircases. They can be used either in stairwells or individually between the different storeys.

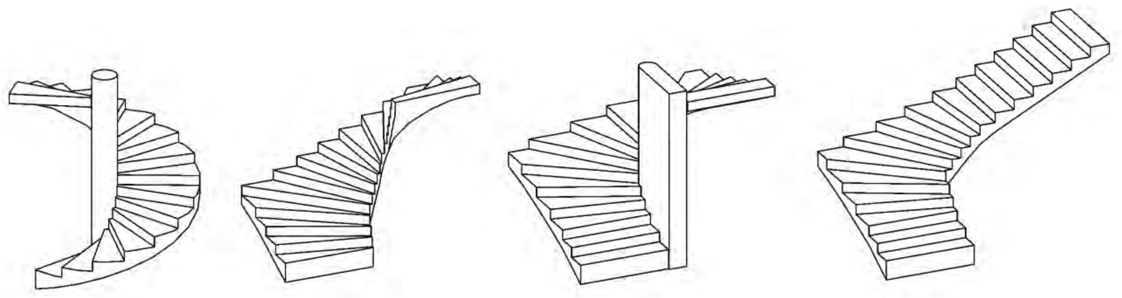


Fig.8.11: Types of monoblock spiral and self-supporting stair units

8.5 Modulation

Precast concrete floors are extremely versatile and can accommodate almost any arrangement of support structure. There are, however, certain guidelines for the proportioning of a building in plan that can be usefully employed to simplify the construction. Totally precast floor units are generally modulated on a 300-millimetre base. The most common dimensions are 600, 1,200 and 2,400 millimetres. Elements for composite floor systems are sometimes made to specific dimensions. When planning a building for design with precast-concrete-floor elements, it is advisable to modulate dimensions to suit the commonly available element widths.

Composite beam-and-block floors are less sensitive to modulation. The required coverage can be achieved by varying the beam spacing, either by using beams in pairs or special infill blocks. Where beams are positioned at reduced centres it may not be possible to use flush soffit blocks unless special sizes are available. With careful modulation, these situations can be minimized, restricted to the edge of the building or avoided completely.

In a simple structure all the floor elements should preferably span in the same direction, simplifying the layout and, in the case of prestressed elements, limiting the number of camber clashes within a bay. Where exact modulation is not possible it may be necessary to produce a special unit, cast to a smaller width or cut to the desired width from a standard module. Narrow strips of in-situ concrete may also be used, designed to span in a direction transverse to the precast floor span. In many instances this in-situ strip can be usefully incorporated into the connection and tying system (Fig. 8.26).

At floor supports, the precast floor elements may conflict with beam/column intersections. To counter this it is possible to detail the beams to be wider than the columns, allowing the floor units to remain plain-ended (Fig. 6.41a). In this case, the floor modulation becomes independent of the column spacing and is, thus, simplified. When beams are not wider than the column width it may be necessary to form notches in the floor units (Fig. 6.41b). It is preferable that the longitudinal floor joints coincide with column positions to facilitate the notching of the units and this should be considered when detailing the building layout. A number of variations may be required to suit the particular layout of individual buildings but the economics of the floor construction is maximized where plain ends can be used.

The use of prestressed tendons in precast components may not be effective in very short floor units. Therefore, precast units used on short spans (of less than 2 metres) may need to be designed as normally reinforced-concrete sections. At the apex of a tapered floor area it may prove difficult to accurately produce skew-cut ends that maintain the correct floor coverage; these areas are often detailed as in-situ concrete when the span falls below 2 metres.

Changes in floor level across a building may be accommodated by L-beams, or by building up one side of an inverted T-beam ledge. Where the difference in floor levels exceeds the thickness of the floor beam, a solution is to use two L-beams above each other. This is often used in, for example, split-level car parks.

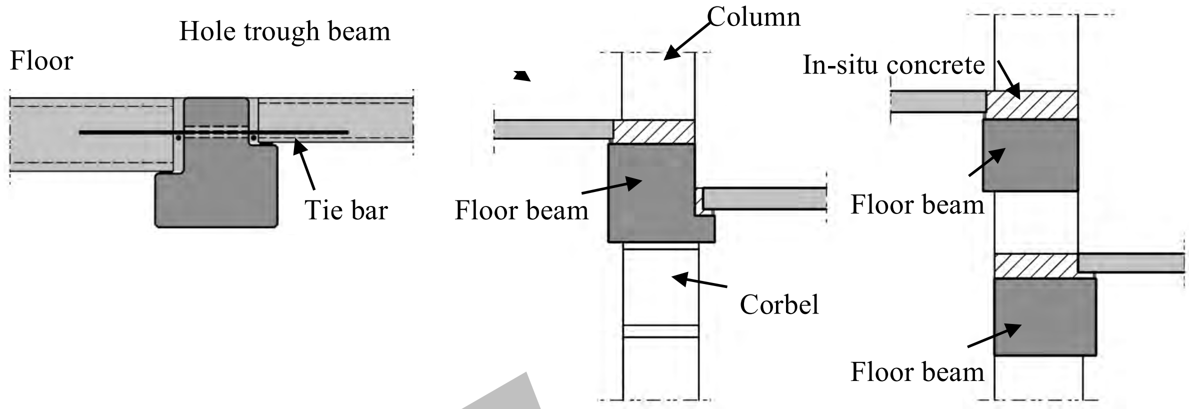


Fig. 8.12: Solutions for changes in floor level and floor-support level

Floor-slab modulation is a useful approach to precast concrete design but is not necessarily a requirement. Other factors may dictate the spacing of supports without compromising the use of a precast flooring solution. Awkwardly shaped sites are common for inner-city redevelopment projects but despite the necessity of constantly changing span length and direction, precast flooring is, typically, still a viable option.

8.6 Design of precast floor elements

8.6.1 General

The design and calculation of a precast floor and roof is carried out in two steps, namely, the design of:

- The individual slab units
- The whole floor

The individual slab units are dimensioned with respect to flexural capacity and shear resistance, and may or may not be in combination with torsion when applicable. The punching-shear resistance for high-concentrated loads must also be checked. Finally, the deflection is calculated and limited to recommended values. Other design criteria include fire resistance, acoustic and thermal performance, durability, handling and construction methods.

This section gives information on specific design rules for precast floor components, as far as they are not covered by the classical design procedures for reinforced and prestressed-concrete members. More detailed information on performance data is available in the production guides and technical literature of the manufacturers.

Precast floor and roof elements are most commonly produced in prestressed concrete. The units are designed in accordance with national or international standards and other selected literature published, for example, by the *fib* Commission on Prefabrication, [9] and [15], and the

PCI [24]. The non-composite floor and roof slabs are normally calculated as single spanning units, although partial continuity may sometimes be incorporated.

In general, precasters provide standardized load/span tables giving the allowable variable load in relation to the span length and the reinforcement. The curves are calculated according to the requirements for bending and shear, and deflection. Where relevant, information is also given concerning the expected pre-camber, or hog, of the elements due to prestressing and self weight.

8.6.2 Prestressed hollow-core slabs

Generally, prestressed hollow-core units have no reinforcement other than the longitudinal prestressing tendons anchored by bond. 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 load distribution, among others.

As in any prestressed concrete element, the design shear capacity is calculated for two conditions: the uncracked section near the support (shear tension) and the cracked section in flexure (flexural shear). The latter occurs when the shear force exceeds the shear compression capacity and a single flexural crack initiates the shear failure. In this area (zone 2), anchorage failure must be considered.

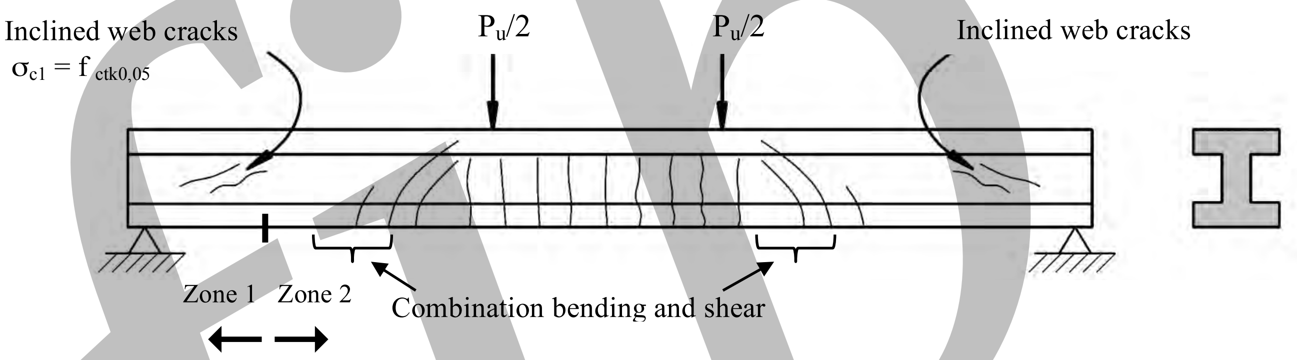


Fig. 8.13: Cracking zones and pattern in a reinforced or prestressed concrete member

The shear capacity of hollow-core slabs in the region that is, in the ultimate loading state, uncracked in flexure, can be calculated from:

$$V_{Rd,c} = \varphi \frac{I b_w}{S} \sqrt{(f_{ctd})^2 + \beta \alpha_l \sigma_{cp} f_{ctd}}$$

where

- I is the second moment of area;
- S is the first moment of area above and about the centroidal axis
- b_w is the width of the cross-section at the centroidal axis
- α_ℓ $= l_x / l_{pt2}$
  is the degree of prestressing transmission ($\alpha_\ell \leq 1{,}0$)

l_x is the distance of the considered section from the starting point of the transmission length

l_{pt2} upper value of transmission length (see EN 1992-1-1:2004 [2], Expression 8.18)

σ_{cp} $= N_{Ed}/A$
is the full concrete compressive stress at the centroidal axis

f_{ctd} $= f_{ctk0,05}/\gamma_c$
is the design value of the concrete tensile strength

φ = 0.8 reducing factor

β = 0.9 reducing factor referred to transmission length.

The shear capacity of the member in the region that is, in the ultimate load state, cracked in flexure, can be calculated with the classical formulas (see also [15]).

8.6.3 Ribbed-floor elements

Ribbed elements are prestressed longitudinally both for capacity in flexure and in shear, and to control deflections. When required, shear reinforcement is also placed in the webs in anchorage zones. The flanges are reinforced using welded fabric to control shrinkage cracking and to ensure the horizontal distribution of loading to the webs. The units are designed according to the classical rules for prestressed concrete, for example, in Eurocode 2 Part 1-1[2].

8.6.4 Floor-plate floors

Precast floor plates may be either prestressed or reinforced. They can also be designed with concrete or lattice ribs or with a mix of both. Where used, lattice girders are manufactured from high tensile steel bars to stiffen the elements during transport and erection. The longitudinal bars in the lattices may be ignored in design for the serviceability condition but they may be included in the design for the ultimate limit state. Design for the serviceability limit state has to take into account real support conditions and temporary support props, and also real loads on the precast part and after pouring the in-situ screed. This is particularly important for the control of cracking.

8.6.5 Beam-and-block floors

If the blocks used are considered to be ‘non-structural’, the beam-and-block floor normally is completed with a reinforced-concrete topping. In this way, a composite floor structure is created. This topping can also be taken into account for load distribution and diaphragm action. The reinforced or prestressed beams are designed for bending and, in combination with the topping, for shear, by the producer. Guidelines about the design are available from the product manufacturers. See also the FIP Guide to good practice *Composite floor structures* [22]. In the serviceability-limit-state design the influence of temporary supports has to be considered.

8.7 Design of the complete floor

While the design of the whole precast floor structure concerns the analysis of bending moment and shear force capacities, the most important aspect is to achieve a stable and coherent structure out of individual elements. The most important objectives are:

- Structural integrity
- Diaphragm action of the floor or roof for the transmission of horizontal actions
- Transverse distribution of concentrated loads

8.7.1 Structural integrity

Floor systems consisting of individual precast concrete units should be connected together to form a tying system, either with or without a cast-in-situ structural topping over the whole floor surface. The design of the tying systems is extensively dealt with in Section 4.6.

8.7.2 Diaphragm action

Floors made of hollow-core units act very well in horizontal diaphragm because of their edge profile and the grouting of the longitudinal joints between adjacent units. It is advisable to limit the design value of the average horizontal shear stress in the longitudinal joints between hollow core units to 0.1 N/mm². The shear stress, calculated on the effective depth of the joint is seldom critical, so that hollow core floors normally provide sufficient diaphragm action without a structural topping, but the units must be restrained from moving apart.

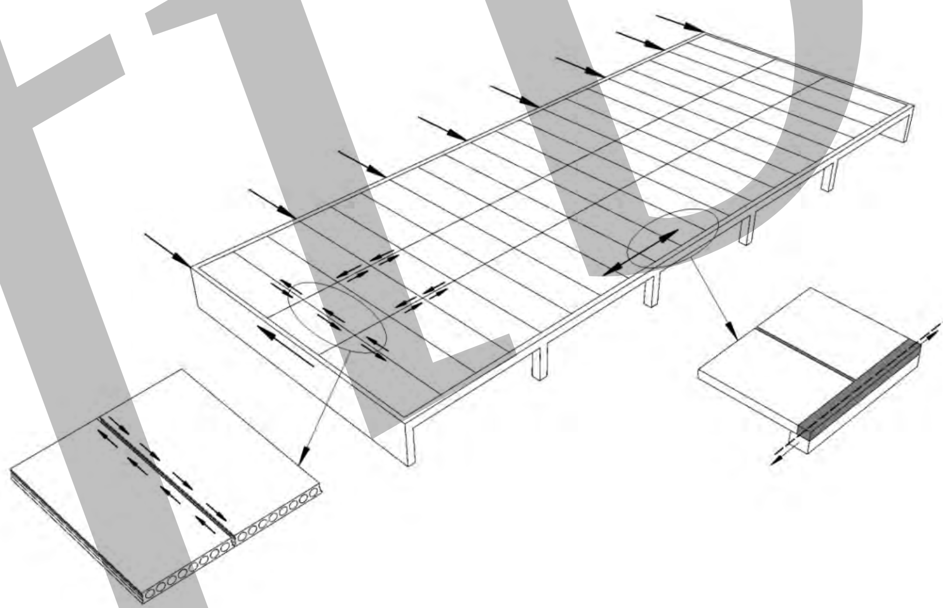


Fig. 8.14: Tie connections in a hollow-core floor to achieve diaphragm action

The peripheral tie reinforcement is designed 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 and design can be made with the help of strut-and-tie models.

In precast floors using TT-units without structural topping, shear transfer between the elements is secured by bars welded to plates fully anchored in the top flange of the unit.

The diaphragm action may also be achieved 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 (see Section 4.4).

The positioning and minimum proportioning of ties required by Eurocode 2 is given in Section 4.6.2 of this bulletin.

8.7.3 Transverse distribution of concentrated loads

8.7.3.1 General

Floors usually not only carry uniformly distributed loads but also concentrated line or point loads, for example, partition walls. When these loads are to be taken up by the underlying slab alone, it would lead to larger dimensions for the considered unit and, consequently, for the entire floor than what would be strictly necessary for the uniform variable loading. In reality, however, the concentrated load is spread over a number of neighbouring elements.

When a simply supported slab field is loaded with uniformly distributed loads, each slab unit deflects similarly. The situation is different when a slab field is subjected to concentrated loads, such as line loads or point loads. The slab unit on which a concentrated load is applied deflects. Since the slab units are connected by grouted joints and transverse ties, members 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 just the directly loaded slab unit.

Much research has been carried out on the subject of transverse load distribution on hollow-core and ribbed floors. The results show that the concentrated load is distributed over several adjacent units, almost as in a monolithic floor. The *fib* Commission on Prefabrication has worked out an analytical calculation model for hollow-core slab floors - [15] and [19]. 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 only transmit shear forces but no bending moments.

The magnitude of the vertical shear forces in the joints depends on the torsion stiffness of the units, the longitudinal and transverse flexural stiffness of the elements and how well the lateral displacement of the slabs in relation to each other is restricted. Even in the case of cracked joints, shear forces will be transmitted across the cracks in the grout due to the presence of lateral compressive stresses originating from the torsion of the elements and the shear-friction mechanism. In any case, shear transfer capacity should be controlled by transverse tie reinforcement at the support, which gives the necessary forces perpendicular to the joint when the adjacent elements tend to separate. The required resistance of the transverse tie reinforcement shall be at least equal to the total vertical shear force, which has to be transmitted across the longitudinal joints.

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

- The longitudinal joints between the elements should be designed to take up vertical shear forces. During erection, a good quality concrete should be used to fill up the joints.
- The lateral displacement should be limited.

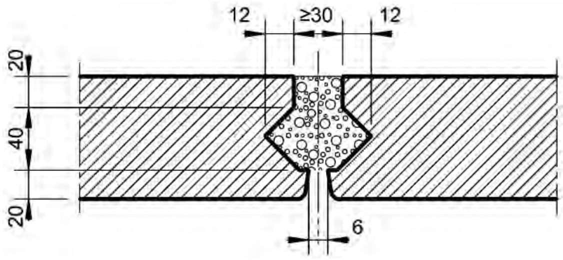


Fig. 8.15: Joint profile in ribbed floor

The load distribution is also achieved by use of a structural topping. The structural topping is commonly applied in ribbed-floor elements, because of the fact that the joint thickness is too small to transfer vertical shear forces by the joint mortar alone.

8.7.3.2 Practical design of load distribution

The determination of the possible load distribution can be done either by simple 'deemed-to-safety' rules or by more complicated analytical calculations. In many cases, the simplistic approach will suffice. The concentrated load is then assumed to be distributed over an effective width equal to up to five precast units or over a width equal to one quarter of the span on either side of the loaded area. When this distribution is not sufficient, analytical calculations or graphs may be used. They are, however, specific to each type of precast floor.

- Hollow-core floors

  When a floor is loaded with a uniformly distributed load, each unit deflects similarly and can be treated as a one-way slab. The case is different with line and point loads. The unit on which the concentrated load is applied deflects; adjacent slab units deflect also because the slabs are connected with grouted joints and the effect of a concentrated load is distributed to a wider area on the floor.

  The hollow-core units behave like plates connected with hinges after the joints are grouted. The transverse distribution of concentrated line and point loads on hollow-core floors is quite effective.

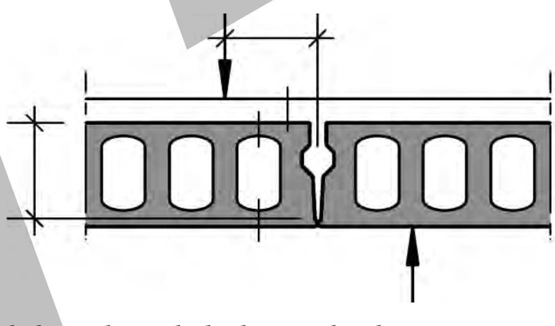


Fig.8.16: Transfer of vertical shear through the longitudinal joint

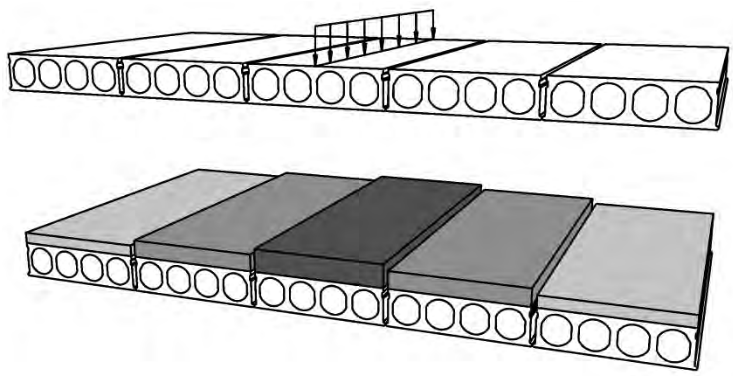


Fig. 8.17: Load distribution for linear load in middle of slab field

The transverse load distribution can be determined with the help of graphs (see [15] and [19]) when the following conditions are fulfilled:

- The longitudinal joints between the slabs are capable of taking up vertical shear forces (these vertical shear forces will cause torsion in the slab); the edges should have a longitudinal shear key to transfer the vertical loads. The joint should have a minimum width of 30 millimetres at the top and should be filled with a good quality grout or concrete.
- The lateral displacement of the slab units is limited.
- The diaphragm reinforcement is adequate for the in-plane bending moment of the floor.

Examples of graphs showing practical load distribution factors for 1.20-metre-wide hollow-core units are given in Figures 8.18 to 8.20. They are based on analytical calculations and tests.

The graphs are almost independent of the thickness of the units and the shape of the voids since the load distribution is mainly governed by the relation of the torsion and flexural rigidity of the units. The graphs should only be used for the determination of the distribution factors of bending moments for point loads less than 100 kN, and not for shear, since the load distribution at the region near to the support may be less effective.

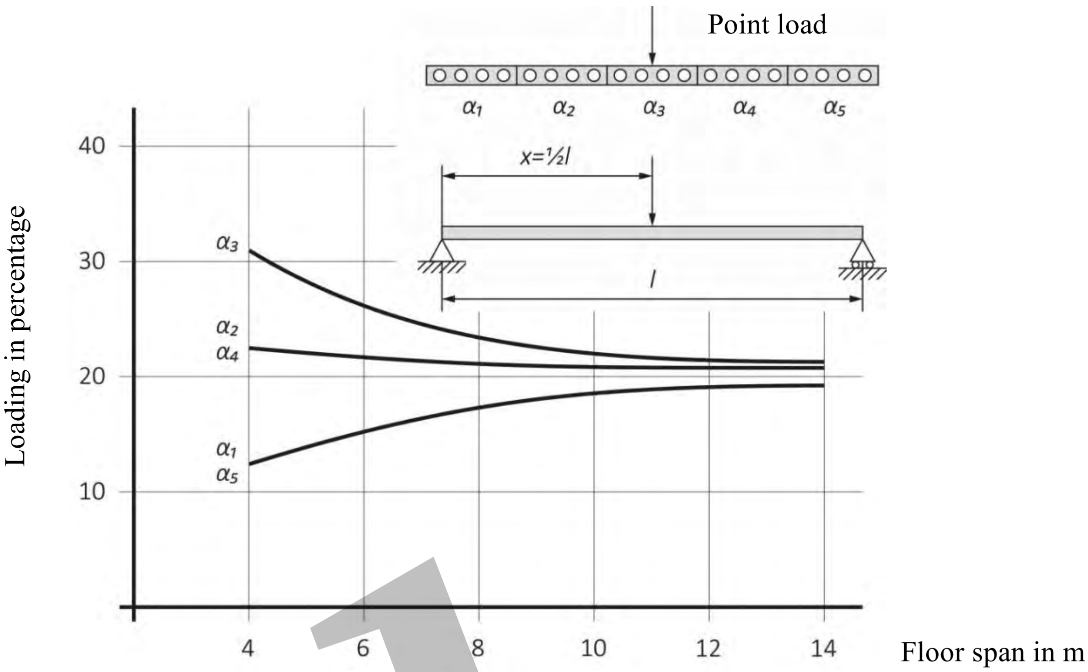


Fig. 8.18: Load distribution factors for hollow-core floors with 1.20m-wide units, for point loads in the centre area of the slab field (valid only for moments). The graph is a copy of Figure C2 from EN 1168 [4].

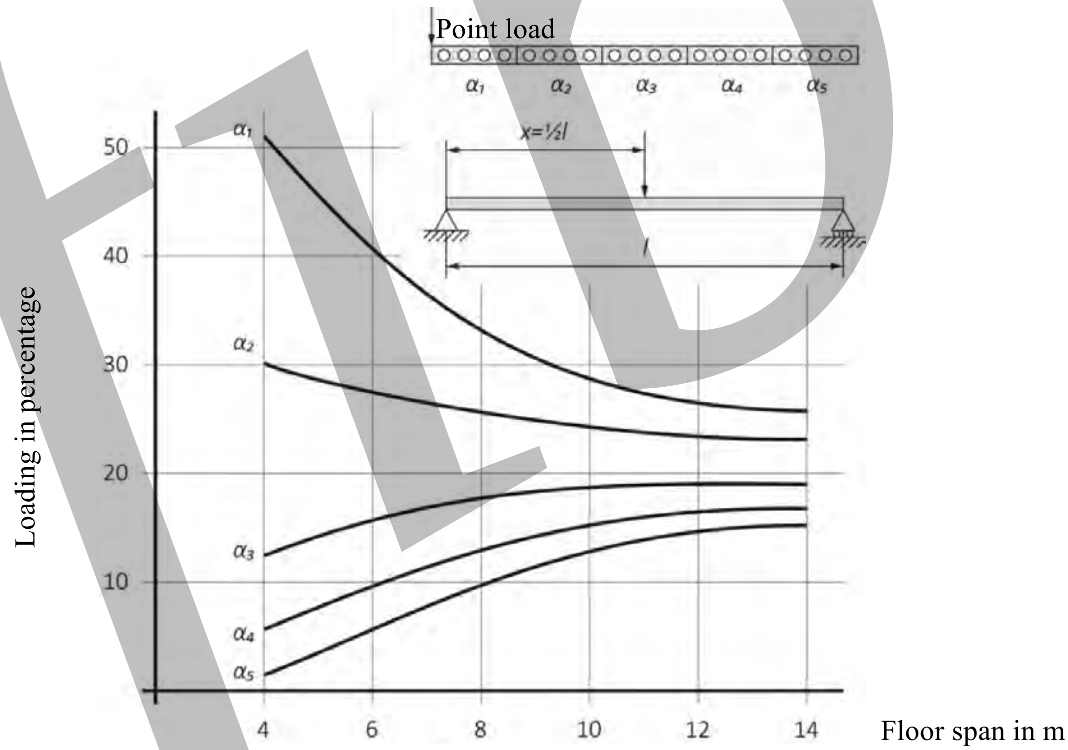


Fig. 8.19: Load distribution factors for hollow-core floors with 1.20m-wide units, for point loads at the edge of the slab field (valid only for moments). The graph is a copy of Figure C3 from EN 1168 [4].

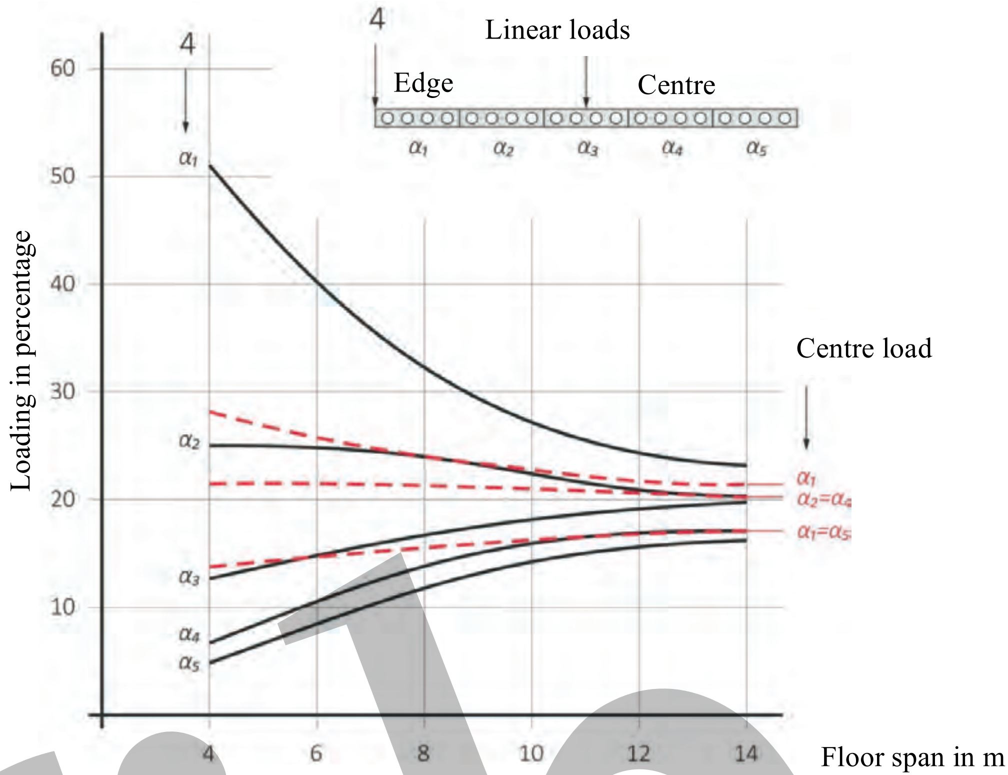


Fig. 8.20: Load distribution factor for hollow core floors with 1.20m-wide units for concentrated linear loads (valid only for the moment distribution). The graph is a copy of Figure C1 from EN 1168 [4].

- Ribbed floors

  Transversal load distribution in double-tee floors with thin flanges (40/50 millimetres) normally requires a structural-concrete topping with transverse reinforcement. It is preferable to also provide transverse bars in the top flange of the elements, which should be connected by welding after erection (Fig. 8.21a). When double-tee units have sufficiently thick floor flanges to be used without topping, the transverse load distribution is possible through the grouted indented joints (Fig. 8.21b) and welded mechanical connectors.

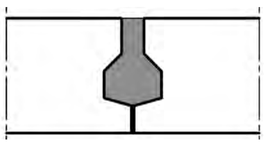
a.  b. 

Fig. 8.21: Welded and grouted connections between double-T floor elements

8.7.4 Cantilevering floors and balconies

Cantilevers may be formed in a number of ways:

Beams cantilevering over columns can support floors and balconies. Cantilever beams require columns to be spliced at every floor level. This design may also require a greater number of beams and may need an additional edge beam at the end of the cantilevers.

Fig. 8.22: Examples of precast cantilevering beams over columns

Certain types of floor units, such as double-tees, may be designed to cantilever directly over edge beams. Hollow-core units are not recommended for direct cantilever action other than for small spans up to about 2.00 metres. The slabs are designed with prestressing tendons at the upper and the lower part of the cross section. The cantilevering action can also be taken up by a reinforced structural topping anchored in concreted open cores (Fig. 8.23). When using any type of floor, the manufacturer must be consulted because the floor units are normally designed for use as simply supported spans.

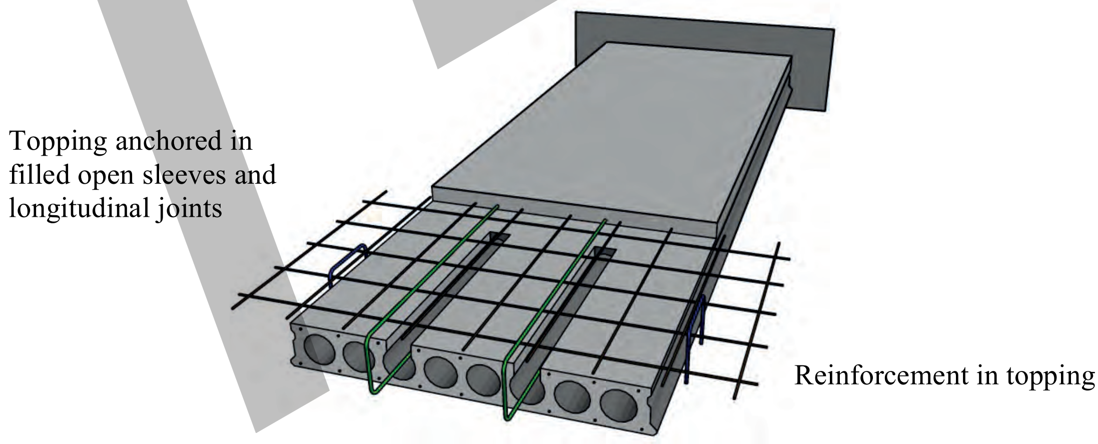


Fig. 8.23: Cantilevering hollow-core slab

8.8 Examples of typical connections

8.8.1 General

The following types of floor connections will be considered:

- Support connections
- Connections at longitudinal joints
- Lateral connections at non-supported floor edges

To ensure that precast floors perform satisfactorily, it is most important to conceive and design the connections properly. The following section gives recommendations and examples for the design and detailing of typical floor and roof connections.

The essential objectives to be met by support connections include:

- Connecting the units to the supporting structure
- Transferring tensile forces to the stabilizing system
- Establishing structural integrity and achieving diaphragm action and the transverse distribution of concentrated loads
- Balancing the effects of creep, shrinkage, temperature changes and differential settlements

8.8.2 Support connections

8.8.2.1 General

The detailing of connections at the support depends on the type of floor unit and the material of the supporting structure: concrete, steel or masonry. The practical points to be considered include:

- Minimum support length (in relation to tolerances)
- Evenness of the contact zone along the support
- Rotation capacity
- Prevention of spalling
- Tie arrangement
- Degree of restraint of the floor units

For hollow-core units and floor plates, the bearing stresses are rarely critical. However, for ribbed soffit units, the loading can be very high and the support zone rather small, for example, when double-tee units are supported on their webs.

Rigid neoprene strips, wet mortar bearing, fibre cement plates or similar materials are used to localize the support reaction and improve the supporting conditions when bearing surfaces are uneven, or when the contact stresses are high. In light loading, for example, in domestic buildings, they are not always structurally necessary; floor units are sometimes laid directly onto the supporting structure. In other cases, the use of bearing pads is always advisable.

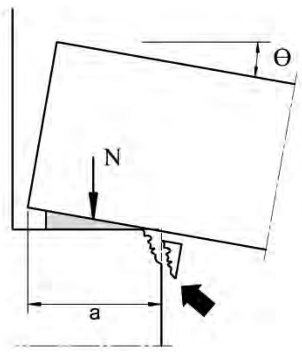


Fig. 8.24: Nominal floor support length

In double-tee construction it is preferable to locate the support at the webs, even when the unit ends with a protruding top plate. In this way, the support forces are introduced directly into the webs.

There is a possibility to design hollow-core slabs with indirect support but more checks to the connection at the supports are required (see *fib* recommendations for hollow core slabs [15] and [19]). Ribbed floors are normally designed with indirect supports with reinforcement in the in-situ part. This is one of the main advantages with these kinds of floor designs.

8.8.2.2 Tie arrangements at the support

The purpose of tie connections at the support is to ensure the transfer of vertical and horizontal forces from the floor to the adjacent structure, for both normal actions and unintended loading. The connection must therefore satisfy the requirements of force transfer, structural integrity, deformability and ductility. The detailing of the longitudinal, transversal and peripheral ties is critical in this context; however, there are different practical solutions, depending on the type of floor and supporting structure.

In hollow-core floors, the longitudinal tie bars are placed either in the grouted longitudinal joints or in concreted sleeves (Fig. 8.25). The latter are made in the top flanges of the units during manufacture. Tie bars placed in longitudinal joints need an anchorage length of about 1.0 to 1.50 metres because of the less effective anchorage conditions than for concreted sleeves, where an anchorage length of 0,6 to 0,8 m normally suffices.

Unless the supports are designed for moment continuity, the connecting reinforcement bars should preferably be placed in the middle of the cross section rather than in the upper section. This helps to avoid undesirable restraining moments at the support. Placing the reinforcement close to the bottom does not sit well with the design philosophy for structural integrity, as explained in previous sections. Therefore, the best place is in the middle of the cross section.

At intermediate supports, the longitudinal tie bars are made continuous over the support structure, whereas at edge beams the tie bars are directly anchored to the transverse tie beam or to the supporting structure that has the tie-beam function (Fig.8. 27),

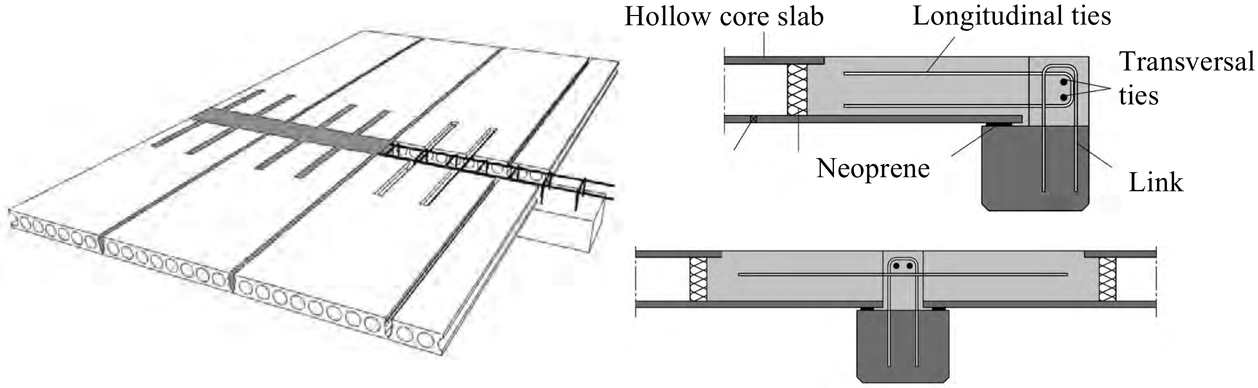


Fig.8.25: Support connections for hollow-core floors

8.8.3 Connections at lateral joints

The main function of lateral joints between precast floors and beams or walls is to transfer horizontal shear forces between the floor and the adjacent stabilizing components. Recesses for connections may be formed in hollow-core floor units by removing part of the top flange (Fig. 8.26, left). Reinforcement bars and cast-in-situ concrete are place intermittently along the edge of the slab, usually at 2.4-metre centres.

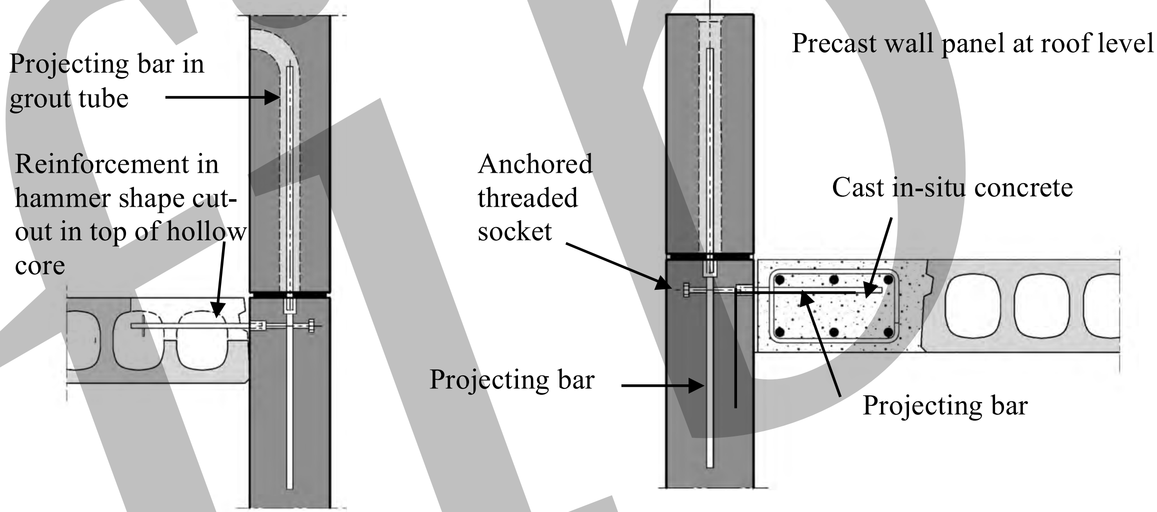


Fig. 8.26: Connections at lateral joint of hollow core floors and roofs

Continuous in-situ joints or topping screed may be used where a stronger shear connection is required. These may be formed either by using a soffit unit, or may extend to the full depth (Fig. 8.26; right). The slab is sufficiently flexible to accommodate the differential vertical movements caused by temperature fluctuations and loads.

8.8.4 Hollow-core units clamped between walls

Floor units are usually designed on the assumption that they are simply supported. This is the logical consequence of the general design philosophy of keeping connections simple and to provide stability by limiting the number of stabilizing components. However, unintended restraining effects may appear, for instance, due to heavy wall loads on the ends of hollow floor units (Fig, 8.27).

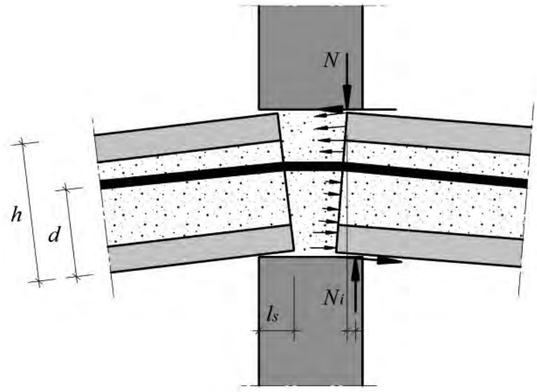


Fig. 8.27: Possible causes of unintended restraint of hollow core floors at the support connection

If support rotation is prevented by the support connection, restraint stresses can appear when the element is loaded. Restraint stresses can also occur due to intrinsic effects, such as shrinkage, creep and thermal strains. Even if the unintended restraint is ignored in the design of the elements themselves, the consequences must be evaluated and appropriate measures should be taken to avoid possible problems.

Since hollow core elements normally have no top reinforcement and no shear and transverse reinforcement, it is necessary to carefully consider the risk that cracks caused by unintended restraint may reduce the shear capacity of the element. The restraint moment can be reduced by limiting the load on the wall, i.e. limiting the number of floors or providing soft joint fill that prevents wall load from entering the end zones of the hollow core unit(s), see Fig. 8.28-left. To avoid direct loading from the wall and facilitate crack formation in the preferred location, the hollow core unit(s) could be provided with slanted ends as shown in Fig. 8.28-right.

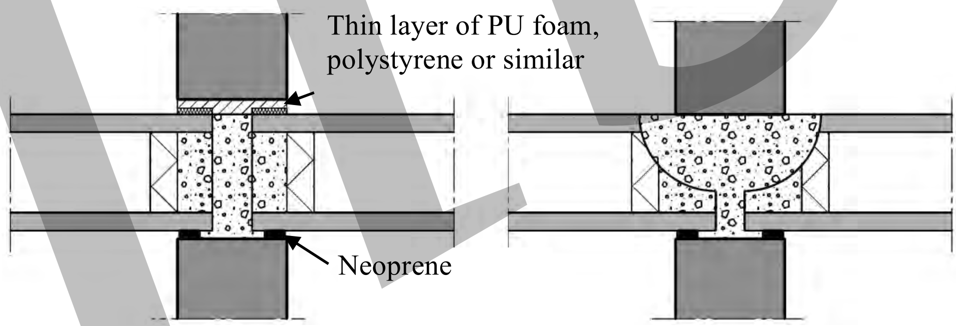


Fig. 8.28: Measures to limit or avoid restraint moment at the support connection of hollow core elements, a. use of a soft fill, b. slanted ends

Detailed information about the design of hollow core units regarding possible clamping effects between wall units is given in [12] and [15].

In ribbed floors, the continuity between units and the supporting structure is obtained by direct anchoring of protruding bars from the units into the tie beams, or by welding (Fig. 8.29).

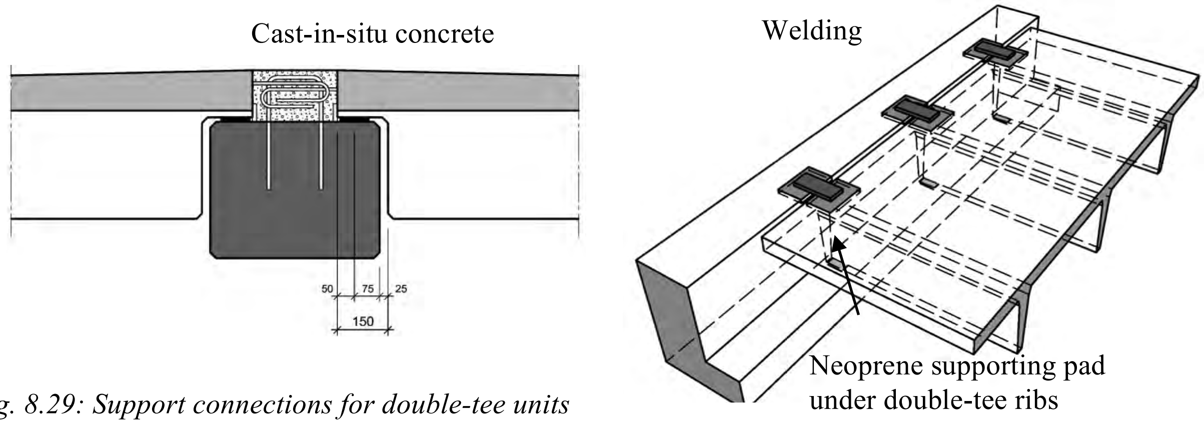


Fig. 8.29: Support connections for double-tee units

The connection between precast floor units and the supporting structure may also be obtained by means of a structural topping. The reinforcement of the topping may be continuous over the internal beams and lapped with projecting reinforcement in the floor beams.

Connections between composite floor-plate floors and supporting members present few problems. Continuity can be provided by lapping the mesh with reinforcement projecting from the supporting beams or walls. The mesh must be ductile grade steel.

Lateral connections of double-tee units to walls may be formed either by using projecting reinforcement from the wall into the topping on the double-tee floor or by welding cast-in steel plates. Typical examples are given in Figure 8.30.

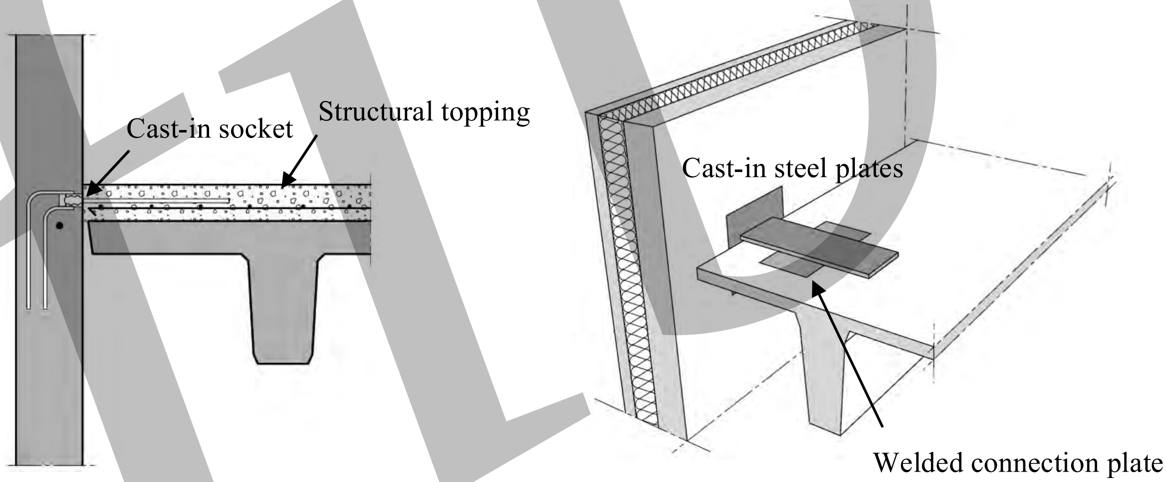


Fig. 8.30: Lateral connections between double-tee floors and walls

8.8.5 Support connections on steel beams

Figure 8.31 shows examples of support connections on steel beams. In slim floor structures the tie-bars are placed either over the steel profile or through holes in the web. When fire resistance is required, all exposed parts of the steel beams should be protected by adequate fire insulation.

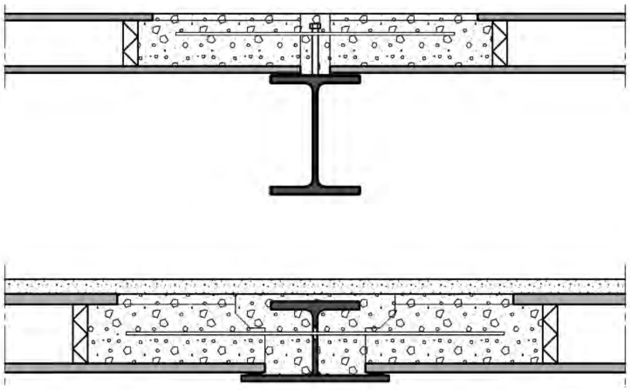


Fig.8.31: Floors supported on steel beams

A special solution for slim floors is achieved with the so-called hat beam. It is a hollow steel beam made from welded steel plates with a wide under flange on which the hollow-core floor elements are supported (Fig. 8.32). Another solution concerns a hollow steel-concrete composite beam made from welded steel plates with holes in the sides. The hollow-core floor units sit on the wide bottom flanges of the beam and the voids in and outside the beam are completely filled with in situ concreted after the erection of the floors. After the concrete has hardened, the beam acts as a composite structure.

Fig. 8.32: Hollow-core floor supported on hat beams

9 Architectural concrete façades

9.1 General

The term 'architectural concrete' refers to precast units that are intended to contribute to the architectural effect of the construction through design, finish, shape, colour, texture and quality of execution. Architectural concrete is a high-quality building material that offers a range of top-quality finishes such as limestone or granite, complex brickwork detailing and masonry profiles reproduced in reconstructed or simulated stone, all of which are features that would be prohibitively expensive if carried out on site with conventional methods. It is used for the façades of all sorts of buildings, such as apartments, offices, commercial buildings and educational and cultural facilities.

9.2 Precast façade systems

Depending on their function within the building, architectural concrete cladding can be designed as load-bearing or non-load-bearing, simple-skin or double-skin. The most common structural systems are described below.

9.2.1 Load-bearing façades

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. The most classical example is the sandwich façade. Sandwich units are composed of two concrete leaves with a thermal insulation in-between. Figure 9.1 illustrates how this load-bearing function is achieved.

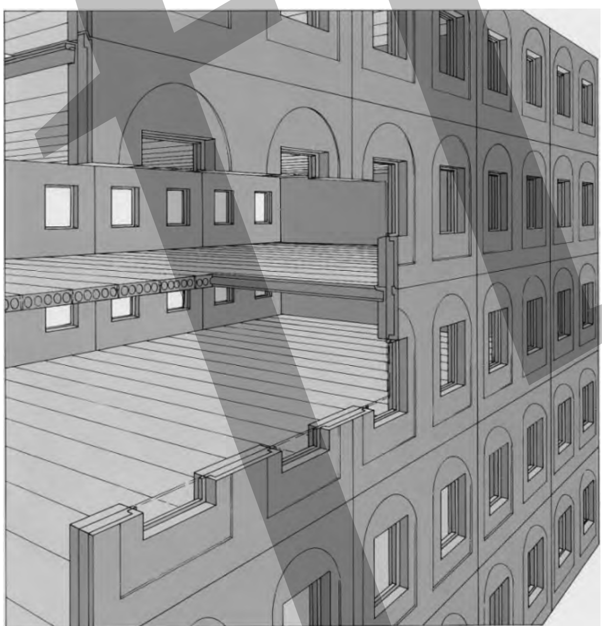


Fig. 9.1: Load-bearing sandwich-façade elements

The figure shows a framed façade element carrying the integral vertical load of the floors and the structure above it. The façade element can be a sandwich panel or a twin-skin façade.

As previously mentioned, these façades can, in principle, also fulfil a horizontal stabilizing function like any precast-concrete shear wall. When this is the case, shear connections between the elements are typically required.

The façade can also be composed of load bearing spandrel panels (Fig. 9.2). Here the spandrel elements act like beams, transferring vertical loads to the columns. They can be executed as sandwich units, with the exterior cladding in architectural concrete. Another possibility is to precast only the interior skin of the spandrel beam and to finish it on site with masonry or any other added finishing material.

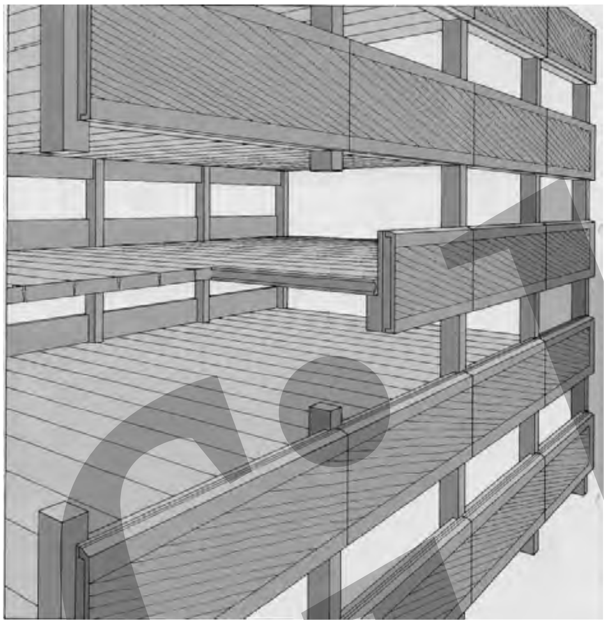


Fig. 9.2: Load-bearing spandrel units

9.2.2 Non-load-bearing façades

Non-load-bearing façades fulfil only an enclosing function. The units are fixed to the structure of the building either individually or as self-supporting units. In the first case, the exterior columns or beams and floors of the building structure support the self weight of the cladding elements. In the second case the façade is supporting itself and the elements are only anchored horizontally to the structure. In principle, the shape of the units can be designed without any restriction. Non-load-bearing sandwich elements are commonly used in skeleton structures or for the sidewalls of load-bearing façades. Single-skin elements are mostly used for facings of columns, spandrel panels, and so forth.

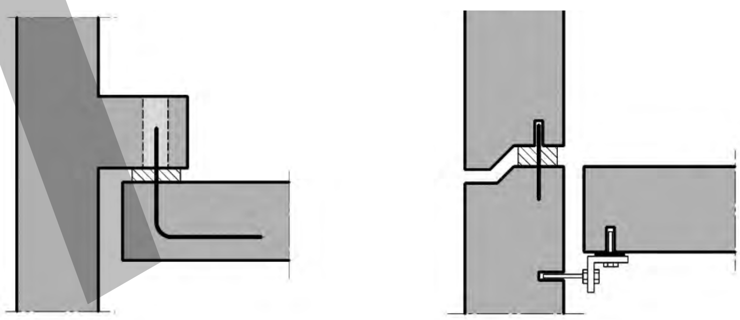


Fig. 9.3: Principle of self-supporting and non-bearing façades

9.2.3 Sandwich panels

Precast insulated sandwich panels are typically composed of three separate layers: an outer layer of reinforced precast concrete, an insulation layer and an inner layer of reinforced or prestressed precast concrete. The layers are mechanically held together using various types of connectors. Precast insulated sandwich panels are mainly used for façade walls but sometimes also for internal walls. They can be designed as cladding or as a structural element for the building and can have a variety of architectural finishes. Coloured concrete, polished or differently treated concrete surfaces, even graphics and pictures can be applied on the outer leaf of panels.

Precast sandwich panels are able to reduce building energy costs, improve long-term performance and make use of more environmentally friendly concrete.

Non-composite sandwich panels are identified as panels where the inner and outer concrete layers (often of different thicknesses) act independently. The inner layer is typically the structural layer and is designed to accommodate handling and in-service loads, while the outer layer is commonly the thinner non-structural layer that is attached via the connectors. These connectors allow the layers to move independently of each other, allowing for thermal expansion and contraction without affecting the structural integrity of the wall.

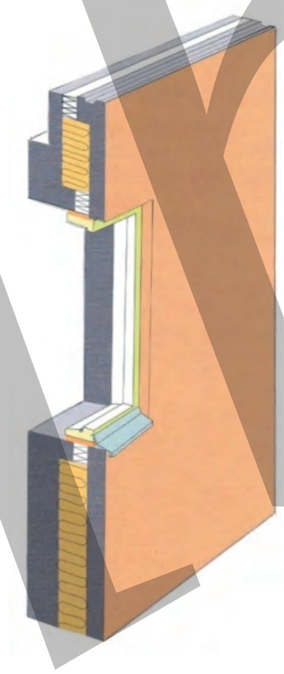


Fig. 9.4: Principle sketch of a sandwich façade panel

Fig 9.5: Buildings with precast insulated sandwich panels

The longer sandwich panels have been produced in individual countries, the more architects and designers are coming to realize what the potential this type of building element is. In most countries non-ventilated sandwich panels, without air gaps, are the preferred solution. However, it is also possible to incorporate a cavity between the outer leaf and the insulation. Its principal role is to prevent the penetration of rainwater into the insulation and the inner surface. Humidity evaporates in the ventilated cavity or is evacuated at the horizontal joint (Fig. 9.6).

Fig. 9.6: Sandwich panels

Left: Without air void
Right: With air void

9.2.4 Twin-skin façades

A twin-skin façade incorporates sandwich-façade construction in which the two concrete leaves, namely, the inside and the outside leaf, are fabricated and erected separately. The load-bearing leaf of the façade consists of simple framed panels placed with the smooth moulded side towards the interior of the building. The precast floor units are supported on these elements. In a following step, airtight joint sealing is applied and the insulation layer is attached to the exterior face of the framed panels. Finally, the exterior cladding is erected and can be in precast concrete or another type of cladding material.

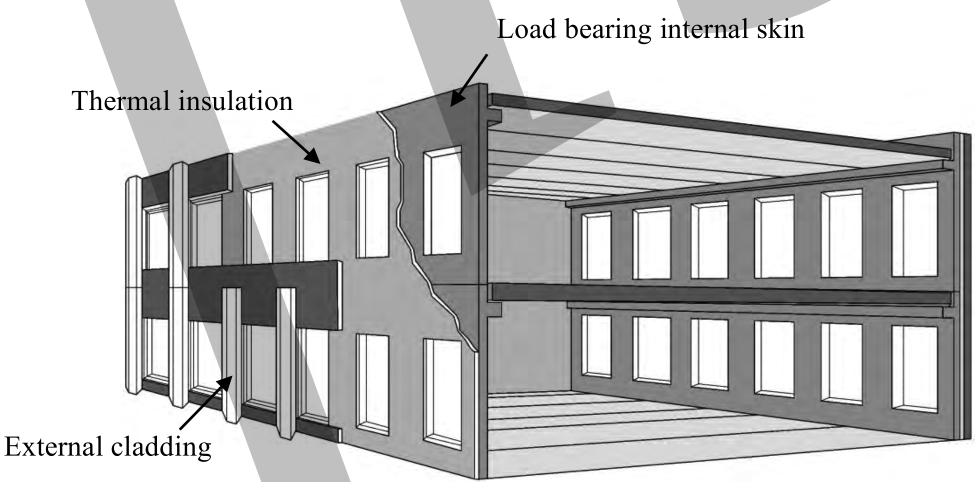


Fig. 9.7: Principle of twin-skin façade system

The solution offers several advantages over classical sandwich panels:

- Large flexibility in the design of the exterior façade. Complete freedom with respect to size, shape and materials used.
- Possibility to use other materials for the exterior cladding.
- Precast units are simple. Internal panels are rectangular storey-high elements with large repetition. Exterior cladding is mostly made of flat panels. Connections are very simple
- Insulation is continuous over the façade without any single weak thermal point, including at panel joints
- Façade comprises a cavity between exterior cladding and insulation
- Outlook of façade can be totally different from one building to another, without major difference in the interior structure and elements used. Joint pattern in façade is unobtrusive

Fig. 9.8: Application of twin-skin façade system

A drawback to the solution is that it needs more individual precast units, which means more handling, storage capacity, transport, connections, and so forth. However, it is quickly compensated for by the simplified production of individual units and the great freedom in architectural design for the façade.

9.2.5 Special elements

Architectural concrete can also be used for decorative purposes inside buildings. As a matter of fact, all precast components can be made in architectural concrete, if necessary. There are numerous such examples, such as balcony units, cornices, parapets, plinths, string courses, special columns (Fig. 9.9), floors with decorative soffit profiles, suspended decorative ceilings, internal walls, central cores and polished stairs.

Fig. 9.9: Reconstructed stone precast concrete columns for entrance of apartment building

9.3 Structural stability

Precast architectural façades are usually designed as simple cladding panels that do not greatly contribute to the horizontal stability of the building. In such cases, stability is provided by central cores and/or shear-wall action. However, when using the twin-skin façade method, the internal skin of the façade can be designed to resist in-plane horizontal forces from wind, earthquakes or other actions.

9.3.1 Stability provided by cores and shear-wall action

The load-bearing façade elements usually support only the vertical loads from the floors and the structure above them. The horizontal forces acting on the façades are transferred to the stabilizing structure (cores or shear walls) by the diaphragm action of the floors. The connections between the façade elements and floors are designed as hinges in the direction perpendicular to their plane.

9.3.2 Stability provided by the façade

When the façade is composed of load-bearing walls with sufficient in-plane stiffness, they can also ensure the horizontal stability of the building. The precast floors function as diaphragms between the front walls and sidewalls and ensure the coherence of the system. The vertical joints between the concrete walls should be able to transfer shear forces. This may be obtained, among other ways, through grouting or welded connections.

9.4 Principles of design and dimensioning of the units

9.4.1 General considerations

Precast claddings in architectural concrete are mostly executed in reinforced concrete. In some cases small, partial prestressing will be applied, for example, to limit tensile stresses during handling. Full prestressing is seldom used for façade panels.

The calculation of façade panels is, in principle, not different from normal reinforced-concrete structures. Cracking should be avoided as much as possible because it affects the aesthetic appearance. For this reason larger limitations will be imposed on serviceability stresses. Possible additional stresses due to deformations from various actions should be taken into account.

The following construction phases should be taken into consideration during the design of the elements:

- manufacture, storage, transport and erection
- in service, after hardening of the connections

The forces that come into action during these phases are quite different. In addition, the concrete strength of the elements is much lower at the demoulding stage than in service. Recommendations are given below for the most important points.

9.4.2 Actions during the various construction phases

The most critical actions nearly always occur during the demoulding of the units. Not only the self weight of the units but also the adhesion to the mould and the dynamic loading at stripping are important factors. In order to take these into account, the self weight of the units should be increased by a coefficient of 1.5 to 2.0, based on the complexity and the overall dimensions of the units. For example, for framed panels a coefficient of 1.5 may be used and for plates a coefficient of 2.0. The lifting points should be placed in judiciously in order to limit bending moments during stripping and handling, especially where long units are concerned.

To take account of dynamic loading during handling and transport, a coefficient of 1.25 may be applied.

For the calculation of the panels in the serviceability limit state outside of the normal acting forces, special consideration should be given to indirect effects from volume changes or restrained deformations at the supports. Those forces sometimes dominate the normal loading on connections (see Section 9.5).

9.5 Other design aspects

The elements in architectural concrete can be subjected to deformations due to temperature movements, shrinkage, creep and support movements. Precast structures are able to take up these deformations even better than cast-in-situ structures.

9.5.1 Thermal deformations

Deformations due to the differences in temperature between parts of a construction should be carefully evaluated. Such temperature differences may appear between the interior elements of the structure, which are almost constant in temperature, and the external parts, such as the façades and roofs, which are exposed to climatic conditions. This movement will vary based on the type of structure, panel size and climate. To avoid the curvature of large panels and greater concentrations of stresses, the connections of cladding elements should be designed in such a way that thermal movement between the façade elements and the supporting structure is possible. To cope with this movement, the fixings must safely hold the panels in position and not become structurally affected. Using synthetic washers, spacers and oversized holes is the normal way to allow for such movement. For sandwich panels, the connections between the two panel leaves should enable the differential in-plane movement of the leaves (see Section 9.9).

The range of temperatures that should be taken into consideration will also affect the colour of the external wall skin and the orientation of the elements. In moderate climatic conditions, external surface temperatures could range from - 20 °C in winter to + 60 °C in summer for dark finishes, while the internal temperature of occupied buildings will vary from + 5 °C in winter to + 30 °C in summer. The following total temperature differences should be taken into account in calculating the differential movement in temperate climates:

- Δt = 60 ° C between the average maximum summer and minimum winter temperatures of the façade and roof elements, taking into account that erection and assembly will normally not be done at outside temperatures below 0°C
- Δt = 40 ° C for the maximum difference between the internal and external skin of sandwich elements
- Δt = 20 ° C for the difference between two opposing façades in summertime

9.5.2 Creep and shrinkage

The volume changes attributable to the creep and shrinkage of the hardened elements are usually small compared with thermal movements. However, attention must be drawn to the possible effects of creep during the hardening of the concrete, when the shape of the mould is hindering volume changes. A typical example is the internal mould of a framed element. Cracking can be avoided by removing the element from the mould as soon as possible and by adding more construction reinforcement. The possibility of crack formation at the internal corners of framed elements and changing from large to smaller cross sections should be given special consideration.

9.5.3 Settlement of the supporting structure

Façade elements that are supported on a reinforced concrete beam follow the initial and long-term deflection of the beam. In order to limit deflection as much as possible the supporting points of the façade elements should be placed as close as possible to the supporting points of the beam or the floor.

The size of the supporting beam or floor should be determined in such a way that the long-term deflection will be limited to an acceptable value, for example, $\ell/1500$.

9.6 Shape and dimensions of the units

9.6.1 Mould considerations

An important property of concrete is its mouldability, which allows a wide range of possible shapes to be created. Concrete shapes are not limited to volumes enclosed within plane surfaces; they may also be rounded. Panel profiles can provide the simple 'streamlined' elegance of a modern building or the classical cladding profiles typical of traditional natural stone cladding.

Elements in architectural concrete are generally cast in unit moulds or on tilting tables. Different materials can be used for the construction of formwork for architectural concrete:

- Timber formwork is eminently suited to average reuse and works for simple and smooth contact surfaces
- Steel formwork is good for multiple reuse
- Concrete moulds are appropriate for multiple reuse and for moderately complex forms
- Plastic forms are suitable for moulding complex and curved components
- Rubber forms are used for components, such as sculptural ornaments, that are difficult to demould

Timber is often preferred because of the relative cost of the basic material, the availability of craftspeople trained in traditional building techniques, the ease with which timber can be worked and fastened and the capacity to reuse the moulds for the casting of variants. Timber can be formed into sturdy, robust moulds capable of withstanding the considerable stresses imposed by the pouring and the vibration of concrete. Usually a timber mould should have a trouble-free life of up to 30 to 40 castings but there have been instances where 100 castings were made from the same mould, albeit with some repairs to the mould.

Moulds usually comprise fixed parts and removable parts that allow for the stripping of the concrete units after hardening. The fixed part comprises a basic carcass faced with a liner and ensures rigidity and stiffness,.

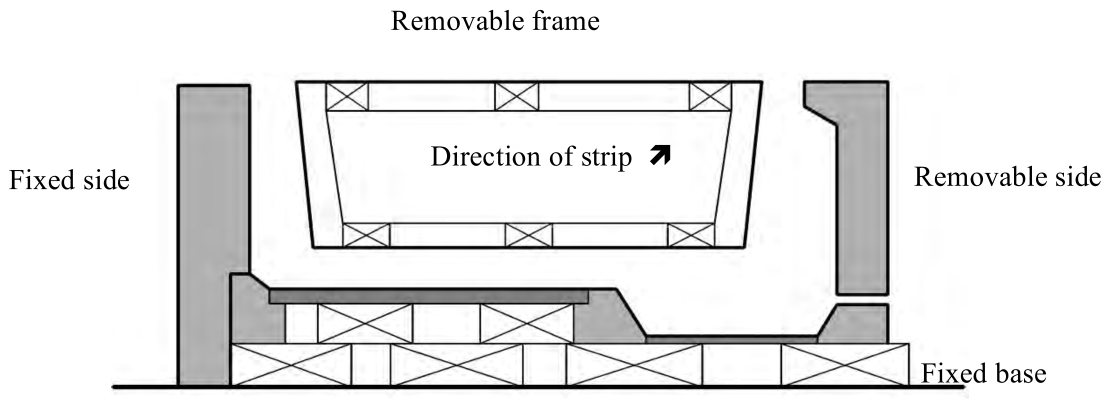


Fig. 9.10: Typical mould construction

To allow for the casting of a minimum number of elements in a single mould, elements are classified by family. The mould is built in such a way that transformation to variant elements is possible. The largest units are always cast first so as to avoid differences in the surface texture of successive units.

The shape of the units should be designed in such a way that the mould has a maximum number of fixed parts. Fixed moulds prevent cement leakage, are more accurate and produce higher-quality elements.

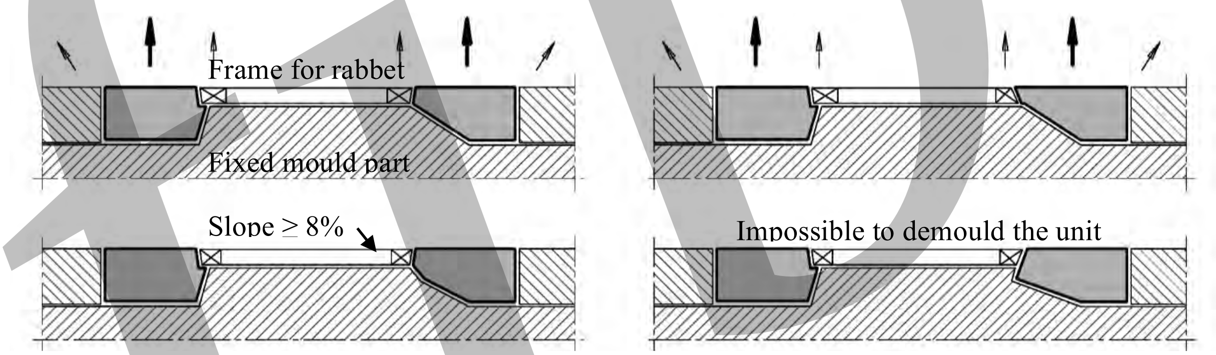


Fig. 9.11: Mould

Any profile or feature can be built in during the design and assembly of a mould for a basic family of elements. The mould designer or mould maker does, however, require early outline drawings of such a family of elements in order to accommodate variety in the form of returns, variations in end detail and the like. Whilst changes may be introduced in the basic section or profile of the mould by the insertion of pads or stools during the construction process, changes in section that take the form of projections beyond the basic profile are expensive and should be avoided whenever possible.

The manufacturer will ask to use 'lead' or 'draw' on features, striations and re-entrant surfaces, as well as around formers. For most purposes these should be pitched at 1 in 4 to allow mould stripping without damaging the young concrete elements. Generally, for all surfaces from which the element is to be stripped without dismantling of the mould, the mould should have a minimum lead or draw of 1 in 12.

It is preferable to design façade units for a maximum number of castings in the same mould whenever possible. Figure 9.12 shows spandrel façade units for a car park in Bologna, Italy (see also Figure 1.5). Despite a large number of different units, most of the elements were cast in the same basic moulds. This allowed not only economy of production but also flexibility in casting.

Fig. 9.12: Some of the peripheral spandrel beams bear the floor on sloped line supports and others are non-bearing. All have the same height and outer shape and are partially prestressed to prevent cracking. Due to the irregular layout, they differ in length and are inclined at various angles in plan. In spite of the large number of different sub-types, they were cast in the same moulds.

9.6.2 Preferential dimensions of the elements

The preferred dimensions of façade panels are storey height and a multiple of a basic module, for example, 300 millimetres in width. The latter is normally dictated by the planning or structural grid of the building design. The maximum overall dimensions of load-bearing and non-load-bearing elements are typically governed by the requirements for handling and transport. As a general rule, the weight of the elements should be limited to 10 tonnes. This corresponds to the normal lifting capacity at most precasting plants. However, one should also take into account the handling at the building site, which is determined by the place and the capacity of the crane. For example, for centrally placed lifting cranes the elements at the corner of the façade are generally at the greatest distance and the weight to be lifted may be limited by the lifting capacity of the crane.

When it comes to normal transportation the general rule is that one of the element's two main dimensions should not be larger than 3.60 metres.

The thickness of façade elements is dictated by structural design, the provision of adequate cover to reinforcement and the need for the adequate safeguarding of the elements against cracking at the time of stripping. To ensure good concrete compaction and to guarantee the proper placing of reinforcement and adequate concrete cover, in particular for fire resistance and durability, the depth *a* of framed units (Fig. 9.13) should fall between h/10 and h/15, *h* being the total height of the unit. For single-skin concrete panels, a minimum concrete thickness should be considered in order to prevent cracking and deformation during manufacture.

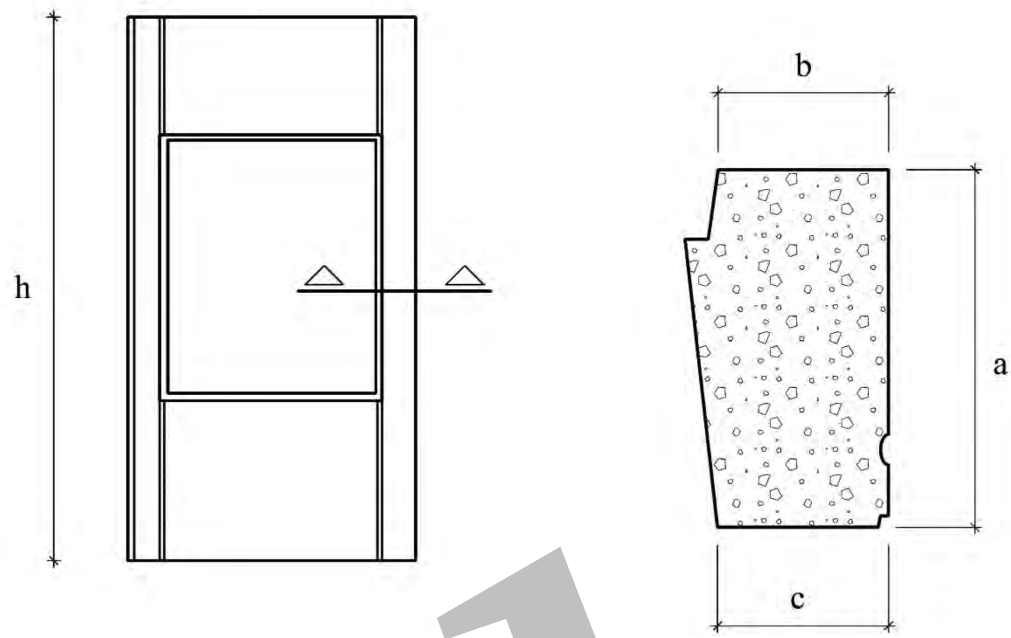


Fig. 9.13: Minimum column dimensions

- Column width

  To obtain good concrete compaction and guarantee the proper placing of reinforcement and adequate cover, in particular, for corrosion, the following minimum dimensions should be adopted (Fig. 9.13):

  $b \geq 100$ mm
  $c \geq 120$ mm

  Comment: The cracking of architectural concrete elements should be avoided as much as possible as it negatively affects the decorative aspect of the elements. Cracks are more easily perceived on smooth surfaces than on exposed aggregate surfaces.

  Experience shows that for framed elements, for example, the existing regulations for reinforced concrete could lead to the cracking of the mullions, especially in places where an abrupt change in section from the mullions to the panel might already cause a concentration of stresses.

- Thickness of flat panels

  Unless the minimum concrete thickness of flat panels is justified by calculations or tests, the indications for the minimum and optimal values of elements of uniform thickness are given in Figure 9.14 as a function of the largest panel dimension.

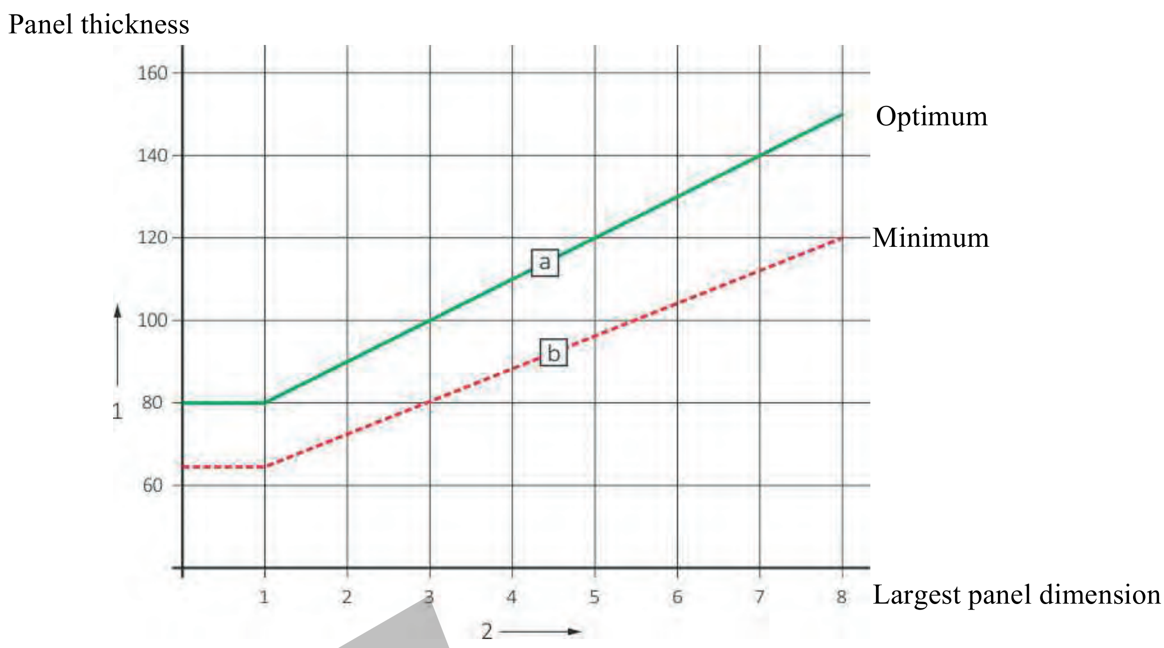


Fig. 9.14: Recommended thickness of flat panels

9.6.3 Modulation

As already explained in Chapter 2, modulation is an important factor in the design and construction of buildings. For façade units, this standpoint is more moderated. Modulation is certainly desirable, but should not constitute an obstacle to the architectural concept of the building. Modulation in connection with industrial production is therefore not imperative, but may have an influence on the cost of the elements.

Modulation axes should preferably be kept on the inside of the façade. Figure 9.15 shows examples of solutions for façade corners: separate corner units (a. and b.), a diagonal corner intersection (c.) and integrated corners (d., e. and f.). Solution c. is not recommended because of the risk of damage to the panel edges and difficulty to obtain a straight and regular joint profile.

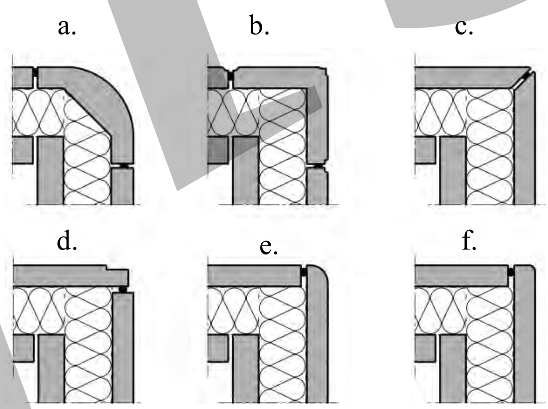


Fig. 9.15: Possible solutions for façade corners

9.7 Surface finishing

Concrete need not always be grey and rough. The precasting industry has developed concrete mixes, and moulding and surface finishing techniques that add a highly refined aspect to concrete units.

9.7.1 Texture

Concrete surfaces can be produced in a wide variety of textures, ranging from completely smooth to very coarse. Various techniques exist for all degrees of detail and every type of surface.

Etching or tooling is used to obtain the texture of fine natural stone. The texture of the concrete is completely flat but not shiny. The coarse granulation of the concrete is not visible. Acid etching is not a very environmentally friendly process and in some European countries restrictions are imposed on this kind of treatment in the open air.

Fine washing with a surface retarder and sandblasting are frequently used techniques for lightly exposed aggregate finishes. The result is a smooth surface on which the fine aggregates of the concrete are visible. Fine washing with a surface retarder ensures that the aggregates remain smooth but sandblasting obtains a slightly dull aspect. A surface finish ranging from slightly to entirely matt can, therefore, be achieved. Sandblasting is regularly used for the reconstructed-stone finishing of architectural cladding panels.

More heavily exposed aggregate finishes display the characteristic structure of concrete. This appearance is obtained by washing the surface with a water jet. For sections on the topside of the cast, this is done before the concrete hardens. For the sections of the concrete that come into contact with the mould, a retarder is used. Once it is removed from the mould, the concrete is washed. The texture depends on the size and type of aggregate used, with every possible variant from rounded to crushed shapes available.

Wet grinding and polishing is currently used for top-quality architectural concrete finishing. It competes well with polished natural-stone facing and offers greater advantages, not only related to cost but also to design. Ground surfaces are divided into three classes according to their degree of gloss: matt, glossy and brilliant. The process is carried out with modern automatic or semi-automatic machinery.

9.7.2 Colour

The range of natural colours that can be used in architectural concrete is virtually the same as for natural stone. When the texture is fine, the colour is mainly influenced by the colour of the fine aggregates, while coarser granular structures allow for a greater choice of colours.

It is also possible to add colour pigments to the concrete to create special effects. Inorganic pigments are more stable than organic ones. A wide gamut of pigment colours is available on the market. Surface conditions, such as moisture content, vibration at casting and hardening, may influence the intensity of the colour of the concrete. Consequently, slight colour variations between the elements may appear, especially when darker colours are used.

9.7.3 Faced panels

Precast façade panels can also be faced with other materials, such as natural stone, bricks and ceramic tiles. These materials are typically placed at the bottom of the mould and cast into the units. Materials that are small in size, such as ceramic tiles, can be fixed in a durable manner simply by bonding with the concrete. Special fixing is needed for larger-sized materials such as natural stone. The stone is fitted with anchors made of stainless steel and a bond breaker is used to allow for the differential expansion of both materials. Natural stone has a different expansion coefficient than concrete. In addition, the temperature of the stone will be higher than that of the concrete since it will constitute the external skin.

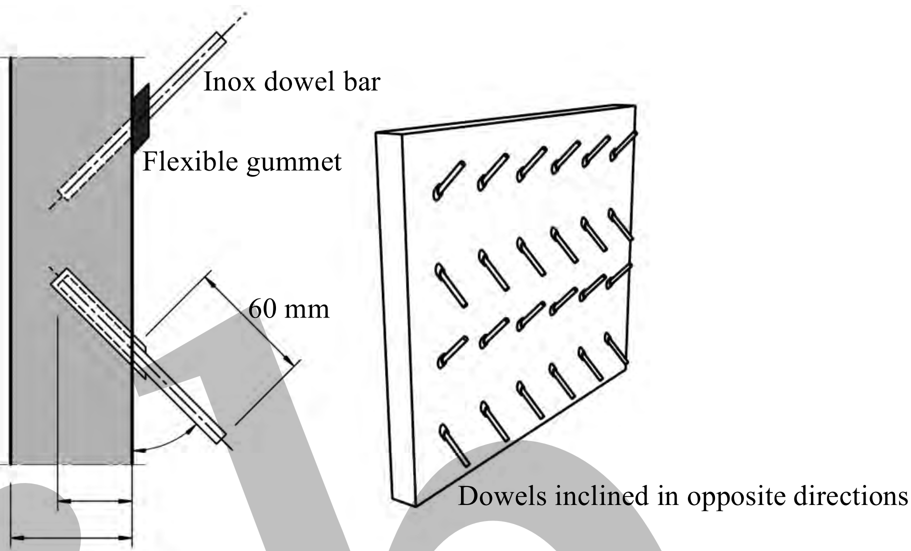


Fig. 9.16: Fixing and layout of granite-faced panel

9.8 Thermal insulation

There are several solutions for the insulation of precast architectural concrete façades. The first solution is the 'sandwich' panel (Fig. 9.4). The insulation is incorporated inside the concrete panel. The elements are relatively flat, with a thickness that usually ranges from 70 to 90 millimetres for the external skin and 120 to 160 millimetres for the internal skin, depending on whether the element is non-load-bearing or load-bearing.

It is also possible to incorporate a cavity between the outer leaf and the insulation. Its principal role is to prevent rainwater from penetrating to the insulation and the inner surface. Moisture evaporates in the ventilated cavity or is evacuated at the horizontal joint. Figure 9.18 shows one way of incorporating a cavity.

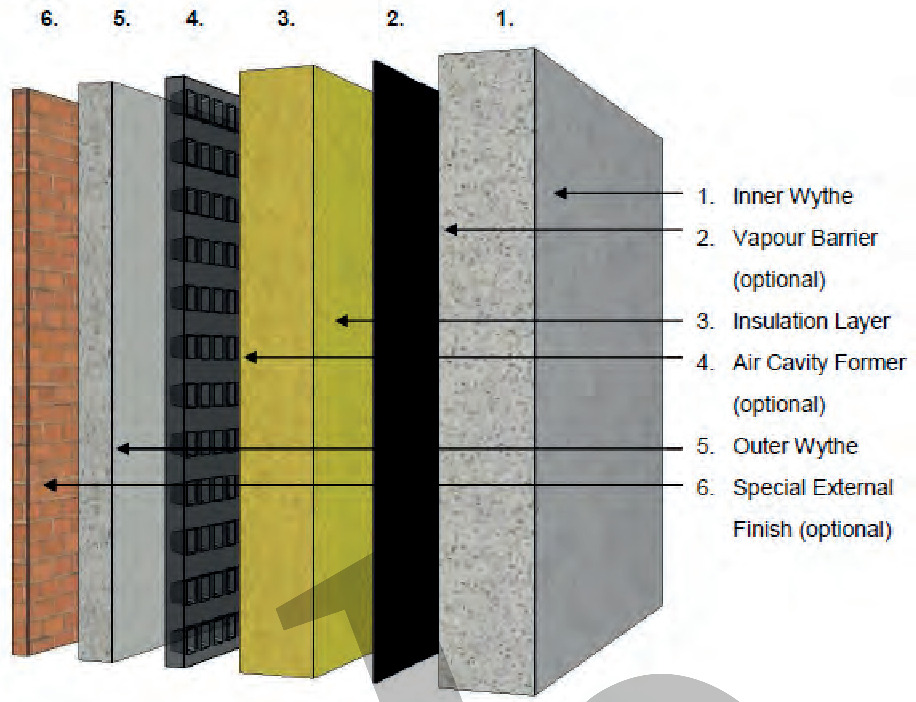
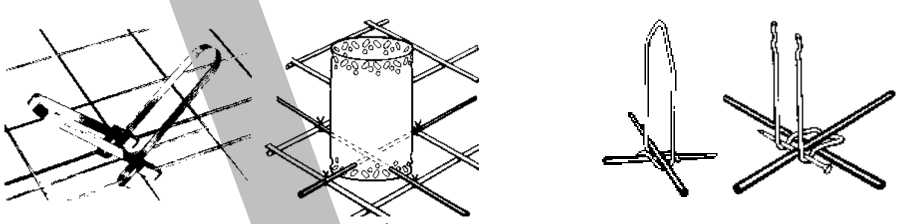


Fig. 9.17: Sandwich panel with aerated cavity

The exterior leaf of a sandwich panel should be fixed to the internal one in such a way that it is free to expand and contract. The connections between the two concrete leaves of the sandwich panel should fulfil mechanical requirements and contribute to ductility and durability. Two basic solutions exist: special connector systems and the diagonal reinforcement between leaves.

- Special connector systems are usually composed of bearing anchors, torsion anchors and spacers.

  The bearing anchors (Fig. 9.18a and b) support the weight of the external concrete leaf and wind action. Torsion anchors are needed when the bearing anchors do not procure sufficient rigidity in the transverse direction (a). The role of the spacers (c) is to take up horizontal actions and to maintain the correct distance between the two concrete leaves without restraining the lateral movement.



a. b. c.

Fig. 9.18: Typical sandwich panel connectors

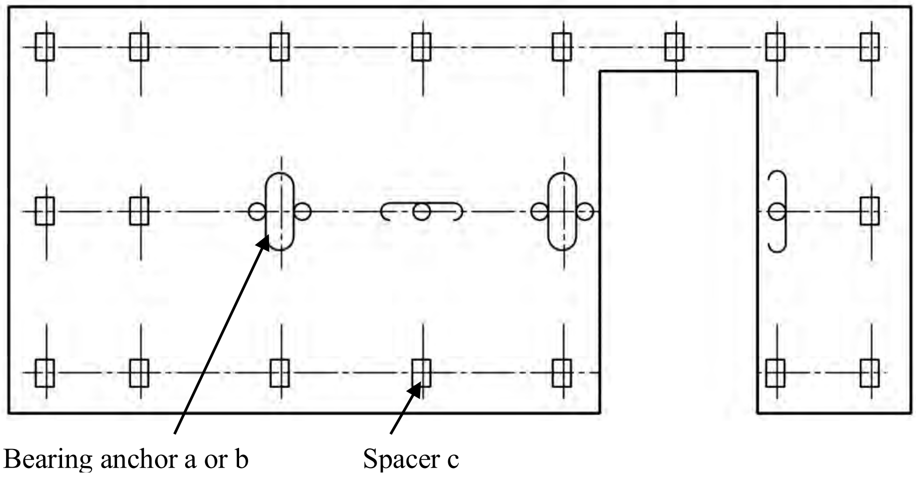


Fig. 9.19: Position of the different anchor types in a sandwich panel

Special connectors are normally used for sandwich panels with a relatively small space between the concrete leaves (up to 100 millimetres). Directives for the design and application of sandwich panel connectors can be found in producers' catalogues.

- Diagonal reinforcement between the two concrete leaves is mainly used in sandwich panels with a large insulation thickness (100 to 150 millimetres). Diagonal reinforcing strips in stainless steel are placed in a vertical position at regular intervals, typically about 600 to 1,200 millimetres, depending on the weight of the suspended concrete leaf and the capacity of the reinforcing. They come in different sizes and lengths (Fig. 9.20 and 9.21).

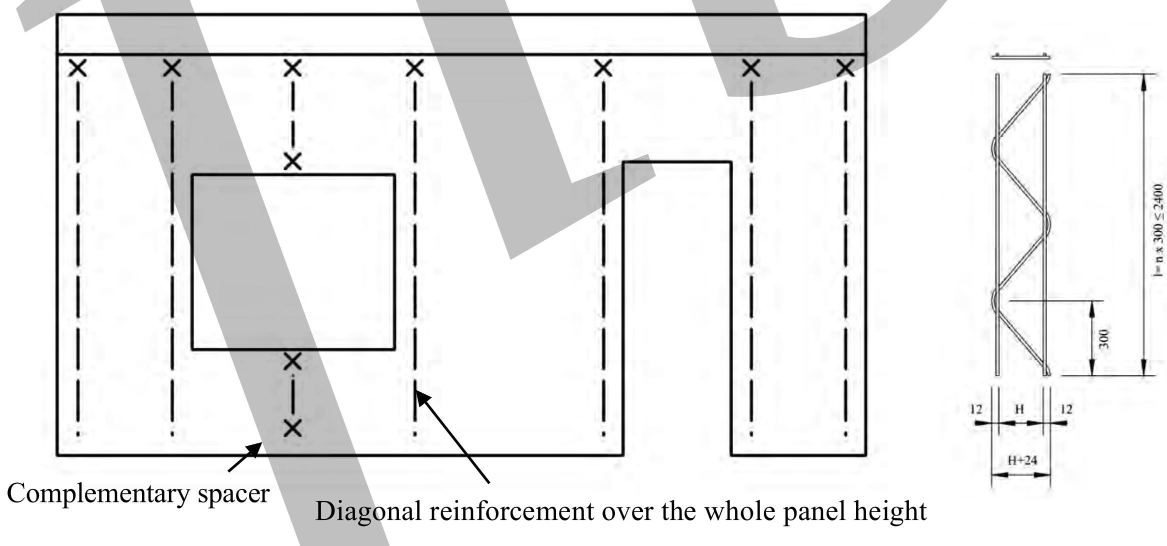


Fig. 9.20: Continuous diagonal fixing of the two leaves of a sandwich panel over the full panel height.

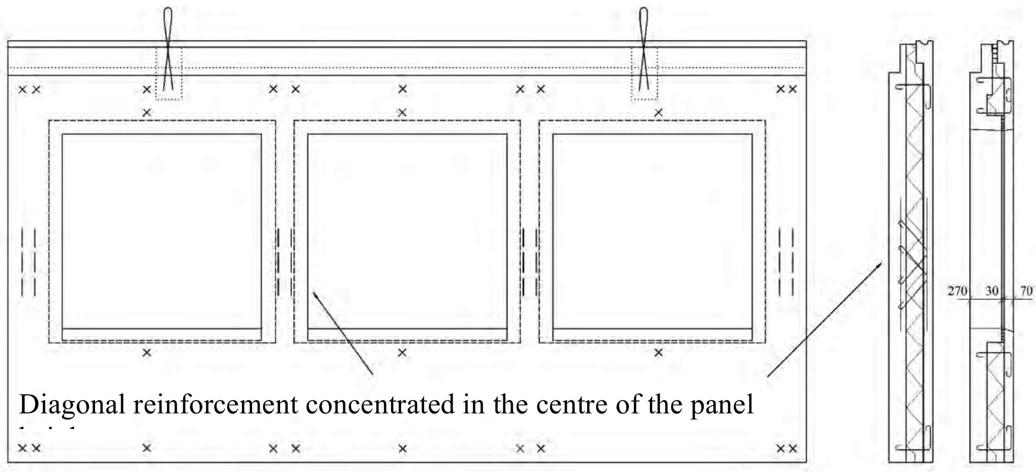
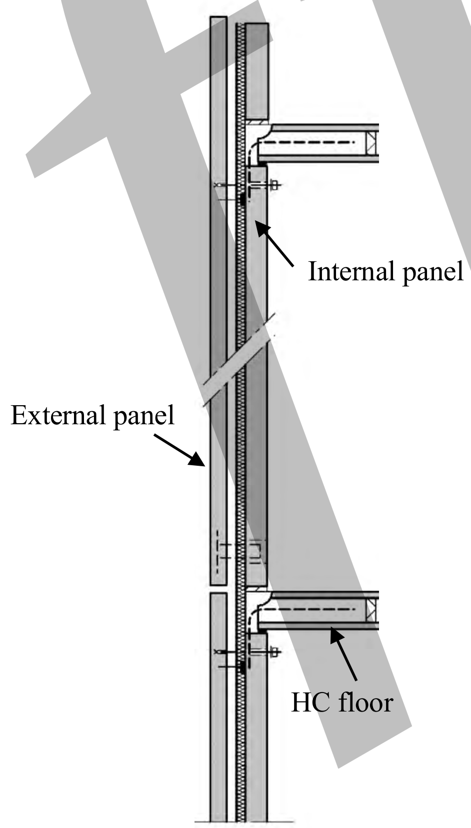


Fig. 9.21: Diagonal sandwich panel fixing concentrated in the panel centre

Complementary bars are placed at the perimeter of the sandwich panel to strengthen the connection. The connection system is able to absorb deformations due to thermal expansion and contraction because of the small diameter of the bars and sufficient width between the two concrete leafs.

A variant construction method of the sandwich façades is the 'twin-skin' method (Fig. 9.22). (See also Section 9.2.4.)



First step: Erecting the internal façade panel with the smooth as-cast side inwards

The floor elements are placed either on top of each panel or on a haunch. After the erection of all the storeys the thermal insulation is fixed to the outside of the panels with the help of mechanical anchors.

Second step: Erecting the exterior façade skin

When it is executed in architectural concrete, the cladding may be self-bearing or fixed to the interior skin. Since the design of the outer skin is independent of the internal skin, the size and shape may be completely different. For example, the panel height may cover two storeys.

Fig. 9.22: Assembly steps for twin-skin façade system

A second possible solution for insulating architectural concrete façades consists of the application of an insulating layer over the whole inside of the cladding. Afterwards a lining of, for example, plasterboard or brick masonry is placed over the insulation. However, this may reduce the thermal mass of the concrete.

9.9 Panel fixings

Fixings form a vital part of precast-concrete construction. Excellent literature with numerous examples and application studies that deals with connections and fixing details for precast architectural façade elements is available. In the next section the classical types of connections and fixings for architectural concrete panels are described.

9.9.1 Types of fixings and applications

- Connections with projecting reinforcement

  The mechanism of this connection uses the force transfer between the lapped reinforcement and dowel action. The elements to be connected have projecting bars that overlap in a judicious way within the cast-in-situ joint.

  Examples of connections with projecting reinforcement are shown in Figure 9.23. This type of connection is often used to connect load-bearing façade elements and floors. It is also a possible solution for the fixing of non-load-bearing elements. Specific advantages of this type of connection are: large tolerances, good economy and resistance to corrosion and fire. The main disadvantage is that the fixing does not provide immediate stability and must therefore be supplemented with temporary supports or other fixings during construction.

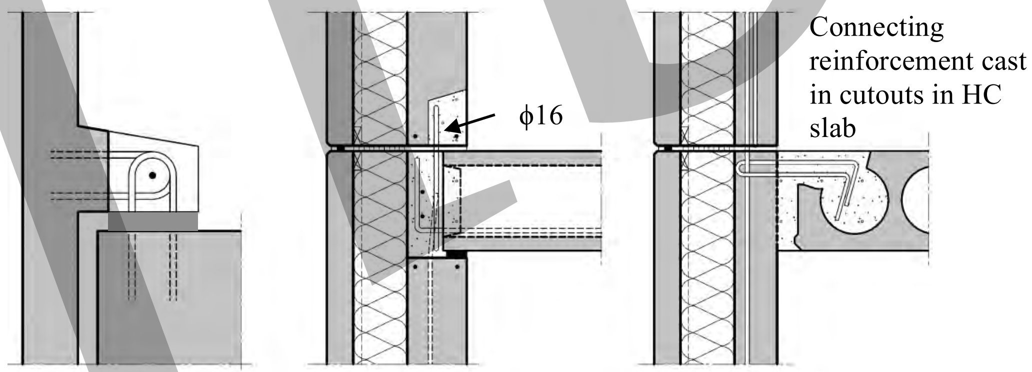


Fig. 9.23: Connection with projecting reinforcement and in-situ concrete

- Bolted connections

  Bolted connections are normally used for non-bearing façade elements. Among the existing fixings used are cast-in bolts, cast-in anchor rails and cast-in insert threaded bars. Figure 9.24 shows some solutions.

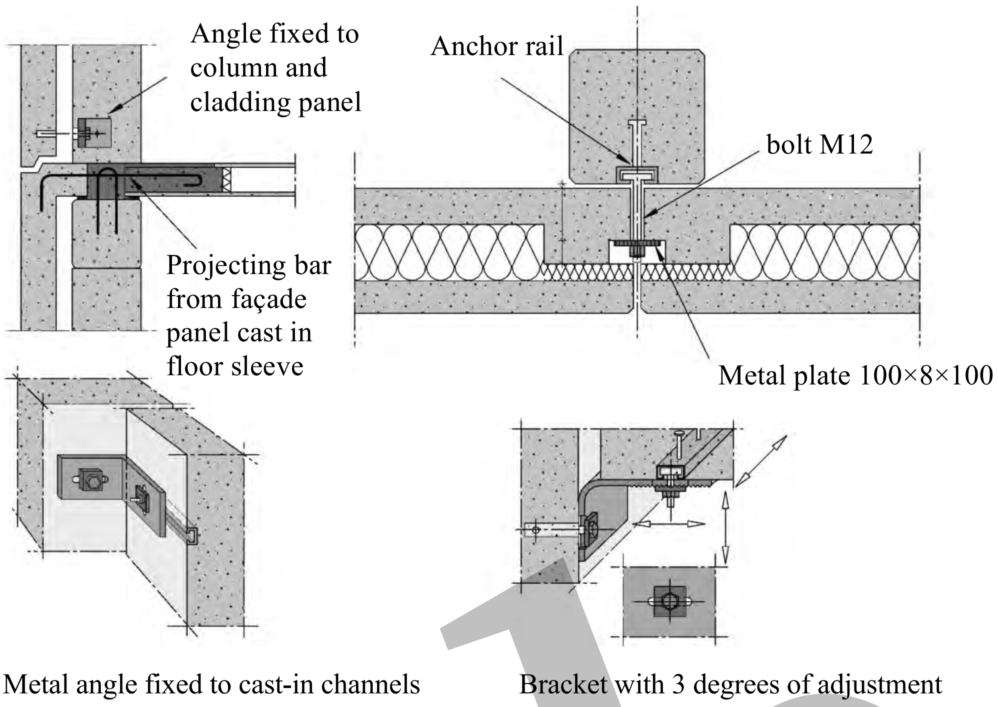
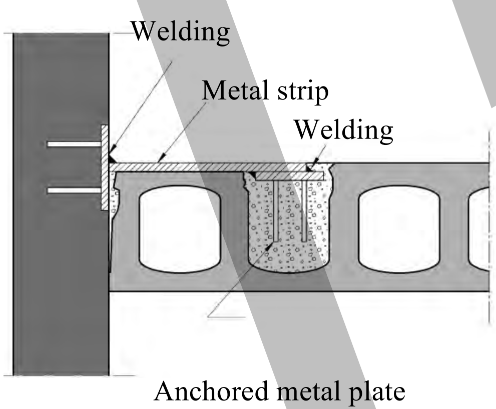


Fig. 9.24: Examples of bolted connections

The bolted connection is demountable and provides immediate fixity. However, to overcome constructional deviations, tolerances in all three dimensions must be accommodated.

- Welded connections

Welded connections are frequently used in the USA and Canada but more seldom in Europe. They are efficient and may easily be adjusted to varying field conditions. Their strength and reliability depend on workmanship. In Europe the more stringent regulations for welding on site together with the risk of bad weather conditions limit this application.



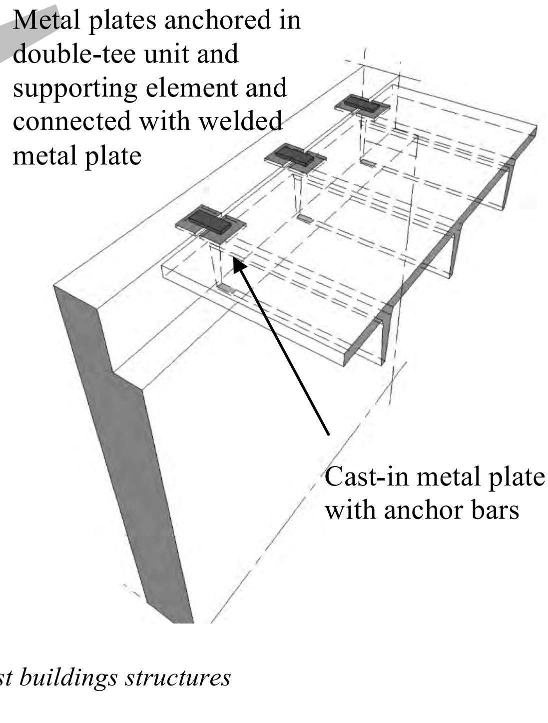


Fig. 9.25: Welded fixings

Anchor plates are widely used for welded connections in combination with flat metal straps, reinforcing bars or metal studs welded to the plate. The exterior surface of the plate is normally flush with the concrete face and provides a welded area for connection to the support system.

Cantilevering balcony units in architectural concrete can be anchored to the floor structure with systems that are able to break the so-called thermal bridge with the floor. Top reinforcement and compressive studs below take up the cantilever action.

9.9.2 Designing to cope with differential movement

The consulting engineer should advise the precast concrete panel designer of the predicted structural movement, namely, the shortening of the columns and the deflection of the beams. The thermal movement of the precast concrete panel should be predicted by the designer/manufacturer of the panel. Load-bearing fixings may have to cope with differential thermal movement only.

To cope with this movement, the fixing must safely hold the panel in position and not be structurally affected. PTFE washers, spacers, and oversized holes are typical means of dealing with this movement.

9.9.3 Tolerances

All fixings must allow an adjustment in three directions. Slotted or oversized holes are chosen to cope with the following deviations:

- Tolerance required for the location of the socket or channel in the precast panel
- Tolerance required for the location of the channel cast into the structure and the deviation in the position of the structure and post-drilled fixings.

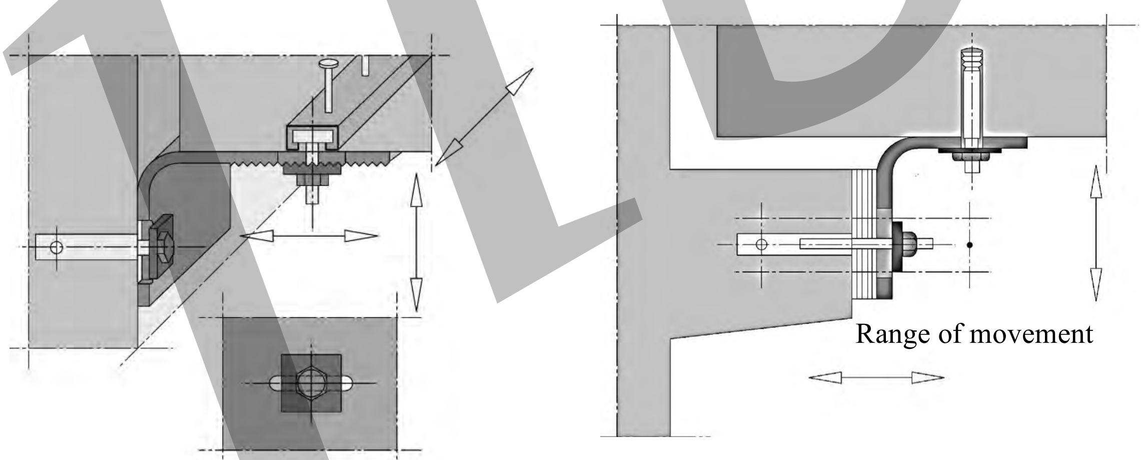


Fig. 9.26: Brackets with three degrees of adjustment

Packing shims are generally included to cope with tolerances; only the right sized shims and no more than the specified maximum number per fixing should be installed.

9.9.4 Durability

Once most fixings of architectural concrete cladding are installed they can no longer be inspected. As well as meeting requirements for mechanical strength and ductility, exposed fixing materials should have sufficient resistance to physical and chemical deterioration.

The methods and metals used depend on the importance of the fixing and how accessible it is for inspection. In a dry climate, fixings embedded in concrete only require the minimum concrete cover prescribed in durability standards.

When the concrete is exposed to weather and the thickness of the concrete cover is too low to guarantee that it will be waterproof, the fixings must remain free of corrosion.

Load-bearing and restraint fixings that are not embedded in concrete are generally produced from one of the following corrosion-resistant metals: copper, aluminium bronze, phosphor bronze or stainless steel. The AISI Ti 316 (American standard equivalent to DIN 12371) and AISI 316 types can be used.

Care should be taken to avoid harmful bi-metallic contacts that could cause galvanic corrosion. Danger from bi-metallic contact can be avoided by the use of insulating washers and sleeves.

In addition to the requirements for stability and durability, a number of construction criteria are also very important for the design of good connections.

- Adopting, whenever possible, the same type of anchorage for all the cladding and avoiding dimensions that are larger than necessary. Serial work saves costs and improves execution.
- Preferably placing projecting bars on the upper side of the mould to avoid issues with demoulding.
- Ensuring that fixings take into account the structural tolerances of the building and of the precast units during manufacturing.
- Allowing for the three-way adjustment of all fixings, of whatever type, so that panels can be adjusted and levelled.
- Allowing for adequate clearance between the cladding and the structural elements, normally measuring a minimum of 25 to 30 millimetres. These clearance gaps will usually not be visible in the finished building and can thus be made as large as practical consideration demands, within reasonable limits.

9.10 Drip grooves

Precautions should be taken for the removal of run-off water from the façade elements. Backward sloping surfaces, for example, unless they are designed not to be subjected to water flow, have a tendency to greater streaking because they usually attract partial water flow from above. To this end, a drip groove should be provided. The groove protects the soffit from being stained but, far more importantly, it stops the water from causing random staining on the protected wall below.

Drip grooves should be located in such a way that sound concrete can be introduced into the space between the panel face and the drip. The section of the drip groove must allow mould stripping without damage either to the mould or the drip. Drip grooves should be a minimum of 20 millimetres, preferably 25 millimetres, wide and should form an angle of at least 40-45° with the soffit at the front arris. Otherwise, water is liable to be blown across in high wind conditions.

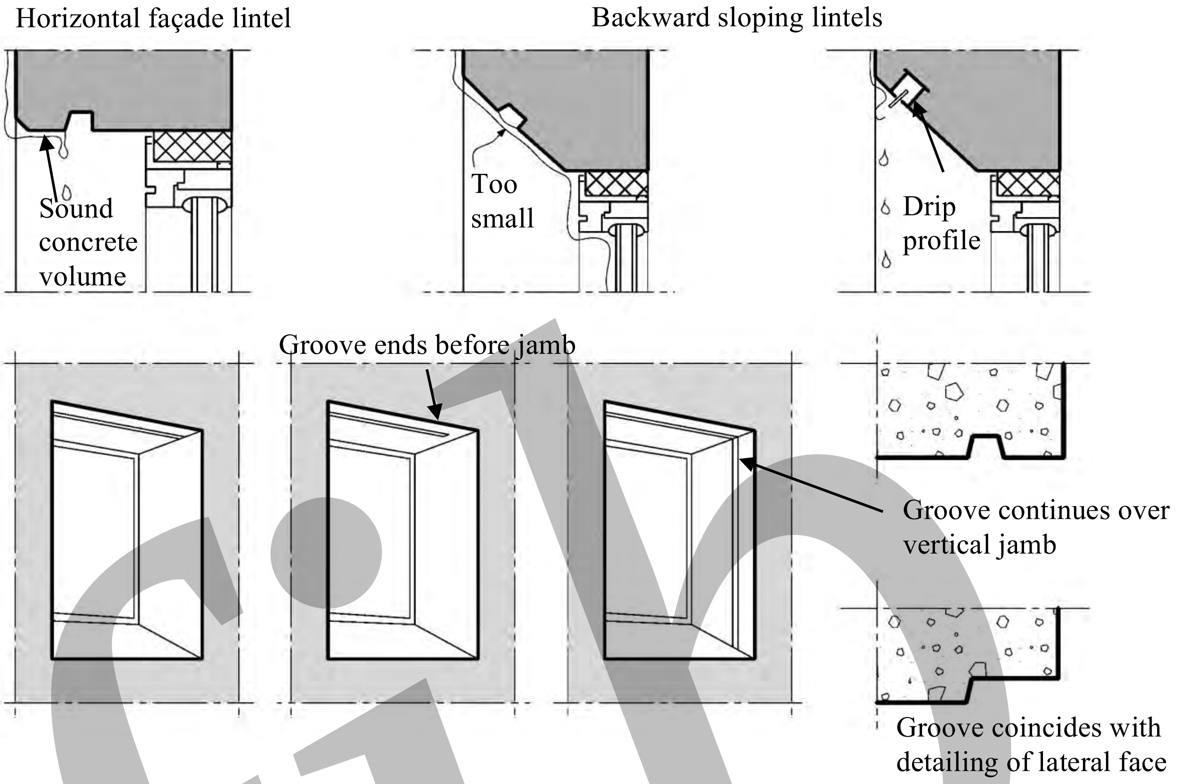


Fig. 9.27: Guidelines for positioning of drips for the removal of dirty rain water from the façade

It is advisable to detail a drip groove at the head of a window to prevent water from running down the glass and causing stains. The drip groove detail should trace the complete water path. If the groove continues to the vertical faces of the window opening, there is a danger that the dirty water will cause stains on the abutting vertical wall. In such a position, it is advisable to stop the groove approximately 50 millimetres from the wall. An alternative is to have a groove larger than the drip groove in a corresponding position on the jamb to contain and guide the water and conceal any staining it may cause.

9.11 Weathering joints

9.11.1 Appearance of weathering joints in façade

Joints are inherent to precast structures. Just as with natural stone cladding, the joints have to be seen as a logical step in the design of precast architectural concrete cladding. Various alternative solutions are available to obtain a good aesthetic outlook, for example, false joints and very pronounced or more discreet joint profiles.

Another approach consists of concealing the joints in the façade. Various ways of doing this exist. Joints are far less obvious when they form a corner or when they are situated at the dividing point between two different materials. Another technique, among others, consists of interrupting continuous joints by staggering the elements or inserting decorative elements over the joints.

There are many good examples of joints being incorporated seamlessly. This does require studying the elements thoroughly at the design stage, which is often neglected. Usually, only the minimum dimension for waterproofing is taken into consideration, with little concern for the visual aspect.

The examples below of buildings in Finland and Germany show how joints can be hidden very effectively.

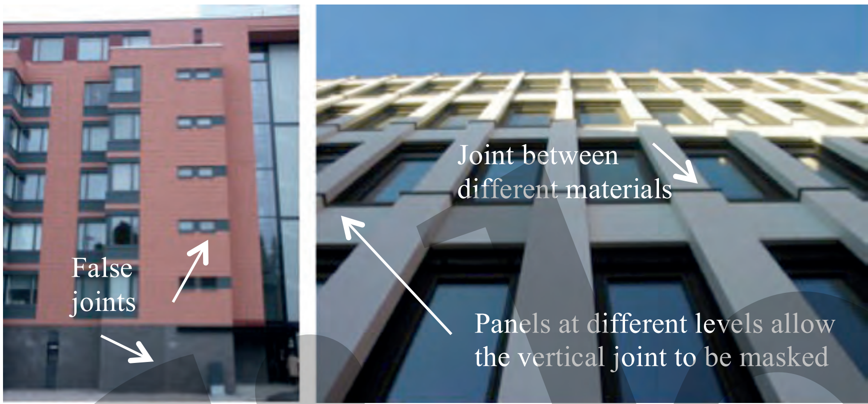


Fig. 9.28: Some architects do not accept joints between the panels

9.11.2 Design of joints

In precast concrete cladding systems weather tightness is dependent on the effectiveness of seals at joints between the individual panels and between the precast cladding and other elements of the façade. The main function of the joint sealant is to provide a flexible weatherproof connection between the panels that allows for movements to occur at the joint as a result of the thermal and moisture expansion and contraction of the panels themselves, along with movements in the whole structure.

The shape and dimension of the joint profile should be designed in such a way that it does not constitute a local weak point in the elements, creating a risk of damage to the corners. Joints that are too complicated in shape are also difficult to construct and may reduce the watertightness of the structure.

9.11.2.1 Types of weathering joints

Two types of watertight joints have been used successfully

- Face-sealed joints: in this type of joint the penetration of both air and water is prevented by a single seal close to the face of the panels. The most common and, generally, most effective method is to use gun-applied polysulphide or silicone sealants. The seal material is intended to bond to the sides of the joint and should be sufficiently flexible to accommodate the movement that occurs without splitting or loosing adhesion. The joint width is developed from a consideration of the maximum and minimum width to be encountered in practice (tolerances and movement) and the movements that will take place.

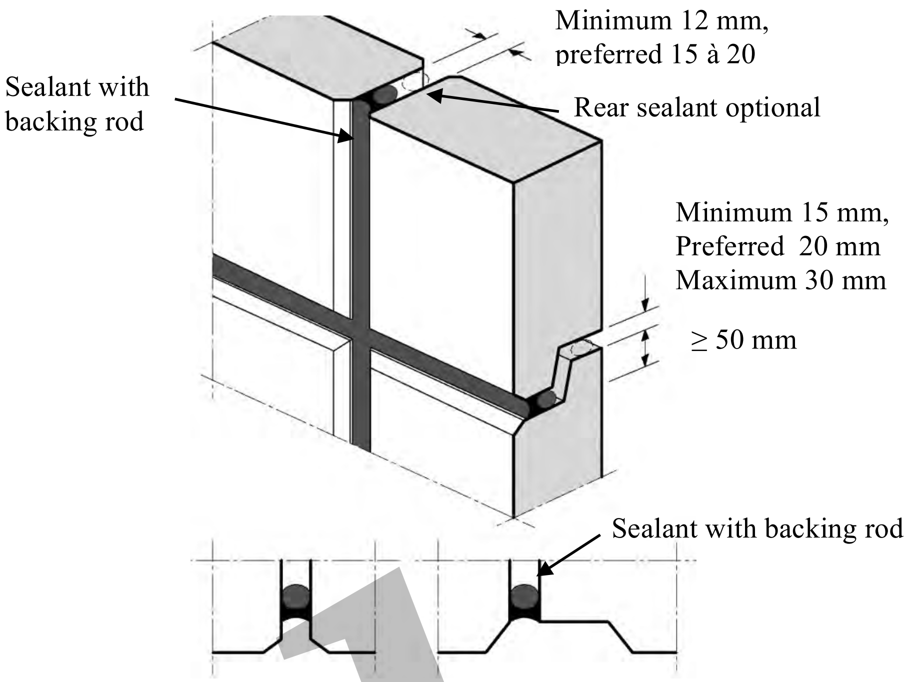


Fig. 9.29: Examples of face-sealed joint

There are no issues with seal continuity when gun-applied sealant materials are used. Since the product is installed in a paste-like consistency, no variation in joint size or defects in the joint edges occur. Once the sealant has cured or set, it provides a continuous barrier exactly fitted to the joint shape and bonded to the edges.

Face-applied seals allow freedom of design for panel shapes. The edge profiles of panels can be very simple. Normally, face seals are applied when the fixing of the cladding has been completed. External access is required for the application of the seals, and joint faces need to be dry and free of frost.

Setting the seal slightly back in the joint to obtain better protection against wind, rain and ultraviolet light is recommended. The effectiveness of these joints depends on the continuity of the adherence to the concrete and the elasticity of the sealant material. This implies that the joint edges should be regular and plain, and certainly not washed out with retarder or similar additives.

Since they are directly exposed to weather, sealant materials will change over the years and some maintenance and/or repair must be anticipated. However, since the seal is near the face of the panel, inspection and maintenance can be carried out easily.

- Open-drained joint or two-stage joint (Fig. 9.30): This type of joint deals with air and water penetration in two ways. A first barrier prevents the large-scale penetration of water so that only limited amounts will pass to the zone between the primary barrier and the air seal at the back of the joint. The protection afforded by this first barrier blocks the dynamic wind force from projecting water into the air seal. The air seal plays a vital role in joint function and its deterioration could lead to water infiltration. For this reason, using gaskets or foam strips may be unsuitable because of the risk of discontinuity at the junctions or of gaps occurring due to defects in joint surfaces, allowing pathways for air and water penetration. Gun-applied sealants provide the most secure air seal.

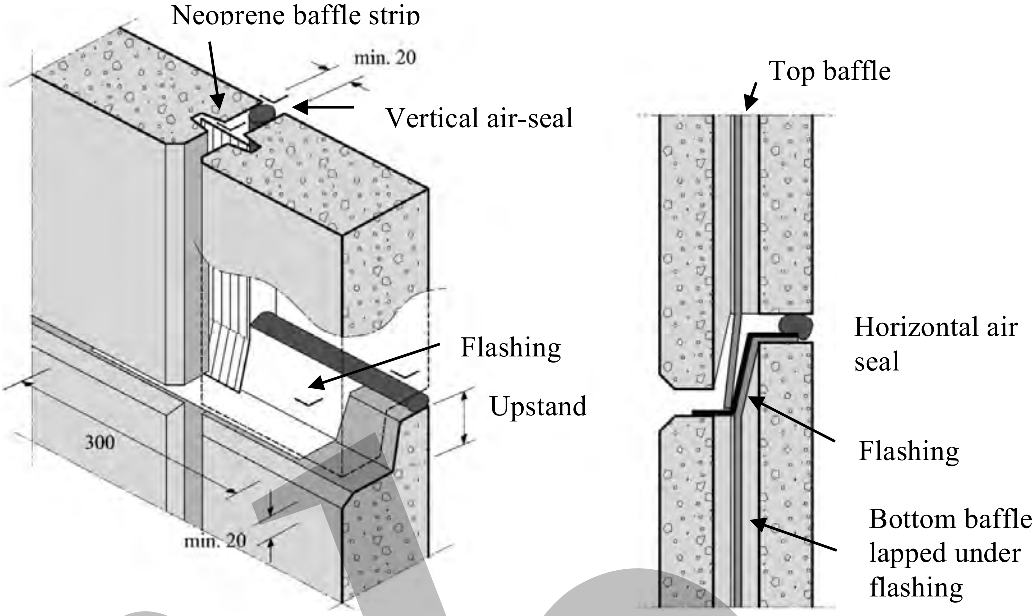


Fig. 9.30: Open drained joint

The first barrier in the vertical joint comprises an expansion chamber to slow down the wind pressure and a baffle to conduct the rainwater downwards. The horizontal joint is arranged with an overlap of panels to provide the primary barrier. The height of the upstand should be 50 to 70 millimetres to prevent water penetration by wind pressure. Flashing is placed over the vertical joint at the intersection of the vertical and horizontal joint. The air seal is placed at the back of the joint.

In open-drained joint systems, the different jointing stages occur during erection and are not always easy to perform. Special care is needed at the intersection of vertical and horizontal joints because weak points in the system exist. The inclined grooves in the joint edges should be sufficiently parallel to allow for the insertion of the baffle. Movements due to variations in the temperature and hygrometry of the panels generally do not affect the effectiveness of the joints. The inspection of open-drained joints is rather difficult, especially at the intersection of horizontal and vertical joints. Repairing leaking joints is also not easy. Flashing at the intersection of horizontal and vertical joints is mostly inaccessible and, thus, irreplaceable. The same applies to baffles. Repairs to those parts of the joints that are accessible from the outside may be possible with sealant or adhesive strip flashing.

To date most of the weathering joints in façade cladding are executed with face-sealed joints. In principle, these joints are more susceptible to ageing than open-drained joints, because the seal is exposed to wind, rain and ultraviolet light.

The combination of both above systems can also be used to advantage. The horizontal joint is then arranged with a continuous upstand and an air sealant. The joint can be left open or be face-sealed. The vertical joint is always face-sealed (Fig. 9.31). Face sealant joints eventually require maintenance.

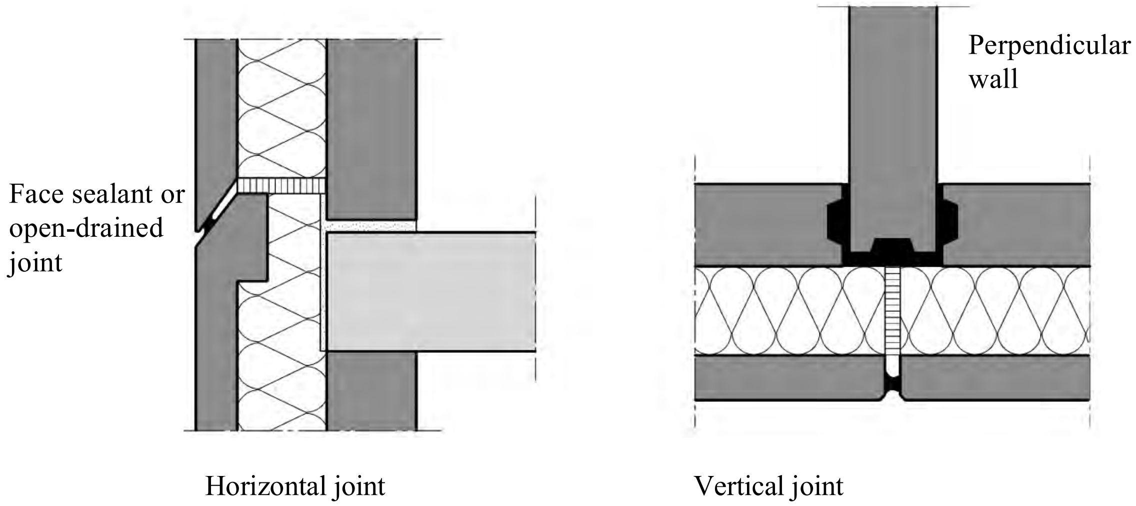


Figure 9.31: Typical joint configuration in sandwich façade cladding

In twin-skin façade systems, the weatherproof joints are sometimes placed at the level of the interior façade skin before the exterior cladding is erected. The joints between the exterior cladding units are then left open. The insulation material has, of course, to be watertight.

9.11.2.2 Joint width and filling

In order to design the correct joint width for a particular sealant it is necessary to know the amount of movement that will occur at the joint and what tolerances are allowed in the manufacture and fixing of the components making up the joint.

Commonly, the main reason for movement is the expansion and contraction of the components resulting from changes in temperature, moisture and shrinkage. The influence of temperature on the concrete cladding depends on the colour of the concrete and the orientation of the façade. A good elastic sealant can deform up to 25 per cent.

To allow for the correct application of the sealant, the actual joint width should be a minimum of 8 and a maximum of 30 millimetres. The table below gives an indication of the minimum nominal joint width in function of the width of the element for common types of sealant (polysulphide, polyurethane or silicone sealants). More information on joint types and sealants is available in *Precast concrete cladding* [55].

Table 9.4: Recommended joint width and depth for face-sealed joints

Component width (m)	Minimum nominal joint width (mm)	Minimum joint depth (mm)
1.80	12	8
2.40	12	8
3.60	14	8
4.80	15	10
6.00	16	10

10 Constructional detailing and dimensional tolerances

10.1 General

Prefabrication involves using specific solutions for a number of construction details such as supports, corbels, openings and cutouts in the units. They should be adequately detailed at the design stage. To this effect, it is very important to get a good insight in the force transfer in these parts of the construction since it constitutes the basis for the good dimensioning of these details and the arrangement of the reinforcement. In this chapter the design of the most important construction details in prefabricated structures are discussed.

10.2 Support connections

10.2.1 General requirements

The integrity of bearings for precast members shall be ensured by:

- effectively reinforcing the elements above and below the bearing
- preventing loss of bearing through movement
- suitably limiting bearing stress

Unless sliding bearings are used, the load-bearing capacity of supports for precast units can be seriously affected by the splitting or cracking of the connected components due to horizontal forces. These may originate from shrinkage, creep and thermal deformations. They could also result from faults during erection, inclinations, and so forth.

Where large rotations are likely to occur at the edge of flexural member supports, suitable bearings capable of accommodating these rotations should be used. The rotations may also throw the line of action of the loads on to the extreme edges of the bearings, especially when hard supporting pads are used. In such cases allowance should be made for the consequent increase in bending moments or local bearing stresses.

Bearings shall be dimensioned and detailed in order to ensure correct positioning, accounting for production and assembling tolerances.

10.2.2 Support length

The nominal support length of a simply supported precast member is the sum of the net bearing length plus all applicable tolerances. A distinction should be made between isolated and non-isolated members. Isolated members are those for which, in case of failure, no alternative means of load transfer is available, for example a simply supported beam. A typical example of a non-isolated bearing is a floor where a local weakening of the support capacity may be compensated by the transverse load distribution of the whole floor structure.

The net bearing length is determined by the allowable stresses in the contact zone between the supported and the supporting member. Supporting pads are used to even out the contact stresses. The supporting pad should be positioned at a certain distance from the edge of the supporting construction to avoid the splitting of the edge. The same provision should be taken to avoid the splitting of the lower corner of the beam end. In addition, allowance should be made for tolerances on the length of the supported member and on the distance between the supporting construction at both ends of the element.

According to Eurocode 2 Part 1-1 [2], the nominal length a of a simple bearing, as shown in Figure 10.1, is determined by the following equation:

$$a = a_1 + a_2 + a_3 + \sqrt{\Delta_2^2 + \Delta a_3^2}$$

where

a_1 is the net bearing length with regard to the bearing stress

b_1 is the net bearing width

a_2 is the distance assumed ineffective beyond the outer end of the supporting member. The usual value of a_2 ranges from 10 to 15 mm for linear supports (floors) and from 10 to 25 mm for concentrated supports (beams).

a_3 is the distance assumed ineffective beyond the end of the supported member. The normal value ranges from 5 to 15 mm.

Δa_2 is an allowance for tolerances for the distance between supporting members

Δa_2 = span length / 1200

Δa_3 is an allowance for the length of the supported member. Δa_3 = member length/2500.

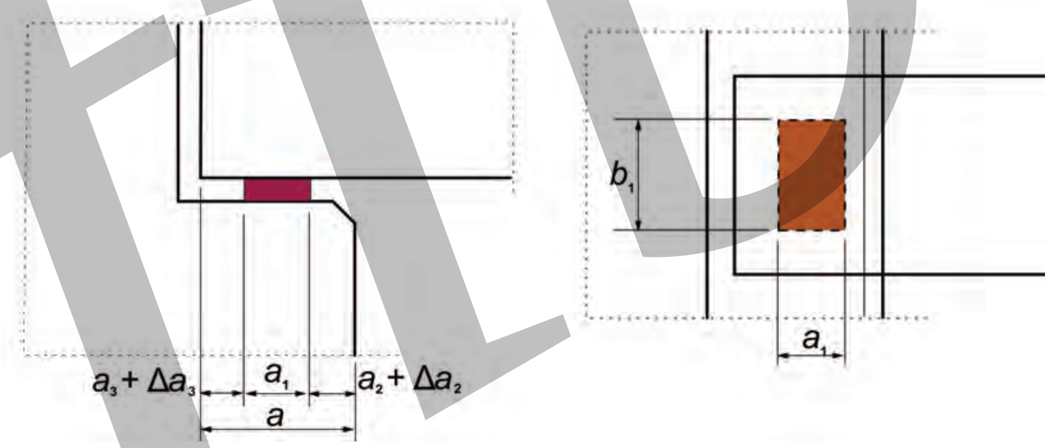


Fig. 10.1: Example of bearing

The nominal support length of isolated members should be 20 mm greater than for non-isolated ones. The following table gives indicative values for the nominal support length "a" for simply supported precast floors and beams in relation to the material of the supporting structure. The enlargement of the support length for isolated members has been taken into account.

Table 10.1: Indicative values for minimum nominal support lengths

Supported element	**Supporting structure**	**Slab thickness *h* or beam length *ℓ***	**Minimum nominal support length**
Hollow-core floors	concrete/steel	$h \leq 250$ mm $h > 250$ mm	60 - 70 mm 100 - 130 mm
	masonry	$h \leq 250$ mm $h > 250$ mm	100 mm 120 mm
Floor planks	concrete with propping without propping	- 	30 mm 50 mm
	masonry with propping without propping	-	40 mm 50 mm
Ribbed floors	concrete	$\ell \leq 15$ m	150 mm
Secondary roof beams	concrete	$\ell \leq 8$ m	140 mm
Floor beams	concrete	$\ell = 12 - 20$ m	200 - 230 mm
Roof beams	concrete	$\ell \leq 24$ m $\ell \leq 40$ m	195 mm 225 mm

10.2.3 Half joints

Half joints are used to limit the total construction depth of beams and ribbed floors. Half joints may be designed using strut-and-tie models. The Eurocode 2 recommends two alternative models (Fig. 10.2). A combination of the two models is usually applied for precast concrete components. The reinforcement is dimensioned according to model *a*, but additional inclined stirrups are placed to limit the cracking in the corner of the cantilever.

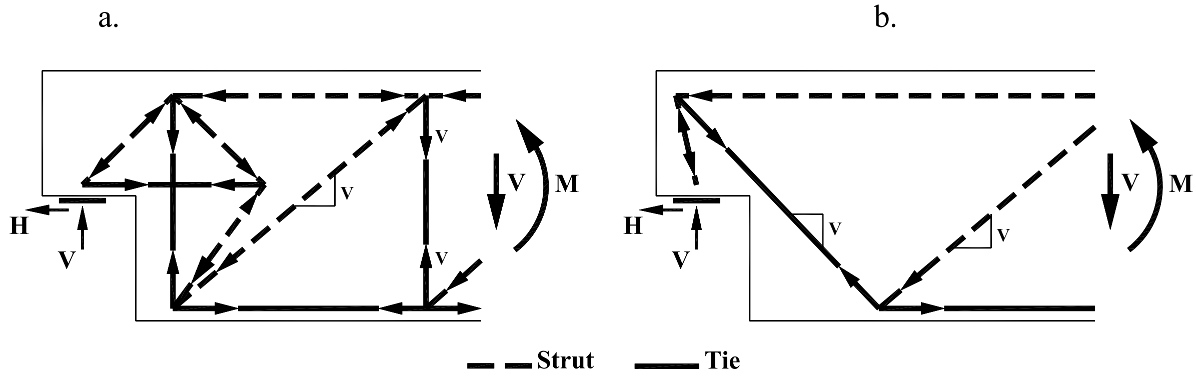


Fig.10.2: Indicative models for reinforcement in half joints

Figure 10.3 illustrates good reinforcement detailing for half joints in beams. A_s takes up the tensile component resulting from the bending moment in the cantilever and the horizontal force in the support connection. A_{sb} are horizontal stirrups to take up transverse stresses from the support reaction and forces from the dowel action in a pinned connection. A_{sa} are vertical stirrups at the end of the beam just before the cantilever. They are designed to transfer the vertical support action to the longitudinal tensile reinforcement in the beam. They are also taking up the transverse tension forces from the transfer of prestress at the end of the beam, together with the horizontal stirrups A_{se}. Finally the inclined bars $A_{s\alpha}$ are placed in the inner corners of the half joint to limit the possible crack width and to prevent the possible corrosion through cracks. All these reinforcing bars should be adequately anchored.

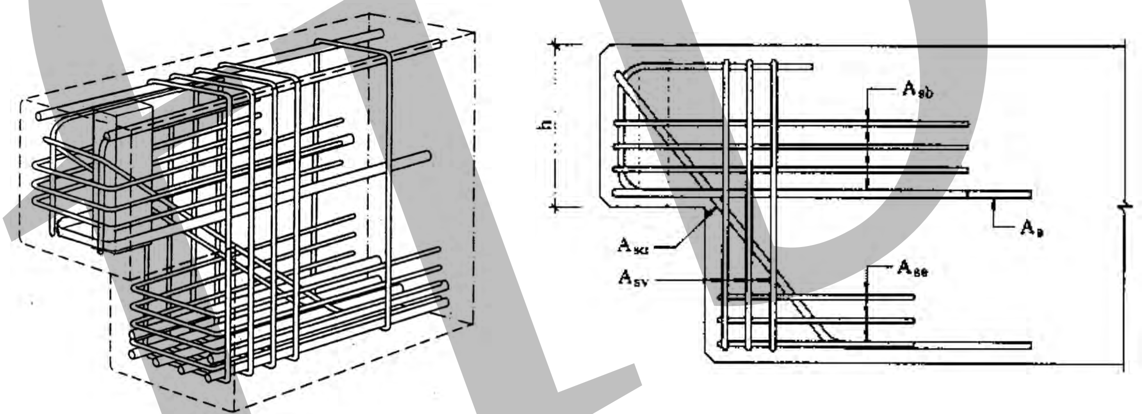


Fig. 10.3 Good reinforcement detailing in a half joint

The above detail is also applicable for both parts of half joints in a beam-to-beam connection, as illustrated in Figure 10.4.

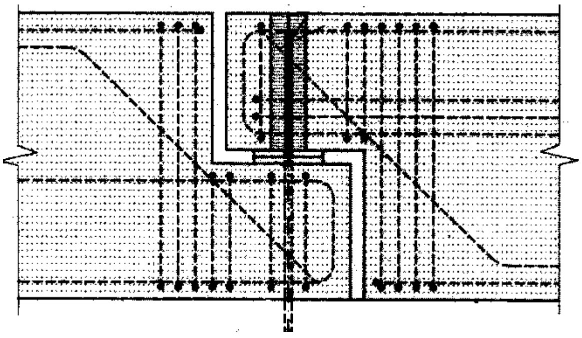


Fig. 10.4: Symmetrical half joints in beam connections

10.2.4 Beam boot support

There is a tendency to keep the boot height of L-shaped and inverted-tee beams as small as possible sa as to limit the total floor construction thickness. The normal height of a boot is 150 millimetres, but 100 millimetres also exists. Figure 10.5 illustrates the force transfer in an inverted-tee floor beam and Figure 10.6 gives an example of the detailing of stirrups.

In principle, the support reaction from the floors on the beam ledge must be transferred to the compressive zone of the beam. This can be done in the following way:

- The boot functions as a short cantilever that could be dimensioned according to the strut-and-tie method. The compression strut in the boot is supported by the prestressing tendons in the beam. The horizontal tie force of the strut is taken up by the transverse links in the boot.
- The vertical component of the strut force is transferred to the compression zone at the top of the beam by the vertical legs of the stirrups in the beam.
- The shear force from the floor load is taken up by the vertical concrete section of the boot.

Figure 10.6 shows examples of the arrangement of prestressing tendons and stirrups in inverted-tee beams.

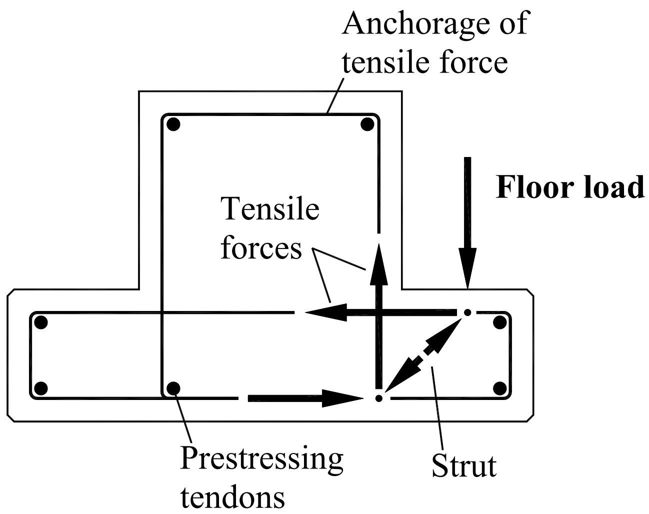


Fig. 10.5: Force transfer in the boot of an inverted-T beam

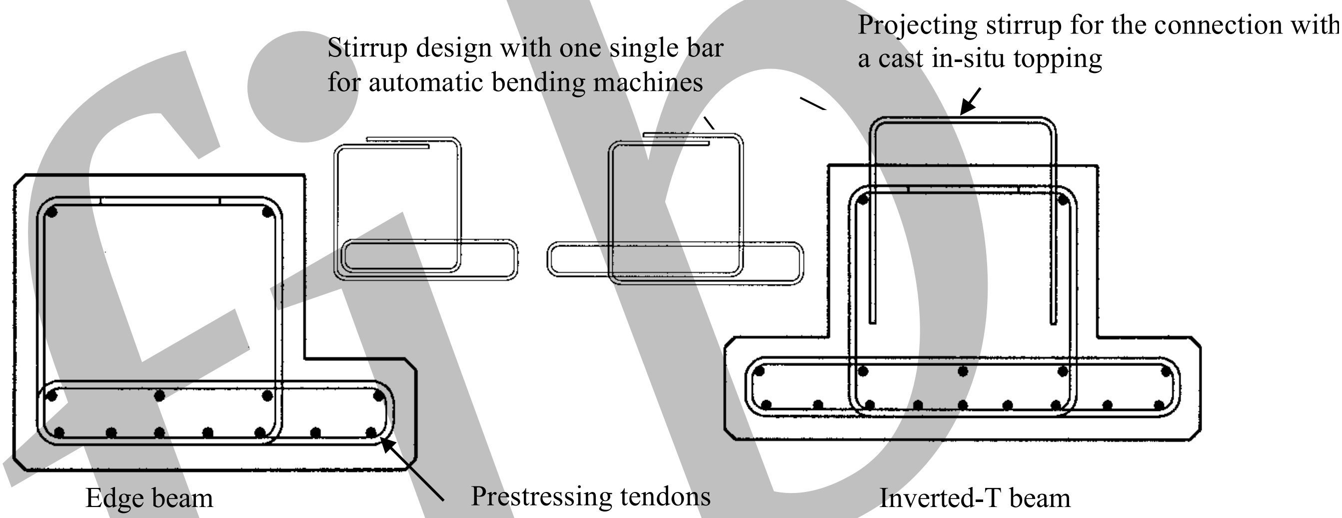


Fig. 10.6: Examples of detailing of reinforcement in boot floor beams

10.3 Concrete corbels

10.3.1 General

Corbels are used in prefabrication for beam-column and beam-beam connections, but also for floor-wall supports. Figures 10.7 and 10.8 show different types of corbels. Type *c* is difficult to produce because of the complex moulding and reinforcement in the column, and should only be used when absolutely necessary. The recommended alternative is to use 4 single corbels.

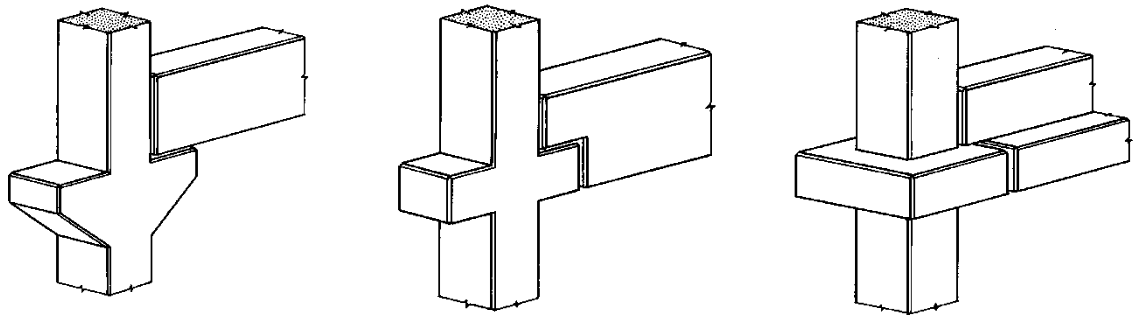


Fig. 10.7: Examples of column corbels

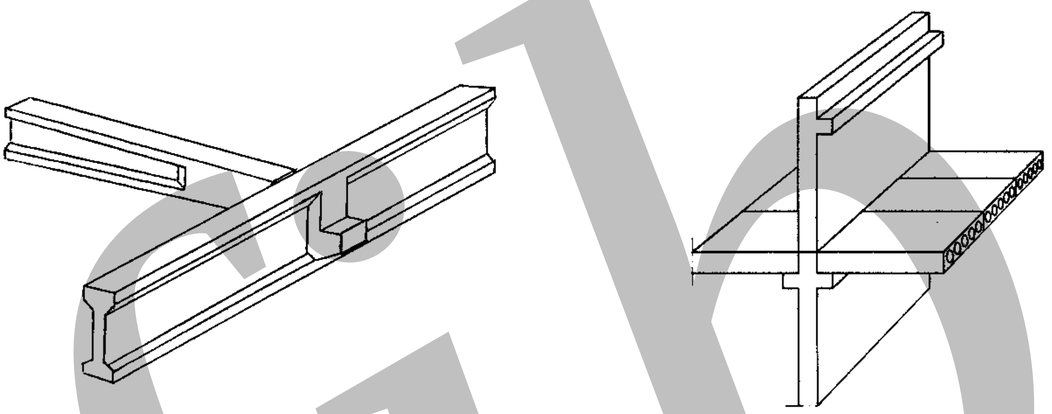


Fig. 10.8: Examples of beam and wall corbels

10.3.2 Corbel design

Recommendations for the dimensions of column corbels are given in Figure 10.9. The distance a_0 from the load point to the face of the support should not be greater than d, the effective depth of the corbel. The depth h_1 of the face of the corbel should not be less than half of the total depth. The length of the corbel l should not be larger than 0.7 h. For reasons of standardization, it is recommended to standardize l to 300 or 400 millimetres.

$a_0 \leq d$	recommended value: 0.4d to 0.6d
$h_1 \geq a_0$	minimum 0.5 h
$l < 0.7$ h	usually $l = 0.5$ h but preferably modulated to 300 or 400 mm

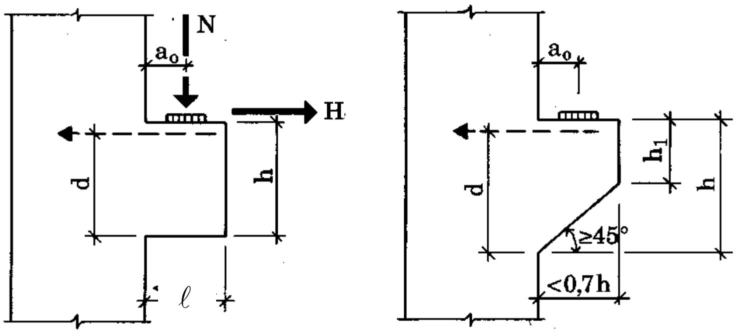


Fig. 10.9: Recommended corbel dimensions

Column and beam corbels are usually designed using strut-and-tie models, as described in Eurocode 2 EN 1992-1-1, section 6.5 [2]. The inclination of the strut is limited by $1.0 \leq \tan\theta \leq 2.5$ (θ Fig. 10.10).

In addition to the vertical loading N, the corbel shall also be designed for a complementary horizontal force induced by creep shortening, thermal expansion and contraction due to temperature changes, particularly in the case of long prestressed beams. In the absence of more detailed calculations, H may be taken equal to $H_d = 0.15\ N_d$. Other horizontal actions caused by overhead cranes or other actions, should be accounted for in the design.

Figure 10.10a shows the stress trajectories from a loaded corbel into the column. The corresponding stresses are taken up by an inclined compressive strut F_c and a horizontal tie force F_s. The width of the compressive strut is a function of the design compressive strength of the concrete. The tensile reinforcement should be duly anchored into the column. In the case of double corbels at opposite sides of the column, the tie reinforcement continues over the two corbels. For a single corbel, the main reinforcement should be fully anchored at the opposite side of the column and lap with the longitudinal column reinforcement. More detailed information about the arrangement and detailing of the reinforcement is given in Section 10.3.3.

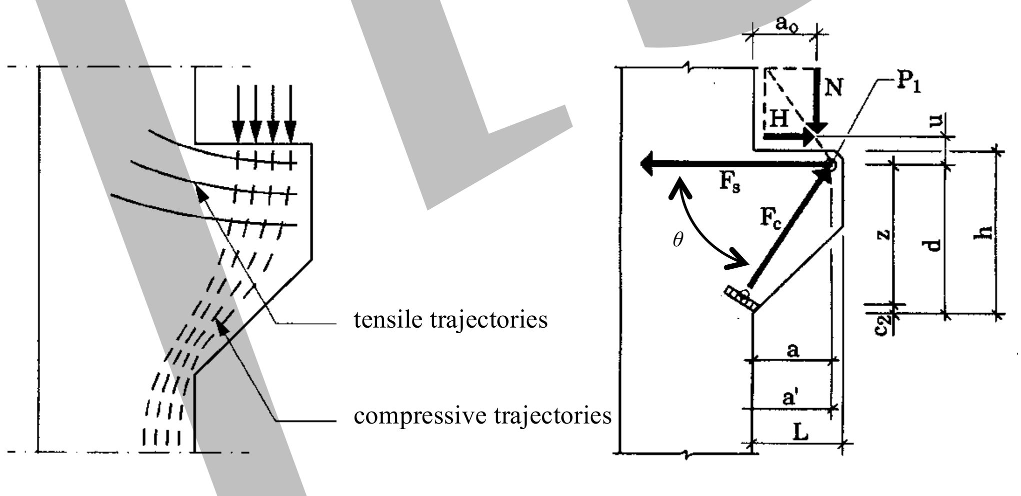


Fig 10.10: Stress trajectories and strut-and-tie model in concrete corbels

Wall corbels are usually designed as a short cantilever in bending and shear. If measures are taken to obtain a uniform distribution of the bearing pressure of large floor units, for example, with mortar, neoprene or similar pads, the design bearing width b_1 (Fig. 10.1) may be taken as the actual width of the bearing. Otherwise the support should be designed for a concentration of the bearing stresses over 600 millimetres per floor unit.

10.3.3 Detailing of corbel reinforcement

There are several critical points to be considered in the practical detailing of corbel reinforcement. A first problem concerns the short distance between the line of application of the vertical loading and the edge of the corbel. The tensile component of the strut-and-tie model should be adequately anchored beyond the load line. However, this can be difficult because of the large bend diameter of the reinforcing bars, which leaves the upper corner of the corbel unreinforced.

There are different ways of anchoring the main steel in the top corner of the corbel:

- by welding the main steel to a bar across the front face of the corbel (Fig. 10.11a);
- using a horizontal U-shape stirrup in the top of the corbel (Fig. 10.11b).
-

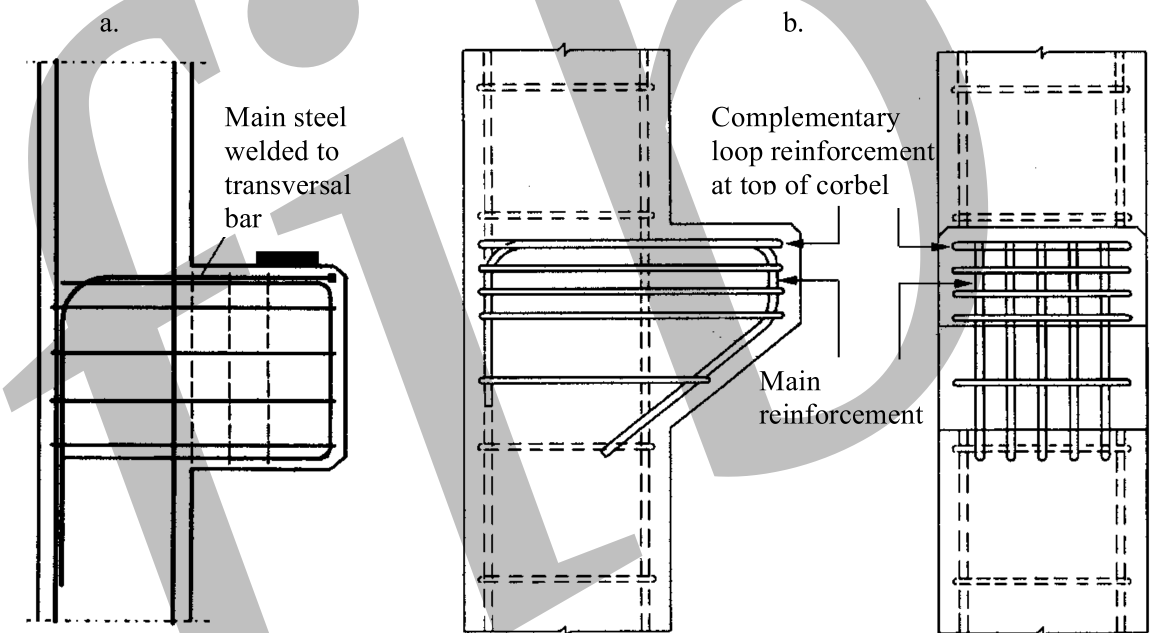


Fig. 10.11: Anchorage of main corbel reinforcement

A second possible complication concerns the reinforcement of multi-planar corbels, particularly if the level of the shoulder is at the same level in the two directions. The congestion of the horizontal tie back may be relieved either by varying the height of the top of the corbel by at least 50 millimetres or by casting in a special steel insert to transmit the tie forces as necessary.

A third problem is related to the bearing stresses in the bend of the reinforcement. Using small diameter bars, for example, ones that are smaller than 25 millimetres in diameter, can solve this. Elliott [41] stipulates that the disturbed region in the column may extend for a distance equal to about 1.5 h above the top of the corbel and 1.5 h cosθ below the root of the corbel. Additional links should be provided in the column in these regions.

A fourth problem with multi-planar corbels is that the mould must be specially shaped and built for each project. This is especially hindering when the units are cast on long-line production systems. When the corbels are only at one side of the column, they are positioned at the top face of the mould, thus enabling the use of classical standard mould equipment. However, when the corbels are at more than one face, the standard mould system has to be modified to insert the corbel moulds. To overcome this problem the columns may be cast in a two-step process. In step one, the column is cast without corbels other than at the top face of the mould. At the place of the other corbels, fully anchored threaded couplers are placed in the mould at the level of the tension reinforcement to receive the corbel reinforcement after the column has been stripped from the mould. To improve the transfer of shear stresses, an approximately 20-millimetre-deep recess is made in the column at the place of the corbel or a retarding agent is applied to expose the aggregate, but not to disturb it. In step two, the corbel reinforcement is inserted into the threaded couplers, together with the needed stirrups and the corbels are cast as a second stage, possibly a few days later.

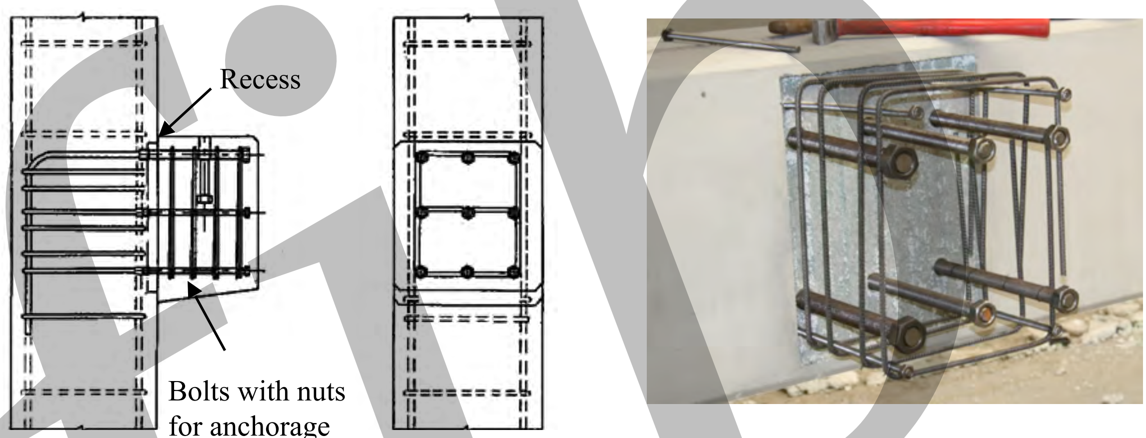


Fig. 10.12: Example of a two-step corbel

10.3.4 Hidden corbels

There is a growing tendency to use hidden steel corbels in beam-to-column connections. The advantage of the solution is that the intersection between beam and column is neat, without an underlying corbel. The connection is also attractive from an aesthetic point of view. Various solutions exist on the market, with some examples of possible solutions given in Figure 10.13.

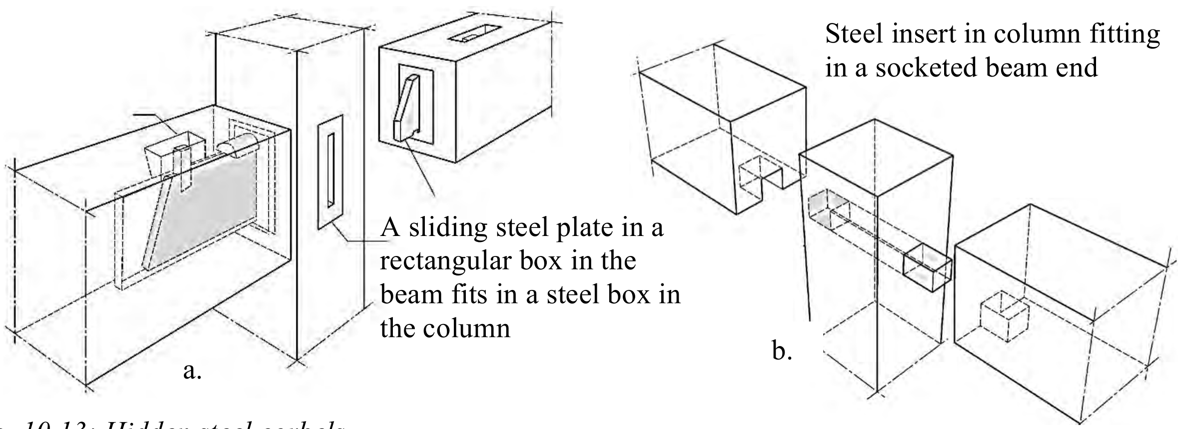
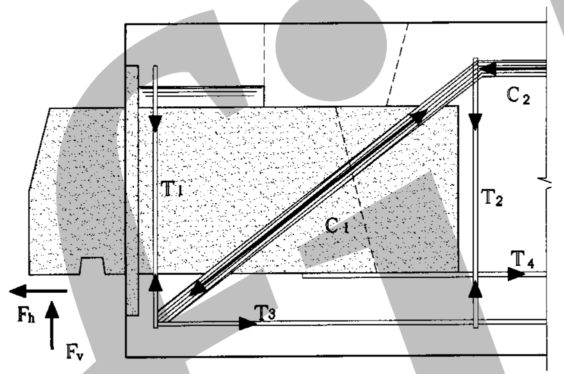


Fig. 10.13: Hidden steel corbels

The corbel with the steel billet is structurally efficient as well as being simple to manufacture and execute but is clearly visible below the beam sight lines and may require concealment in internal rooms. The simplified calculation is based on the strut-and-tie method (Fig. 10.14). Allowable bearing capacities are available in literature [8].



F_v	Vertical support load
F_h	Horizontal support load
T_1	Resulting tensile force at the front end of the beam
T_2	Resulting tensile force at the rear end of the beam
T_3	Resulting tensile force at the bottom of the beam
T_4	Tensile force caused by friction in the column
C_1	Resulting compressive struts on each side of the beam
C_2	Resulting compressive force in the top of the beam

Fig. 10.14: Strut-and-tie model for design of the corbel of Figure 10.13a

10.3.5 Beam corbels

Corbels in prestressed beams are often manufactured in two stages. In a first step, the beams are cast in standard moulds on long-line prestressing beds and the corbels at the side of the beams are cast in a second step. Just like for two-step column corbels, fully anchored threaded couplers are placed in the mould at the level of the tension reinforcement to receive the corbel steel after the beam has been stripped from the prestressing bed. For beams in reinforced concrete, corbels are directly provided in the mould, at least when the production series is large enough to compensate the additional mould cost.

It is of the utmost importance that the vertical reaction of the beam corbel is transferred correctly to the compressive zone of the beam. This can be achieved through stirrups runnning under the corbel and being anchored in the compressive zone of the beam (Fig. 10.15). On both sides of the corbel additional vertical links are placed in the beam over a distance corresponding to a possible crack at 45°.

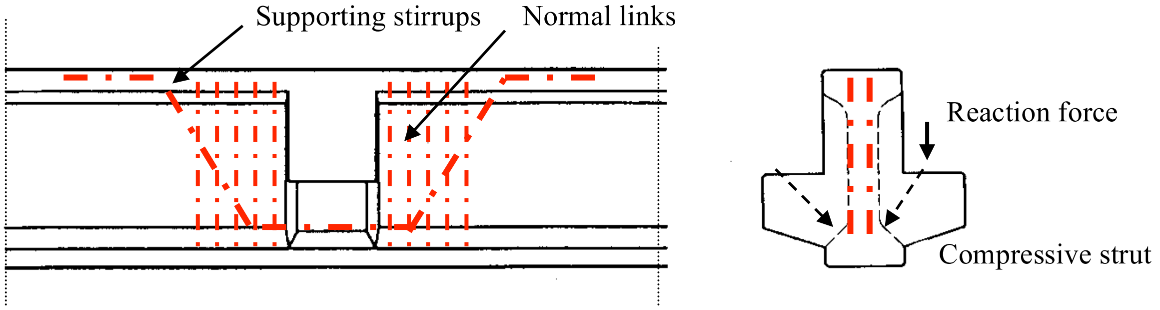


Fig. 10.15: Supporting reinforcement for beam corbels

Figure 10.16 shows a practical example of the reinforcement of a two-step corbel on a rectangular beam.

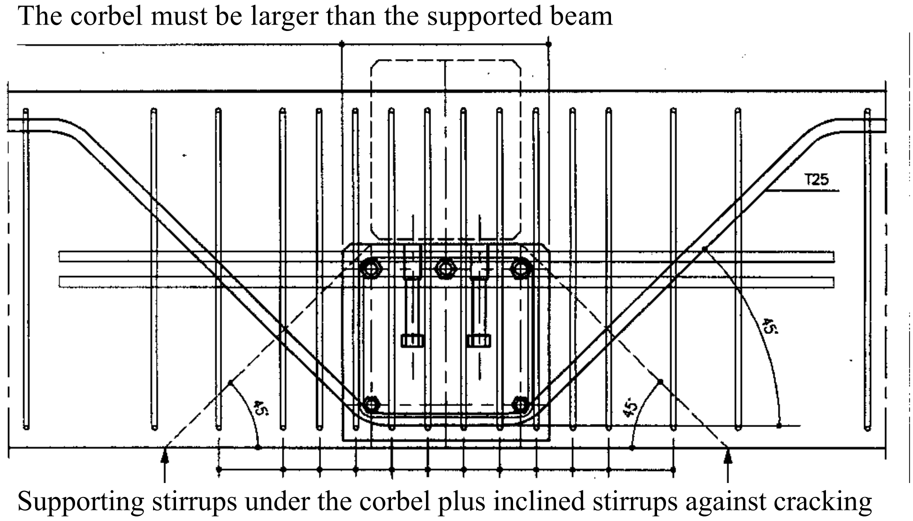


Fig. 10.16: Beam corbel reinforcement - front view

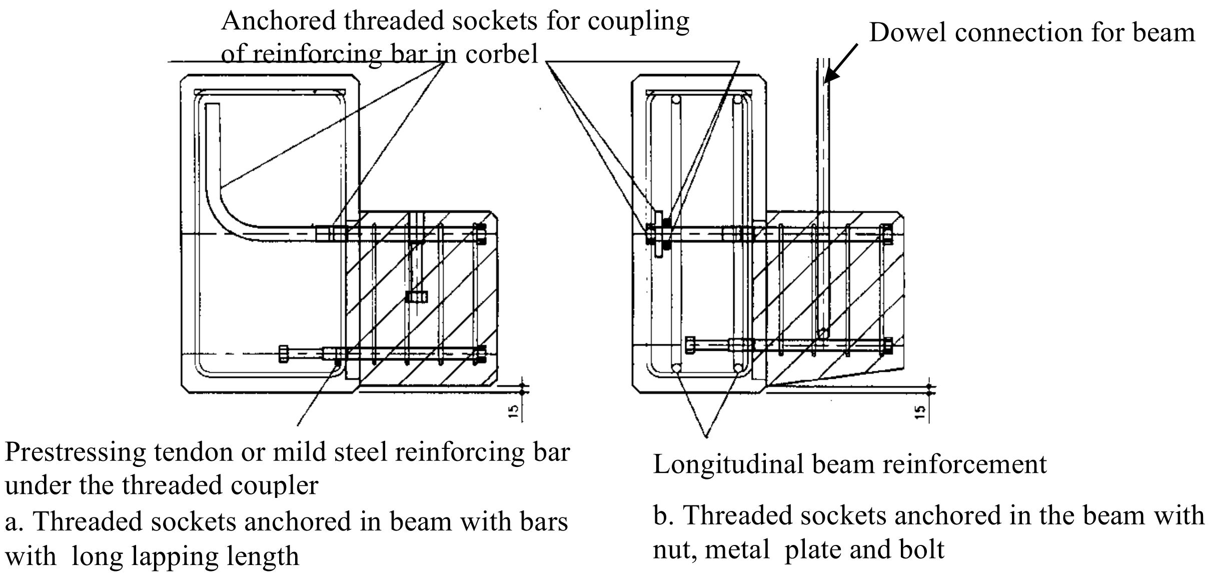


Fig. 10.17: Beam corbel reinforcement - cross section

10.4 Design of openings and cutouts

10.4.1 Holes and cutouts in beams

Large openings are sometimes required in the webs of beams or the ribs of double-tee slabs to enable the placement of ducts and pipes directly under the floor soffit. They should be designed carefully. The size of the openings is limited by the flexural and shear capacity at the place of the openings. The verification should be carried out for all the steps in production, transport, erection and service.

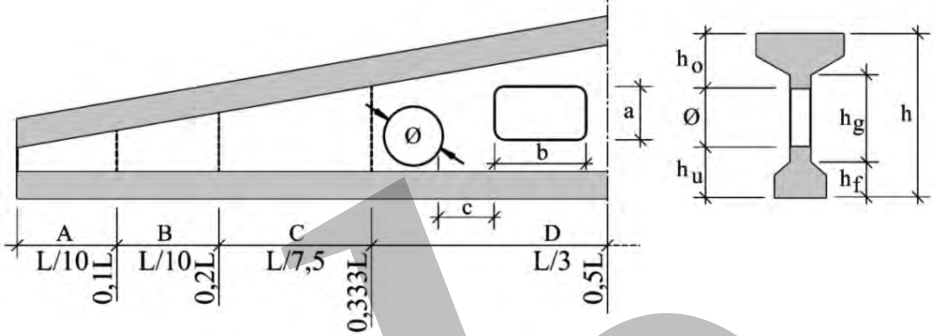


Fig. 10.18: Openings in roof beams

Normally, the remaining height of the cross section above and under the opening should be the least of 2/10th of the total beam height and 200 millimetres. The diameter of round openings should be limited to 2/3rd of the height of the beam web. Table 10.3 gives indicative values for the dimensions of openings in the web of a roof beam with sloped pans (Fig. 10.18).

Table 10.3: Recommended dimensions of openings in roof beams

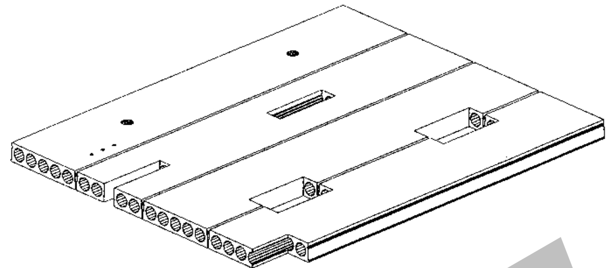
	A	B		D
Diameter round openings	≤ 100 mm	< h/4	< h/3	< h/2
Rectangular openings a b	- -	< h/5 < h/4	< h/4 < h/3	< h/3 < h/2
Free distance c	≥ h		≥ 2 Ø of 2b	

For larger openings, it is recommended to consult the precaster. The calculation of the resistance of the beam above and under the opening can be made on the basis of a truss model or according to the Vierendeel theory.

10.4.2 Openings and cutouts in floors

Openings and cutouts in precast floors can be produced in different places and in a variety of sizes and dimensions. The points to be taken into account at the design are the stability of the units, the possibilities for manipulation, the visual aspect (rough or sawn edges) and the cost.

Cutouts in hollow-core floors are achieved in one or two ways depending on the size. Small holes of less than approximately 300/400 millimetres in size may be formed in the precast unit during the manufacture stage and before the concrete has hardened. 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. The dimensions are normally limited to the values given in the table.



Length/width (mm)	HC 180 - 300	HC 400
- Corner - Front - Edges	600/400 600/400 1000/400	600/300 600/200 1000/300
- Centre round holes square openings	 Core minus 20 mm 1000/400	 ⌀ 135 1000/200

Figure 10.19: Holes in hollow-core elements

Where the openings are too large to be incorporated within the hollow-core unit, trimmer angles or cast in-situ trimmer beams are used to carry the floors at the edges of large holes (Figure 10.20). The self-weight of the units is transferred to the adjacent units through the trimmer angles, whereas the superimposed loading is transferred through the grouted longitudinal joints, provided that adequate peripheral and internal ties enclose the whole floor. The floor units on either side of the hole must be reinforced sufficiently to carry the additional loading.

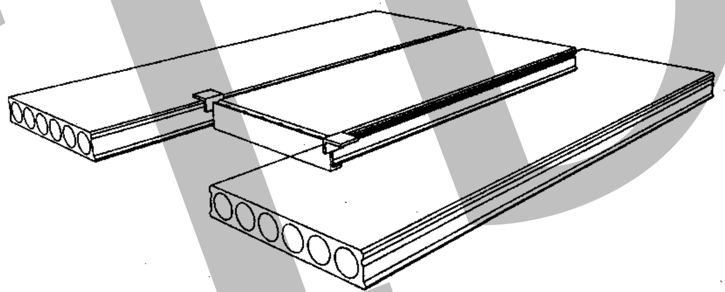
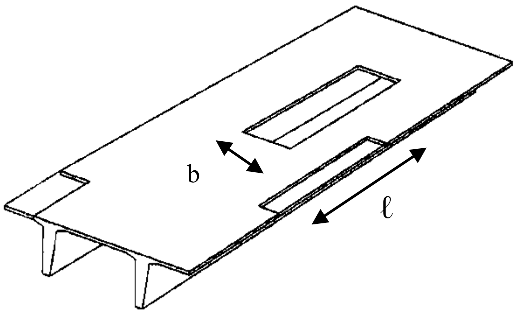


Fig.10.20: Use of trimmer angles or beams for large voids

One note of caution when using hollow-core slabs in wet and cold climates is that water may penetrate into the hollow cores during erection. If this water is allowed to gather and expand by freezing, for example, during the construction of the building, there is a possibility that the bottom flange will split off the unit. A simple remedy is to drill weep holes in the bottom of the slabs, usually during manufacture on the beds. They are situated at about 1 to 1.5 millimetres from each end of the units and during erection it should be verified that they remain open.

Holes in ribbed-floor elements may be formed in the positions shown in Figure 10.21. In no circumstances should vertical holes be formed through the webs of double-tee units. The circular holes through the webs are possible above the prestressing tendons to provide passage for services.



ℓ/b (mm)	TT-2400	TT-3000
- Centre	1000/630	1000/930
- Edge	1000/320	1000/460
- Corner	1000/320	1000/460

Fig. 10.21: Voids in 2.4-m and 3.00-m-wide double-tee units

10.5 Special reinforcement

10.5.1 Anchorage zones of prestressed components

The transfer of prestress causes tensile and splitting stresses in the anchorage zone of pretensioned components. These stresses are to be taken up by transverse reinforcement. Figure 10.22 gives a schematic presentation of the reinforcement in the anchorage zone of a prestressed beam.

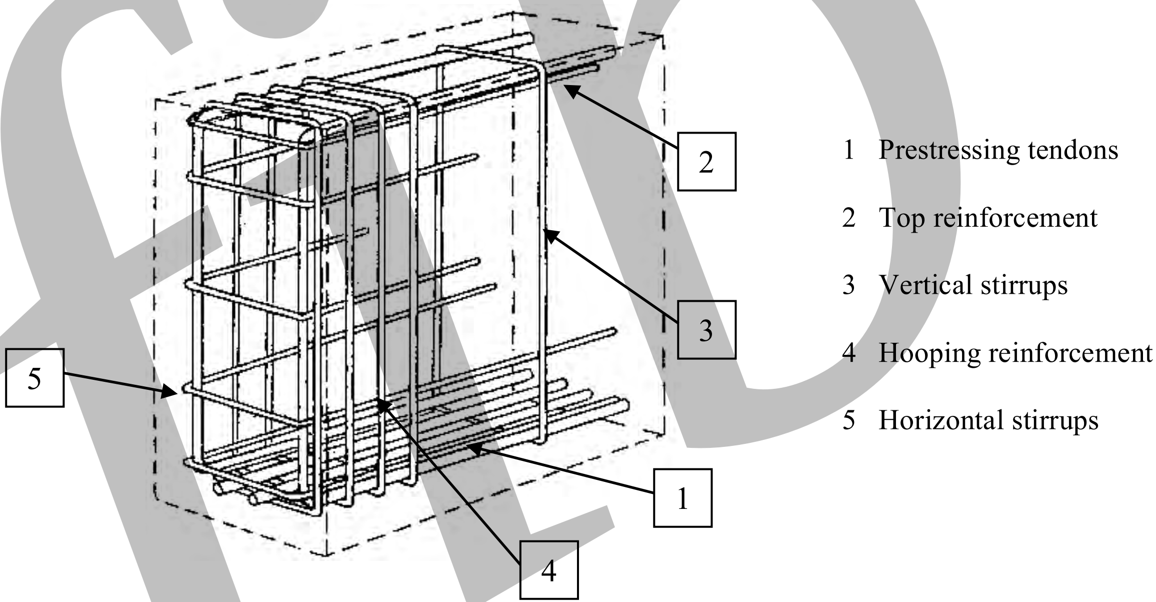


Fig. 10.22: Arrangement of the reinforcement in the end zone of a prestressed beam

The following types of reinforcement can be distinguished:

- Pretensioned tendons. The transmission and anchorage length can be calculated according to Section 8.10.2 of Eurocode 2 Part 1-1 [2]
- Top reinforcement to take up the tensile forces from:
  - The eccentricity of the prestressing force

- The cantilevering of the beam end with respect to the place of the lifting hooks during handling. Because it a temporary situation, higher steel stresses are allowed. The calculation may be made in the following way:

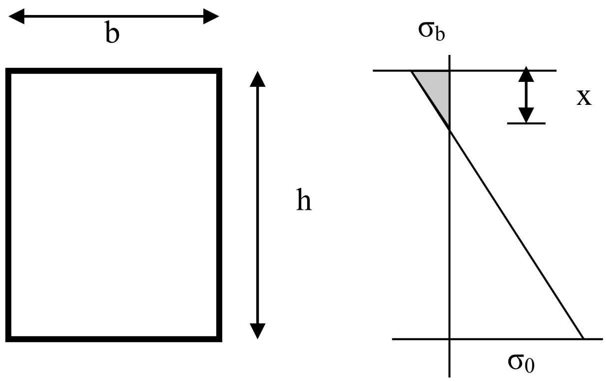


Fig. 10.23: Calculation model for handling reinforcement of a beam

$A_s = [b \,.\, x \,.\, (\sigma_b / 2)] / 300\ \text{N/mm}^2$

where b: width of the beam;
x: height of the tensioned zone;
σ_b: tensile stress in the upper fibre of the beam due to prestressing + cantilever action of the beam beyond the lifting hook.

- Vertical stirrups to take up the shear force in the beam
- Transverse reinforcement to take up the splitting stresses from the transfer of the prestress. The tensile forces due to concentrated forces could be assessed by a strut-and-tie model or other appropriate representation (see Section 6.5 of Eurocode 2 Part 1-1 [2]). The magnitude of the transverse reinforcement is in the order of 11 per cent of the total prestressing force.

$A_{sw} = 11\%\ x\ [N \,.\, A_p \,.\, f_{po}] / 300\ \text{N/mm}^2$

10.5.2 Transverse reinforcement of column ends

Columns should be provided with transverse reinforcement at the end to take up transverse tensile stresses due to:

- Concentrated loads at the support of beams
- Splitting stresses at compression connections
- Transfer of prestress in prestressed column
- Complementary horizontal forces due to creep and thermal deformations of long beams (see Section 10.2.2).

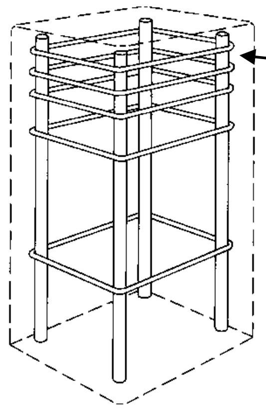


Fig. 10.24: Transverse reinforcement in columns

10.5.3 Walls and façade elements

Plain concrete walls should have a minimum area of construction reinforcement:

- Transverse reinforcement in the form of links (Fig. 10.25)
- Diagonal angle reinforcement against possible shrinkage cracks during curing/hardening of the components in the mould
  Window and door moulding frames may indeed hinder the shrinkage of the concrete around it and may cracking when the units remain in the mould for too long
- Stiffening of cantilevering parts of walls during handling and transport, for example, small doorposts
  Usually large reinforcing bars or steel angles are used to stiffen the cantilevers. After erection they are cut or removed, unless they are hidden in the floor finishing.

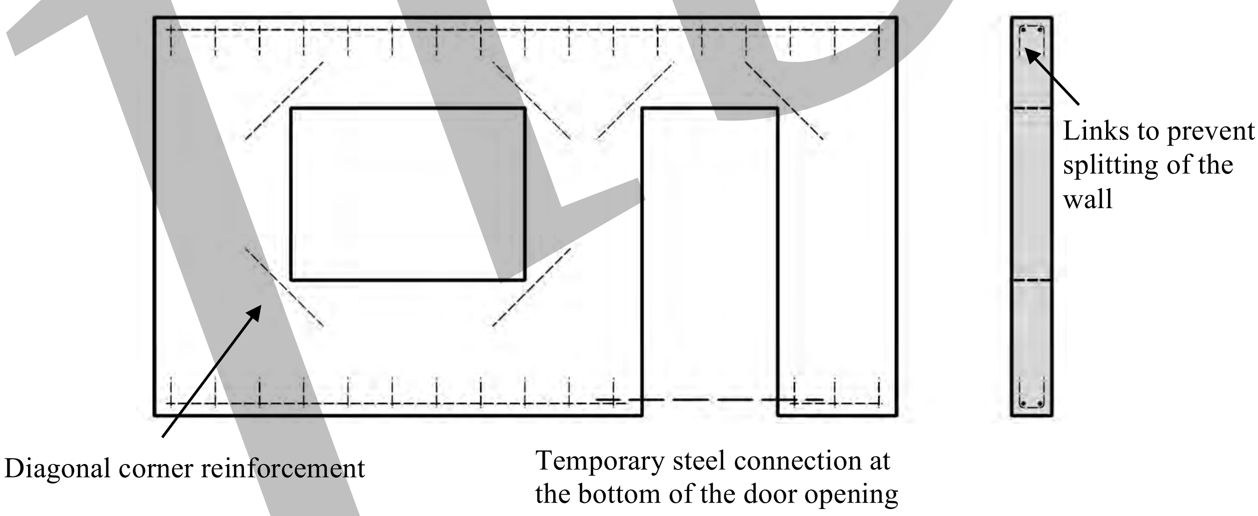


Fig. 10.25: Constructional reinforcement in precast walls

10.6 Dimensional tolerances

10.6.1 General

No manufactured element can be produced to an exact size nor can a structure be assembled and erected to exact dimensions of grid and height. Three kinds of inaccuracy arise in precast buildings for which dimensional tolerances should be established as a part of the design: those of manufacture, those of setting out and in-situ construction, and those of erecting and positioning the elements.

Allowed dimensional tolerances are divided into structural and non-structural (compatibility) ones. Structural tolerances are, for example, on the sectional and principal dimensions, and the position of the reinforcement. Non-structural tolerances are, for example, on the position of openings.

Tolerance values are expressed in millimetres and refer to deviations with respect to the nominal values indicated in design documentation. They are generally expressed in plus and minus values, for example, on the length of an element $\ell \pm x$ mm. These positive and negative values need not necessarily be identical. Indeed, for some dimensions, such as the thickness of a slab, the plus value is usually taken larger than the minus value, for example $h + x$ mm and - y mm. This is not only due to the fact that deviations in plus are more obvious here but also because of the impact of the thickness on the load-bearing capacity of the unit.

Dimensional deviations are not always a consequence of execution; they can also be inherent to physical phenomena, such as shrinkage or elastic deformation due to prestressing. In the latter case, the manufacturer will often apply a certain over-length especially for long elements.

10.6.2 Types of tolerances

In the following the most important product tolerances are described for different types of elements.

- Linear elements

  Tolerances for linear elements, such as columns and beams, are specified for the following parameters:

  - Structural tolerances, such as the length, width, flange and web thickness of I-shaped elements, the angle deviation of a surface, the bow deviation of column faces, the deflection of beams and the skew of the vertical plane of beams

  - Non-structural tolerances, such as the size of holes and openings and the overall position of holes and inserts

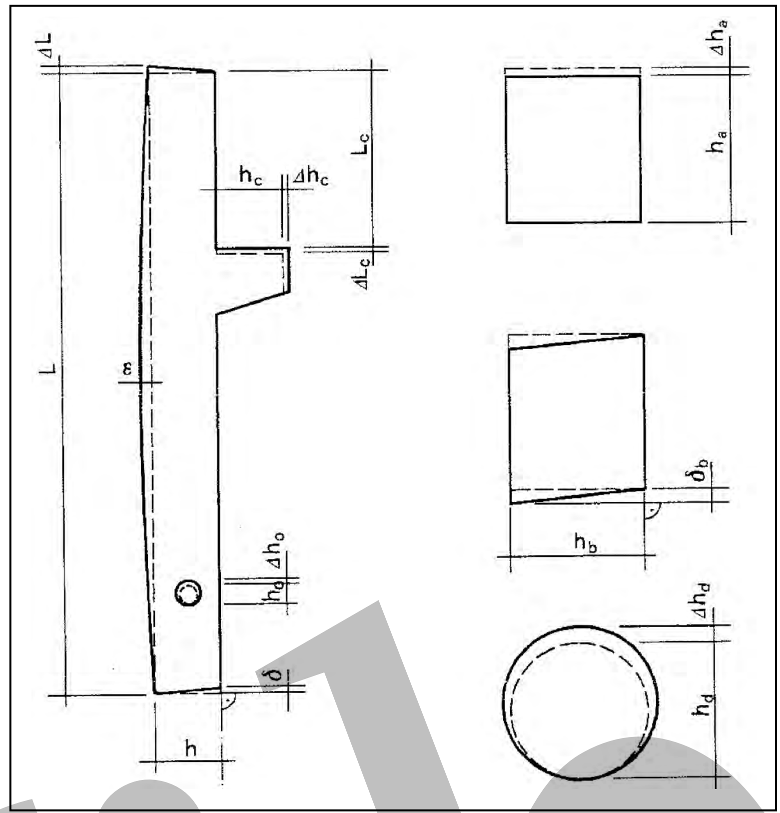


Fig. 10.26: Example of possible dimensional deviations for columns

- Walls, façades
  - Structural tolerances, which are length, width, height and diagonal deviation (skewness)
  - Non-structural tolerances, such as the evenness of surfaces, warping, the straightness of edges, the size and position of openings, and the overall position of inserts

- Floor elements
  - Structural tolerances, which are length, width, lateral bow, depth, flange thickness and web thickness
  - Non-structural tolerances, such as planarity, the angle deviation of webs, the size and position of openings, and the overall position of inserts

11 Fire resistance

11.1 General

The structural behaviour of a concrete building exposed to fire is a complex phenomenon because of the large number of intervening parameters. The design and calculation methods related to the analysis of the global structure during 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. It should enable the application of a specific design philosophy extending beyond the simple verification of the fire resistance of single concrete elements, as it is often the case now.

The requirements with respect to the fire resistance of a building structure are laid down in national regulations. They specify how long a structure shall resist a normalized fire – generally the ISO 834 Standard fire curve for buildings. A more severe hydrocarbon curve also exists that was specifically developed for fire tests on the fire protection materials used in petrochemical installations and oil platforms.

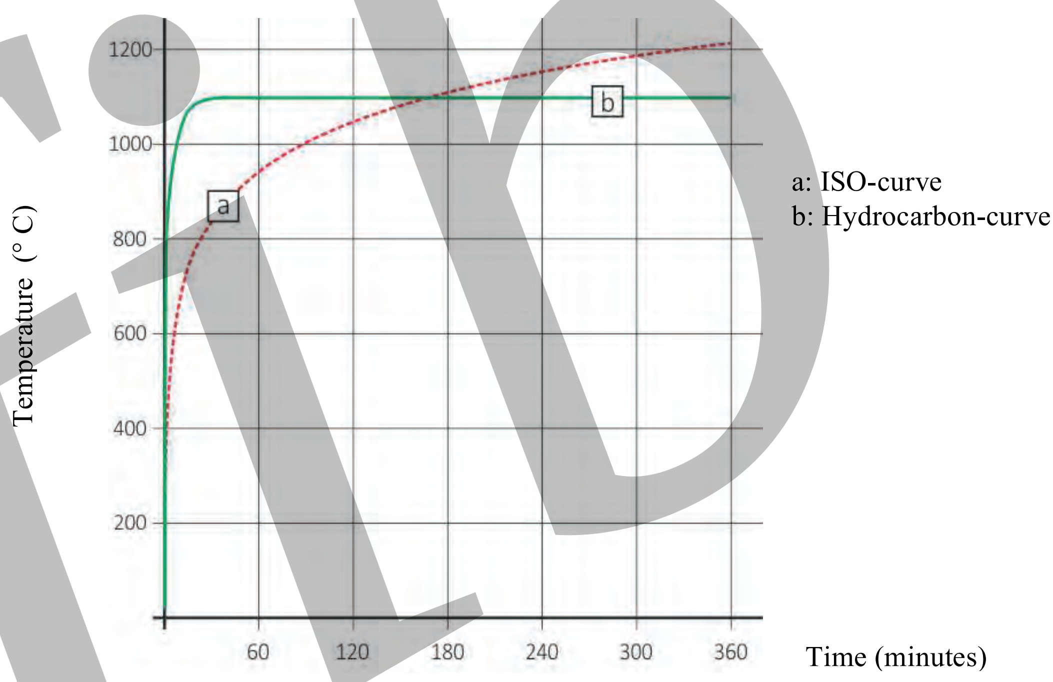


Fig.11.1: Standardized temperature curves

11.2 Basic requirements

The capacity of a concrete structure to maintain its load-bearing function during the relevant fire exposure is expressed as follows:

$$E_{d,fi}(t) \leq R_{d,fi}(t)$$

where

$E_{d,fi}(t)$ is the design effect of actions in the fire situation at time *t*

$R_{d,fi}(t)$ is the corresponding design resistance at elevated temperatures.

The basic criteria for a concrete structure, to comply with the above conditions are as follows:

- Stability symbolized by the criterion *R*
  It is assumed to be satisfied where the load-bearing function is maintained during the required time of exposure.

- Thermal insulation by criterion *I*
  It may be assumed to be satisfied where the average temperature rise over the whole of the non-exposed surface is limited to 140 K, and the maximum temperature rise at any point of that surface does not exceed 180 K. These temperatures are to be seen as serviceability limit states related to the occupation.

- Flame tightness or integrity, by criterion *E*
  It may be assumed to be satisfied where the separating function, namely, the ability to prevent fire spread by the 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.

According to their function in a building, members shall comply with criteria *R*, *E* and *I* as follows:

- Load-bearing only: mechanical resistance (criterion *R*)
- Separating only: integrity (criterion *E*) and, when required, insulation (criterion *I*)
- Separating and load-bearing: criteria *R*, *E* and, when requested, *I*

Chapter 11 will mainly deal with criterion *R*.

11.3 Fire actions

11.3.1 Decrease in material performance

When a fire occurs in a building, the temperature rises fast, at least when there is enough combustible material and oxygen. The exposed construction members will heat up according to the thermal conductivity of the materials: very fast for unprotected steel, rather slowly for concrete. Two phenomenons occur simultaneously: a reduction of the material performances and thermal dilatation. Table 11.1 gives as guidance the stress relation of concrete and reinforcing and prestressing steel, as a function of the material temperatures. They are taken from Eurocode 2 ‘Design of concrete structures - Part 1.2: General rules - Structural fire design’ [3]. More data about stress-strain relationships are available in this document.

In unprotected steel structures the temperature will rise rapidly over the whole exposed cross section because of the high thermal conductivity of the material. As a consequence, after about

15 minutes of heavy fire, a critical limit state will be reached in which the material strength is reduced to half and the safety margins disappear. Plastic hinges appear everywhere and the structure may collapse.

In concrete structures, the situation is completely different. The heating progresses much slower and in a heterogeneous way over the cross section and the 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. However, the surrounding concrete retards this phenomenon. At a certain temperature the reinforcing steel is no longer capable of taking up the acting stresses and failure occurs. This temperature is called the critical temperature. For a concrete cover on the reinforcement of, for example, 25 millimetres, the critical temperature of the reinforcing steel (500 °C) will occur after about 90 minutes of ISO fire exposure. For a concrete cover of 35 millimetres the critical temperature will be reached after 120 minutes. This data is taken from Eurocode 2, part 1-2 [3]. 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 millimetres to obtain the same fire resistance.

Table 11.1: Material performances of concrete and reinforcing and prestressing steel

Temperature θ [°C]	**Concrete** (compression)		**Reinforcing steel**		**Prestressing steel class B**
	Silicious aggregates	**Limestone aggregates**	**Hot rolled**	**Cold worked**	
	$f_{c,\theta}/f_{ck}$	$f_{c,\theta}/f_{ck}$	$f_{sy,\theta}/f_{yk}$	$f_{sy,\theta}/f_{yk}$	$f_{py,\theta}/(0.9\,f_{pk})$
20	1.00	1.00	1.00	1.00	1.00
100	1.00	1.00	1.00	1.00	0.99
200	0.95	0.97	1.00	1.00	0.87
300	0.85	0.91	1.00	1.00	0.72
400	0.75	0.85	1.00	0.90	0.46
500	0.60	0.74	0.78	0.70	0.22
600	0.45	0.60	0.47	0.47	0.10
700	0.30	0.43	0.23	0.23	0.08
800	0.15	0.27	0.11	0.11	0.06
900	0.08	0.15	0.06	0.06	0.03
1000	0.04	0.06	0.04	0.02	0.00
1100	0.01	0.02	0.02	0.03	0.00
1200	0.00	0.00	0.00	0.00	0.00

Note: the table is taken from EN 1992 Part 1-2 [3]; since the data fall under the responsibility of the European Member States, slightly different values may be found in the national application document (NAD) of each EU-member state.

In addition to the decrease of the material performances, the structure will be subjected to thermal expansion: beams and columns mainly in longitudinal direction, floors and walls 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 seldom the fire resistance of the individual members.

11.3.2 Thermal expansion

When a fire occurs locally in the centre of a large building, the thermal expansion will be hindered 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, cumulated deformations.

The longitudinal expansion of beams or ribbed slabs will be considerably larger than for plain slabs. Beams and webs are exposed on three sides to fire. The thermal gradient will be more uniform over the whole cross section. The plain slab will expand less in the longitudinal direction but deflect more because of the differential temperature gradient from the bottom to the top.

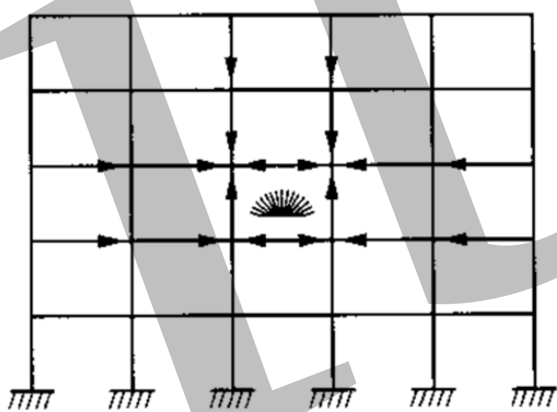


Fig. 11.2: Compressive forces in a frame structure due to hindered thermal expansion

During an intense short fire, the dilatation effect will probably be much less than when the fire lasts longer and is less aggressive, because the concrete needs time to warm up. This shows that the ISO fire curve may not necessarily be the most unfavourable thermal action.

The most critical situation is when the fire covers a wide surface area, resulting in large cumulated deformations. It is not unrealistic to assume that, for example, in a large open storage hall the cumulated longitudinal deformation of successive bays in direct line during a long fire may amount to 200 millimetres and more (Fig. 11.3).

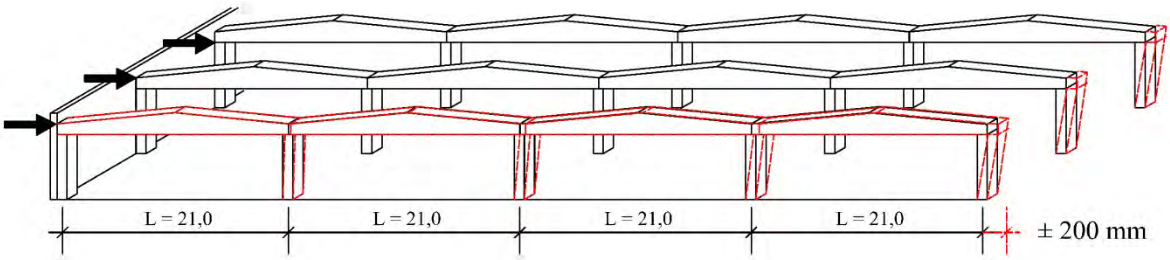


Fig. 11.3: Scheme of a real fire case where, due to blocking of the deformation at one side of the building, the thermal dilatations cumulated in one direction and induced the collapse of the exterior column.

11.3.3 Transverse deformation of 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 variation over the cross section, the exposed underside will expand much more than the cooler upper side. This will force the member to deflect. In some cases, the deflection will cause differential internal stresses over the cross section (see Section 11.6.4).

In addition to the internal stresses, the deflection of the elements also gives rise to a serious increase of support moments. During a fire, simply supported solid slabs and hollow-core floors will deflect due to the expansion of the bottom fibres (Fig. 11.4). However, in a continuous floor structure, the deflection will be hindered by the continuity reinforcement at the top of the slab and the support moment will increase in function of the moment capacity of the top reinforcement (Fig. 11.5). The support moment will create additional compressive stresses at the bottom of the floor and tensile stresses in the top reinforcement above the support.

Fig. 11.4: Thermal deformation of two simply supported hollow-core floors

As a result, the stresses in the top reinforcement above the support will increase and in some cases even yield.

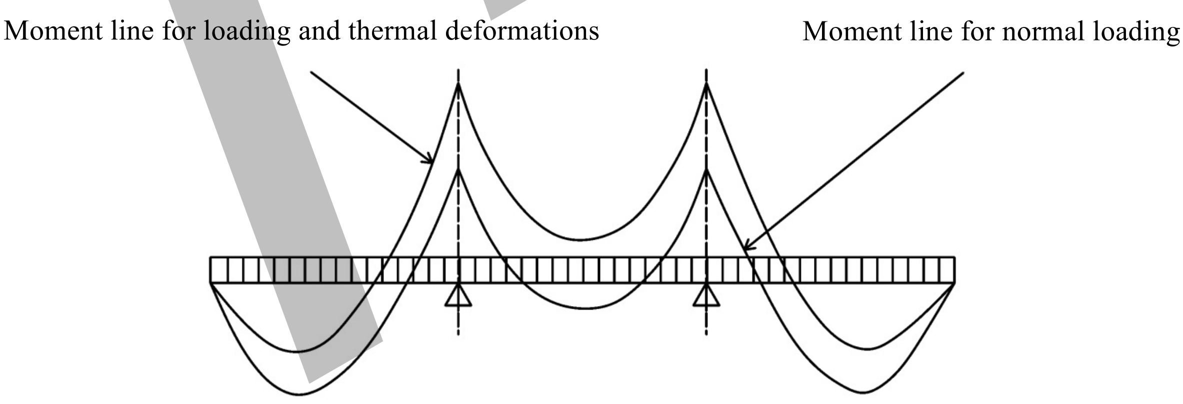


Fig. 11.5: Increase of support moment for continuous structure

11.4 Global structural analysis

According to Eurocode 2 Part 1-2 [3] and most national standards, the analysis of the global structure of a building or parts of it should take into account the relevant failure mode in fire exposure, the temperature-dependent material properties and member stiffness, the effects of thermal expansion, and deformations. However, the code does not stipulate how to include these effects in the design. The analysis of fire safety is henceforth restricted to the verification of the single construction members. Practically, it means that the dimensions of the cross section and the minimum concrete cover on the reinforcement of individual concrete elements for a given ISO fire exposure time should be respected.

However, experiences in real fires show that the instability of concrete structures seldom occurs because of the decrease of material performances at elevated temperatures but nearly always because of the inability of the structure to take the imposed thermal deformations. Fortunately, concrete structures possess not only high fire resistance but also have large redistribution capacities due to robustness and structural integrity. Consequently, the failure of concrete buildings due to fire occurs seldom.

Although very little fundamental research has been carried out in this field, it should yet be possible to outline a design philosophy and guidelines based on practical experiences in real fires and model simulations.

First, the designer should pay more attention to the general behaviour of the building when exposed to fire instead of looking only at the individual components, as is commonly the case. The shape and dimensions of the building as well as the static system are, indeed, very important in the behaviour of a building during a fire. The possibilities for thermal expansion, deformations and blocking forces should especially be taken into consideration. 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 dilatation joints, very large expansion may occur. Not only the distance between the dilatation joints but also their width should be studied from this point of view. In a first approximation, the construction should be verified for an expansion equal to 100 to 150 °C. It would immediately show where the bottlenecks are and how to solve them.

In a multi-storey building with a stabilizing stair and elevator shaft, the most favourable situation will be met when the core is placed in the centre of the structure, enabling an even dilatation of the surrounding floors in all directions. Figure 11.6 shows a structural layout typical for a precast building, which would probably react positively for 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 the more 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. In conclusion, the design should allow for movement where possible, to avoid the incompatibility of deformations due to thermal expansion.

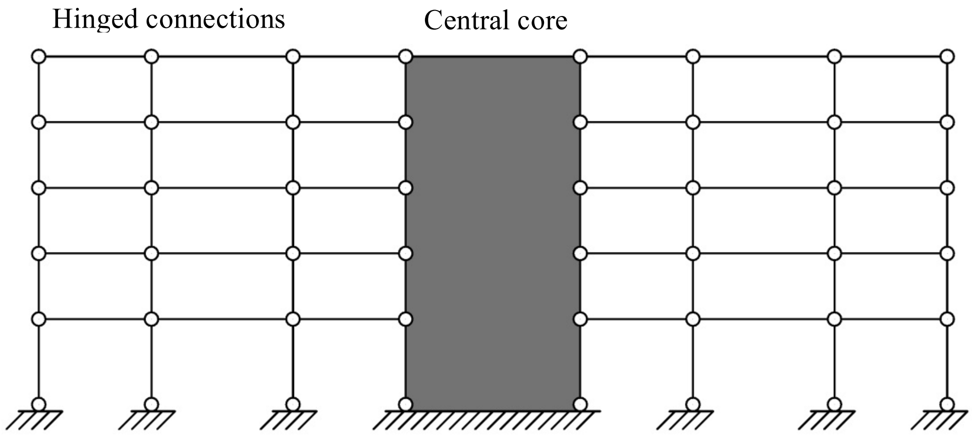


Fig. 11.6: Favourable stability layout with respect to thermal expansions

11.5 Member analysis

According to the Eurocode 2 Part 1-2 [3], fire resistance can be determined by tabulated data, calculation or fire tests. Today designers commonly adopt the first option.

11.5.1 Assessment by tabulated data

Tabulated data give recognized design solutions for the standard fire exposure up to 240 minutes. These tables have been developed on an empirical basis confirmed by experience and the theoretical evaluation of tests. They give the minimum dimensions for the cross section of the components and the minimum concrete cover to the main reinforcement. Series of tables are available in specific literature (see Eurocode 2 Part 1-2 [3], among others). They include columns, load-bearing and non-load-bearing solid walls, beams and solid and ribbed slabs. The information given in this section is mainly taken from Eurocode 2 Part 1-2 [3] and CEN Product Standard EN 1168 [4].

Tabulated data in the Eurocode are based on a reference load level η_{fi} = 0.7. The latter is a reduction factor for the design load level for the fire situation, defined as follows:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}}$$

where G_k is the characteristic value of a permanent action

$Q_{k,1}$ is the principal variable load

γ_G is the partial factor for the permanent action

$\gamma_{Q,1}$ is the partial factor for variable action 1

ψ_{fi} is the combination factor for frequent load values

η_{fi} = 0.7 corresponds to the generally adopted combination for accidental actions, namely: $\gamma_{G=1}; \gamma_{Q,1} = 1$ and $\psi_{fi} = \psi_1 = 0.5$ (for domestic buildings, residential areas and office areas)

The values of the axis distance are established for normal reinforcing steel, with a critical temperature of θ_{crit} = 500 °C. The critical temperature of the reinforcement is the temperature at which the failure of the member in a fire situation is expected to occur at a given steel stress level. For prestressing steel, the required axis distance *a* for beams and slabs should be increased by:

10 mm for prestressing bars

15 mm for prestressing wires and strands

This requirement is based on the assumption that the critical temperature for prestressing bars is 400 °C and for strands and wires 350 °C. Since the quantity of reinforcing is generally somewhat larger than strictly needed at ambient temperature for the load-bearing function, the average steel stress may be smaller than the theoretical critical temperature. Consequently the theoretical axis distances given in the tables may be adjusted. The Eurocode 2 – Part 1-2 [3] provides a calculation method to adjust the axis distances in Section 5.2 (5).

Where reinforcement is arranged in several layers (Fig. 11.7) the average distance a_m should not be less than the axis distance *a* given in the tables.

$$a_m = \frac{A_{s1}a_1 + A_{s2}a_2 + \ldots + A_{sn}a_n}{A_{s1} + A_{s2} + \ldots + A_{sn}} = \frac{\Sigma A_{si}a_i}{\Sigma A_{si}}$$

where A_{si} is the cross sectional area of steel bar (tendon, wire) '*i*'

a_i is the axis distance of steel bar (tendon, wire) '*i*' from the nearest exposed surface

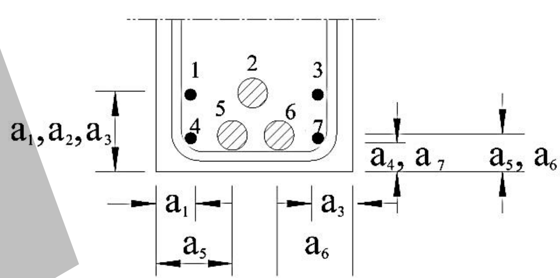


Fig.11.7: Dimensions used to calculate average axis distance a_m.

The values in the tables apply to normal-weight concrete (2000 to 2600 kg/m³) made with siliceous aggregates. If calcareous aggregates or lightweight aggregates are used in beams or slabs the minimum dimensions of the cross section may be reduced by 10 per cent. The same is valid for non-load-bearing walls with calcareous aggregates. The reason for the reduction lies in the endothermic reaction of limestone at higher temperatures. However the reduction is not applicable for columns.

When tabulated data are used, no further checks are required concerning shear and torsion capacity and anchorage details.

11.5.2 Example of simplified calculation method

The verification by calculation is based on the ultimate limit state (ULS) method. The load-bearing capacity is calculated with reduced material characteristics corresponding to their temperature at a given fire-exposure time. The method is extensively described in the Eurocode 2 Part 1-2 [3]

The method is based on the hypothesis that concrete at a temperature above 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. This method is applicable to a reinforced and prestressed concrete section with respect to the axial load, bending moments and their combinations. For a rectangular beam exposed to fire on three sides, the effective cross section in the fire situation will be in accordance with Figure 11.8. The method is valid for a minimum width of the cross section given in Table 11.2.

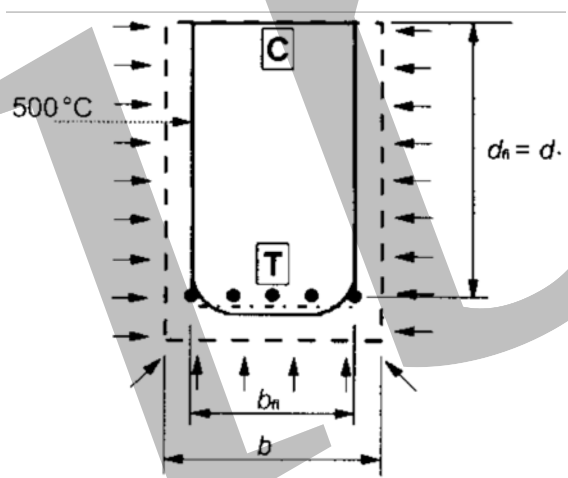


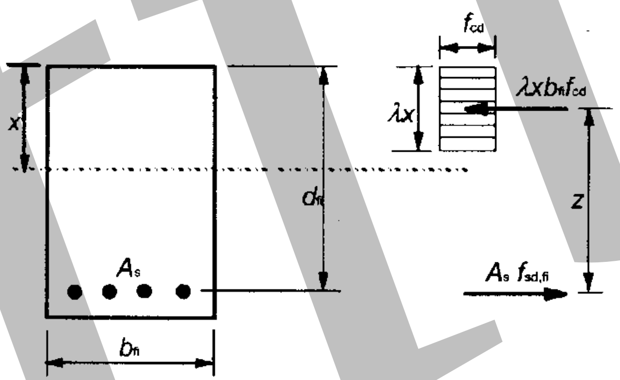
Fig. 11.8: Reduced cross-section of a reinforced concrete beam with fire exposure on three sides

Table 11.2: Minimum width of cross-sections as function of fire resistance

Fire resistance	R60	R90	R120	R180	R240
Minimum width 'b' cross-section	90	120	160	200	280

The temperature profiles given in the Eurocode 2 Part 1-2 [3] or other specific literature may be used to determine the 500°C gradient in the cross sections exposed to a standard fire. On the basis of the reduced cross section approach, the procedure for calculating the resistance of a reinforced concrete cross section in the fire situation may be carried out as follows:

- Determining the isotherm of 500°C for the specified fire exposure
- Determining a new width b_{fi} and a new effective height d_{fi} of the cross section by excluding the concrete outside the 500°C isotherm
  The rounded corners of isotherms can be regarded by approximating the real form of the isotherm to a rectangle or a square (Fig. 11.8).
- Determining the temperature of the reinforcing bars in the tension and the compression zones
  The temperature of the individual reinforcing bar can be evaluated from the temperature profiles given in handbooks or specific literature and is taken as the temperature in the centre of the bar. Some of the reinforcing bars may fall outside the reduced cross section (Fig. 11.8). Despite this, they may be included in the calculation of the ultimate load-bearing capacity of the fire-exposed cross section
- Determining the reduced strength of the reinforcement due to temperature (see Table 11)
- Using conventional calculation methods for the reduced cross section for determining the ultimate load-bearing capacity with strength
  Figure 11.9 shows the calculation model of the load-bearing capacity of a rectangular cross-section.



b	original width
b_{fi}	width of effective cross-section
h	original height
h_{fi}	height of effective cross-section
d_{fi}	effective height of effective cross-section
z	lever arm between tension reinforcement and concrete
λ_x	see conventional calculation method for ULS (usually 0.8 x)

Fig. 11.9: Stress distribution at ULS for a rectangular concrete cross-section

For T-shaped cross sections, the compression zone at ULS is often within the reduced flange area. In this case the T-shaped cross section is calculated with b_{fi} the width of the flange and z and d_{fi} according to Figure 11.9.

Fire is considered to be an accidental action. As a result, reduced values for the applied loading corresponding to the frequent part of the characteristic loading may be used. Indeed, the probability of the occurrence of the full characteristic loading during a fire is negligible. In Eurocode 0: EN 1990: *Basis of structural design* the reduction factor for the frequent value of the loading for residential and office buildings is $\psi_1 = 0.5$. For accidental actions, lower values than for normal design situations for the partial safety factors may also be used. The Eurocode recommends values of $\gamma_s = \gamma_m = 1.0$.

The 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. When using the simplified calculation method for elements in which the shear capacity is dependent on the tensile strength, special consideration should be given where tensile stresses are caused by non-linear temperature distributions, for example, in hollow-core slabs (See section 11.6.4).

The method is not applicable for columns in which the structural behaviour is significantly influenced by second order effects under fire conditions. Eurocode 2 Part 1-2 [3] gives detailed information on the subject.

11.5.3 Analysis by testing

Tests are usually performed on simply supported elements, with an imposed loading corresponding to the frequent load combination. Many tests and associated research work on precast elements, such as beams, columns, double-tee elements and hollow-core slabs, have been carried out since 1960. This work covers a wide range of cross sections together with variations in reinforcement quantities and position, cover to soffit and lateral surface, and so forth. A large number of test results are available and tests are not performed as frequently as before. In addition, because of the limited size of test furnaces (normally up to 4 to 6 metres in length) and the fact that spans of precast units range from 6 to 50 metres, tests are not really relevant; other methods are needed for the assessment of the fire resistance of precast concrete elements.

For simply supported flexural members the ultimate limit state for the stability criterion ‘R’ is generally obtained when the deflection reaches L^2 / (400d), L is the clear span of the test specimen, in millimetres, and d is the distance from the extreme fibre of the cold design compression zone to the extreme fibre of the cold design tension zone of the structural section, in millimetres. For columns, the ULS corresponds conventionally to a certain value of the contraction, or when a given rate of contraction is exceeded. Precise data are available in national standards.

11.6 Fire resistance of components

The following sections give specific data on the fire resistance of precast concrete elements. They are mostly taken from Eurocode 2 Part 1-2 [3].

11.6.1 Columns

The fire resistance of reinforced and prestresssed columns is influenced by several parameters, including:

- Size and slenderness of the columns
- Load level
- First order eccentricity
- Concrete strength and aggregate type
- Mechanical reinforcement
- Axis distance of the reinforcement

11.6.1.1 Dimensioning of the cross-section

Standards usually prescribe a minimum column cross section based on the aforementioned parameters. In precast structures, however, the design of the column section must also consider actions during the manufacture and erection of the units. This results in minimum cross sections of 300/300 or even 300/400 millimetres.

11.6.1.2 Detailing of the reinforcement

It is always recommended to respect a balance between the size of the main reinforcing bars and the column cross section, especially for more slender sections, namely, smaller than 300 millimetres. The distance between the stirrups should also not be too great. The reason is evident from the following experiment.

During a fire test on a column of 280 x 280 mm² section and 3.94 metres in height in high-strength reinforced concrete C90 (Fig. 11.10), failure occurred after 96 minutes of ISO fire due to the buckling of the longitudinal reinforcing bars. The column was reinforced with 4 bars ϕ 25 and an axis distance of 30 millimetres. The concrete mix comprised 510 kilograms of cement and 51 kilograms of silica fume per m³. The moisture content at the test was 2.2 per cent. The test load was 1500 kN and the load eccentricity 28 millimetres. After 90 minutes of fire exposure, the column shortened over time. In this way, the test load was transferred gradually from the concrete to the reinforcing bars, finally leading to the buckling of the latter. Figure 11.11 clearly shows the buckling of the main reinforcing bars in between two successive stirrups.

A similar column in prestressed concrete, reinforced with 4 strands of ϕ ½ inch failed only after 124 minutes of fire exposure. Here a similar increase of the shortening was also observed but in contrast to the reinforced column, the load was not transferred to the reinforcing strands since they followed the same shortening as the column. This appeared from the fact that failure here was due to the crushing of the concrete. All other parameters were, indeed, the same.

As a conclusion, for the fire resistance of slender columns, it is better to use smaller bars to get a better distribution of the compression between concrete and steel, and to place the stirrups at closer spacing.

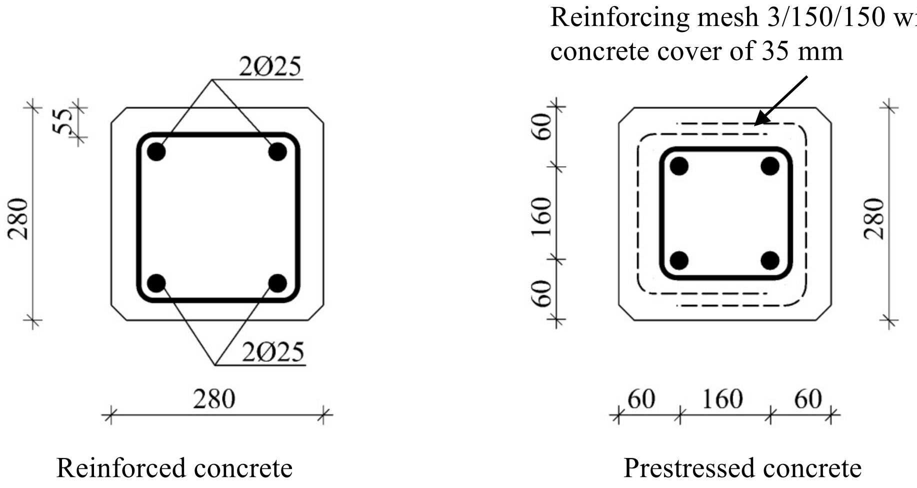


Fig. 11.10: Cross-section of the test columns

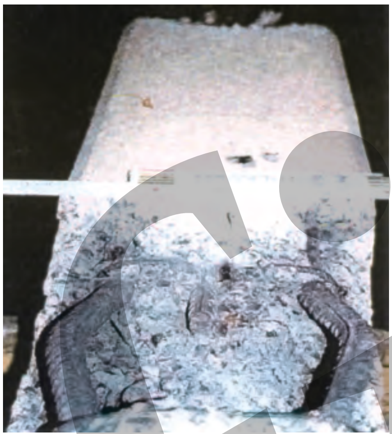


Fig. 11.11: Failure pattern of the column in reinforced concrete

11.6.1.3 Assessment of the fire resistance of columns

In the past many tests have been performed worldwide on concrete columns. Complex computer programs now exist that allow the calculation of the fire resistance of columns, both in reinforced and prestressed concrete, and that take into account the above parameters, including buckling. However, it is not possible to include all this information in tabulated data. As a result, the following design rules are limited to braced structures.

According to the Eurocode 2 Part 1-2 [3], the fire resistance of columns may be assessed by two methods, an empirical one and an analytical one. They also belong to the so-called tabulated data. In view of simplification and harmonization, these two methods may be replaced in the next revision of EN 1992-1-2 by only one unique method.

Method A is based on the following equation:

$$R = 120[(R_{\eta.f_i} + R_a + R_l + R_b + R_n)/120]^{1,8}$$

where

$$R_{\eta,f_i} = 83.[1,00 - \mu_{f_i} \frac{(1+\omega)}{(0.85/\alpha_{cc})+\omega}$$

$\mu_{fi} = N_{Edfi}/N_{Rd}$ load factor at fire

$R_a = 1.60\ (a - 30)$

$R_l = 9.60\ (5 - \ell_{0,fi})$

$R_b = 0.09\ b'$

$R_n = 0$ for n = 4 (corner bars only)

$= 12$ for n > 4

a is the axis distance to the longitudinal steel bars (mm); 25 mm $\leq a \leq$ 80 mm

$\ell_{0,fi}$ is the effective length of the column under fire conditions; 2 m $\leq \ell_{0,fi} \leq$ 6 m

The effective length of a column under fire conditions $\ell_{0,fi}$ can in all cases be taken as equal to ℓ_0 under normal temperature conditions. For braced structures where the fire resistance is higher than 30 minutes, the effective length $\ell_{0,fi}$ may be taken as equal to $0.5\ell_0$ for intermediate storeys and $0.5\ \ell_0 \leq \ell_{0,fi} \leq 0.7\ \ell_0$ for the upper story. In these expressions ℓ_0 is the actual length of the column (between the storey level axis's).

$b' = 2Ac\ /\ (b+h)$ for rectangular cross sections

$= Ø_{col}$ for circular cross sections

b' is limited to: 200 mm $\leq b' \leq$ 450 mm; $h \leq 1.5\ b$

ω denotes the mechanical reinforcement ratio at normal temperature conditions:

$$\omega = \frac{A_s f_{yd}}{A_c f_{cd}}$$

α_{cc} is the coefficient that takes into account the long-term effects on compressive strength. The value for normal actions lies between 0.8 and 1.0. In the above expression, the value $\alpha_{cc} = 0.85$ has to be used (normal temperature) since in the expression account already has implicitly been taken for the fire situation. In fire conditions the design value of the compressive concrete strength is determined based on $f_{cd,fi} = \alpha_{cc,fi}.\ f_{ck}/\ \gamma_{c,fi}$ to be compared to $f_{cd} = 0.85\ f_{ck}/\gamma_c$ at normal temperature.

In Table 5.2a the Eurocode 2 Part 1-2 [3] gives conservative minimum dimensions (ω = 0 and $\ell_{0,fi}$ = 3 m) for the width of columns and the axis distance to the main reinforcement of the columns with a buckling length of $\ell_{0,fi} \leq$ 3 metres. Method A is only applicable for columns with a maximum first order eccentricity comprised between 0.15h and 0.40 h. The following symbols are used:

b and h: width and height of the column cross section

$\mu_{fi} = N_{Edfi}/N_{Rd}$ load factor at fire

n: number of longitudinal reinforcing bars

a: axis distance reinforcement to concrete surface

Method B is also only applicable for braced structures. This method is based on analytical calculations where the second order effect has been taken into account together with the reduction of the mechanical characteristics of concrete and reinforcing steel in relation to the temperature.

11.6.2 Beams

The fire resistance of precast reinforced and prestressed beams is usually assessed by tabulated data. The values given in Table 11.3 are applicable where the following rules are used:

- Beams are exposed on three sides, namely, the upper side is insulated by slabs or other elements
- The tabulated data are valid for the cross sections shown in Figure 11.12
- For beams of varying width (Fig. 11.12b) the minimum value *b* relates to the centroid of the tensile reinforcement

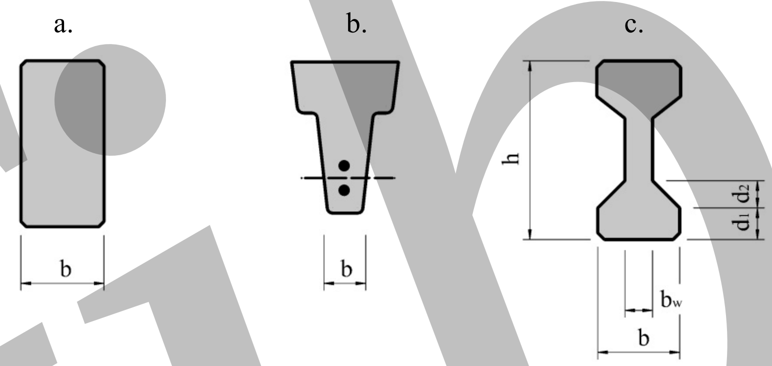


Fig. 11.12: Definition of dimensions for different types of beam sections

- The underflange of I-beams should have sufficient mass to keep the temperature of the reinforcement below the level accepted for the establishment of the table. This condition is met where:
  - The effective height d_{eff} of the bottom flange is not less than:
    $d_{\text{eff}} = d_1 + 0.5\, d_2 \geq b_{min}$
    where b_{min} is equal to 220 mm for R120 and for R180
  - When the actual width of the bottom flange b exceeds the limit 1.4 b_{w} (b_{w} denotes the actual width of the web, see Figure 11.12.c), and $b.d_{\text{eff}} < 2b^2_{\text{min}}$, the axis distance to the reinforcing or prestressing steel should be increased to:
    $$a_{eff} = a(1.85 - \frac{d_{eff}}{b_{\min}}\sqrt{\frac{b_w}{b}}) \geq a$$
    where d_{eff} and b_{min} are defined above

For most standard precast I-beams with a width b ≥350 mm, $b.d_{\text{eff}}$ is larger than $2b^2_{\text{min}}$ and the above rule does not apply.

- Temperature concentrations occur at the bottom corners of beams. For this reason, the axis distance to the side of the beam for corner bars (tendon or wire) in the bottom of beams with only one layer of reinforcement should be increased by 10 millimetres for widths of beams up to that given in Column 4 of Table 11.3, for the relevant standard fire resistance.
- 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 $A_c = 2b^2_{min}$ where b_{min} is given in Table 11.3.

Table 11.3 gives minimum values for the axis distance of the reinforcement to the bottom and the sides of simply supported beams, together with values for the width of the beam and the web. The table is taken from Eurocode 2 part 1-2 [3] – Table 5.5. In case of modification of the standard, the requirements of the latter are to be followed.

Table 11.3: Dimensions and axis distances for simply supported reinforced and prestressed beams

Standard fire resistance	**Minimum dimensions (mm)**				
	Possible combinations of a and b_{min} where a is the average axis distance of the tensile reinforcement and b_{min} the width of the beam				Web thickness b_w (*)
1.	2.	3.	4.	5.	6.
R 60	b_{min} = 120 a = 40	b_{min} = 160 a = 35	b_{min} = 200 a = 30	b_{min} = 300 a = 25	80 - 100
R 90	b_{min} = 150 a = 55	b_{min} = 200 a = 45	b_{min} = 300 a = 40	b_{min} = 400 a = 35	100 - 110
R 120	b_{min} = 200 a = 65	b_{min} = 240 a = 60	b_{min} = 300 a = 55	b_{min} = 500 a = 50	120 - 130
R 180	b_{min} = 240 a = 80	b_{min} = 300 a = 70	b_{min} = 400 a = 65	b_{min} = 600 a = 60	140 - 150
R 240	b_{min} = 280 a = 90	b_{min} = 350 a = 80	b_{min} = 500 a = 75	b_{min} = 700 a = 70	160 - 170
a_{sd} = a + 10 mm; a_{sd} is the axis distance for the corner bars (or tendon or wire) to the side of beams with only one layer of reinforcement. For values of b_{min} greater than given in Column (4) no increase of a_{sd} is required.					
For prestressed beams the axis distance should be increased by 10 mm for prestressing bars and 15 mm for prestressing wires and strands (see§ 11.5.1).					
(*) The values to be used are specified in the National Application Document					

11.6.2.1 Specific requirements for non-rectangular beams

Purlins and secondary roof beams often have a trapezoidal or I-shaped cross-section and are usually in prestressed concrete. In many cases, the beams have only stirrups at their end zones. When a fire resistance of more than 30 minutes is required, it is necessary to provide stirrups over the whole length of the element, to prevent horizontal cracking especially when the units have an abrupt change in the cross-section as shown on Figure 11.13.

The temperature in the smaller part is much higher than in the larger top part of the section. As already discussed in Section 11.3.3, the deformation of the cross-section will not be directly proportional to the temperature gradient, and shear stresses will occur at the change of the section. Fire tests on such beams without stirrups over the whole length of the elements have demonstrated that during a test, horizontal cracks may appear at the variation of the profile. To prevent this, stirrups are necessary over the whole length of the beam.

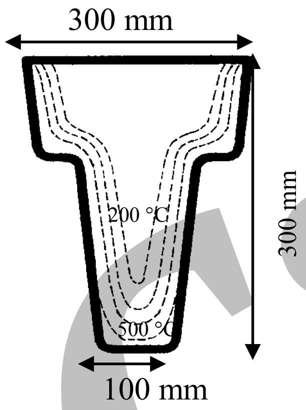


Fig. 11.13: Temperature gradients in a purlin after 60 minutes fire exposure

11.6.2.2 I-shaped beams

Large I-shaped beams exposed to a long and severe fire will show a much more complex stress distribution in the cross section. Figure 11.14 shows the isotherms after 1 hour of fire exposure in a I-shaped beam of 1,125 millimetres in height, 360 millimetres in the width of the top and bottom flanges and 120 millimetres in web thickness.

The average temperature in the top flange will be much lower than in the bottom flange. This will lead to additional shear stresses in the web.

In addition, due to the differences in temperature inside the web itself, thermal stresses will occur. Figure 11.14 shows that after one hour of fire exposure, the temperature in the outermost concrete amounts to about 600 °C, while in the central zone the temperature remains under 400 °C. The exterior part will tend to expand more than the inner one and hence create compression in the exterior zone and tension in the inner one. Due to the phenomenon of load-induced thermal strain (LITS), a certain redistribution of stresses will occur.

It is obvious that vertical stirrups are important to take up the shear stresses between the web and the flanges.

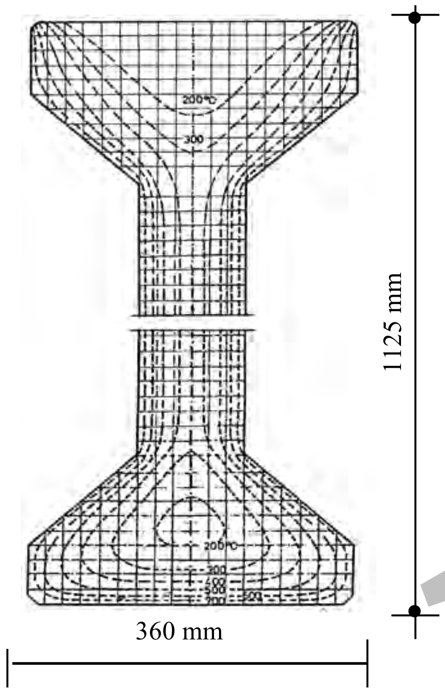


Fig. 11.14: Temperature gradients in a I-shaped beam after one hour fire exposure

11.6.3 Walls

Precast partition walls need only meet the criteria for thermal insulation ‘E’ and integrity ‘I’. To this effect, the minimum wall thickness should not be less than that given in Table 11.14. If calcareous aggregates are used, the indicated values may be decreased by 10 per cent. To avoid excessive deformation and the subsequent failure of the integrity between wall and slab, the slenderness ratio fi/i should not exceed 50.

Table 11.4: Minimum wall thickness of non-load bearing walls (partitions)

Standard fire resistance	**EI 60**	**EI 90**	**EI 120**	**EI 180**	**EI 240**
Minimum wall thickness (mm)	80	100	120	150	175

Precast load-bearing reinforced concrete walls have usually standard thicknesses ranging from 140 to 300 mm. The fire resistance of 60 minutes is met without further measures. Higher fire resistance may be assumed if the data given in Table 11.5 are applied.

Table 11.5: Minimum dimensions and axis distances for load bearing reinforced concrete walls

Standard fire resistance	Minimum dimensions (mm) wall thickness / axis distance for			
	$\mu_{fi} = 0.35$		$\mu_{fi} = 0.70$	
	wall exposed on one side	wall exposed on two sides	wall exposed on one side	wall exposed on two sides
REI 90	120/ (*)	140 / (*)	140 / (*)	170 / (*)
REI 120	150 / 25	160 / 25	160 / 35	220 / 35
REI 180	180 / 40	200 / 45	210 / 50	270 / 55
REI240	230 / 55	250 / 55	270 / 60	350 / 60

(*) The normal concrete cover should apply
$\mu_{fi} = N_{\mathrm{Ed,fi}} / N_{\mathrm{Rd}}$ where $N_{\mathrm{Ed,fi}}$ is the design axial load in the fire situation
N_{Rd} is the design resistance of the wall at normal temperature

11.6.4 Prestressed hollow-core floors

Hollow-core slabs are among the most researched precast products in the world. Since the 1970s, more than one hundred fire tests have been carried out on prestressed hollow-core floor units in laboratories in Europe, the USA and Japan. The results show that when the tests are carried out on floors with normal connections to the supporting structure, a fire resistance of 2 to 3 hours is obtained for both bending and shear loading.

Today the hollow-core slab is the only precast element for which rules for not only the bending capacity but also the shear capacity of the slab under fire conditions have been calculated. This chapter has mainly been based on the approach followed in the European product standard for hollow-core slabs, EN1168:2005+A3:2011 [4].

The fire resistance given for a hollow-core element (load-bearing function) is valid when installed in a floor structure with a tying system specified in EN 1992-1-1[2], unless additional measures are taken.

The increase in the temperature in a hollow-core slab is the determining parameter and is described in Section 11.6.4.1 below.

For the separating function of a hollow-core slab floor, additional measurements for the insulation for minimum thickness (see Section 11.6.4.4) and the integrity of the joints (see EN 1992-1-2, 4.6 [3]) may be needed. The topping or screed cast directly on the precast unit may be taken into account when assessing the fire resistance of the floor for the separating function.

11.6.4.1 Temperature in the slab

The following assumptions about slab temperatures are valid for slabs exposed to fire from the underside (soffit):

- Unless a more accurate thermal analysis is made, below level $a_{50\%}$ - the level at which the total web width is equal to the core width - (Fig. 11.15), the temperature could be assumed to be equal to the temperature of a solid slab (Fig. 11.16).

- Above that level a linear interpolation can be assumed between the temperature at the $a_{50\%}$ level and the temperature at the top of the floor. Since the 'I'-criterion (insulation) may be assumed to be satisfied where the average temperature rise over the whole of the non-exposed surface is limited to 140 K, and the maximum temperature rise at any point of that surface does not exceed 180 K, the considered maximum temperature in this approach is taken to be 160°C (=20°C+140°C).

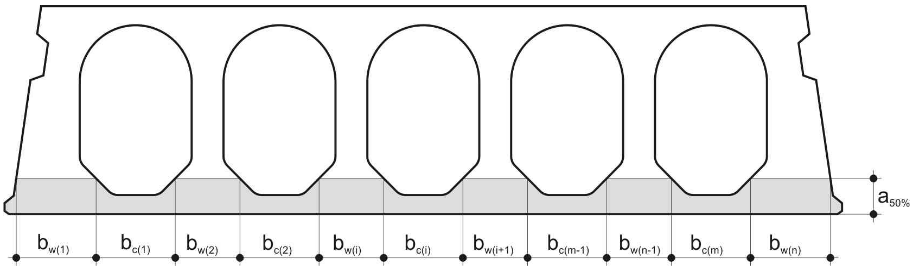


Figure 11.15: Area where solid slab temperatures may be assumed

$$a_{50\%} = \text{level on which } \sum_{i=1}^{n+1} b_{w(i)} = \sum_{i=1}^{n} b_{c(i)}$$

where (Fig. 11.15):

n	= number of cores
$b_{w(i)}$	= width of web number 'i' at considered level
$b_{c(i)}$	= width of core number 'i' at considered level

Figure 11.16 gives temperature profiles within the lower part of the cross section of hollow-core units.

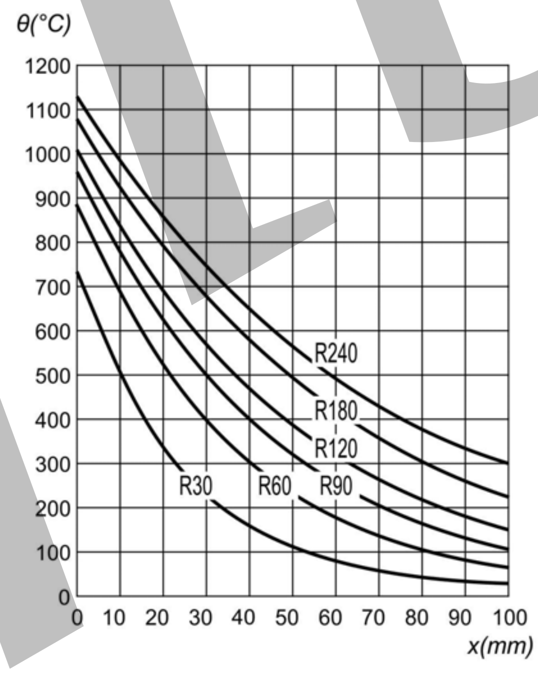


Fig. 11.16: Temperature distribution in the lower part of hollow-core elements during standard ISO fire; x is the distance from the soffit to the considered point

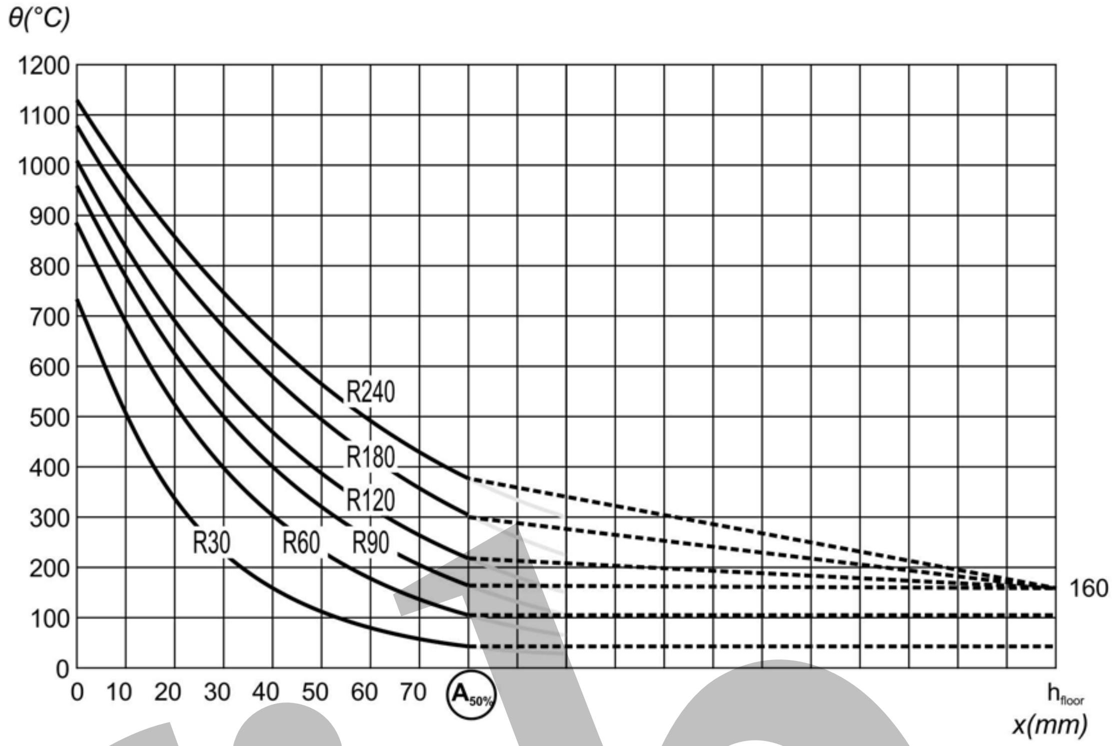


Fig. 11.17: Assumed temperature distribution in the floor during standard ISO fire (x is the distance from the soffit to the considered point).

11.6.4.2 Bending capacity

Fire resistance where bending failure is involved is governed by a decrease in the strength of the prestressing tendons in relation to the temperature. Full-scale tests have shown that the temperature change within the vicinity of the prestressing reinforcement is practically independent of the slab thickness.

The fire resistance where bending failure is involved may be determined by using simplified calculation methods (see Section 11.6.4.1 and EN 1992-1-2 clause 4.2 and Annex B [3]).

The temperatures in the compression zone are in the order of 100 to 300 ˚C. At these temperatures concrete has 90 to 95 per cent of the compressive strength left. As $\gamma_C = 1{,}0$ and $\alpha_{cc} = 1{,}0$ (NDP) in fire design, the depth of a concrete block decreases and the internal lever arm increases. This will result in a more favourable fire design. Thus, the concrete strength on the compression side may be assumed to be unaffected where the slab thickness fulfils an insulation criterion, namely, the same internal lever arm as in normal temperature design can be assumed to exist.

11.6.4.3 Shear and anchorage capacity

The shear capacity of prestressed hollow-core slabs during fire is affected by the increase of the tensile stresses in the webs due to the temperature gradient and the decrease of the bond strength, which will consequently lead to an increase of the anchorage length.

Assessment by calculation

A calculation method for shear and anchorage failure has been published by the European Committee for Standardisation (CEN) in EN1168: 2005+A3:2011 [4]. This standard provides an empirical formula that has been validated by FEM calculations and fire tests in laboratories.

Table 11.6 gives values related to the following assumptions:

- Prestressed hollow-core slabs with strands cut at the ends of the elements
- Support length of 70 millimetres
- Section of 1.88 cm^2/m of longitudinal tie reinforcement placed in approximately the middle of the slab (placed in joints between the slabs and/or in filled cores)

This table demonstrates the shear capacity under fire conditions ($V_{Rd,c,fi}$) as a percentage of the shear capacity in ambient (cold) conditions ($V_{Rd,c,cold}$). Since $V_{Rd,c,cold}$ is the shear resistance given by the simplified shear tension model, Table 11.6 is only useable with the simplified expression according to EN 1168 clause 4.3.3.2.2.3 [4]. The influence of the concrete in the filled cores with embedded tying reinforcement should not be taken into account in this shear tension model.

Table 11.6: Example of the shear capacity under fire conditions ($V_{Rd,c,fi}$) as a percentage of the shear capacity in ambient (cold) conditions ($V_{Rd,c,cold}$)

$V_{Rd,c,fi}/V_{rd,c,cold}$ (%)	**Slab thickness [mm]**				
Fire resistance	**160**	**200**	**240-280**	**320**	**360-400**
REI 60	70 %	65 %	60 %	60 %	55 %
REI 90	65 %	60 %	60 %	55 %	50 %
REI 120	60 %	60 %	55 %	50 %	50 %
REI 180	45 %	50 %	50 %	45 %	45 %

11.6.4.4 Insulation

Assessment by tabulated data

Table 11.7 gives the minimum thickness *h* of the slab related to the fire resistance of insulation. The slab thickness in Table 11.7 corresponds with the minimum floor thickness given in Table 5.8 of EN 1992-1-2 [3] for solid slabs, and has been calculated according to the following conversion equation for hollow-core slabs:

$$t_e = h\sqrt{A_c/(b \times h)}$$

where

t_e is the effective thickness

h is the actual slab thickness

A_c is concrete area of the concrete section
b is the width of the slab

The minimum slab thickness given in Table 11.7 is based on a minimum concrete area of 0,4 bh.

Table 11.7: Tabulated data for minimum thickness of the slab related to fire resistance of insulation.

Minimum dimensions	***Required fire-resistance class***			
	REI 60	REI 90	REI 120	REI 180
Slab thickness [mm]	130	160	200	250

Note: Where a concrete topping or a screed is used, the thickness of the non-combustible layer may be taken into account in the fire resistance of the floor for the separation function.

11.6.5 Ribbed floors

The fire resistance of double-tee floors depends on the cross section of the webs, the thickness of the flange and the axis distance of the reinforcement. Table 11.11 applies for the minimum dimensions of the cross section and corresponding axis distances of the main reinforcement in function of the required fire resistance.

11.7 Fire resistance of structural connections

The principles and solutions applied for the fire resistances of structural components also remain valid for the design of connections: minimum cross-sectional dimensions and sufficient cover to the reinforcement. The design philosophy is based on the large fire 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 in neoprene since they are protected by the surrounding components. Other considerations are related to the ability of the connection to absorb possible large displacements and rotations. Some considerations regarding specific connections are given hereafter.

11.7.1 Pinned connections

Simply supported connections perform excellently during a fire because of the 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 the semi rigid behaviour. After a certain horizontal deformation, an internal force couple is created between the dowel and the surrounding concrete, giving additional stiffness in the ultimate limit state. This is normally not taken into account in the design, but nevertheless constitutes a reserve capacity to safety.

11.7.2 Floor-beam connections

The connections between precast floors and supporting beams are within the colder zone of the structure and, hence, are not typically affected by 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, or a type of hairpin connecting reinforcement.

In the case of restrained support connections, the Eurocode 2 Part 1-2 [3] prescribes the prevision of sufficient continuous tensile reinforcement in the floor itself to cover possible positive and negative moment alterations.

11.7.3 Steel inserts and connections

Mechanical connecting devices should be protected to the same degree as other structural members. Steel parts embedded in concrete will undergo 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 and steel angles.

11.7.4 Joints

Joints between precast elements must be detailed in such a way that they comply with the required criteria for stability, integrity and insulation.

Longitudinal joints between precast floor and wall elements generally do not require any special protection. The condition for thermal insulation and structural integrity is a minimum section thickness (unit plus finishing screed for floors) according to the required fire resistance. The joint should remain closed; this may be achieved with tie-reinforcement in the perimeter joint. Joints between firewalls and columns should be connected over the full length, or through a special profile of the column cross section.

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