Bulletin 101

fib CEB-FIP

PCI Precast/Prestressed Concrete Institute

Precast Concrete in Tall Buildings

State-of-the-art Report

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Task Group 6.7

December 2021

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The *fib* Task Group 6.7 includes the following members:

Convener and editor: George Jones (CDC Ltd, UK and Ireland)

Carlos Chastre Rodrigues (Universidade Nova de Lisboa, Portugal), Andrej Cholewicki (Consultant, Poland), Ned Cleland (Blue Ridge Design Inc, USA), Hugo Corres Peiretti (Fhecor Ingenieros Consultores, Spain), Tom D'Arcy (CEG, USA), Wit Derkowski (Cracow University of Technology, Poland), Iria Doniak (ABCIC, Brazil), Mischa Falger (BAM Advies & Engineering), Antonello Gasperi (Consultant, Italy), SK Ghosh (S. K. Ghosh Associates Inc., USA), Simon Hughes (Westkon Precast Pty. Ltd., Australia), Simon Kelly (Kelly Structural Design Services, Ireland), Stefano Pampanin (Sapienza University of Rome, Italy), Lasse Rajala (Sweco Structures Ltd, Finland), Spyros Tsoukantas (Consultant, Greece), Pieter van der Zee (Ergon Structural Concrete, Belgium), Dick van Keulen (Delft University, Netherlands), Bin Zhao (Tongji University, China).

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This report was drafted by Task Group 6.7 "Precast Concrete in Tall Buildings" within the *fib* Commission 6.

Authors:

George Jones (Convener / Editor)
CDC Ltd, UK and Ireland

Ned Cleland
Blue Ridge Design Inc., USA

Kim Elliott
Consultant, UK

Stefano Pampanin
Sapienza University of Rome

Lasse Rajala
Sweco Structures Ltd, Finland

Jan Vambersky
Jan Vambersky BV, Netherlands

Pieter van der Zee
Ergon Structural Concrete, Belgium

Dick van Keulen
Delft University, Netherlands

Contact details of Task Group members can be found in the members only section of the *fib* website, www.fib-international.org

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In Chapter 12:
Jones, with individual case study contributions from:

12.2 Kaare K.B. Dahl (Ramboll, Denmark)
12.3 Pieter van der Zee (Ergon, Belgium)
12.4 van Keulen Dick (Ingenieursstudio DCK, Netherlands)
12.5 Kazuhito Sugaya (Sumitomo Mitsui, Japan)
12.6 Kazuhito Sugaya (Sumitomo Mitsui, Japan)
12.7 Lasse Rajala (Sweco Rakennetekniikka Oy, Sweden)
12.8 Francisco Graziano (Pasqua & Graziano, Brazil)
12.11 Mischa Falger (BAM Advies and Engineering, Netherlands)
12.13 Simon Hughes (Westkon Precast Pty. Ltd., Australia)
12.14 Simon Hughes (Westkon Precast Pty. Ltd., Australia)
12.15 Bin Zhao (Tongji University, China)
12.16 Hugo Corres Peiretti (Fhecor Ingenieros Consultores, Spain)

Foreword

Population density increases resulting from a general movement from rural to urban habitats, together with a soaring demand for access to sought after property sites in big cities and resorts, has resulted in increasing numbers of taller buildings over recent decades.

In view of their current popularity several references have been written on the design of tall buildings in steelwork and structural concrete. However, it was felt in fib Commission 6 for Prefabrication that there was not an up to date reference available for the use of precast concrete in tall buildings, that brought together in a single document the modern applications of precast concrete in tall building construction. Task Group 6.7 was therefore set up in 2014 to address this issue and to prepare a "State of the Art Report" on the subject. This report would focus on how to integrate precast concrete into tall buildings and aims to capture the interest and influence professionals and all parties involved in tall building construction through a single reference without being unduly theoretical in approach. The outcome is this bulletin.

Bulletin 101 is divided into four parts. The first four chapters introduce the reader to the benefits that can be achieved with precast concrete and how it can be integrated into any building as individual elements mixed with other construction forms or as precast systems themselves. Shafts, stair and service cores, division walls and floors are all parts of any functioning tall building and can be provided in precast concrete to act as the structural framework also.

The next four chapters cover the individual "building blocks" in precast concrete, i.e. floors, columns, walls and stairs. Their application to tall building construction is described with particular attention given to design and detailing and production methodology. There are then three chapters on areas of specific interest. These are building facades, precast in seismic zones and construction itself. The Bulletin then concludes with numerous case studies that illustrate the applications and benefits previously described. The Group particularly wanted to use case studies from as many different regions as possible, and we believe this has been achieved with examples from Europe, North and South America, Australia, Japan, the Middle East and China.

We are also pleased to have had close cooperation with PCI throughout the drafting process and that the Bulletin will be published by both the *fib* and PCI.

The Committee is grateful to all active members of the Task Group for sharing their knowledge and experiences. Our thanks go in particular to George Jones who successfully led and convened the task group in an inspiring way.

Stef Maas
Chair of Commission 6
Prefabrication

Harry Gleich
Chair of PCI
Technical Activities Council

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1. Introduction

1.1 Background

There has been considerable worldwide growth and advances in the construction of tall buildings over recent years. The variation in use of such buildings has been remarkable, from the most highly specified premium hotels and apartments to affordable and social housing.

Tall buildings use many structural materials to form their framework and shell, and generally mix those materials to deliver the building required by its owner. Precast concrete has been used by architects and engineers to good effect in this process, from whole frameworks to facades. In addition, precast elements are regularly mixed with both structural steelwork and cast in place concrete to achieve efficient building solutions.

This report aims to demonstrate how precast concrete can be effectively integrated into tall buildings using modern materials and techniques and draws on the experience and expertise in the global concrete industry to deliver this aim.

The document is not intended to be a manual for the design of tall precast concrete buildings per se. The design approach to tall concrete buildings has already been covered in *fi* bulletin 73[13]. The intention is to show how precast concrete can be beneficiall used in tall buildings. Where there are design aspects that are specifically related to precast concrete, such as connections between elements and achievement of structural robustness, these will be covered in the relevant chapters.

1.2 What is a Tall Building?

The definition of a tall building is not as simple as relating it to its height or number of storeys. A tall building is generally recognised as one whose proportions, context, or behaviour is different from other buildings because of its “tallness”, and consequently has significant influence on aspects of the design and construction.

The Council on Tall Buildings and Urban Habitat (CTBUH)[24] has produced several explanatory documents relating to the criteria for the defining and measuring of tall buildings. They have isolated three categories and define a "tall building" as being a building that exhibits some element of "tallness" in one or more of those categories, which are:

- Category a): Height relative to context

This category relates to the context in which the building exists. For instance, a 14-storey building in Hong Kong or Dubai may not be considered tall but would be in a European provincial location if surrounded by low rise buildings several storeys lower. Fig. 1-1 illustrates the effects of context.

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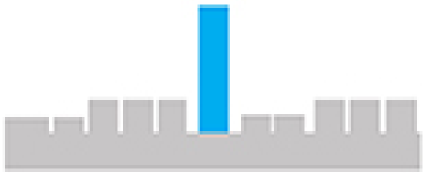
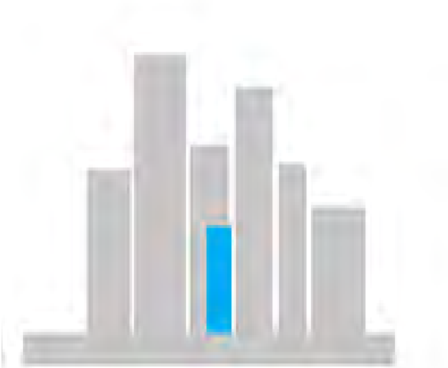


14 storey "tall building" in provincial location



14 storey building amidst "tall buildings

Fig. 1-1: Example of height relative to context.

- Category b): Proportion

There are numerous buildings which are not particularly high but are slender enough to give the appearance of a tall building, especially in low urban backgrounds. Conversely, there are numerous large footprint buildings that have many floors, but their slenderness ratios (height / plan dimension) are relatively small and would not be considered tall buildings, as illustrated in Fig. 1-2.

Lender building in urban landscape

Large footprint building

Fig. 1-2: Example of proportion.

- Category c): Tall Building technologies

If a building contains technologies which may be considered a consequence of being "tall" (such as specialised vertical transport technologies, special wind bracing systems etc.), then the building can be classed as a tall building.

As we can see from the categories above the number of floors is normally a poor indicator of defining a tall building. However, CTUBH suggest that a building of 14 or more storeys, or over 50 metres, could be used as a threshold for considering a building as a "tall building" subject to the categories described above.

1.3 The Application of Precast Concrete in Tall Buildings

Tall buildings are often characterised by a few common parts. These parts are in turn governed by several factors such as occupancy (residential, office, retail etc.), fire strategy, vertical transport systems (lifts), visual appearance, services (water, electricity etc.) and structural system. Nonetheless, the building parts themselves can be generalised as follows:

- The lift and stair core;
- Escape stairs and landings;
- Isolated vertical loadbearing elements; columns and walls;
- Floors; slabs and beams;
- External façade.

All these general building parts can be provided in concrete, and more specificall in precast concrete. In addition, many tall buildings have layouts that repeat over many storeys, even though the basic building footprint may be relatively complex. This increases the attraction of factory produced concrete elements. These advantages will be dealt with in greater detail in later chapters, but examples of buildings with non-regular and curved elevations and edge profiles, difficult to construct insitu, are shown as an example in Fig.1-3 as precast solutions.

The general building parts listed previously can be further subdivided into precast concrete elements that make up those parts, for instance:

1. The lift and stair core: Wall panels (although often limited to the lower end of the tall building range, slip formed insitu concrete is generally the preferred method of construction for taller buildings). These are typically flat panels but can be moulded as “L” or “U” shaped elements to give greater rigidity at corners and aid stability during installation.

2. Escape stairs and landings: Precast concrete stair flights and landings are currently the most used means for providing escape stairs in tall buildings. They remove a lengthy site operation required for insitu construction and are a robust and fire resistant solution compared to other materials, such as steel and timber. Close interaction with the slipforming operation can result in the stairs following close behind and remove a critical activity from the programme.

3. Isolated vertical loadbearing elements: Precast concrete columns and walls are becoming widespread in their application to tall building construction. Many tall building layouts typically have a central core with isolated columns and walls (often referred to as blade columns) remote from the core and around the building perimeter. Tight floor cycle times (the construction time from floor to floor) and limited crane availability mean that precast columns and walls are often essential to achieve critical targets. Depending on the crane lifting capacity vertical elements can be produced in single storey or multi storey heights and can interact with both insitu and precast concrete flooring systems. High strength concrete is an added advantage to maximise structural capacity in relatively small sections.

4. Floors: Horizontal diaphragm action is often essential in the stabilisation of tall buildings. Lateral actions applied to the building are transferred to the stabilising vertical element (normally the core) via the diaphragm action of the floor slab. Hence, the floor system not only transfers the gravity loads to the vertical elements, but also the horizontal actions. Precast flooring systems therefore must be designed for these conditions and are often used compositely with insitu toppings and stitches. Flooring systems can be either flat slab or beam and slab, and many types of flooring systems are available; reinforced, prestressed, voided and with the option to incorporate services in the prefabrication process before arriving on site.

5. External façade: Nowadays, there can be an initial tendency to use full panel glazing for external facades in tall buildings, but precast concrete architectural panels are still in widespread use and appeal to architects because of the broad range of colours, textures and shapes that are available. Precast concrete façade panels are generally non loadbearing and transfer their own self weight and applied horizontal actions to the building structure. They can be fabricated with windows and services already fixed, and with insulation sandwiched between an external aesthetic façade and an internal structural wythe.

The above is a brief introduction to the use of precast concrete in tall buildings. The following chapters will describe the benefits of using precast concrete more fully, will then show how precast concrete can be used in mixed construction with other materials, and then describe where precast is used as the main structural medium in framing solutions. The various precast elements described above will then be examined to show how they can contribute to tall building construction. There follow chapters covering seismic zones and installation. At the end of the document there are several case studies that will further illustrate the topics previously covered.

The aim is to produce a state of the art report describing the many possibilities for precast concrete in tall buildings, and perhaps showing uses, applications and benefits that readers may have been unaware, have not previously considered, and therefore may be useful in future projects.

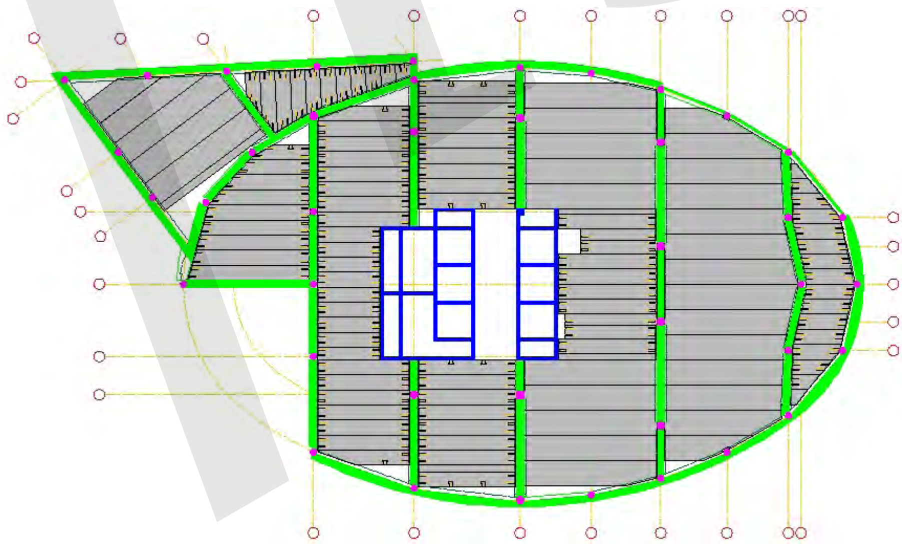


Fig. 1-3: Non regular external building edges but ideal for a precast solution due to repetition at each floor level, Central Plaza, Brussels (courtesy of Ergon Structural Concrete).

2. Benefits of Precast Concrete in Tall Buildings

2.1 General Project and Design Benefits

The benefits of precast concrete have been widely documented and include strength, integrity, versatility of form and use, compatibility with other structural forms, speed of construction, off site dependability, assured and improved quality, durability, security, fire resistance, sound and thermal insulation, energy conservation, and increased certainty. These benefits can be realised in tall buildings, but the design philosophy of any chosen system should be closely followed where possible and the precast system not solely used to copy or modify an initial insitu solution. Basic aspects of any precast system should include:

1. Standard solutions from modular design (element widths of 3 M, 6 M, 12 M or 24 M, where M = 100 mm). The producer will often have standard mouldage based on modular design. Customised elements outside these parameters may result in further work and extra cost; however, taller buildings have the advantage of many repetitive floor levels and may merit customisation. Modern automated and robotic systems also accommodate non-repetitive and customised solutions.

2. Standard products that have been developed by the precast producer(s); again, repetition of floor levels in taller buildings may mean it is attractive to develop variants to standard product to suit the building.

3. Details, particularly at connections, should be as simple and repetitive as possible. The more complicated a detail the more likely it is to go wrong and the longer it will take to achieve. It is likely that standard details used for lower rise buildings will have to be modified and / or strengthened for taller buildings, but the same basic principle still applies.

4. Taking account of dimensional tolerances. Absorb tolerances at connections using mortar beds or grouts, or insitu stitches where necessary.

5. Using insitu infills where appropriate. Do not force the precaster to precast areas that are obviously unsuited to the system.

There are many cases where precast concrete has been chosen as a feasible form of construction, but only at a late stage in the planning process. This is often because concept designers only consider insitu concrete and structural steelwork as the primary structural forms. This may have arisen because precast design has been carried out by producers employing in house specialist engineers and the concept designers are not familiar with the aspects of the precast systems mentioned above. Such gaps in training and outlook should be remedied by the organisations involved.

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It is often when the commercial and construction expertise of the contractor is brought to the table that other options are investigated. However, in many such cases precast concrete is restricted to the substitution of components already detailed as steel or insitu concrete and many of its benefits are lost in the rush to commence construction and achieve unrealistic delivery dates.

There is, however, a developing trend where the contractor has its own in house precast specialist, and often a precast production capability, and assesses the suitability of all materials regarding quality, cost, and buildability at tender stage. Precast concrete has a place in this assessment, which has seen the development of mixed or hybrid solutions in tall buildings. Precast concrete, insitu concrete and structural steelwork are used together to achieve the most beneficial solution using the respective properties and advantages of each material.

Often an insitu concrete slip or jump formed core would be used to provide general building stability (example in Fig. 2-1a), but then infilled with precast concrete flight and landings as the core construction progresses so that the core is fully accessible at the end of the slip form operation. This choice utilises the monolithic properties of insitu concrete to achieve early building stability and then the speed, offsite dependability and quality of the precast flights and landings for early access.

Fig. 2-1a: Nido Spitalfields, London, using precast concrete blade columns.

Fig. 2-1b: Shell Building, Melbourne.

The remaining floor areas may be insitu reinforced or post tensioned flat slab construction to provide a two way spanning solution and account for the column grid, or precast floors with insitu toppings to provide diaphragm action.

Any balconies projecting outside the building shell could be precast concrete with thermal insulation barriers to avoid cold bridging; again, off site dependability, quality, speed, and safety being important factors. The remaining vertical elements outside the core could be precast concrete to minimise the floor cycle time, so important in the early completion of tall buildings, i.e. the time taken to move construction from floor to floor.

Examples of buildings using a slip formed core with precast vertical elements outside the core and insitu floor slabs are shown in Fig. 2-1a at Nido Spitalfields, London, and the Shell Building, Melbourne in Fig. 2-1b, which has external polished granite spandrels and columns combined with insitu 15 m span post tensioned floors and insitu stability core.

An idealised floor plan is shown below in Fig. 2-2 that will be used to illustrate several later examples of benefits that can be achieved. It could be considered as typical of a residential building on a confined city centre site, with a central circulation core, and "blade columns" outside the core. The term "blade column" is one that is often used, however, these isolated elements are often between 200 and 300 mm thick, to fit into division wall locations, and exceed the 4:1 ratio of horizontal length to width that defines a wall. They, therefore, sometimes attract significant in plane moments due to their relative stiffness in that direction and their design should take account of that. If the building shown in Fig. 2-2 was to be used for office accommodation then the "blade columns" would most likely become conventional square or round columns to allow an open plan space.

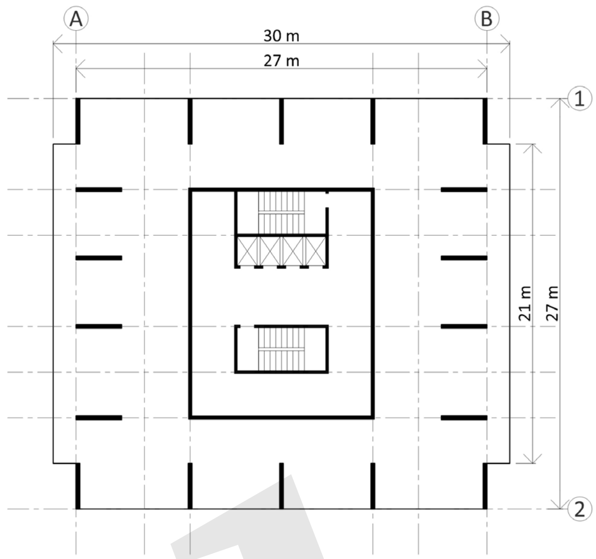

Fig. 2-2: Idealised floor plan.

In the previous examples we considered the benefits of using precast concrete in mixed solutions. Precast can also be used as a total framing solution, although not common for the type of tall building in Fig. 2-2, they are more common in cellular buildings as in Fig 2-3 and 2-4.

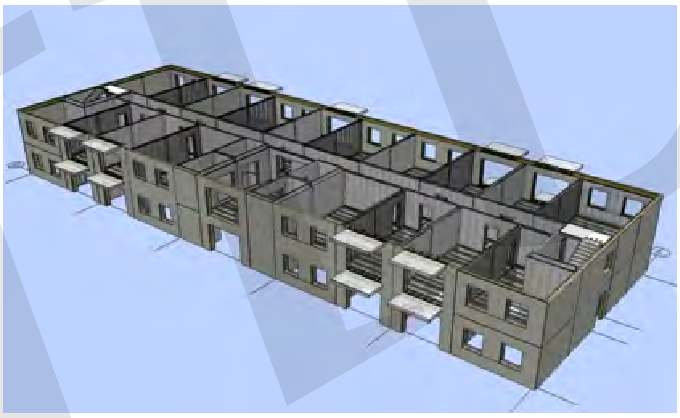

Fig. 2-3: Typical storey of precast concrete cellular building.

Fig. 2-4: Elevation of precast concrete cellular building.

Buildings of this type normally have relatively small individual dwelling units and could be used for low cost social and affordable housing. Due to the high number of individual dwellings there is a high concentration of walls. These buildings are generally regular in both shape and appearance, and they ideally require a high degree of sound insulation. Precast concrete is an optimum solution and would provide cost and speed benefits through standardisation and repetition.

Unlike the previous examples, where lateral stability is provided mainly by a stiff central core, in the case of cellular buildings all the walls can contribute towards a general building stiffness to resist horizontal actions.

Precast concrete sandwich panels are used to insulate the external façade, as shown in the cross section in Fig. 2-5 where thick mineral wool is sandwiched between concrete wythes.

Floors can also be used as part of the building thermal fabric with either underfloor heating or coolant pipes cast into precast ceilings to minimise air conditioning.

Fig. 2-5: External sandwich panel after production.

Due to relatively short spans in taller buildings slabs do not necessarily need to be two way spanning and precast flooring elements are an ideal solution to span between fixed lines of support at wall locations. Joints can be designed to provide both shear transfer and tying systems (see later chapter on floors and diaphragms). However, a screed covering the precast flooring units is often specified to ensure level consistency and could be used as a structural topping to meet structural integrity requirements.

We generally think of the superstructure when considering tall buildings, but a major part of the building is at ground level and below. Typically, the basement areas are used for car parking and a regular grid of columns is preferred to suit parking and manoeuvrability guidelines. This often means that the column grids must be adjusted at the level where the occupancy changes from car parking to residential / retail / office. This change in occupancy leads to changes in the structure where loads are transferred between vertical elements. Large beams are often needed that would require substantial temporary works and propping if poured insitu. To overcome these issues self-supporting precast beams and slabs act compositely with insitu concrete infills and toppings to give a monolithic transfer system. An example showing precast U beams and slabs is shown in Fig. 2-6:

Fig. 2-6: Precast U beams and slabs at a transfer level.

Column starter reinforcement can be provided along the span of the composite transfer beam and inside the insitu beam infill. The load path along the transfer beam between the two offset columns is then complete.

Vertical basement elements in tall buildings also present challenges for various reasons. Columns normally must resist substantial compressive axial forces due to the large number of supported floors above, but also should have as small a cross section as possible to assist the car parking arrangement and maximise the number of car spaces. High strength concrete, up to 120 N / mm² cube strength, is sometimes used in columns to achieve this aim. The columns are often directly jointed together at floor levels with projecting corbels and nibs to support the adjoining floor and to avoid a zone of weakness through the weaker concrete in the floor plate.

The perimeter wall can also present challenges due to high water pressures in multi storey basements, and difficulties in gaining access to the back face of the wall due to the proximity of site boundaries, and confined excavations and working space.

Fig. 2-7 shows construction of a multi storey basement with high strength precast columns and perimeter walls in view.

Fig. 2-7: Precast basement perimeter wall and high strength columns, Dublin, Ireland.

In Fig. 2-7 the precast wall acts compositely with insitu concrete backfill to achieve a composite construction. Notches have been formed in the precast walls to provide end supports for precast beams that will be installed later. High strength precast columns can be seen with projecting corbels and nibs to support beams and slabs, respectively. The benefits of strength and versatility can be seen in this example.

There are often areas with access difficulties at high levels that make insitu construction impractical and precast becomes the ideal solution. Typical examples are capping slabs across tall shafts, where temporary support for insitu slabs is both extremely difficult and has obvious health and safety issues. In such cases, self-supporting precast slabs remove those problems.

In the example shown in Fig. 2-8 and 2-9 a complex diamond shape had to be formed at roof level. Edge concrete beams and columns had to be constructed that interacted with a steel framework. The precast part of the structure was modelled in 3D and the model exchanged with both the steelwork specialist and the architect to develop a compatible solution.

Whilst this was a departure from the basic guidance of using simple, standardised, modular systems it was recognised that precast concrete was the only solution for this situation and was proved to be successful.

Fig. 2-8: Corner column completed at roof level, Blackfriars, London.

In Fig. 2-8 two separate columns were joined at the top to form the topmost corner of the roof construction. The steel roof framework joined the precast columns at numerous points via welded connections onto steel plates cast into the faces of the columns. There were also many lengths of cladding channel that had to be incorporated. The degree of accuracy and complexity required was achieved through common 3D modelling between the precaster, steel specialist and architect.

Fig. 2-9: Edge roof beams rising to meet corner column in Fig. 2-8.

Edge precast roof beams spanned between precast columns and sloped from a low point at one end to meet the corner column shown in Fig. 2-8. The beams were insitu stitched over the top of each column with the insitu concrete contained inside the precast beam section, i.e. no site shuttering or propping required. The roof structural steelwork awaiting connection to the precast beams can be seen in the foreground. This is a good example of the versatility of precast concrete in both form and use. A difficult and complex shape and arrangement achieved in a location that would have been almost inaccessible for insitu concrete construction.

We have considered how the benefits arising from using precast concrete can influenc projects and their design, and how those benefits can either be achieved or lost. In the next sections we will investigate the quality and material benefits that arise, and then the construction site benefits.

2.2 Quality and Material Benefits

We all witness the evolution of society in our lives and how it is influenced by global communication, information processing and transfer, industrialisation, automation, and more efficient use of the Earth's limited resources.

However, whereas design methods and management techniques become ever more refined as technology advances the building process itself has continued, to a large extent, to remain a labour intensive, site based operation, particularly with respect to concrete construction. To meet the demands of a modern society building construction needs to move towards industrialisation and remove inherent risks and uncertainties that impact on quality and the efficient and sustainable use of resources. Precast concrete is a factory made product, generally produced remote from the construction site in a facility purpose made for that product. Therefore, the benefits of industrialisation can be achieved.

This happens through the following:

- Consistent working conditions;
- Skilled workers;
- Repeated actions;
- Automation of operations that is achieved through repetition. These include reinforcement preparation, mould assembly, concrete casting, and surface finishing;
- The capability to inspect and reject product before delivery to the construction site;
- The capability to check installation methods on a full scale basis before site delivery;
- The removal of risk due to weather and supply shortages;
- Improved health and safety;
- Higher degree of accuracy and quality;
- Less waste.

Fig. 2-10: Long line slab bed in precast factory.

What are the benefits of industrialisation and factory produced product that can be realised? They can be summarised as:

1. Increased certainty that the quality required is achieved.
2. Time certainty: The process is unaffected by weather and provided the drawings are provided in time to the factory then the components are produced in advance of the construction process; we then have offsite dependability.
3. Value for money: Time and quality certainty brings budget certainty. Financial professionals in the past have failed to recognise benefits other than direct construction costs. This may have cost clients dearly, particularly in relation to early return on investment.
4. Reduction in site mistakes, due to high degree of checking and testing at the factory.
5. Fewer people at the construction site; with consequent health, safety, welfare, and security benefits.
6. Greatly reduced risk of missing the floor cycle time in tall building construction.

To achieve these benefits contributions are required from others. It was mentioned earlier that benefits could be lost through not respecting the basic aspects of a system, but of as much importance is the supply of information from design professionals in a timely manner to allow the precast production drawings to be finalised to meet the casting programme. Factory produced concrete also allows the efficient use of materials. Prestressing provides greater concrete section strength than conventional reinforcement, allowing shallower concrete sections compared to reinforced components. Voids formed in concrete sections such as hollowcore slabs allow up to 45% reduction in slab self-weight whilst maintaining the same strength as a comparable solid section. Greater control over material supply through factory checking systems largely eliminates waste.

The use of concrete with cube strengths greater than 60 N / mm^2 is now commonplace in precast factories. This extra concrete strength assists both vertical elements subject to high compressive forces in their permanent position in the building and prestressed horizontal elements that require a large amount of prestress to resist design vertical actions. High strength concrete columns have major benefits in tall buildings as smaller sections are required, thus increasing the nett floor area in buildings with a small footprint as in Fig. 2-2. Care must be taken if storey height columns are used with an insitu floor plate sandwiched between, as lower strength concrete in the floor may negate some of the advantage in the high strength column. Locally placed high strength patches at column locations can overcome this problem.

Costly, time consuming mistakes on site can be avoided by producing prototype structures at the factory that model the installation procedure and allow accurate estimates of the time that must be allowed in the construction process. In Fig. 2-11 a single storey of a precast walled core with integrated precast landings and flights was set up and installed by the intended installation crew. This system was later successfully used in a building of twenty storeys.

Fig. 2-11: Prototype storey height precast walled core and stairs for Battersea PowerStation Development, London.

2.3 Construction Site Benefits

There are significant benefits in quality and time certainty, plus a greatly reduced skilled labour requirement at the construction site. However, before these can be realised there are several important issues that have to be addressed. These are:

1. Lifting and crane requirements: The siting and size of cranes is vitally important. The point where the crane unloads the delivery vehicle can often be the determining factor in the maximum weight of precast element that can be used. It is often the maximum weight of element that determines the benefit accrued in relation to both time and cost.

2. The interface with connecting structure: The connection detail must be agreed at an early stage and must be as simple as possible. An as built survey of any connecting starters and base levels is essential, preferably before the connecting precast elements are manufactured, so that details can be adjusted to suit if necessary.

3. Early confirmation of builders' work required by following trades, any cast-in items, or penetrations. Free issue materials to be cast into the precast elements need to be delivered to the factory in good time before precast manufacture.

4. A realistic and accurate delivery schedule: This should allow the producer to manufacture the components in a timely manner and avoid unnecessary prolonged storage at the factory.

Early off site inspection and approval of product: It is essential that confirmation is received at the earliest possible stage that the product being manufactured meets the specified requirements.

If we consider the floor plan example in earlier Fig. 2-2 then the contractor may want to precast the "blade columns" outside the core to improve the floor cycle time. There are eighteen elements of that type at each floor level.

Firstly, it must be ensured that the cranes provided can lift the precast elements to each part of the building. In this case the precast elements are the same cross section, 3000 x 250 mm. As the building is residential then a storey height of 3 m can be expected and a flat slab depth of 250 mm for the insitu floo . This means that the precast element weight will be 5.2 tonnes for single storey elements and suitable craneage needs to be provided. This may mean that a bigger crane would be needed compared to pouring the columns insitu and this extra cost becomes a factor in the decision to use precast columns.

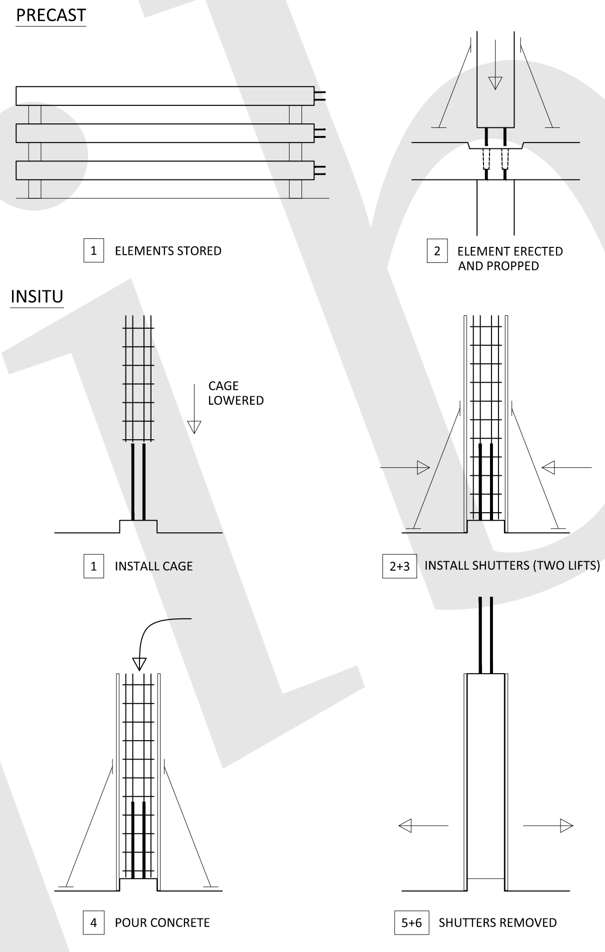

Fig. 2-12: Example of comparable column construction methods.

The main site benefits that would be realised in using precast columns would be a reduction in floor cycle time and required working space (often critical in a tall building with a relatively small footprint). Fig. 2-12 illustrates the number of crane lifts that could be required for precast installation compared to insitu construction of columns, i.e. two lifts for precast (this could be reduced to one if the elements can be installed directly from the delivery trailer) and six for insitu. If we consider, say, 15 minutes per lift then it takes one crane hour longer for each column if insitu is chosen. Therefore, in our example eighteen crane hours are saved per floo . Often double storey height columns are used, and time savings can be doubled. However, in the case we are currently considering the floor cycle time would be reduced by at least two days if there is a single crane and one day if two cranes are available. For a tall building with many floors this saving may be critical. This is perhaps a simplistic viewpoint as the builder may argue that fewer crane lifts are required for formwork installation and removal depending on the complexity of the column plus precast elements have to be lifted further as the building height increases, however, the other operations involved in the construction process must also be considered, e.g. time required to place reinforcement, formwork, and concrete. In that case insitu construction would require more time outside the lifting operations and would further increase the time benefit in choosing precast.

There are also the added benefits of certainty in both quality and strength. The precast element would have already been checked and passed for incorporation into the building and would be close to its eventual design strength having been cast many days before in factory conditions. The comparable insitu column would still have to be inspected once its formwork was removed and must undergo curing and strength gain at the exposed site, with the contingent risks and uncertainty. The types of connection are considered in more detail in later chapters, however special care should be taken in relation to the cleanliness of the joint to avoid accumulation of dirt that would reduce its capacity. This could be accentuated where tie bars are projecting from the bottom of the precast element.

The building in Fig. 2-2 could also have precast concrete balconies along gridlines A and B. The contractor could cantilever the supporting steelwork for the insitu floor to in turn support a precast balcony with a structural thermal break, like those shown in Fig. 2-13 and 2-14. The balconies are then attached integrally with the floor when the insitu slab is poured.

Fig. 2-13: Precast balcony with large edge upstand.

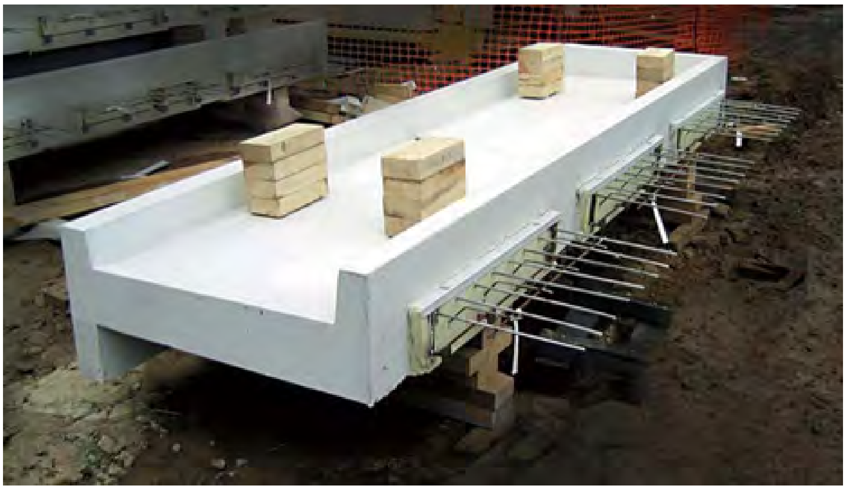

Fig. 2-14: Cantilever balconies with structural thermal break.

Balconies often have many cast-in items and penetrations for balustrades, glass screens, drainage, and services generally. Precast balconies remove the incorporation of this detail off site. Site safety and access is ensured if the perimeter safety screens are extended outside the balcony edge.

Other operations can be moved off site, particularly in relation to façade panels, where elements can be delivered with windows already fixed, external finishes applied, and insulation included. Shapes can be moulded to provide the visual impact required.

The repetition provided in the many storeys of a tall building provides the basis for making precast concrete cost effective. The possibilities for using precast concrete are there to be explored and solutions found, but in general, the modern consensus is that offsite production can only benefit and shorten the construction process.

3. Integration of Precast Concrete into Mixed Construction

3.1 Types of Mixed Construction

Precast concrete can be mixed with both insitu concrete and structural steelwork to provide a technical and commercially viable solution[10].

It is generally recognised that precast concrete currently has its major application in tall building construction when mixed with other materials rather than being the main structural form. There could be many reasons for this but can often be because of the construction culture of the region; the region could have a bias towards either insitu concrete or structural steelwork depending on regional building history. Nevertheless, precast concrete still has its place in the construction process and may bring benefit when integrated into the project that are difficult to achieve with the other two more traditional systems.

There are numerous combinations when using precast concrete in mixed construction. Some of the more well-known ones are:

- Slipformed cores with precast concrete stairs and landings;
- Insitu cores with precast concrete columns, beams, and slabs;
- Precast columns and walls with insitu concrete floors;
- Precast cores with insitu concrete floors;
- Composite precast and insitu concrete floors;
- Precast floor slabs with steel beams;
- Insitu concrete or steel frames with precast concrete facades;
- Precast concrete balconies with insitu and composite floors.

In tall buildings, the cores are often the most important parts from several aspects; structural stability and integrity, their large plan area relative to the overall building footprint, means of access, housing both the lifts and other services. Early completion of the cores releases many other work areas.

Slipformed insitu concrete cores have become popular because of their speed and obvious time benefits. They climb continuously and it is possible to pour a full storey in twenty four hours using shift work. However, there are drawbacks in relation to poor finish, inaccuracy of cast in inserts and relatively large tolerances, up to +/- 40 mm in plan position. There is also the cost and the time required to assemble the steel rig and start the slipform process.

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The cut-off point where slipforming becomes viable is between fifteen and twenty storeys. Below this cut-off, the options are mainly either jump formed insitu cores (where the shutters are moved up a storey at a time and have a cycle of about six days), or precast concrete cores. Depending on available craneage the precast core option can be attractive, as completion of one storey per day for individual shafts is easily achievable below the cut-off level mentioned earlier. Whilst there may not be a structural engineering reason why precast cores cannot be used for buildings greater than twenty storeys high it is often the time taken to handle precast elements at greater heights when compared to pumped insitu concrete that may limit their use. Site imposed, rather than technical, limitations can be the deciding factor.

Fig. 3-1: Slipformed concrete cores at Blackfriars, London.

3.2 Precast Concrete Mixed with Insitu Concrete

3.2.1 Slipformed cores with precast concrete stairs and landings

We have mentioned already that the core is an especially important part of a tall building. The core is often made from insitu concrete for buildings higher than twenty storeys and not only consists of a series of shafts for lifts and services, but also stairs. Vital time gained through slipforming can be consolidated through use of precast concrete stairflights and landings, that can either be installed as the slipform progresses or at the end in a single operation. Fully precast flights and landings can be provided and installed using a minimum amount of grout to complete connections that can be hidden from view.

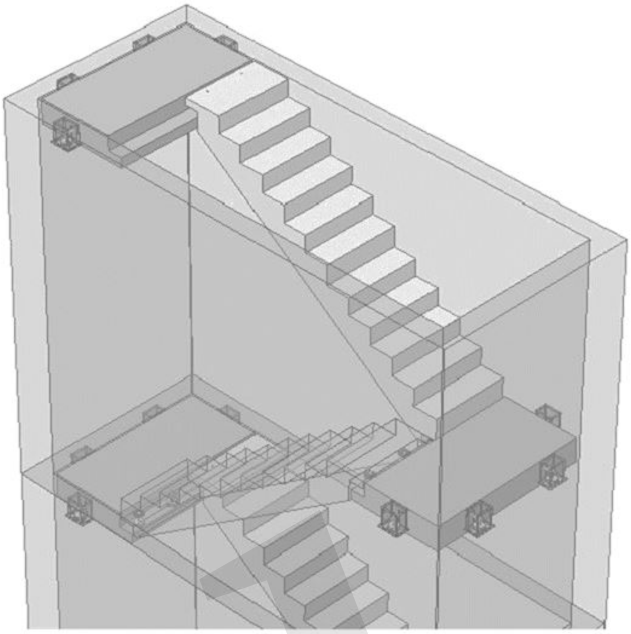

Fig. 3-2: Precast stairs inside a “box” with hidden connectors, courtesy of J & P Building Systems Ltd.

The landing connectors can also be acoustically sealed to minimise sound transfer to adjoining rooms. For a full precast solution, the connections should be carefully designed in seismic areas to account for potential movement.

3.2.2 Insitu cores with precast columns, beams, and slabs

Insitu slipformed or jump formed cores are often used with precast structure in the remainder of the building outside the cores. The cores are relatively much stiffer, particularly if designed as fully integrated boxes with shear transfer at corners, compared to the vertical structure outside the boxes. Columns in office buildings will therefore attract relatively small amounts of lateral loading. However, the building’s tallness may induce secondary effects that need to be considered.

For taller buildings where structural tube systems are adopted for lateral stability purposes the forces acting around the building perimeter need to be carefully analysed and lateral effects dominate resulting in closely spaced rigid elements required to produce a stiff hollow tube. In buildings less than about forty storeys that are stabilised by cores column designs are likely to be dominated by axial gravity forces. The beams and slabs provide the horizontal diaphragm that transfers the lateral forces to the stabilising core near the centre of the building. Precast concrete columns can be moulded to any shape required by the Architect and there are a large variety of beams and slabs that can be used; however, it is often necessary to provide an insitu structural topping over the slabs to complete the diaphragm.

Fig. 3-3: Precast concrete double storey columns and prestressed beams, Brussels (Note insitu concrete topping to complete floor diaphragm).

Fig. 3-4: North Galaxy, Brussels. Columns and beams as shown in Figure 3.3 with insitu stability core.

In the example in Fig. 3-3 and 3-4 circular precast double storey columns are used. Due to high axial loads there is a grouted connection between columns with projecting corbels to move the beam bearings away from the column shaft, avoiding the column bearing onto the beam ends. Projecting links are provided from the beams that anchor the peripheral tie, that also passes through and links the columns. The prestressed beams support double tee slabs that were chosen for their low self-weight compared to span capabilities. Both prestressed beams and double tee slabs act compositely with an insitu concrete topping that completes the lateral diaphragm transferring horizontal actions from the building perimeter to the insitu stability cores.

3.2.3 Precast columns and walls with insitu concrete floors

A commonly used form of construction is to mix precast concrete vertical elements with insitu concrete flat slabs.

This is particularly popular with insitu concrete frame contractors who are looking for minimum floor cycle times in tall buildings, i.e. the construction time from floor to floor outside the cores. Flat slab construction greatly improves floor cycle times when compared with ribbed and waffle slabs.

Shuttering tables for insitu flat slabs that are fast to assemble can be dismantled from completed floors below and moved upwards as the precast elements are installed. For tall buildings with small footprints like the example in Fig. 2-2 a typical floor cycle time would be between 5 and 7 days, precast vertical elements would only take up 1-2 days of the cycle.

The choice of precast concrete vertical elements is often decided by the site crane lifting capacities. It may be necessary with longer walls to stitch component parts together with insitu concrete if the crane capacity is exceeded. This often happens when precast concrete is chosen late in the construction period due to time constraints and craneage has already been installed without considering the impact on precast concrete as part of a mixed solution. When mixed with flat slab construction the precast vertical elements are generally manufactured in single storey lengths and jointed across the insitu concrete floo . Coupling systems or common connecting bars grouted into ducting cast into the precast elements are typical means of connection. Double storey height columns can also be used that have exposed reinforcement connecting two storey height sections.

The insitu flat slab can be either traditionally reinforced or prestressed post tensioned concrete. Post tensioning can reduce slab thickness over many storeys in tall buildings resulting in either an accumulated cost saving through height reduction or an increase in number of storeys. The Shell Building in Fig. 2-1b is an example of this form of construction.

The floor type can be important to precast vertical elements; reinforced slabs are generally more heavily reinforced with passive reinforcement, with the possibility of continuity reinforcement congestion over supports, sometimes making it difficult to accommodate the precast element connection due to the extra area required for couplers and / or ducting. Post tensioned slabs are more lightly reinforced with relatively well spaced prestressing tendons. However, prestressing effects and the method of analysis used in the floor design need to be considered in design and detailing the precast vertical elements.

Fig. 3-5: Battersea Power Station Residential Development, London (Example of precast concrete columns and walls with post tensioned insitu slab, courtesy of Byrne Brothers(Formwork) Ltd.

The columns are idealised in the floor design as either pinned or fixed at their ends, the element detailing needs to reflect this thinking. It may also be convenient for the floor designer, who may not also be the precast designer, to distribute relatively large moments to the columns to maximise the structural efficien y of the slab. Close coordination between designers is therefore required to maximise overall project benefits. A structurally efficien slab should not result in columns or walls that are overstressed.

In tall buildings there can often be a change in occupancy between levels resulting in a change of structural section size in the vertical elements. For instance, in a city centre residential development the basement levels would most likely be allocated for car parking, at ground level there could be reception areas and retailing, with apartments, offices or hotel bedrooms at the upper levels. Therefore, the section could start as circular at the lower levels where a more open arrangement is required, and then change to walls or "blade columns" at higher levels where there is a more cellular layout.

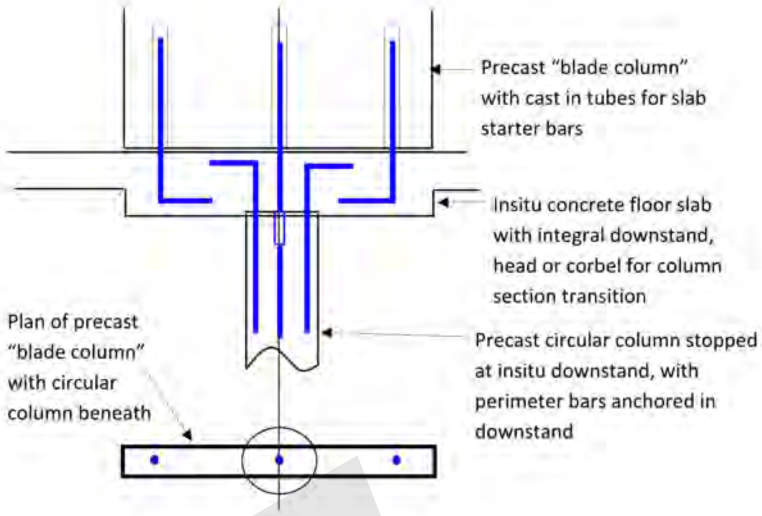

Fig. 3-6: Example of circular section below, changing to "blade column" above.

The transfer of forces at these interfaces need to be carefully considered. The common interface between the two sections may be relatively small and it may be necessary to provide a transition head or corbel between the two different sections. An example is shown in Fig. 3-6. The transition concrete could be heavily reinforced and congested. Therefore, it is normal for the precast element below to stop at the bottom of the transition, the transition would then be poured insitu with starters or couplers provided from the floor slab, and the precast would then resume from top of floor slab.

Details of this type can be varied and complicated depending on the position and size of the elements below and above the floor relative to each other. There may also be instances of "walking columns" where the columns / walls are displaced in plan relative to each other. This is another added complexity and is likely to introduce large forces in the slab transition, which again should be poured insitu. These problems can obviously be avoided if the layout can be efficiently adjusted and should be brought to the Architect's attention at an early stage in the design process.

3.2.4 Precast cores with insitu concrete floors

Precast cores and shafts are often used with both insitu and precast concrete floors for buildings up to twenty storeys, and sometimes beyond, but vertical and horizontal lifting constraints can restrict their use at higher levels.

Fig. 3-7: Example of precast concrete staircore using flat panels.

Fig. 3-8: Twin shaft produced as complete plan section with horizontal joints only.

Precast stair-cores are often provided as flat panels connected at their corners with fully precast flights and landings integrated into the core. It is possible to provide support for the landings through recesses in the panels without use of proprietary steel connectors or insitu stitching. This makes the connection faster to complete and commercially attractive.

Fig. 3-7 shows a flat wall panel being installed in a precast core where the panels are connected at the corners and linked by the landings. Note the top detail of the panel being installed where an upstand acts as a shutter for the insitu slab edge, also couplers for the wall continuity reinforcement and exposed reinforcement for the floor connection can be seen along the top edge of the panel.

In the case of shafts, it may be feasible to provide the full shaft section and therefore avoid any vertical joints between elements.

In tall buildings, there can be large vertical shears concentrated at panel corners in cores and shafts that may require insitu concrete stitches as connections. Fig. 3-8 is an example of a pair of shafts that have been produced as single section segments. In this case four components have been used to complete one storey, but all vertical joints have been removed. Note that the horizontal connection to the insitu floor is through coupled reinforcement on the vertical face of the precast element.

Staircores and shafts can also be formed using precast twin wall (also known as double wall) systems which consist of two outer precast concrete biscuits, each about 65 mm thick, connected by steel lattice trusses. The void between the biscuits is then filled with insitu concrete once the panels are in place. These systems are ideal when thick walls are required that exceed crane capacities if made solid and provide insitu concrete monolithic connections between precast elements and adjoining insitu floors. The main drawback is that there is an additional construction stage introduced that may impact on the construction programme.

The connection between either a precast staircore or shaft and an insitu floor slab is dependent on the floor design, and the connection should reflect the requirements of that design. However, the slab designer should take account of the precast walling system that has been chosen and not develop details that are either difficult or impossible to achieve.

3.2.5 Composite precast and insitu concrete floors

There is a large selection of precast concrete flooring systems that can be used with insitu concrete toppings in tall buildings. These systems are described extensively in Chapter 5.

Fig. 3-9: Flat slab solution using Airdeck precast slabs composite with an insitu topping, courtesy of Airdeck Building Concepts nv.

Flat slab construction is popular in tall buildings because it is fast, generally has simple and repetitive details, provides a flat soffit for attaching and routing services, penetrations through the slab are easily incorporated and can accommodate an irregular support layout relatively easily as it is not reliant on support beams. There are precast flooring systems that can provide flat slab solutions. These tend to be reinforced concrete systems, particularly in tall buildings with small footprints that have relatively short spans, however they can also be used in a post tensioned solution with the structural topping.

Relatively thin precast biscuit slabs (also called wideslab, filigree, filig an and shuttering slabs), generally between 50 and 75 mm thick, have projecting steel lattice trusses produced from similar or the same production systems as twin wall. It is therefore possible to provide a complete solution of walls and floors from the same production machinery.

Temporary propping at calculated spacings is required until the biscuit slab becomes composite with its topping. The biscuits contain the main bottom tension reinforcement. Top reinforcement is provided in the topping to allow continuity over supports and two-way spanning. Void formers can be provided if greater section depths are required for longer spans and result in self-weight reductions of up to 30 %.

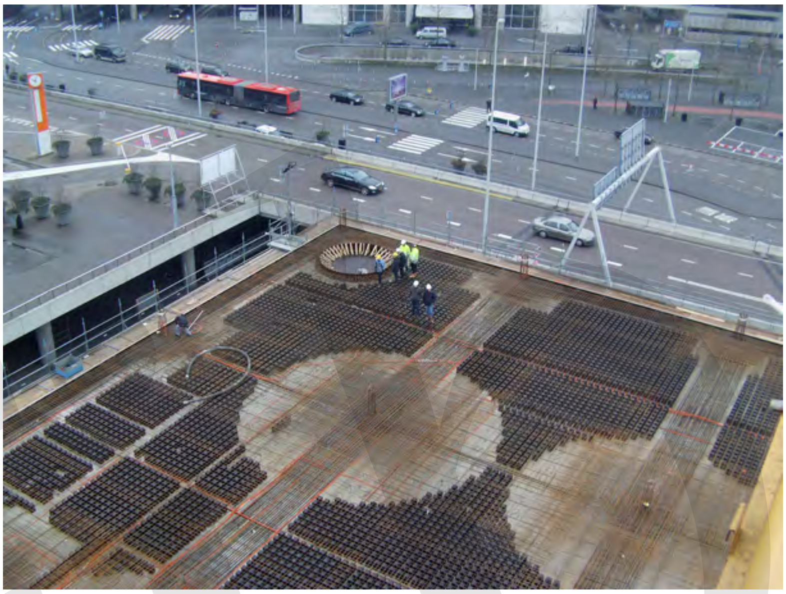

Fig. 3-10: Installed Airdeck precast biscuit slabs with localised void formers in combination with post tensioning covering full floor area (see also Chapter 5). Act as composite flat slab when combined with insitu concrete topping, courtesy of Airdeck Building Concepts nv.

Where composite flat slabs use precast biscuits, and the floor is designed as two-way spanning, finite element analysis can be used to plot the locations of minimum and maximum bending moments and shears. This is then helpful in producing a slab layout so that joints are located close to points of zero moment, bar reinforcement is then placed just above the joints to limit potential cracking at the reduced section.

Where there is a more regular support arrangement, and particularly with longer spans, prestressed hollowcore or double tees, acting compositely with a topping, are an option. However, downstand support beams and ribbed slabs may be an issue in some tall open plan buildings. In predominantly walled residential buildings, where division walls provide support, hollowcore is often the floor of choice as it is mass produced, has a self-weight reduction of between 25 and 30% compared to solid flooring, excellent load / span characteristics and a relatively flat soffit. It can act compositely with a structural topping if required for diaphragm action or robustness. Temporary propping to support the wet topping is often not necessary.

Prestressed downstand beams can become both composite and continuous by the addition of small quantities of insitu infill and topping concrete. Span / depth ratios of up to 30:1 are possible. An example is shown in Fig. 3-11 where the addition of relatively small amounts of reinforcement and concrete achieve the desired result.

Fig. 3-11: Prestressed precast beam intended for composite action and moment continuity.

3.3 Precast Concrete Mixed with Structural Steelwork

3.3.1 Precast floor slabs with steel beams

Precast concrete slabs can be used with steel beams in a similar manner to concrete beams.

In the cases where the steel beam section has a relatively narrow top flange when a UB is the supporting member then additional steel sections or plates may have to be added to ensure satisfactory bearing for the slab. In such cases alternative UC sections with wider flanges are sometimes provided as supports.

Steel / precast slab options like flat slab construction are available using "Slim floor solutions. In these systems, a shallow steel section is provided and the precast slab, normally hollowcore, but biscuit slabs could be used, rests on the bottom flange of the steel support beam. Reinforcement then passes through the beam connecting the supported slabs and the connection at the beam is then concreted insitu.

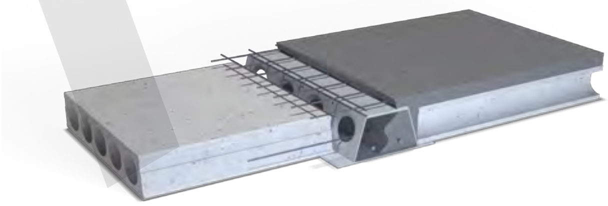

Fig. 3-12: Slim floor example with precast slabs supported on bottom steel flange of DELTABEAM®, courtesy of ©Peikko Group Corporation.

Connecting reinforcement through the steel beam increases the bearing length of the slabs and reduces torsional effects at the support. In hollowcore slabs the connecting reinforcement is concreted into open cores. Further reinforcement is provided inside the webs, and parallel to span, of the steel section to achieve higher fire resistances.

If biscuit slabs are used it gives the potential to mirror flat slab construction more closely through two way spanning and greater continuity by providing designed top reinforcement in the insitu topping. Slim floor systems also have proprietary steel connectors for supports at precast columns that are cast into the columns allowing the option of multi storey vertical elements.

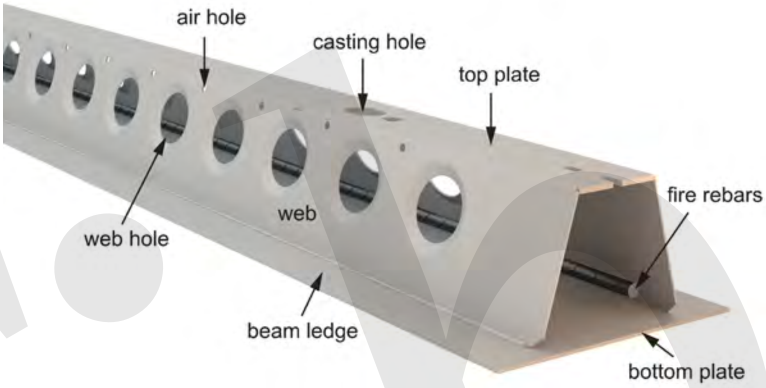

Fig. 3-13: Example of Slim Floor beam, DELTABEAM®, courtesy of ©Peikko Group Corporation.

Fig. 3-14: Flat slab solution using Slim Floor DELTABEAM® and hollowcore slabs, courtesy of ©Peikko Group Corporation.

3.3.2 Precast concrete cores with steel frames

Precast concrete cores can be combined with steel frames. The connections between the precast cores and the steelwork require careful attention to detail to execute the connections correctly and quickly. Types of connection should be minimised and cast in plates with threaded sockets or studs welded to the plates are often a successful option.

Fig. 3-15: Example of connection of steelwork to precast concrete core.

In Fig. 3-15 there is an example of a steel beam connection to a precast core. A steel connection plate has been cast into the precast panel. The plate has projecting studs for a bolted connection, but the plate is recessed so that the studs align with the outside concrete face meaning that a flat mould surface was possible for production, simplifying manufacture.

Note that there is also a steel angle beneath the supported steel beam, this was provided for temporary support to allow early release of the crane, the main connection being executed later and not affecting the programme. Pull out continuity bars have also been provided in the panel for connection to an insitu topping and a proprietary steel channel for connection of a steel support angle for precast slabs. It was possible to install a storey height of core during a night shift ready to receive the connecting steelwork the following morning.

3.4 Other Precast Concrete Components in Mixed Construction

3.4.1 Precast concrete facades and claddings

Precast concrete is used extensively to clad tall buildings. Architects for buildings where insitu concrete or structural steelwork has been fully used as the framing system turn to precast concrete for the outer building skin due to not only its strength, durability, appearance, and versatility but to express artistic intentions, elegance, and three-dimensional sculpturing. It goes beyond the functional attributes of other materials. Chapter 9 deals extensively with the capabilities of precast concrete in this form.

Fig. 3-16 The Carlyle, Los Angeles, USA.

3.4.2 Precast concrete in basement construction

Due to its high strength, speed of installation and flexibility precast concrete elements can be used extensively in basement construction, with insitu concrete often used to stitch them together. Examples of applications are:

- High strength concrete columns used to reduce structural section size where floor space is limited.
- Walls at site boundaries where the proximity of an adjoining site prevents the use of traditional formwork.
- Transfer structures and decks, particularly at locations where there are existing buildings and structure below that must be bridged without temporary propping.
- Non-conventionally shaped structure.

Fig. 3-17: Precast concrete U beams bridging existing structure, note precast panel in background supporting slipformed core.

3.4.3 Precast concrete balconies

There is widespread use of precast concrete balconies in tall building construction. These generally cantilever from the face of the building.

Their benefits have already been described in Chapter 2 and they can be used with either insitu or composite slab construction.

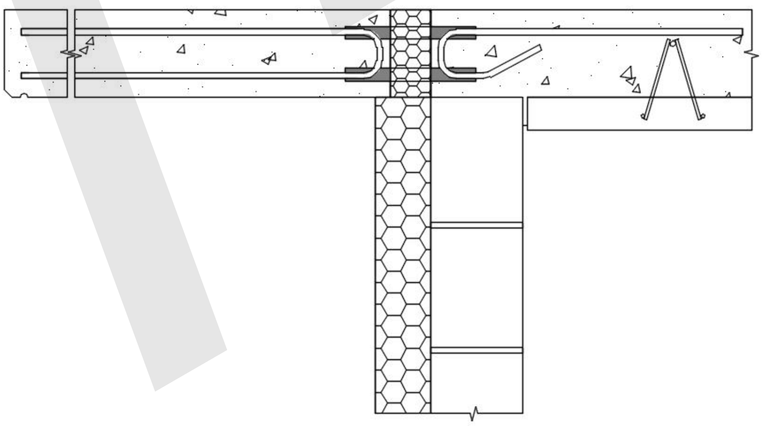

Fig. 3-18: Structural thermal break between balcony and composite floor.

In Fig. 3-18 the "outside" precast concrete balcony incorporates a structural thermal break that anchors the balcony to the "inside" floor slab. Cantilever moments generated in the balcony are resisted via this anchorage into the adjoining structural floor slab. The balcony needs to be installed in advance of the main floor construction but is often temporarily supported by the slab shuttering steelwork. In this illustration, a precast concrete biscuit slab has been used as part of the "inside" composite slab, however, solutions using reinforced and post tensioned insitu slabs or other types of precast concrete flooring in the main building floor are possible. This detail shows one way of achieving continuity of external insulation for the building envelope whilst still providing balcony access and structural continuity with the floor.

In warmer climates, it may not be necessary to provide the thermal insulation detail in Fig. 3-18. In those cases, conventional projecting reinforcement could be used for the structural connection into the floor slab. Where the balcony is in the direction of span of the main floor then precast slabs could be extended and cantilevered over edge supports to provide the balcony.

3.4.4 Precast concrete bathroom pods

Bathrooms can take a significant amount of time to complete during the finishing stages of tall building construction as many different trades are required to work in relatively small, congested areas. A way to considerably reduce this effect on the programme is to use prefabricated bathroom pods. They can consist of thin walled concrete envelopes with all fittings and internal decoration complete so that they are "plug and play" and only need connection to external ducts and wires.

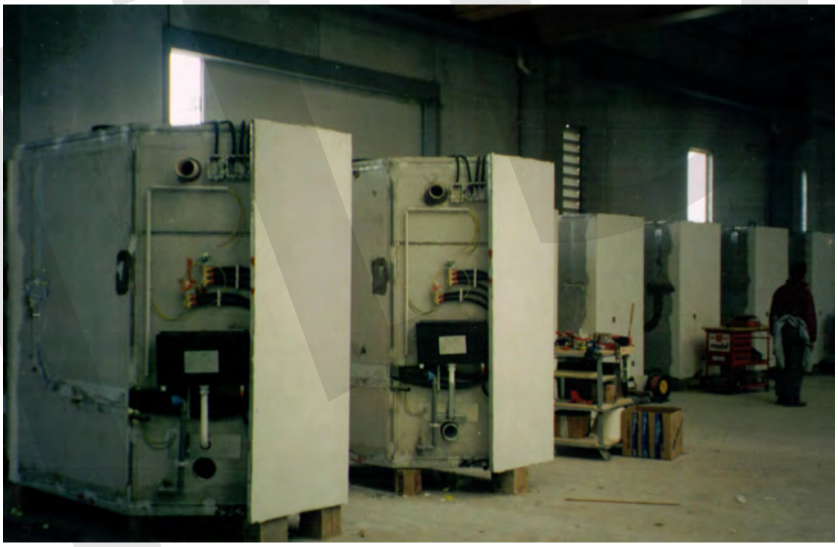

Fig. 3-19: "Plug and Play" bathroom pods assembled in factory conditions.

The prefabricated pods are installed during construction before the next floor closes access. This can extend the floor cycle time, but the subsequent reduction in finishing time should outweigh any other negative time impact. Consistency of level between the bathroom floor and the adjoining floors is normally achieved by screeding the floor after installation of the pod. Tiling, sanitary ware, sinks, showers, baths, and fittings can be chosen to suit the owner's requirements and fitted by craftsmen in a quality assured factory environment. The specification can vary from utilitarian, affordable finishes to high end premium class hotel decoration.

3.5 Interaction with the Client's Design Team

Mixed construction by its nature comprises several engineering parties involved in the design of a project. It is possible that the precast part can often be the majority part of the structure. However, the precast engineer can often be several layers away from the client's engineer. This can result in difficulties.

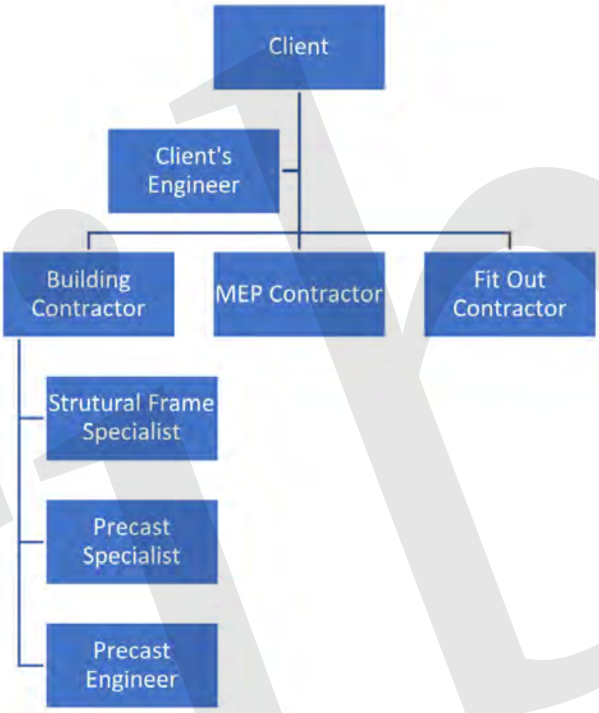

Fig. 3-20: Example of a possible project organisation.

Furthermore, tall buildings also tend to be larger projects that may have experienced several designs and redesigns before reaching the construction stage. Sometimes it can be several years between initial planning and start of construction. Involvement of the precast specialist and engineer may therefore be some way along the timeline as they would be employed through the Building Contractor, who may only be appointed when construction is imminent.

The structural design of the building is a key aspect of the design process and the client's engineer will have intimate knowledge of its structural performance. It is often the case that the client's engineer produces an analytical model that takes account of all the external forces acting on the structure, and from that the design forces and moments acting on each element. Because of the timeframe involved, and the need to consider the effects of global forces acting on each element, it is normally the case that either the client's engineer provides working details to the precast specialist for element production or provides the precast engineer with design forces and moments for each precast element so that element calculations, BIM models and working drawings can be produced for the precast specialist. It is normally the case that the latter option is followed as the precast engineer will be experienced in the precast systems that have been chosen.

If the latter option is adopted then to improve the likelihood of a successful project, and achieve the benefits from precast, clear information received in good time is essential. Key guidelines to follow are:

1. Definition of scope at the earliest stage: A design statement issued by the precast engineer can bring clarity.

2. Clear lines of communication: Easy access to the client's engineer is necessary to avoid several layers of design managers and the consequent possibility of misinterpretation and delay.

3. Clear and unambiguous briefing information: Hundreds of pages of computer output are to be avoided. Design information from the client engineer's frame analysis should be summarised in simple format.

4. Clear approval procedures: Drawings and calculations to be returned with a simple status and comments that are easily understood.

The design statement mentioned above is a document that sets out the precast engineer's interpretation of the precast design scope. It gives the basis for the preparation of calculations and development of working drawings and should be agreed between the two engineers before proceeding with detailed design. It should typically include the extent of the work, the briefing information provided, the relevant building codes and specifications, loading and fire criteria, serviceability limits (deflections, cracking, durability, vibration etc.), structural idealisation of elements and connections, robustness provisions, material characteristics and any other special considerations.

The importance of clear lines of communication is vital for a successful project. Because the precast engineer could be several layers away from the client's engineer there could be many obstacles and pitfalls in agreeing a design. It is advisable that there is direct communication between engineers to avoid, or at least minimise, possible misinterpretation by non-engineering management intermediaries in the process.

The client's engineer should provide design moments and forces for the precast elements in a simple format and not rely on the precast engineer to interpret complex analysis, or isolate critical combinations, that will only extend the design and approval process. The client's engineer will be more familiar with the overall structure and the interaction of elements through the global analysis previously carried out and should more easily identify the critical cases.

It is important that the precast specialist clearly understands when the precast drawings have completed the approval procedure and can be used for manufacture of precast elements.

Comments should be consolidated by the Design Team (there is also likely to be input from the architect and other engineering disciplines and contractors, in addition to the structural engineer) into a single returned document, and submitted calculations and drawings then assigned a status. This status should have a clear definition and advise the precast specialist if manufacture can proceed. If it is not possible to proceed with manufacture, then the reasons for that should be clearly set out together with the actions required by the precaster to remedy that situation.

One of the more common systems for assigning status is:

- Status C: Do not proceed with construction, resubmit for further review.
- Status B: Can proceed with construction if comments / queries are satisfactorily answered.
- Status A: Proceed with construction, submitted documents satisfactory.

There could be an element of risk for the precaster if he proceeds with manufacture based on a status B drawing as defined above, particularly if the queries raised are unclear and could be misinterpreted.

In summary, it is important that there are clear procedures for identifying requirements and producing designs and drawings that satisfy those requirements and lead to the manufacture of satisfactory precast elements that maximise, as far as possible, the bene ts to the project.

4. Precast Concrete Structural Systems

4.1 Introduction

There are many examples where precast concrete elements have been used as structural frames for tall buildings. However, their use as a majority, or total framing, solution is often limited through a lack of familiarity, mainly due to cultural reasons. For instance, in the United States steel construction would be the preferred choice, whereas in Asia and the Middle East it would be insitu concrete. As described in earlier chapters there are numerous advantages to be gained through using precast concrete and these can be further increased and improved upon by a more widespread application for the whole, or main parts, of the building frame. Shorter construction periods can be achieved, labour densities on an already congested building site dramatically reduced, site related risks minimised, and cost certainty improved.

The structural frame not only has to transfer the vertical loads from the building itself into the foundations but also should resist relatively much larger external horizontal forces arising primarily due to wind, but also in the case of seismic action if located in an affected region, special care should be given to internal actions arising by alternating soil movements. In this latter case in moment resisting beam-to-column connections, the ends of the beams must be well anchored by negative and positive starter bars, in their top and bottom, respectively. In tall buildings, deformations should be tightly controlled for the comfort of the occupants and service requirements may be dominant. Second order effects arising through deformations also should be carefully considered as they will most likely become significant.

There are several structural systems used in tall buildings. These can be simplified into four main types:

- Framing systems;
- Shear wall systems;
- Tube systems;
- Dual systems (wall-frame systems).

A Structural System Chart is shown in Fig. 4-1 and has been reproduced from *fib* Bulletin 73[13], first published in 2014. The chart was originally produced by Fazlur R Khan and estimates the number of storeys achievable for each structural system. It indicates that the limit for framing systems is about twenty storeys and that tube systems become necessary for buildings over about fifty storeys. However, these limits are likely to change with advances in technology.

This chapter was mainly authored by Jones, Vambersky, van der Zee, van Keulen.

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"Framing systems" relate to beam and column elements that are rigidly connected as moment resisting sub frames. Each sub frame resists a proportion of the horizontal force relative to its stiffness. "Shear wall systems" utilise internal walls in two orthogonal directions to provide lateral stability. Often the cores and lift shafts are used as parts of the stability system. Where the cores have a large, combined stiffness relative to the columns in a typical tall building footprint the lateral loads are resisted entirely by them and the columns can then be designed to resist gravity loads only.

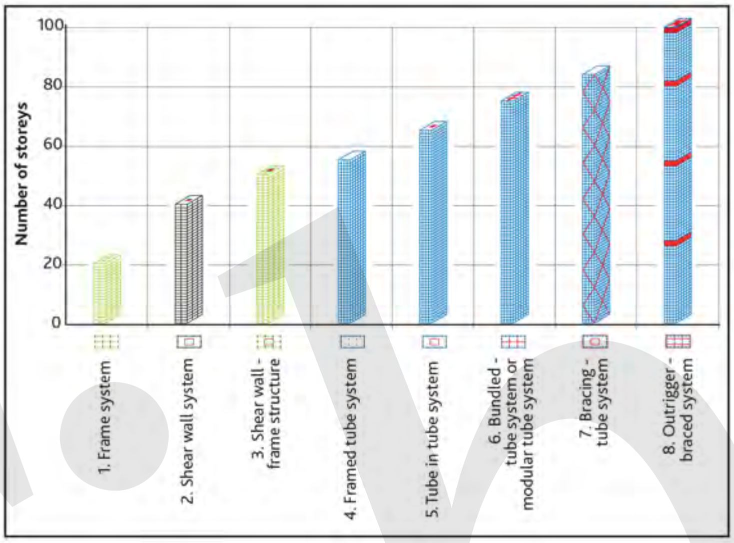

Fig. 4-1: Structural system chart[13].

There can also be a combination of shear wall and framing systems, where internal shear walls act with moment resisting beam and column frames. This could allow an increase in achievable building height before having to switch to a tube system.

In "tube systems", the periphery of the building is typically utilised for lateral stability. The building is idealised as a hollow tube spanning vertically from its foundation. The opposite peripheral walls in each direction act as the tension and compression flanges linked by perpendicular peripheral walls acting as webs.

There are then variations on the simple tube system that utilise other walls and cores inside the building, as can be seen in Fig. 4-1. The façade will have windows that split the walls into beam and column moment resisting subframes in the tube solution. Precast concrete can be ideally suited to tube solutions where visual concrete is required for the façade. Sandwich panel elements can be integrated into the building frame and provide a multi-functional role, i.e. weatherproofing, appearance, thermal insulation, and structural system.

The remainder of the chapter describes how precast concrete elements can be combined to provide the structural systems described above.

4.2 Moment Resisting Frames

In reinforced concrete framed structures resistance to lateral forces depends mainly on the rigidity (moment capacity) of the connections between vertical and horizontal members (this could be different in seismic zones, see Chapter 10). For conventionally framed rectilinear structures with horizontal members on orthogonal axes this is particularly relevant. The lateral resistance of the structure is related to the sum of the moment capacity of all connections and the lever arm between the most exterior elements. The taller the building the greater the overturning moment demand and, in turn, the resistance required, resulting in ever larger and more complex connections. In addition, excessive lateral deflection under service loading (wind or low intensity earthquake) should be controlled by appropriate selection of the lateral load resisting system. Consequently, a limit for a frame only lateral load resisting system is typically reached at about twenty storeys (or lower depending on the wind or seismic intensity) where other structural systems for lateral resistance and stiffness should be considered. The connection between beam and column is often achieved through "insitu stitching".

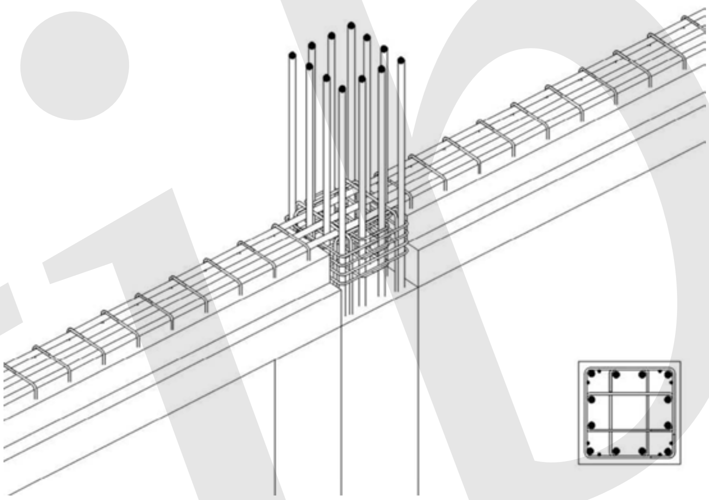

Fig. 4-2: Simple inverted T beam and storey height column. Column links continue through insitu stitch as shown in the 3D view and plan section (example of emulative system, modified after *fib* Bulletin 78[15]).

In Fig. 4-2 simple beams and columns are connected at their juncture with vertical and horizontal continuity reinforcement. In this case the column is storey height and the beams single span. Where moment reversal in the beam at the column support is predicted then the end part of the beam itself can be formed as a U section (similar to Fig. 5-2) to provide bottom reinforcement continuity through the column. If there is enough site lifting capacity then multi storey columns, multi span beams and sub-frames are all possible and can reduce the number of site connections, and / or precast elements, required. Fig. 4-3 and 4-4 show examples of multi storey columns that allow for insitu stitching (emulative approach with wet connections) at intermediate levels, and subframes incorporating both columns and connecting beams.

Fig. 4-3: Multi storey columns with gaps to allow for connection at floor levels (courtesy of Precast India Infrastructures Pvt Ltd, from *fib* Bulletin 78[15]).

In buildings that require concrete on their perimeter, precast columns and beams, or deep spandrels, can be utilised as moment resisting frames to form a tube solution as described earlier. The columns would need to be at closer spacings to provide increased stiffness and would match the windows. The perimeter tube can then be combined with internal moment resisting frames and / or walls to further increase rigidity (framed tube system). Use of internal diaphragms in this manner effectively results in a vertical multi-cell cantilever box beam.

Fig. 4-4: Sub-frame with connections at mid lengths of columns and beams (Biological Science Building under construction at University of Canterbury Campus, New Zealand, photo courtesy of Stefano Pampanin).

Fig. 4-5: Building with precast moment resisting frames in Tokyo (SQRIM system, refer also to case study 12.5).

Systems have also now been developed that do not require insitu stitching at beam and column joints, but instead rely on dry connection solutions. Coupled reinforcement connections are provided for both beam (horizontal) and column (vertical) joints. The connection is inside the precast element and, typically, each bar coupling is achieved using pressure injected grout. The building in Fig. 4-5 used connections of this type.

In seismic zones framing solutions have been sought that can develop stable inelastic behaviour without leading to severe structural damage. Precast Seismic Structural System (PRESSS)[41] technology has developed since the 1990's to achieve this aim. It relies on jointed ductile connections that are completed using unbonded post-tensioned tendons. With dry precast connections, and the potential to use post-tensioning to achieve longer spans and more open space, PRESSS can be greatly beneficial to tall building construction in all regions.

In seismic situations deformation is concentrated at each joint location through the rocking motion between the elements that considerably reduces structural damage. The beam / column connection is illustrated in Fig. 4-6 and 4-7. The post-tensioning is along the beam centroid with the ductile reinforcement near top and bottom edges.

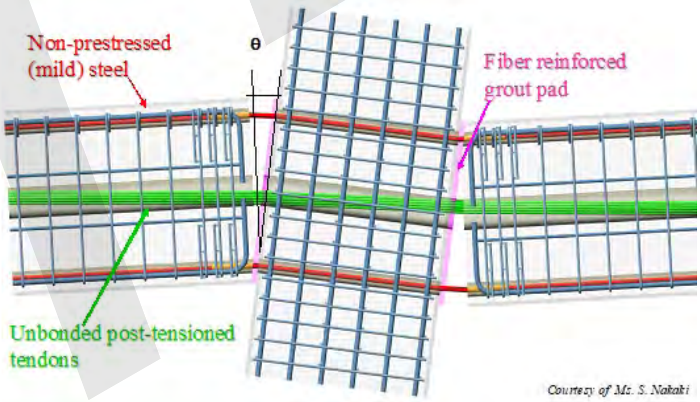

Fig. 4-6: Beam / Column connection using PRESSS technology with internal mild steel dissipater, figur courtesy of Suzanne Nakaki.

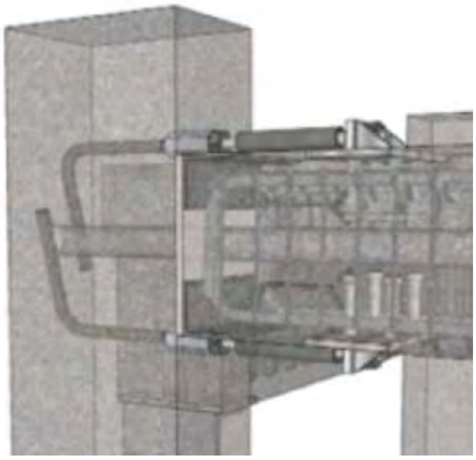

Fig. 4-7: Connection at external column with central unbonded post-tensioning and top and bottom external replaceable "Plug and Play" dissipaters (modified after-Johnson et al 2013).

4.3 Wall Systems

Precast concrete walls can be used as structural lateral and gravity load resisting (framing and stability) systems in tall buildings. They can be applied to both shear wall systems and most tube systems shown in Fig. 4-1. Three illustrative examples are provided in Fig. 4-8:

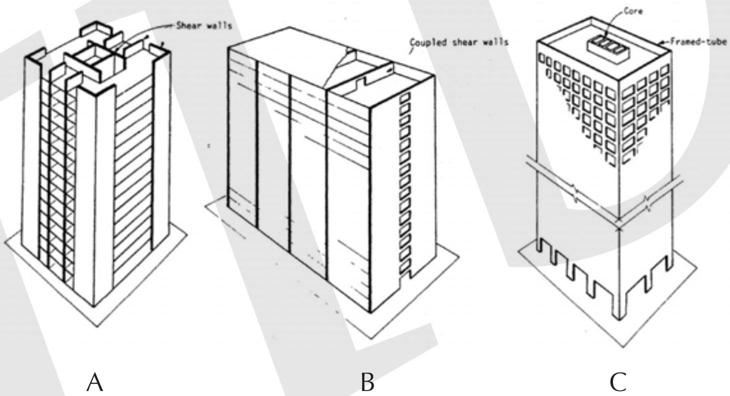

Fig. 4-8: Application of precast walls to tall building structural systems (A-shear walls, B-coupled walls, C-tube in tube) .

In Example A the walls combine to provide a shear wall system, comprising mainly internal walls as only the façade corners are walled. Example B uses the peripheral walls and couples them via internal cross walls. The full building periphery, together with the internal core, is utilised in Example C to provide a tube in tube solution.

In precast systems, there are many discrete elements that must be assembled at the building site into a structural framing solution through a series of horizontal and vertical joints and connections. This is the main difference when compared to insitu construction.

To achieve a successful system, the number of connection details should be few and simple to facilitate speed of construction, but also have enough adjustment to take account of process tolerances, and enough strength to resist the considerable design actions in a tall building. The choice of connection, and the location and number of joints, will often decide if the time benefits of precast concrete are fully realised.

Horizontal connections between wall elements are normally straightforward and typically follow traditional starter bar details with high strength mortar or grout filling the joint. There are several ways of anchoring bars across the joint to complete the connection, but commonly connecting bars are either coupled or grouted into sleeves cast into the precast elements. To aid the construction process it is advantageous at the connection, if design considerations can be satisfied, to have larger diameter bars at greater spacings in a single central row rather than smaller bars at close spacings in each face. Simple decisions of this nature can greatly improve the construction programme.

The type of vertical connection is dictated by the walling arrangement chosen for the building.

There are two types of vertical joints, structural joints, and non-structural joints. A structural joint is required to transfer shear through reinforcement, mortar, or a combination of both. Non-structural joints can be left open if preferred. In a stacked arrangement where the joints are in a vertically continuous line, structural joints are required for the full building height. In an interlocking, or "masonry", arrangement structural joints are not required as the shear stresses are resisted by the elements themselves above and below each joint.

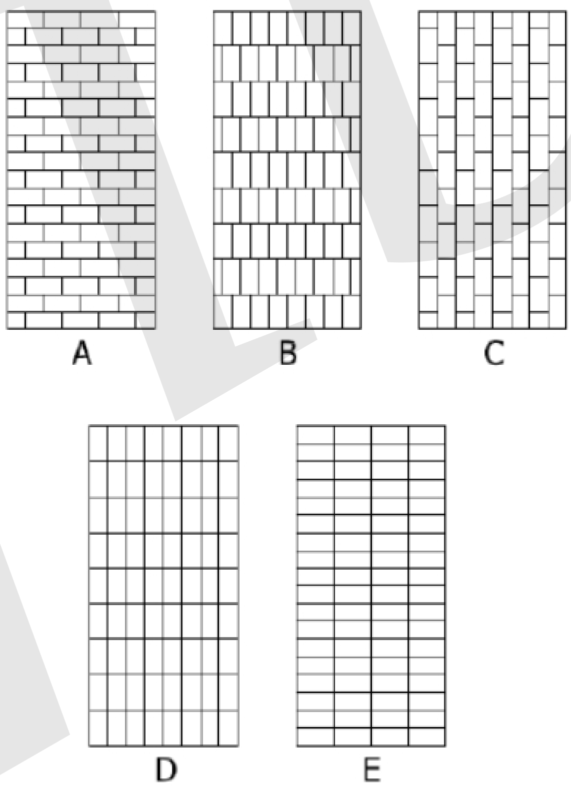

Fig. 4-9: Possible wall arrangements showing joint locations.

Fig. 4-9 shows some possible wall arrangements as follows:

- Arrangement A: Horizontally interlocking masonry configu ation, one storey high horizontal elements;
- Arrangement B: Horizontally interlocking masonry configu ation, two storey high vertical elements;
- Arrangement C: Vertically interlocking masonry configu ation, two storey high vertical elements;
- Arrangement D: Two storey high vertical elements without interlock:
- Arrangement E: Single storey high horizontal elements without interlock.

The vertical connections in Arrangements A and B can be open joints with no structural function. Arrangements C, D and E would require designed structural connections for the vertical joints.

Research has been carried out by van Keulen and Vambersky[27] in 2013 to determine the variables that affect the displacements at the top of a precast concrete shear wall structure compared to those of its insitu monolithic counterpart. The deformation at the top of the shear wall consists of bending and shear components. Their results, using Arrangements A to E, show that the precast connections have little influence on bending deformation but can affect shear deformations. However, they also show that shear deformation effects diminish as building slenderness increases. This, therefore, results in taller buildings (slenderness greater than 5) having shear deformation as a relatively small proportion of the total, see Fig. 4-10 below. They also conclude that horizontal interlocking elements (Arrangement A) generally performed better than the vertical and continuous arrangements.

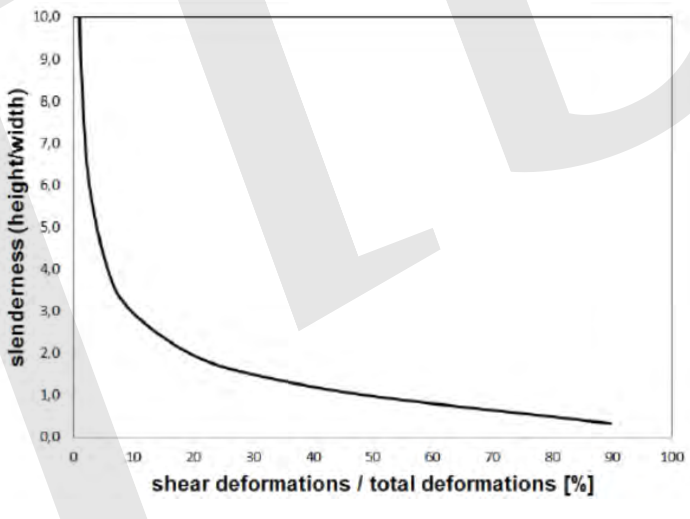

Fig. 4-10: Shear deformation proportional variation with slenderness ratio.

Further research by ten Hagen[29] in 2012 into the feasibility of a tower 200 m high in the Netherlands, constructed using panels with a masonry configu ation (Arrangement A), concluded that the reduction in stiffness of a precast tower, formed in that manner, compared to a similar insitu tower was less than 4 %, with displacements still well within performance requirements. Structural section thicknesses, 400 mm thick main shear walls (increasing to 500 mm thick for critical walls in the lower storeys) used were the same for both precast and insitu options.

If the masonry configu ation is an attractive solution for the stability structure for a tall building, then there are certain basic guidelines that can be followed that will optimise its performance. These are (length is measured horizontal and parallel to the face of the building):

- The overlap between panels in different layers should not be less than 25 % of the element length.
- The length of the smallest element should not be less than its height.
- The non-structural vertical joints should be as evenly distributed along the wall as possible.
- The panels should be as long as practical.

Het Strijkijzer JuBi Towers Erasmus MC

Fig. 4-11: Ex. of buildings in the Netherlands constructed using precast concrete wall systems for lateral stability.

In Fig. 4-11 there are three examples of buildings that use precast concrete wall systems for either total lateral stability or as part of a mixed stability system:

- Het Strijkijzer, The Hague: 42 storeys high and height of 132 m of mostly residential tower. Lateral stability is provided by peripheral shear walls coupled by internal crosswalls.
- JuBi Towers, The Hague: Two towers for government ministries, 41 storeys high and height of 140 m. The stability system consists of a “tube in tube” system of irregular shaped floor plan and several cores. The periphery of each tower consists of precast elements in a stability tube stacked in a masonry configu ation, the internal cores were slipformed.
- Erasmus Medical Centre, Rotterdam: Tower that is part of a university medical centre and hospital. There are 31 floors, and the building is 120 m high. The stability structure consists of a “tube in tube”. The peripheral tube consists of a sandwich panel that incorporates not only a structural element but also insulation and a non-structural façade skin. Peripheral panels are stacked in a masonry configu ation (Arrangement A). The internal tubes also consist of precast concrete walls.

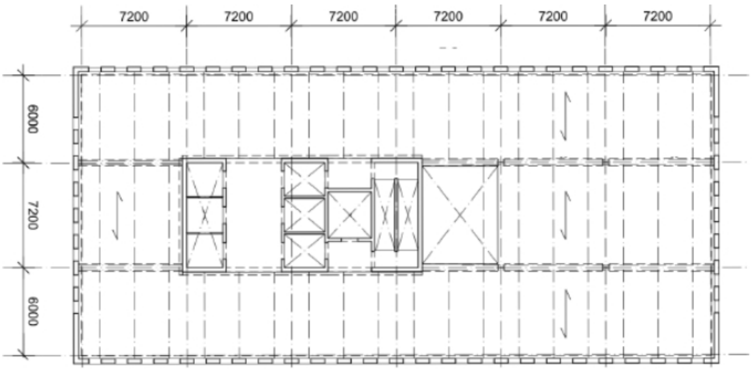

Fig. 4-12: Typical floor plan of Erasmus MC, both peripheral panels and internal cores formed using precast concrete walls.

Fig. 4-13: Sandwich panels used as part of the stability system at Erasmus MC (used in masonry con guration, Arrangement A).

There are three possible ways of jointing precast concrete walls at corners in stability systems (excluding conventional stitching with insitu concrete) that have been researched. These are applicable to both internal cores and the corners of peripheral panels, and are illustrated in Fig. 4-14:

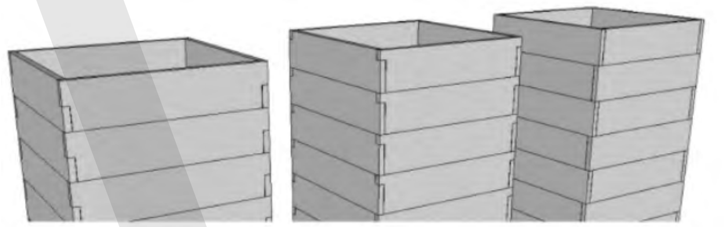

Fig. 4-14: Types of corner connections for precast concrete walls (K.V. Tolsma[30], 2010).

The references in Fig. 4-14 are described as follows:

- IHC: Interlocking halfway connection;
- IACC: Interlocking above ceiling connection;
- SC: Staggered connection.

Research carried out by K.V. Tolsma[30] concluded that the IACC type was considerably weaker than both IHC and SC, with IACC having a resistance about one third the value for the other two connections. However, if considered in absolute terms then the IACC connection can still have its place in tall building construction and was used extensively in Het Strijkijzer (see Fig. 4-11 and 4-15) for connection of internal walls.

IHC and SC were then compared with a monolithic insitu version. Deflection under horizontal forces at the top of each of the three cores was then measured against increasing height. The research showed that there was little difference between the three connection options (IHC, SC and monolithic insitu). The top deflection of the IHC version was 3.3 % greater than the monolithic option, the SC version was 5.9 % greater. Therefore, there is little difference between the performance of the precast system and the corresponding insitu one. A variant to the IACC system is shown in Fig.4-15 with interlocking at both top and bottom of wall panels.

Both through research, and in practice, it has been shown that precast walling systems can be used to provide practical structural stability solutions for tall buildings. Their application greatly increases when the external façade is concrete and when it can be harnessed as part of a stability tube system. Both internal division walls, and circulation cores can also be integrated into a structural precast system using the configu ations and connections previously described.

Fig. 4-15: Precast wall panels with deep nibs for vertical shear connections (used in Het Strijkijzer).

4.4 Robustness and the Avoidance of Progressive and Disproportionate Collapse

Because precast structures consist of many separate elements that are manufactured away from their final position in the building and then assembled into a completed structure special attention must be paid to the connections between the elements and their ability to withstand forces that may be either predictable or accidental (*fib* Bulletin 63[12] is a guide to good practice for the design of precast concrete structures against accidental actions). In this context, the terms "robustness", "avoidance of progressive collapse" and "avoidance of disproportionate collapse" are often used.

Firstly, we should consider definitions relating to these terms:

- Robustness: A quality in a structure, or structural system, that describes its ability to accept a certain amount of damage without the structure failing to any great degree. For example, EN1991-1-7[4] further defines robustness as "The ability of a structure to withstand events like fire, explosions, impacts or the consequences of human error without being damaged to an extent disproportionate to the original cause".
- Avoidance of Progressive Collapse: The avoidance of the sequential spread of local damage from an initiating event, from element to element, resulting in the collapse of several elements.
- Avoidance of Disproportionate Collapse: The avoidance of a collapse, after an event, that is greater than expected given the magnitude of the initiating event. The level of collapse expected for certain events may be given in regulations, but more often is related to public and professional perception. For instance, even the most robust structure may suffer complete collapse if the event is severe enough, and this would not be considered disproportionate.

It is desirable for all structures to have these capabilities to withstand actions and events described above, but this is particularly applicable to tall buildings where the risks and consequences associated with collapse are so much greater.

Strategies for limiting the extent of localised failure and achieving satisfactory robustness may often be included in regulations but should always be considered by the engineer responsible for tall building design. In EN1991-1-7 the potential failure of the structure arising from an unspecified cause is mitigated by one or more of the following approaches:

- Designing key elements, on which the stability of the structure depends, to withstand a prescribed hazard loading.
- Designing the structure so that in the event of a localised failure (e.g. failure of a single member) the stability of the whole structure, or of a significant part of it, would not be endangered. Commonly known as the "alternative load path".
- Applying prescriptive design / detailing rules that provide acceptable robustness, e.g. three-dimensional tying for additional integrity as illustrated in Fig. 4-16.

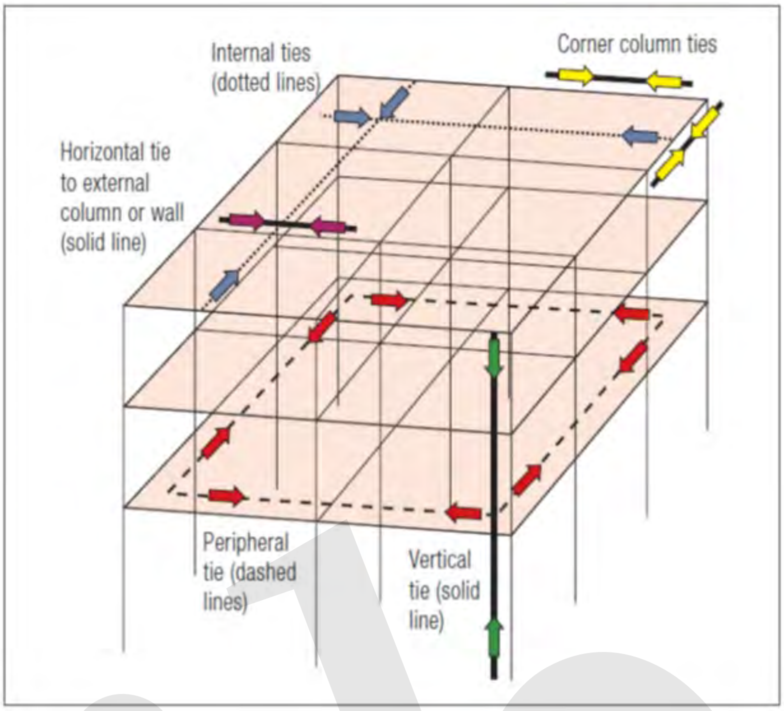

Fig. 4-16: Illustration of three-dimensional tying.

The approach to be used for a building should, in turn, be determined by the amount of associated risk and consequences of failure. For instance, EN1991-1-7 links three consequence classes (CC), as defined in EN1990[1], to the design strategies previously described:

- CC1 Low consequences of failure;
- CC2 Medium consequences of failure;
- CC3 High consequences of failure.

EN1991-1-7 gives examples of categorisation of building type and occupancy and relates them to consequence class. The advice given is that all buildings over 15 storeys are assigned class CC3, i.e. high consequence for loss of human life, or economic, social, or environmental consequences are very great. All tall buildings should therefore be class CC3.

For buildings in CC3 it is advised that "a systematic risk assessment of the building should be undertaken taking into account both foreseeable and unforeseeable hazards". Figure B.1 of EN1991-1-7 gives an overview of a risk analysis through a flow chart.

Advice in relation to the development of a risk analysis for a CC3 building is also given in "Manual for the systematic risk assessment of high-risk structures against disproportionate collapse, October 2013" by The Institution of Structural Engineers (IStructE)[43].

The minimum advised robustness provisions for a CC3 structure would be the intent for an upper risk CC2 structure, normally the prescriptive tying requirements mentioned earlier. However, any additional provisions fl wing from the risk assessment would also have to be implemented.

The IStructE manual also gives several general design principles and procedural recommendations to reduce risk and enhance robustness. The procedural recommendations expand further the advice given in 3.5 of this document and relate to good management and communication practices. The design principles are summarised below:

- A robust design based on ductility and the ability to dissipate energy.
- The development of systematic procedures to identify weaknesses in the structure, and therefore identify the critical elements.
- The provision of alternative load paths and checks for their ability to carry redistributed loads from the loss of a member.
- The consideration, and possible separation, of horizontal and vertical load paths so that accidental horizontal actions will not cause failure of the vertical load path.
- Compatibility between strength based assumptions in the calculation of resistance and provision of the necessary ductility to support those assumptions.
- Design of vertical load bearing elements so that failure is produced in the adjoining slab / beam first, thereby limiting the extent of any damage.
- Continuity through improved connection detailing. However, care should be taken that the structure can resist any increased redistributed loads.
- Assessment of the consequences of loss or damage to the stability system and not just elements in the vertical load path.
- Where there are local severe hazards consider compartmentalisation to avoid spread to the adjoining structure.
- Avoid using brittle connectors.
- Large spans increase the extent of potential damage. If they cannot be avoided, then consider eliminating the hazard if possible.

The general principles relating to robustness and the avoidance of progressive and disproportionate collapse have been considered in this chapter. The following chapters will consider the precast concrete elements themselves and how they are connected to achieve a robust and resilient structure.

5. Precast Concrete Floors

5.1 Introduction

Precast concrete floors can be an integral part of tall buildings. In addition to the normal benefits of concrete (both precast and insitu) they have the added advantages of reduced construction time (time certainty), enhanced structural performance (high strength concrete and prestressing), large span capacity, minimal falsework, reduction in labour and wet trades, and a large selection of different types to suit any given application. Most flooring systems are mass produced under factory conditions and with economies of scale provide a cost effective and fixed price solution (budget certainty).

The main types of precast floor that are generally available are:

- Reinforced filigree slabs (also known as shuttering slabs);
- Voided reinforced slabs;
- Reinforced ribbed slabs;
- Prestressed plate flooring;
- Prestressed hollowcore flooring;
- Prestressed ribbed slabs (single tees, double tees, and channels).

Particularly in tall building construction, precast slabs would be commonly designed to act compositely with an insitu structural topping. The topping not only improves the structural capacity of the floor to resist gravity loads, but also connects the individual precast flooring elements together to act as a plate, or diaphragm, to transfer horizontally applied loads to the vertical stability system (shear walls, stair cores and shafts). The floor slabs can not only act with various types of primary precast beam, but also with structural steel beams and as flat slabs.

The choice of flooring system varies between buildings, and in tall buildings it is often the floor depth and self weight that is critical. Therefore, ribbed slabs that would have low self weight and excellent load / span characteristics may be ruled out because of their relative depth compared to constant depth flooring. In addition, voided slabs that may also have low self weight may not be considered if the spans are relatively short and they have no commercial advantage. Prestressed or reinforced composite plate or filigree flooring may give an optimum solution in tall buildings that often have relatively short spans. Table 5-1 gives a comparison between self weight per unit area of a number of different floor types and and their relative depth where the self weight and depth of a reinforced composite filigree slab is taken as unity. The speed of construction and the capability of achieving the relatively short spans (5 to 8 m) needed in tall buildings is considered equal for each floor type for the purpose of comparison.

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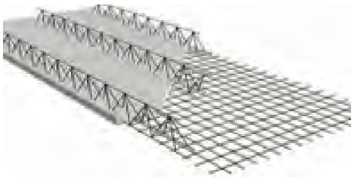
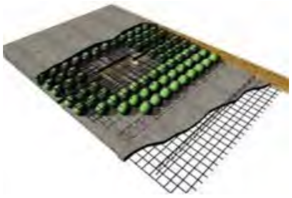
Table 5-1 Relative Comparison of Precast Floor Solutions for Tall Buildings

Slab Type	Relative Self Weight	Relative Depth	Comments
 Composite filigree reinforced slab Composite filigree post tensioned slab	1.0 1.0	1.0 0.6-0.8	Relatively thin precast biscuit with projecting steel lattice. Insitu topping can vary in depth to achieve large spans. Temporary props at about 2.5 m centres. Can be used as flat slab.
 Composite voided reinforced slab	06-0.8	1.0	Similar to filigree slab but voidformers used to reduce self weight for larger spans. A number of proprietary systems are available.
 Composite ribbed reinforced slab Composite ribbed prestressed slab	0.6-0.8 0.4-0.6	1.3-1.5 1.1-1.3	Structurally efficient for span to self weight ratio. However, greater structural depth normally required for ribs compared to other slab options. Temporary propping can be avoided. Primary beam or wall supports required.
 Untopped prestressed plate flooring Composite prestressed plate flooring	1.0 1.0	0.6-0.8 0.7-0.9	Prestressing allows more efficient span to depth ratios compared to reinforced options, plus temporary props minimised. Steel supports required for flat slab solution.
 Untopped hollowcore slab Composite hollowcore slab	0.6-0.8 0.7-0.9	0.6-0.8 0.7-0.9	Similar to prestressed plate flooring but hollowcore voids reduce self weight.

Note: both self weights and depths are relative with reinforced composite filigree slab taken as unity for equivalent structural performance.

Often in tall buildings a flat slab is the desired solution as obstructions caused by downstand beams and slab ribs in the ceiling void are avoided.

5.2 Floor Types

5.2.1 Filigree slabs

They are sometimes known as shuttering slabs, wideslab, half-slab and a variety of trade names. Nevertheless, they typically consist of a thin precast biscuit, between 50 mm and 75 mm depth, that is conventionally reinforced and incorporates a projecting steel lattice as shown below in Fig. 5-1.

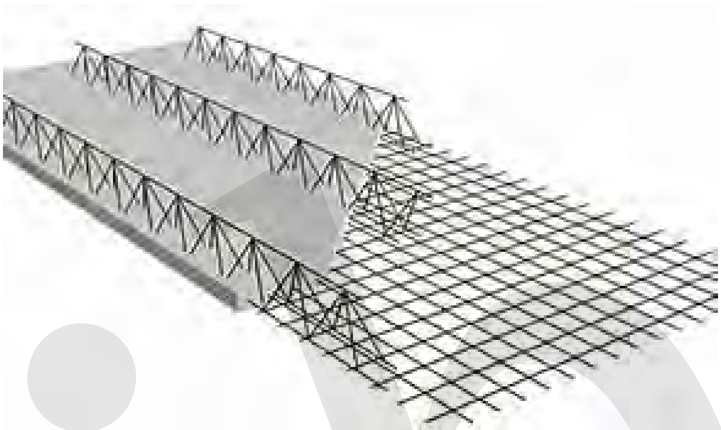


Fig. 5-1: Typical filigree slab.

The lattice provides strength during handling and construction to resist temporary stresses until the slab is acting composite with an insitu topping. The steel lattice is available in different wire sizes and heights from suppliers to suit the design conditions.

Whilst the lattice provides extra strength before composite action can be relied upon there is still likely to be temporary vertical site propping required along the span to suit the lattice capacity, lattice spacing and construction loading. The insitu topping makes up the overall composite slab depth and is part of the construction loading in its wet state. The main bottom tension reinforcement is normally placed in the precast biscuit. However, in circumstances where the reinforcement may be too dense (large bar diameters at close centres in both directions) then the biscuit may be used as permanent shuttering only and not be taken into account as part of the structural section. The main bottom tension reinforcement is then spaced off the top of the biscuit.

This system can be designed as fully continuous over supports with the top tension reinforcement placed over the top of the lattice in the insitu topping. The floor can also be analysed as a two way spanning flat slab. Careful detailing is required to ensure that reinforcement across joints between biscuits satisfactorily resists the design forces.

Because the slabs are relatively light due to the thin precast biscuit they can be manufactured in large areas, with typical 2.4 m widths, and still have precast element weights within tall building tower crane capacities. Filigree systems impose few dimensional restrictions as there is no maximum composite depth and / or span. They can also be used with a similar beam system, as shown in Fig. 5-2, in areas of heavier loading such as at column and wall transfer locations where insitu stitching is needed.

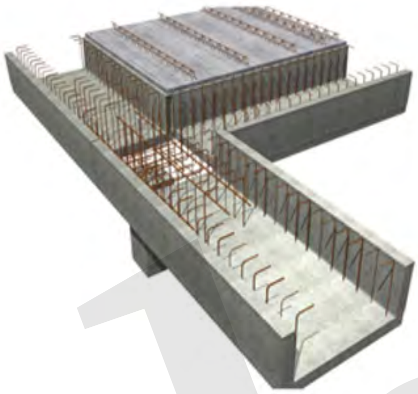


Fig. 5-2: Filigree slab and beam solution in heavy load transfer area.

Filigree slabs can be mass produced on a long line with simple steel moulds, but they can also be accommodated in a twin wall factory (see Chapter 7 also), where the filigree is the first phase of twin wall production. Lifting can be carried out using the lattice provided the supplier's instructions are followed, see Fig. 5-3 below. Note that the slab has to be lifted at the nodal point where the top wire meets the diagonals.

Fig. 5-3: Lifting.

Fig. 5-4: Edge Prop at twin wall.

Fig. 5-4 shows a simple interface between a filigree slab and supporting twin wall. It may be possible to prove that the concrete sections have enough capacity to transfer the forces, but it is still prudent to provide the edge support for safety reasons to avoid risks due to site inaccuracies.

Where there are longer spans and / or heavier load conditions void formers can be incorporated in the top of the precast biscuit to reduce floor self-weight. There are several ways to achieve a voided filigree slab, from simple lightweight solid blocks to proprietary plastic shapes, examples are shown in Fig. 5-5 and 5-6.

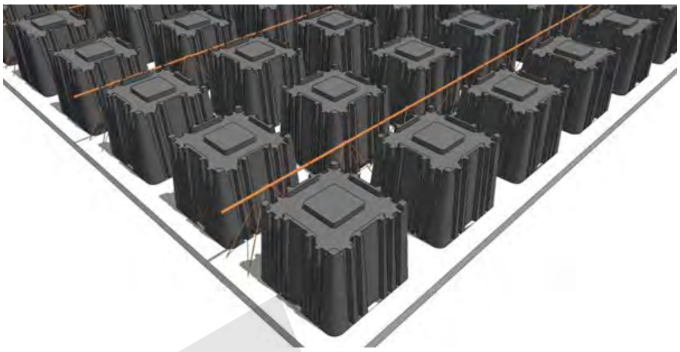


Fig. 5-5: Plastic pots as void formers.

Fig. 5-5 and 5-6 have plastic void formers that have been placed in the factory. In Fig. 5-5 there are square pots, Fig. 5-6 illustrates a sphere arrangement.

Fig. 5-6: Plastic spheres as void formers.

Pipes for cooling systems can also be incorporated in the precast biscuit. Fig. 5-7 has a typical arrangement of coolant pipes fixed to the bottom reinforcing mesh and looped between the steel lattices, saving another site operation.

In Fig. 5-8 a long span slab with polystyrene void formers is lifted at several points. Careful design and detailing is required when using polystyrene due to potential fir issues. In some countries there may be restrictions on the use of polystyrene in tall buildings.

Filigree slabs can also be used as part of a post tensioned floo . The precast biscuit is conventionally reinforced and temporarily propped on site. The prestressing tendons are stressed after the topping has reached its required strength and are grouped together in cast in sheaths as shown in Fig. 5-9, 5-10 and 5-11.

Fig. 5-7: Coolant pipes tied to bottom reinforcing mesh.

Fig. 5-8: Long span slab with solid void formers, lifted at several points.

Postensioning can reduce section depths and be advantageous in seismic zones, however an additional operation is introduced that may prolong the site programme.

Fig. 5-9: End anchorage in post tensioned filigre slab system.

Fig. 5-10: Post tensioning sheaths fixed above filigree slab.

Fig. 5-11: Typical section illustration through post tensioned filigree slab system.

Fig. 5-11 is a typical section of precast filligree slab acting with an insitu topping in a post tensioned floor slab, and illustrates the solution provided in Fig. 5-9 and 5-10. The top unstressed reinforcement is spaced in the top layers by the steel lattice projecting from the filligree slab. The cast in sheaths are in the space between the top reinforcement and the precast biscuit. Each cast in sheath contains several prestressing tendons. The sheaths are located at varying heights above the precast biscuit to provide a designed profile to maximise structural resistance.

5.2.2 Ribbed slabs

Ribbed floors have many advantages when longer spans are required (10 to 25 m) due to their relatively light self weight per unit area and their excellent load / span characteristics. They are more economical as deeper sections over longer spans, often beyond the range of other types of precast flooring.

Typically ribs are at 1.2 m centres and are generally factory produced on a long line as either single tees (1.2 m wide elements) or double tees (2.4 m wide).

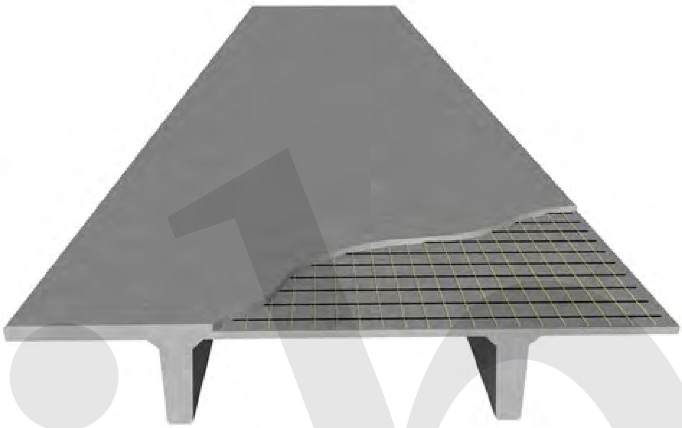


Fig. 5-12: Double Tee slab (typically 2.4 m wide).

Ribbed floors can also consist of precast channels, that are similar to double tees but without the outer top flanges. Channels can give greater strength per unit area.

Rib depths can vary between 300 mm and 1'000 mm depending on local production capabilities. Similarly, rib widths are dependant on available mouldage at the local producer. In taller buildings with higher fire ratings the rib width may also be dictated by fire considerations rather than strength. The flange depth can vary between 50 and 100 mm. Structural toppings are normally required to assist vertical and horizontal load transfer between elements, and for composite action.

In taller buildings, floor spans are typically short due to smaller building footprints and a need for a greater concentration of columns due to heavy vertical loading. Therefore, ribbed floors of depths no greater than 300 to 500 mm would be used. This also means that reinforced ribs are also possible instead of the normal prestressed option. Deeper ribbed floors with depths between 500 and 1'000 mm would only occasionally be used in heavy load and long span circumstances, which are probably more likely to occur in the lower storeys of tall buildings where there may be retail, storage and car parking areas.

There are a number of differences in using ribbed floors compared to flat slabs. One of the main ones concerns the need for supporting beams to transfer the slab end forces to the nearest columns. However, use of precast support beams may not necessarily increase the overall floor structural depth as the rib can be notched and half jointed onto the beam. In the US the notched end of a double tee is described as “dapped”. There are significant stresses at daps requiring special detailing and reference should be made to the PCI design manual[17] when using dapped ends. Also, transverse distribution of services beneath the slab is more difficult. Pipes, cables etc., can be run between ribs but then special measures have to be provided to spread them transversely inside the rib depth. This can be achieved by elongating the end notch in the rib or providing holes through the ribs and / or beams themselves. Locating services below the ribs can lead to an increase in storey height when compared to the flat slab alternative, that can be expensive over the many storeys in a tall building.

Fig. 5-13: An example of fixing services beneath a double tee ribbed floor.

In Fig. 5-13 services are shown between ribs in some positions but also have to be located beneath ribs in order to distribute transversly. Fig. 5-13 also shows the support detail between the double tee rib and the projecting nib of the supporting concrete beam. The ends of the ribs have been notched over the bearing to mitigate the increase in overall floor depth. Note also the corbels provided at columns to support the main floor beams.

5.2.3 Prestressed plate flooring

These are solid sections that are thicker than the biscuits in filigree slabs, and are often used to minimise the amount of temporary propping during construction compared to the filigree solution.

They are normally available in thicknesses between 80 and 300 mm. Whilst plate flooring is also available as a reinforced option, the prestressed version is more popular due to its better load / span properties, which result in thinner floor sections and less temporary propping. The product is sometimes called “wideslab” as it is generally factory produced in long lines in 2.4 m widths, which results in fewer slabs to be delivered and installed in comparison to other 1.2 m precast slab options.

When using the thinner prestressed sections (less than 125 mm), where the centroids of the prestressing strands and the concrete section are relatively close, the beneficia eccentric prestress effects can be minimal, together with the temporary propping advantages over a filigree slab solution. It may therefore be more economic to use a thicker precast prestressed section with a thinner topping to give the same overall floor depth.

Relatively long spans, similar to those achieved with hollowcore slabs, are possible with prestressed wideslab. It may be an attractive solution in areas of higher shear outside the corresponding hollowcore capacity.

Fig. 5-14: Prestressed plate flooring or "wideslab".

Wideslab can be used as either an untopped slab or as part of a composite section. In tall buildings it is more likely for the latter option to be used due to diaphragm action.

In Fig. 5-14 the bottom row of prestressing strands can be seen, the top surface is roughened to enhance resistance at the composite interface and the longitudinal edges of the slab are profiled to both assist in vertical shear transfer and closing of the joint between adjoining slabs.

Prestressed wideslab also provides a flat soffit in both walled construction and when combined with steel beams in open plan areas, where the precast slabs are supported on the bottom flange of the beam.

5.2.4 Hollowcore slabs

Hollowcore slabs are widely used and are probably the most well known precast flooring slab.They are typically prestressed and are usually 1'200 mm wide with longitudinal cores to reduce the floor self weight, this reduction can be between 20% and 40% of the equivalent solid slab. They are generally produced on long line casting beds using extrusion or slide form methods, but can also be manufactured as wet cast in both prestressed and reinforced concrete, using void formers such as dense polystyrene. After hardening the slabs are saw cut to the required length.

Fig. 5-15: Long line hollowcore production.

Hollowcore slab depths are generally between 150 and 400 mm (although greater thicknesses have been produced in some countries), and span to depth ratios of about 35:1 should be achievable. They are often used as untopped elements but in tall buildings a composite section may be preferable to assist diaphragm action and position continuous ties (usually a structural reinforcement mesh) to resist accidental actions. They are designed to be installed and topped without temporary propping.

Fig. 5-16: Slab longitudinal edge profile and end transverse section.

The longitudinal edges of the slabs are profiled as shown in Fig. 5-16. When slabs are placed side by side a shear key profile is formed, and when grouted provides vertical shear resistance and transfer.

Fig. 5-17: Topped and untopped hollowcore planks with grouted shear key.

Fig. 5-18: Proprietory steel DELTABEAM® and hollowcore slabs providing flat soffit through "slim-floor system, courtesy of ©Peikko Group Corporation.

Hollowcore slabs provide a flat soffit solution. In open plan applications the flat soffit can still be achieved in combination with either conventional steelwork framing or proprietory systems as shown in the examples below.

In Fig. 5-18 a proprietory steel beam supports hollowcore slabs. The beam is connected to the slabs via reinforcement through preformed holes. The area around the beam is then concreted to make the slab and beam composite.

5.2.5 Comparable strengths of different floor systems

A number of comparisons are illustrated in the following charts for hollowcore slabs, double tee slabs and downstand beams for simply supported, continuous, composite, untopped and temporarily propped design cases. They are intended to illustrate the potential benefits that can be achieved for different combinations of floor system, analysis model and construction condition. The charts have been developed using Eurocode, a precast concrete strength of 60 N / mm² and an insitu topping strength of 30 N / mm².

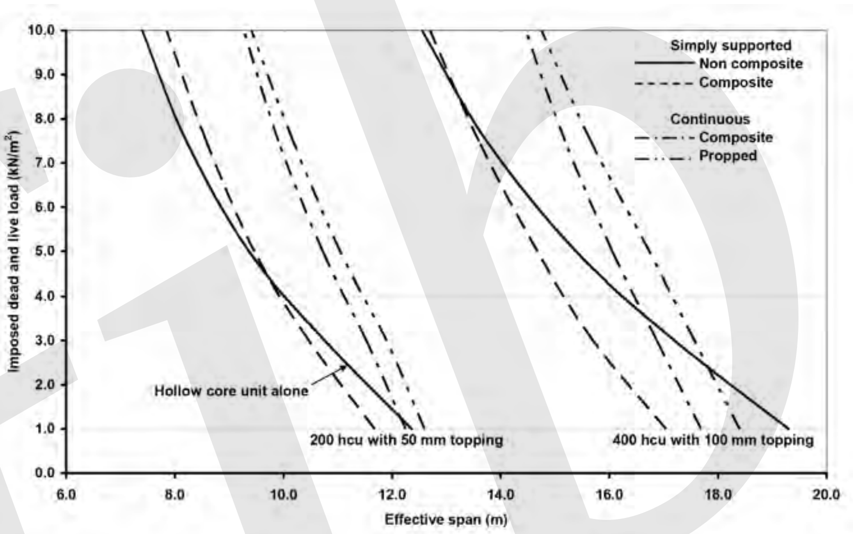


Fig. 5-19: Effective span v Uniform imposed load for hollowcore slabs.

In Fig. 5-19 hollowcore depths of 200 mm and 400 mm are compared for both simply supported and continuous models, untopped and composite sections, and temporarily propped for the continuous model (see also 5.3 for temporary propping). For longer spans it is noticeable that composite action, when compared to the untopped slab, does not improve strength because of the extra topping weight.

Fig. 5-20 compares composite hollowcore slabs with composite double tees. Various depths of each type of slab are plotted. For each slab depth hollowcore achieves greater strength compared to the equivalent depth of double tee. Hollowcore also provides a flat soffit.

In Fig. 5-21 different cases of inverted tee beams supporting hollowcore slabs are considered. As would be expected continuous beams propped during construction give best results.

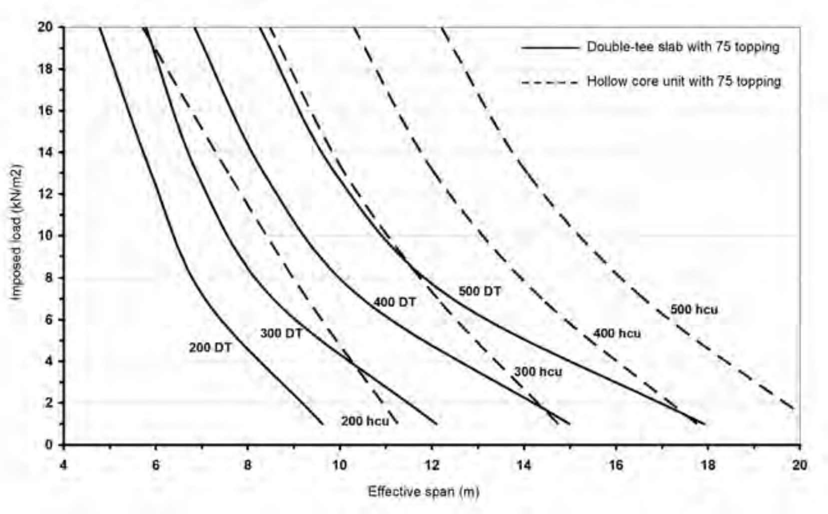


Fig. 5-20: Effective span v Uniform imposed load for hollowcore slabs and double tee slabs.

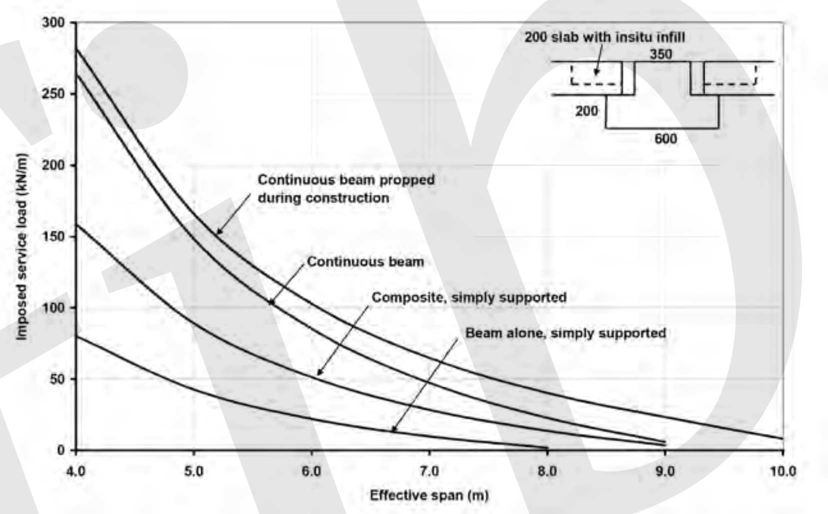


Fig. 5-21: Effective span v Uniform imposed load for different cases of inverted tee beams.

5.3 Design and Detailing

5.3.1 General

Precast concrete flooring elements are installed individually and then connected together via insitu reinforced concrete toppings, edge and end strips or steel fittings. Once forming a structural slab or plate in the building they are required to transfer both gravity and lateral forces to the main vertical structural elements.

When first installed the flooring elements either act simply supported between walls or beams, or continuously over temporary props. The flooring elements have to be designed to resist all forces that act upon them during all phases of production, installation, construction and service.

In the situation where a structural topping is not provided (untopped slab) then the elements span simply supported in the temporary construction case until connected to the main structural framing elements (a requirement that arises in order to provide a building tying system and also to form a diaphragm in order to transfer lateral forces predominantly from wind and seismic actions to the stiffer vertical elements). In tall buildings there is unlikely to be a situation where flooring elements are isolated, indeed most Codes have a minimum requirement that precast flooring elements should at least be anchored into that part of the building that contains the ties. Once the elements are fully connected to the building frame then the structural model may change and restraints that may develop have to be considered in the design.

Where there is a structural topping over the slabs then the final section can be considered composite if roughness criteria at the interface between the two concretes are satisfied (see 5.2.5 for comparisons between composite, untopped, propped and unpropped flooring elements). The roughness criteria would be set out in National Codes. Prestressed slabs, such as hollowcore and double tees, can normally be designed as unpropped during the insitu topping construction phase. If propping is required then it is typically a central midspan prop to control service stresses in the bottom of the section. In the case of reinforced slabs, such as filigree and solid plate elements, it may be necessary to introduce further propping along the span to limit construction phase deflections within acceptable values. Structural toppings not only strengthen the slab through increasing its section modulus and consequently its flexu al resistance but also act as a medium for connecting adjoining elements into a structural plate and provide a means of achieving both moment and tying continuity over supports. In Fig. 5-22 filigree slabs are supported by a delta beam. The structural topping contains the tying reinforcement that passes through the delta beam and links the slabs on each side of the beam. In Fig. 5-23 a precast beam supports hollowcore slabs. There are projecting links from the precast beam and cranked rebar to be concreted into open slots in the hollowcore to complete the connection.

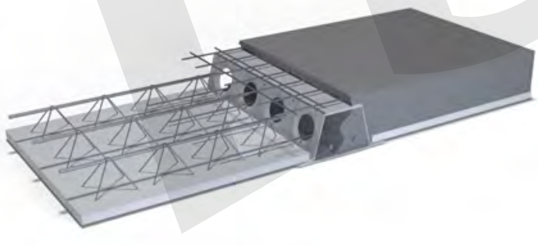


Fig. 5-22: Filigree slab supported by DELTABEAM®, courtesy of ©Peikko Group Corporation.

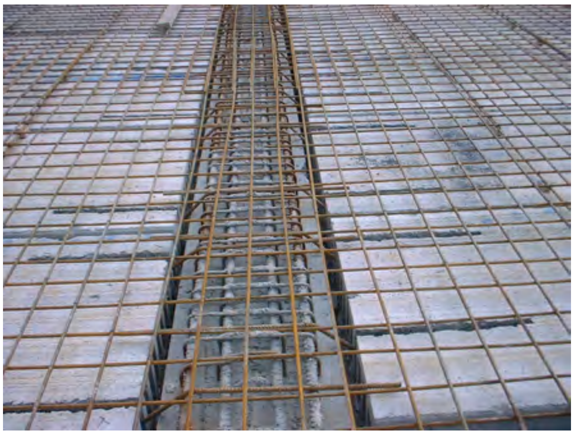


Fig. 5-23: Hollowcore slabs with open cores at ends for reinforcement connection over precast concrete beam.

5.3.2 Floor diaphragms

Horizontal forces, generally arising from wind or seismic actions, are transmitted to the building vertical stability system by the floor acting as a horizontal deep beam (or some other appropriate model), known as a floor diaphragm. The term diaphragm meaning that the vertical depth is very small compared to the floor area. Note that we will be covering aspects generally relating to floor diaphragms in this section. The special issues that affect buildings subjected to seismic actions are more specifically covered in Chapter 10.

A precast concrete diaphragm is a diaphragm consisiting of precast flooring components with optional insitu concrete strips along some or all boundaries and with or without an insitu topping over the precast components.

Any type of floor construction may be designed to function in this way, but particular issues arise in precast concrete floors which comprise individual units (untopped slabs), such as hollowcore or double tee flooring, because of the localised way in which they are connected together, as shown in Fig. 5-24.

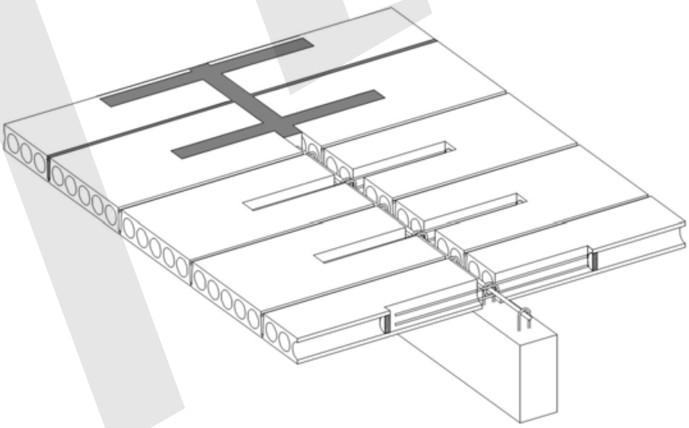


Fig. 5-24: Untopped hollowcore slabs with small areas of insitu concrete along slab boundaries providing structural connections.

If the floor is a solid construction with a relatively large depth of insitu topping, such as a filigree or prestressed wideslab, that acts compositely then these localised areas are no longer present and the horizontal forces are distributed across the full floor area.

Horizontal reaction forces at each level are transferred to the foundations by the vertical stability system (see Chapter 7). Where the distance between the vertical bracing elements is relatively large, the floor has to be designed as a plate and resist in plane shear forces and bending moments.

When the longitudinal gaps between the individual precast units and the strips between the slabs and beams are filled with insitu grout or concrete (if the strips and gaps are wide enough) the completed floor diaphragm is then as shown in Fig. 5-25. A "ring beam", or series of "ring beams" is formed around the precast floor units using small volumes of insitu concrete reinforced with either rebars or unstressed strand to effectively clamp the slabs together and ensure diaphragm action. With the reinforced concrete ring beams in place the precast slabs become part of a self sufficient floor diaphragm. There are, however, many situations where a structural topping is required for the overall benefit of the floor slab design, i.e. additional strength and stiffness plus catenary advantages in the event of accidental actions. In such cases the floor diaphragm is provided entirely by the topping concrete, with its own reinforcement designed to resist in plane forces and moments

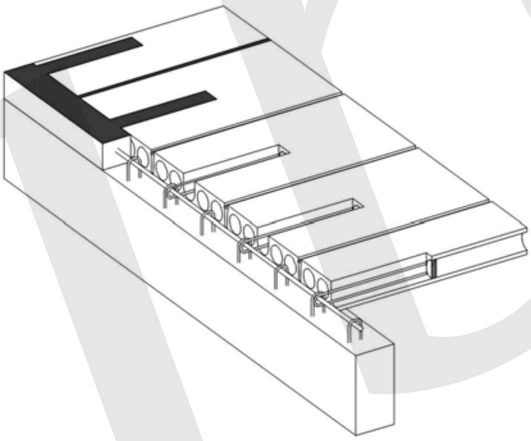


Fig. 5-25: Example of "ring beam" around untopped hollowcore slabs with tie reinforcement in edge insitu concrete strips.

There are a number of possible ways of idealising the floor diaphragm for analysis purposes. Some examples are shown in Fig. 5-26.

In the three examples in Fig. 5-26 the lateral forces are transferred horizontally to stiff shear walls at the ends. Typically a tension-compression couple is developed along the outer edges perpendicular to the applied lateral force. The slabs between the outer chords provide shear resistance.

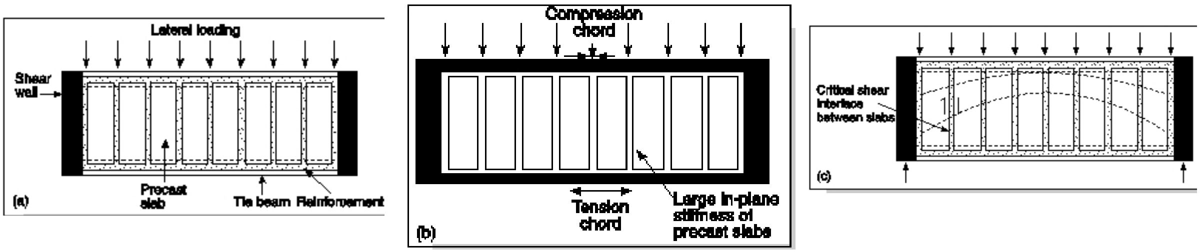


Fig. 5-26: Examples of structural models for floor diaphragms.

Strut and tie models can also be used for analysis of precast diaphragms and can be adapted for different structural layouts and horizontal loading conditions.

Two simple examples of vertical bracing positions are shown in Fig. 5-27. In the one on the left there are shear walls at each end of the building, in the second one on the right the building is stabilised by a core and shear wall inside the floor area. The reactions at the bracing elements are determined via shear wall distribution theory (see chapter 7) and shear force and bending moment distributions in the diaphragm can then be deduced accordingly with the point of maximum moment occuring at the point of zero shear.

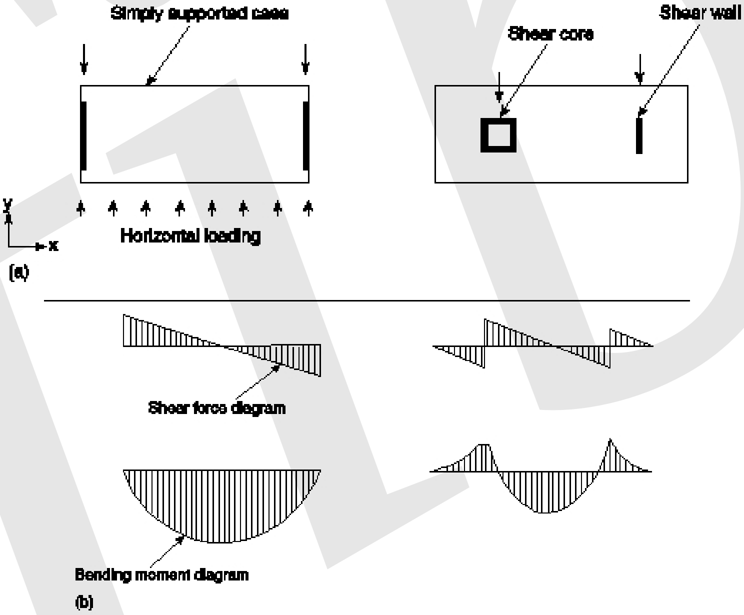


Fig. 5-27: Examples of vertical bracing positions in simple floor diaphragm.

The same approach can be applied to a floor plan with mutiple shear walls. Fig. 5-28 illustrates a floor plan with three shear walls or vertical stiff elements A, B and C. The horizontal reactions in the shear walls (HA, HB and HC) are calculated using Eq. 7-3 in chapter 7. The design diaphragm shear forces and bending moments can then be calculated using statics. The maximum diaphragm bending moment occurs where the shear force is zero between walls, there may be several points of zero shear that have to be checked. However, the maximum moment is normally between the walls that are furthest apart.

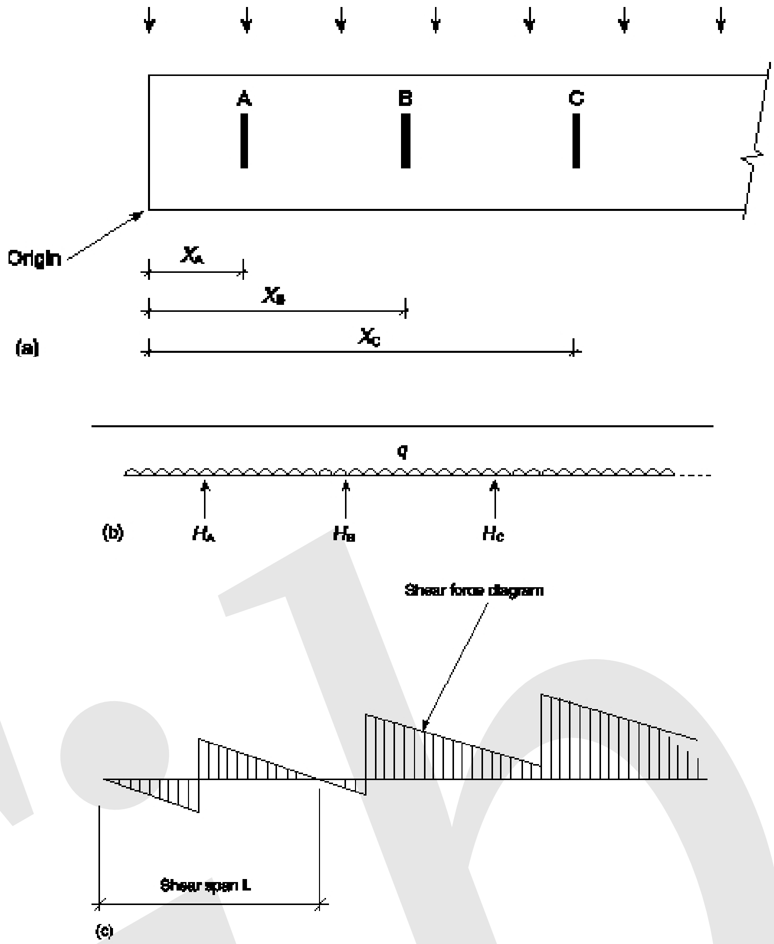


Fig. 5-28: Example of diaphragm with multiple shear walls.

Once the floor diaphragm is subjected to horizontal bending as shown in Fig. 5-29 internal equilibrium is maintained by tension and compression chords. The forces in the chords have to be mobilised by a mechanical connection between the slabs and the chords. The tie force in the chords, T_b, is given as M_h / z where M_h is the applied diaphragm design moment and z is the lever arm.

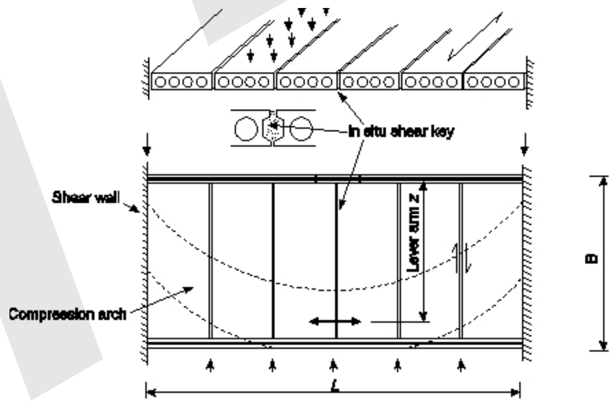


Fig. 5-29: Resistance to horizontal forces in a precast diaphragm.

The value for z depends on the aspect ratio for the floor and the magnitude of the bending. Maximum values for z / B at points of maximum bending are 0.9 where B is less than 0.5 L, and 0.8 for greater values (Bruggeling & Huyghe 1991, and Walraven 1990).

If the joints between precast floor elements are considered cracked then shear resistance in the diaphragm is provided through a combination of aggregate interlock, dowel action and the insitu placing of rebar. Fig. 5-30 illustrates aggregate interlock and dowel action.

Aggregate interlock may be separated into two distinct phases, i.e. "shear wedging" where the inclined surfaces each side of the crack are in contact, and "shear friction" where the surfaces are being held in contact by the normal stress generated by the transverse tie force T_q, which is the tie force due to shear and is equal to V_h / μ' where μ' is an effective coefficient of shear friction and wedging combined (a minimum value of 5 is suggested).

If there are a number of floor bays, n, then T_q is shared equally between each number of chords, i.e. $n+1$. Therefore, for combined bending and tension, the maximum tie force in the chord is given as; $T_h = T_q + T_b = V_h / ((n+1)\mu') + M_h / z$

The elasticity of the tie reinforcement enables a normal stress, σ_n , to effectively clamp the units together.

Dowel action applies through kinking and shear capacity of the reinforcement in the chords.

Shear wedging relies on the adhesion and bond at the precast / insitu interface and is exhausted when the width of the interface crack is sufficient to cause an increase in the tie force T_q. It is influenced by the surface roughness of the slabs and shrinkage of the joint infill material.

Shear friction can also provide a large shear resistance. It is present when both crack width and tie force T_q are increasing. It too is also influenced by the surface roughness of the slab edge, but more by the amplitude of the crevices than by the profile. It is exhausted when the crack width exceeds a certain value, roughly equal to the amplitude of the surface crevices.

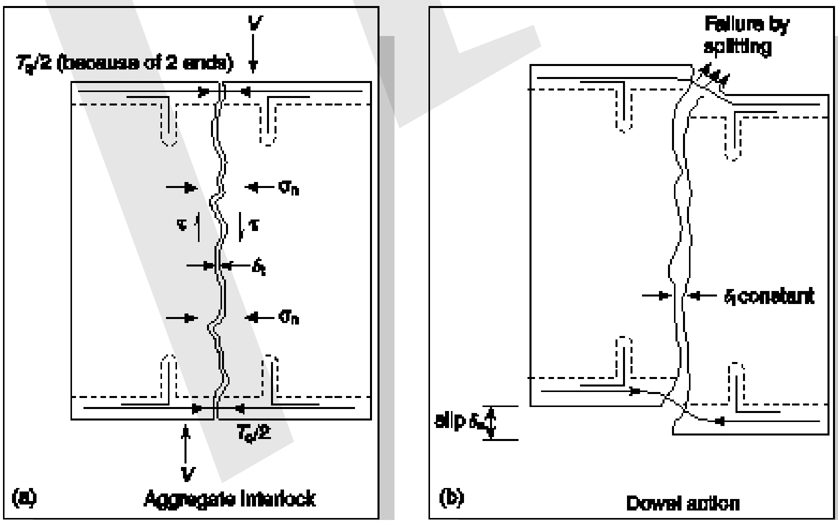
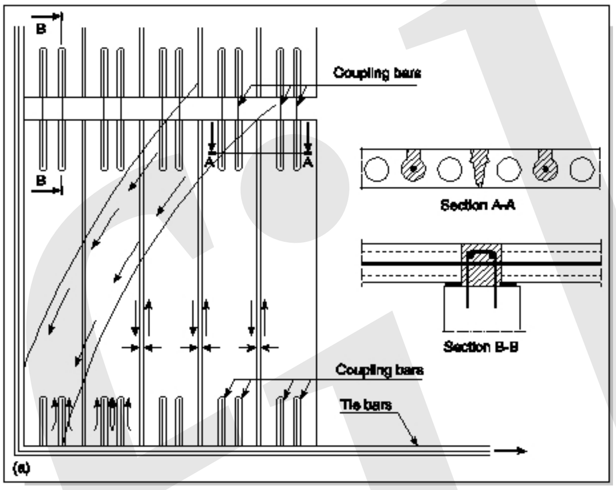


Fig. 5-30: Illustration of aggregate interlock and dowel action.

Dowel action provides lower strength than aggregate interlock but has greater deformation capacity and ductility. It is influenced by the ability of the tie steel in the chords to resist shear forces by bending and kinking, and is dependent on the manner in which the tie rebar is anchored into the diaphragm. The edge profile of the precast slab has no influence on dowel action.

Examples of the site fixed reinforcement that assists in developing the shear resistance described above are illustrated in Fig. 5-31. The reinforcement is interconnected between beams forming the chords and the hollowcore slabs. Preformed slots are provided in the ends of the hollowcore slabs and rebars are insitu concreted into them (a). These bars pass through projecting links from the chord beams into corresponding slots in hollowcore slabs on the other side of the beam. Where slabs are parallel to beams (b) two edge cores are exposed and connecting rebar from the beam is concreted into them insitu.



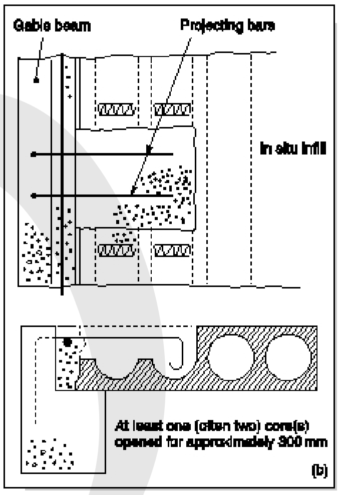


Fig. 5-31: Examples of site fixed reinforcement in untopped hollowcore diaphragms (adapted from "Prefabrication with Concrete", Bruggeling and Huyge, Balkema Press, 1991).

Examples of common slab edge profiles are shown in Fig. 5-32. These profiles allow infill mortar to be placed in the joints and are used mainly for vertical shear transfer.

Fig. 5-32: Examples of edge profiles used mainly for vertical shear transfer.

When there are shear reversals, e.g. in seismic or strong wind conditions, the grouted joint can be damaged and shear resistance reduced closer to that of a plain joint (i.e. uncastellated). This problem can be overcome by a joint profile more appropriate to horizontal cyclic loading such as a continuous sequence of curved wedges, as shown in Fig. 5-33, that provide controlled sliding.

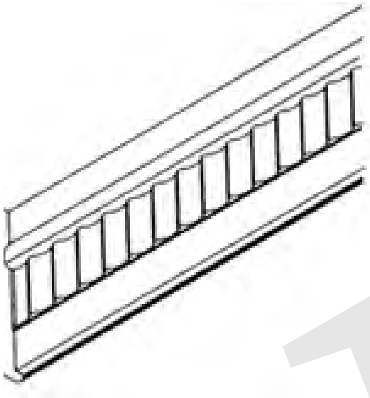


Fig. 5-33: Waved profile along the edge of a hollowcore slab.

In this case the adjoining joint surfaces would move apart whilst sliding on each other making the joint expand transversely and thus mobilising the tying system, which then becomes the limiting factor in diaphragm resistance to horizontal actions.

It is essential that any such profile as described above should have its behaviour validated by test (refer to paper by Menegotto and Monti[48] for testing of the waved profile already described).

The alternative to providing a precast diaphragm as previously described is to use a structural topping as the diaphragm. This situation may arise because:

1. The precast section is too thin, e.g. filigree biscuits, double tees with thin flanges
2. The end of the precast units cannot be effectively tied to the supporting structure to transfer the diaphragm forces to the vertical stability elements.
3. There is no horizontal shear transfer mechanism between individual floor units.
4. The design shear forces exceed the capacities of the precast connections (a possibility in tall buildings) that are likely to be governed by the floor depth primarily designed to resist gravity loading.

The resulting floor is designed on the basis that the precast flooring units provide restraint against lateral (in this case vertical) buckling in the relatively thin topping. In other words the precast floor acts as a permanent shutter in the diaphragm design. The horizontal diaphragm shears are resisted entirely by the insitu topping. Continuity of reinforcement in structural toppings is always extended to the shear walls or cores. Designers should be careful not to allow large voids next to external shear walls, and to ensure that if an external wall adjacent to a prominent staircase is used then a sufficient length of floor plate is in physical contact with the wall. Reinforcement in the topping is uniformly distributed over the full area of the diaphragm and is normally designed according to beam theory. See example in Fig. 5-34 using British Standards.

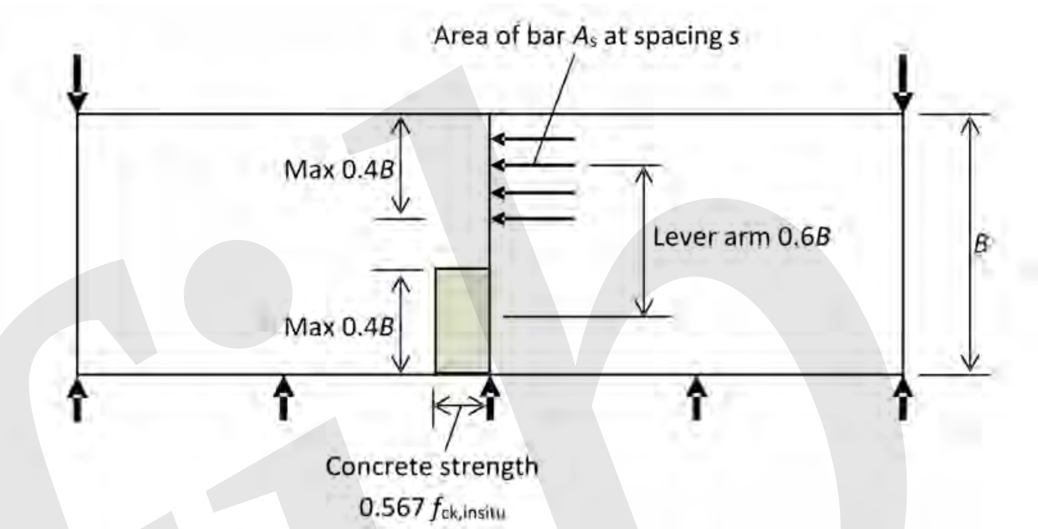


Fig. 5-34: Resisting forces in a topped diaphragm.

6. Precast Concrete Columns

6.1 Introduction

Precast concrete is used extensively for columns in tall buildings, and can be combined with other components, such as floors and cores, in both precast and insitu concrete to give a fast, high quality solution that satisfies structural, architectural and construction requirements.

Most shapes are possible in precast concrete, from simple rectilinear sections to curved, elliptical, hexagonal, and so on (see Fig. 6-1 and 6-2 for examples), also with varying cross section along their length to produce flared heads and bases. Economy in precast construction is often achieved through repetition, and in taller buildings this is most likely to be possible where numerous levels / storeys are the same. Complicated shapes can be shown to be both cost and time beneficial.

Generally, columns and walls are differentiated on the basis of the cross-sectional ratio of the longer side (length) to the shorter side (thickness). Where this ratio is less than four (ratio of three in US) then the section is designed and detailed as a column, where it is four or more the rules for walls apply. This designation is important as often different methods of analysis, fire requirements and reinforcement detailing are applicable. Additionally, the methods of manufacture for precast columns will be different compared to walls. Precast walls lend themselves to long line mass produced floor slab methodology. Walls and floors can often be made with the same production machinery. Columns relate more to the methods used in beam production.

As with most precast components, column design follows similar rules to those applicable to insitu reinforced concrete. However, there are differences, and these mainly relate to the connections between the column and the adjoining precast and / or insitu elements, and to its temporary stability before completion of the connections. The interaction between precast columns and the adjoining structure also requires attention, for instance, the relationship with lower strength concrete slabs sandwiched between higher strength columns, the differential in elastic shortening and the post tensioning of supported slabs.

Higher strength concrete is also more readily accessible in precast production factories. In tall buildings, that often have a small footprint, columns with higher strength concrete can be provided with a smaller, more structurally efficient cross section that increases valuable free floor space. However, although concrete mix strengths as high as 105 N / mm^2 may be possible limitations may be placed on their use, for example, in relation to special measures to limit spalling in fire situations, the effective height of the compression zone in flexure and the calculation of ultimate bond stresses. These items need to be considered as the concrete strength increases.

This chapter was mainly authored by Jones, van der Zee.

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Fig. 6-1: Elliptical cross section with projecting top connection bars.

Fig. 6-2: 900 mm diameter single storey circular columns with cast in tubes for connections at floor levels.

6.2 Design and Detailing

6.2.1 General

In tall buildings, the walling systems, whether they form cores, shafts, or individual shear walls, have a far greater degree of stiffness relative to the columns. Consequently, the columns would only attract a minimal horizontal force, and therefore contribute a relatively small proportion towards the lateral stability of the building. In those circumstances moments due to gravity loading would most likely be dominant and provide the design combination.

There are instances where lateral stability is provided through moment resisting frames, and in those cases, moments generated from lateral loading would most likely be critical. However, structures braced by wall combinations are more common in taller buildings.

Columns are typically subject to bending about two orthogonal axes, in addition to axial loading. In tall buildings moments due to slenderness and second order effects may be significant. A common design approach to this combination is to analyse the section separately about each principal axis and calculate a moment of resistance, M_{Rd}, in the respective direction corresponding to the design axial force, N_{Ed}, and design moment, M_{Ed}. A relationship as shown in Eq. 6-1 (example taken from EN1992- 1- 1[5]) below should then be satisfied:

$$\left(\frac{M_{Edz}}{M_{Rdz}}\right)^n + \left(\frac{M_{Edy}}{M_{Rdy}}\right)^n \leq 1.0 \qquad (6\text{-}1)$$

In Eq. 6-1, y and z are the orthogonal axes, and n is an exponent that is related to the utilisation of the axial capacity of the section, i.e. N_{Ed} / N_{Rd}.

6.2.2 Idealisation of end connections

Precast columns are typically bedded at the bottom either onto an insitu floo , infill, or onto another precast element such as another vertical element (column or wall) or floor beam. Coupled or lapped reinforcement provides continuity with the concrete surface normally bedded using high strength low shrink grout. The top of the column is similarly bedded if connected to a precast element. Where the column is supporting an insitu floor then the top of the column normally penetrates the slab soffit shutter and then becomes integral with the floor slab when poured.

The magnitude of moment due to gravity loading and delivered to the column from its connection to the supported floo , is dependent on the rigidity of the connection. Generally, connections are modelled as fixed at both ends of the column so that the column is bent in double curvature. The main perimeter reinforcement would be continuous through the floor and connect the precast columns above and below the floor slab. The stiffness of the connections at top and bottom of the column would be approximately equal if the section size is maintained. To limit the number of connected bars at the joint it is often preferable to have the corner bars of larger diameter so that the design resistance is achieved through the four corner bars only. Smaller bars, but at maximum spacings to satisfy Code detailing rules, are then placed around the perimeter through the height of the column.

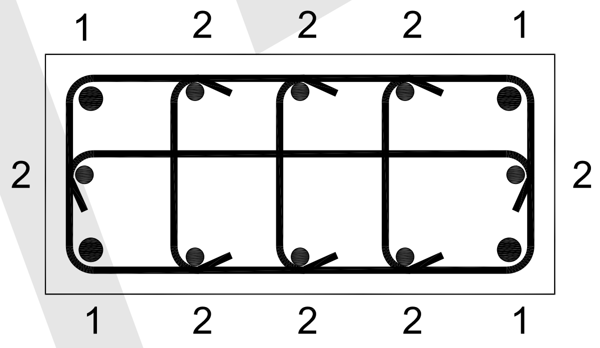


Fig. 6-3: Typical rectangular column section – Bars mark 1 indicate four corner bars contributing to design resistance at floor joint, bars mark 2 are smaller bars in height of column to satisfy Code detailing rules.

However, when using post tensioned floors the importance of the stiffness of the column and its connections increases and can influence the floor design, particularly where the level of prestress is high, and the floor is long. If the restraint provided by the column is so severe that flexing of the column prevents shortening of the floor then the bottom of the column may have to be idealised with reduced rigidity and freed in the temporary condition. This measure would affect the connection detail at both the top and bottom of the column. Note that this detail, often represented as centrally located reinforcement, not only reduces the moment capacity of the section, but the axial load capacity is also reduced as the perimeter reinforcement terminates at the bottom of the column. The design moment at the top connection is, in turn, increased, and may require reinforcement to be bent into the slab to allow the next column above to have the desired reduced rigidity.

In tall buildings where it is generally important that the maximum column capacity is achieved, and where floor lengths are relatively short, then reducing connection stiffness at the base of columns to assist post tensioned floors should be avoided. Some producers and contractors, however, still prefer this detail as only a single bar connection is required on site and are prepared to accept the resultant loss in capacity and / or possible increased concrete section size to achieve greater potential time efficiencies.

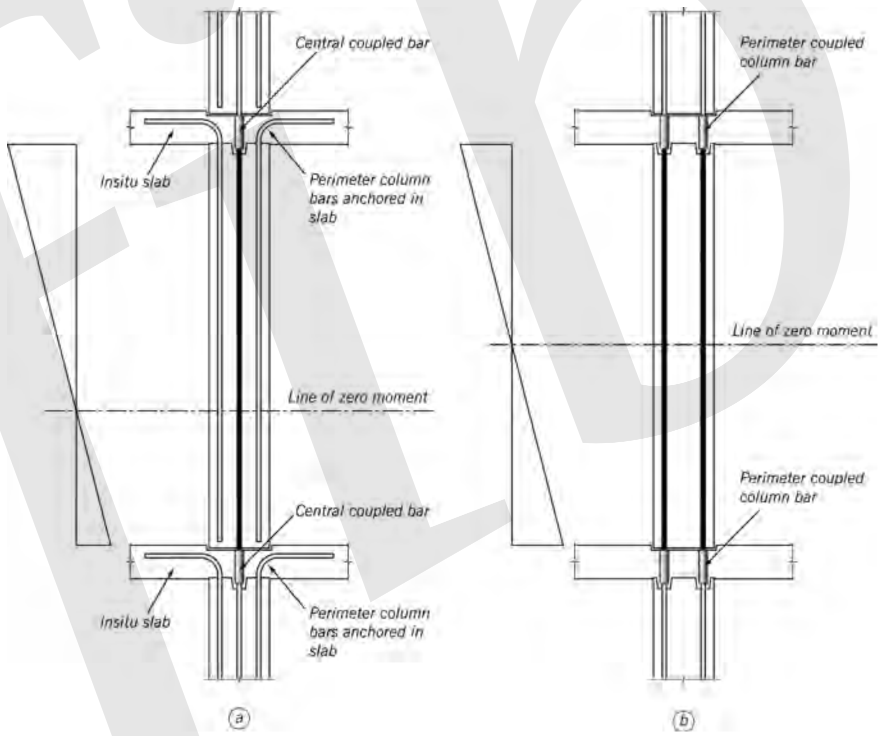


Fig. 6-4: Comparison of columns with reduced rotational stiffness at bottom connection, a), and approximately equal stiffness at top and bottom connections, b).

Top connections for both columns a) and b) in Fig. 6-4 behave in a similar manner to their insitu concrete counterparts and can be analysed accordingly. The bottom of column b) is also designed like a normal reinforced concrete column, but a nominal allowance should be made for potential loss of contact area, normally 10 % is considered reasonable, unless it can be shown that full grout coverage of the bearing area is achieved (possible simplified approaches would be to limit the structural capacity utilisation to 90 % at the bedded joint or reduce the concrete strength by 10 %). At the bottom of column a), the capacity is limited by the curtailment of the perimeter reinforcement as mentioned previously. The central connecting bar, as shown in Fig. 6-5, provides compressive resistance whilst the full section is compressed. Care must be taken to avoid accumulation of dirt in the bedding area and inside the coupler or cast in tube that would lead to reduction in the connection strength.

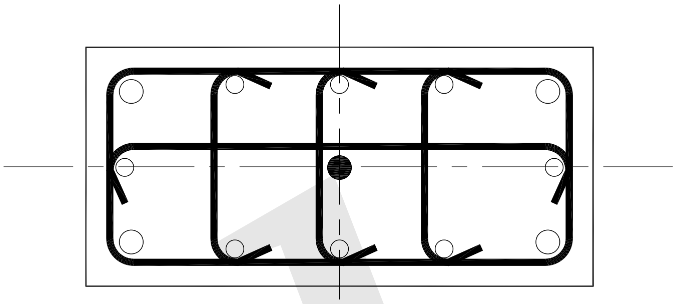
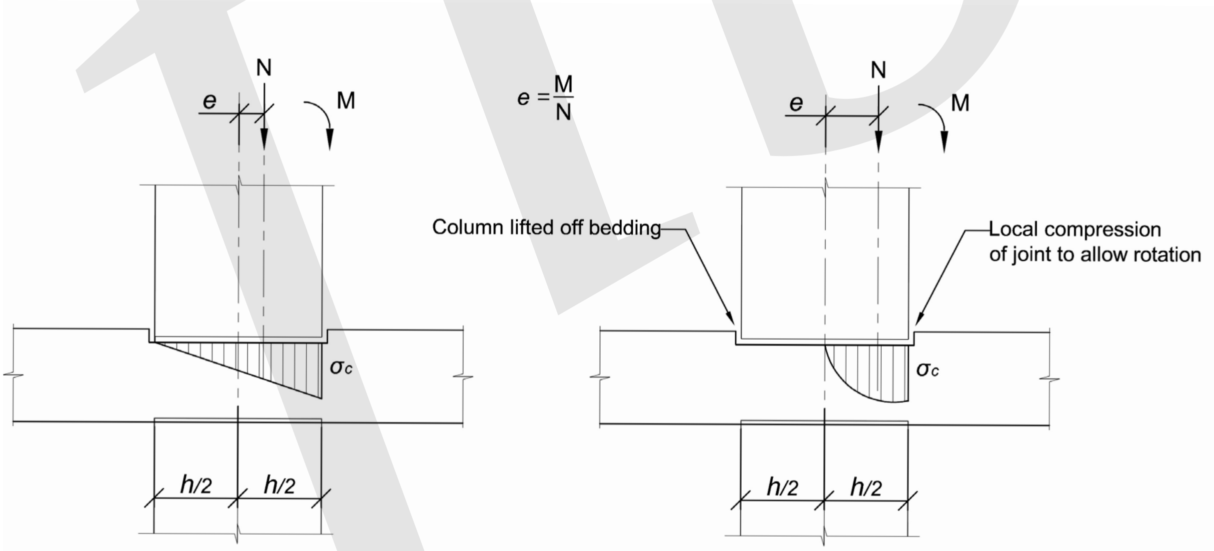


Fig. 6-5: Central bar at bottom connection – The perimeter reinforcement is curtailed at the bottom of the column and a central large diameter bar completes the connection.

Some additional bending capacity can be provided as the joint opens and tensile resistance is mobilised in the bar, however, increasing eccentricity will cause further joint rotation and a reduction in rotational stiffness. If this detail is used for larger eccentricities then the changing rotational stiffness needs to be carefully modelled to avoid crushing and / or excessive joint opening.



Eccentricity, e, at uncracked limit

Eccentricity, e, beyond limit

Fig. 6-6: Comparison of load distribution across bottom bedded joint with central reinforcement as load eccentricity, e, increases.

6.2.3 Section analysis

Precast column section analysis for combined bending and axial force for a double reinforced section is the same as an insitu section. For example, for a rectangular section with the reinforcement symmetrically located about orthogonal centrelines equations can be derived by resolving forces and taking moments about the mid depth of the section, *h/2*, as illustrated in Fig. 6-7.

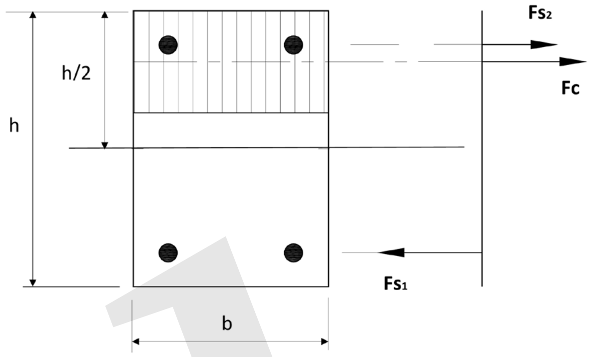


Fig. 6-7: Basis of analysis for a symmetrically reinforced section.

The symbols used in Fig. 6-7 are:

- Fs_1: The force in the reinforcement in the tension zone
- Fs_2: The force in the reinforcement in the compression zone
- F_c: The compressive force in the concrete

If a central rebar solution at the bottom joint is adopted (see Fig. 6-5) then a different approach is required and is dependent on whether the joint is fully compressed or if tension develops (see Fig. 6-6). The section would first be analysed under service conditions to check if the eccentricity ($e = M/N$) is less than $h/6$. If no tension is developed then a simple bearing stress check is undertaken using a rectangular stress block at ultimate limit state to ensure that the axial resistance is greater than the design axial force ($N_{Rd} > N_{Ed}$). The area of concrete, A_c, providing compressive resistance is reduced to take account of the eccentricity. The reinforcement provides compressive resistance only in this case.

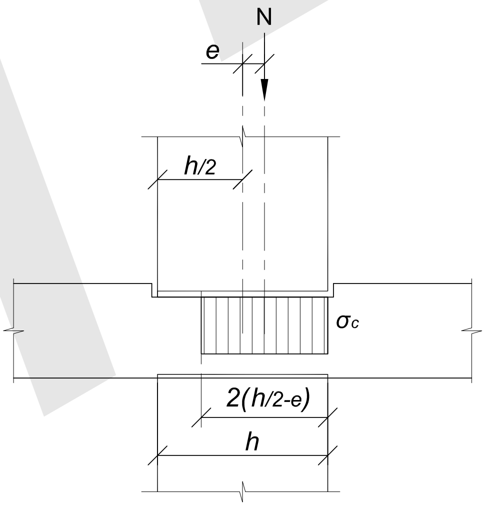


Fig. 6-8: Simple bearing check when $e \leq h/6$.

$$N_{Rd} = A_c \cdot f_{cd} + A_s \cdot f_{yd} \tag{6-2}$$

$$A_c = 2b\left(\frac{h}{2-e}\right) \tag{6-3}$$

A_c may also have to be reduced further to take account of bedding methods at the joint as mentioned previously, together with the area of reinforcement.

In circumstances where tension is developed in service conditions the analysis becomes more complex if a central rebar solution is sought at the bottom joint as the section would have to be remodelled to take account of the joint opening and a reduction in structural stiffness. Remodelling the column joint will have the effect of redistributing column moment to the adjoining structure, most likely slabs or beams, which must then also be rechecked. This is normally done by adjusting the column section in the model. However, unless the precast designer has access to the structural building analysis model, this option may not be available and a more conventional solution using four corner bars to resist the original design moment may have to be used. This situation often occurs in the top storeys where axial loads are relatively smaller compared to moments.

6.2.4 Higher strength concrete columns with normal strength slabs

Higher strength concretes have been developed over recent years, and these have become increasingly popular in columns and walls of tall buildings where reduction in concrete section size is very attractive when available floor space is at a premium.

High strength concrete is commonly defined as strengths greater than 60 N / mm². However, strengths of 95 N / mm² and greater can be produced and used in tall buildings. It is generally either impractical or commercially not viable to use the same strength in insitu concrete floor slabs. It has been mentioned previously that concrete columns are often used with insitu flat slabs, and, in turn high strength columns may be used

General guidance is that the stresses can be transferred satisfactorily when the difference between the strength classes is limited[36]. ACI318[7] suggests that the full column strength may be used for all columns provided it is no more than 1.4 times the slab strength. However, if the difference is greater than this value then the effective strength of the joint concrete, f_{ce}, is reduced. The relative amount of restraint provided by the confining slab concrete means that there are different factors applied to interior, edge and corner columns. For columns near the edges of buildings, but where the slab reinforcement extends a full anchorage beyond the column, then the column can be considered internal.

ACI318 advises the following equation for interior columns, where f_{cc} is the column strength and f_{cs} is the slab strength:

$$f_{ce} = 0.75 f_{cc} + 0.35 f_{cs} \tag{6-4}$$

In addition, f_{cc} should not be greater than $2.5 f_{cs}$ to ensure that there is adequate restraint to the concrete beneath the column. ACI suggests that for design of edge and corner columns floor joints, either the concrete between upper and lower columns should be the same strength or the slab concrete strength should be used.

Research by Ospina and Alexander[37] has proposed the following relationship for interior columns that introduces the effect of the ratio between column and slab dimensions (h/c), where h is the overall slab depth and c is the smaller column dimension. The ratio h/c must also be greater than or equal to 0.33. In addition, fce must not be greater than either f_{cc} or $2.5f_{cs}$, and not less than $1.33f_{cs}$:

$$f_{ce} = \left[\frac{1.4 - 0.35}{\left(\frac{h}{c}\right)}\right] f_{cs} + \left[0.25\left(\frac{h}{c}\right)\right] f_{cc} \tag{6-5}$$

For corner columns Lee and Mendis[38] concluded from research that there is the following relationship:

$$f_{ce} = k \cdot f_{cs} \tag{6-6}$$

k is a factor that varies from a minimum of 1.1 when h/c = 1.0 to a maximum of 1.7 when h/c = 0.3. There is no specific guidance for edge columns but Eq. 6-6 could be conservatively applied as there will be additional confinement for an edge column compared to a corner column.

Eurocode[5] also provides guidance by increasing the loaded area in the slab when assessing stresses, as shown in Fig. 6-9:

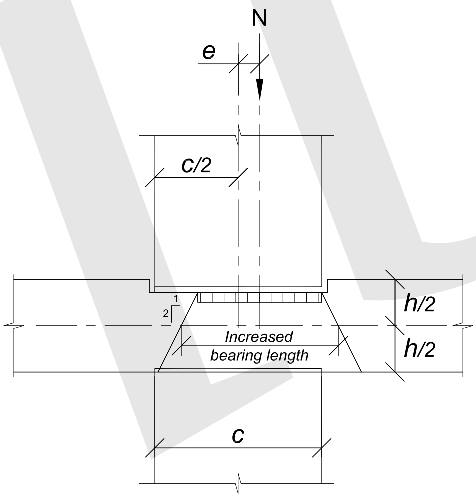


Fig. 6-9: Increased bearing length at slab mid height.

The increased bearing length is based on a distribution in a ratio of 1:2 through the thickness of the slab taken at its midheight as shown above. The benefit is limited to a maximum of $3f_{cs}$: or f_{cc}:. The enhancement is also restricted by any slab edge that may be near.

The effective strength of the slab concrete can also be increased by using designed confinement reinforcement in the slab to avoid bursting of the concrete. Internal columns, and set back edge columns, would have ring reinforcement immediately beneath the main top and above the bottom slab reinforcement layers. Concrete Society Technical Report 64[36] advises that A_{ring}, the total area of the ring reinforcement is calculated as follows:

$$\frac{A_{ring}}{R \cdot h} \geq \frac{\left[0.4 f_{cc} - 0.45 f_{cs}\right]}{f_{yd}} \tag{6-7}$$

f_{cc} is limited to 2.5 f_{cs} and R, the radius of the ring, should be no closer to the column than $h/4$. Column links within a distance c above the slab should also at least be equal to A_{ring}.

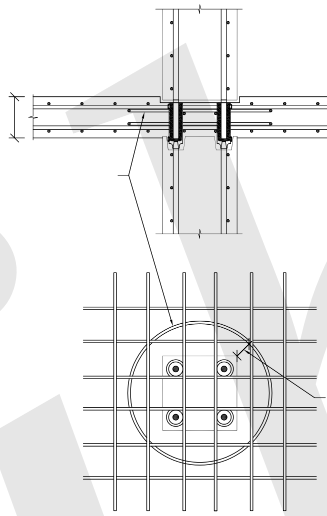


Fig. 6-10: Arrangement of slab confinement reinforcement.

For large columns in thin slabs the containment link size can be relatively large compared to the slab reinforcement and cause congestion. Alternative solutions in those circumstances may be advisable in the form of localised higher strength slab concrete or precast columns passing through the slab that bypass the lower strength slab concrete, and have the column-column connection outside the slab. Detailing of this connection may also have its own problems in effectively transferring the forces across the joint.

6.2.5 Concrete section change between floors and walking columns

The column section size often changes across floors. This could arise solely because of increased axial forces at lower levels that require a larger section resistance, but is also often due to a change in occupancy. For instance, basements normally used as car parks require individual rectangular or circular column sections at regular locations to assist car parking arrangements, then at ground floor there may be retail and reception areas where it is possible to continue the basement arrangement, or perhaps space columns further apart to achieve more open space. Moreover, at first floor the occupancy may change again to either residential or office, where columns may be replaced by walls in the former case, and in the latter, the column arrangement may change to suit the office layout, which is often not the same as a retail / reception layout.

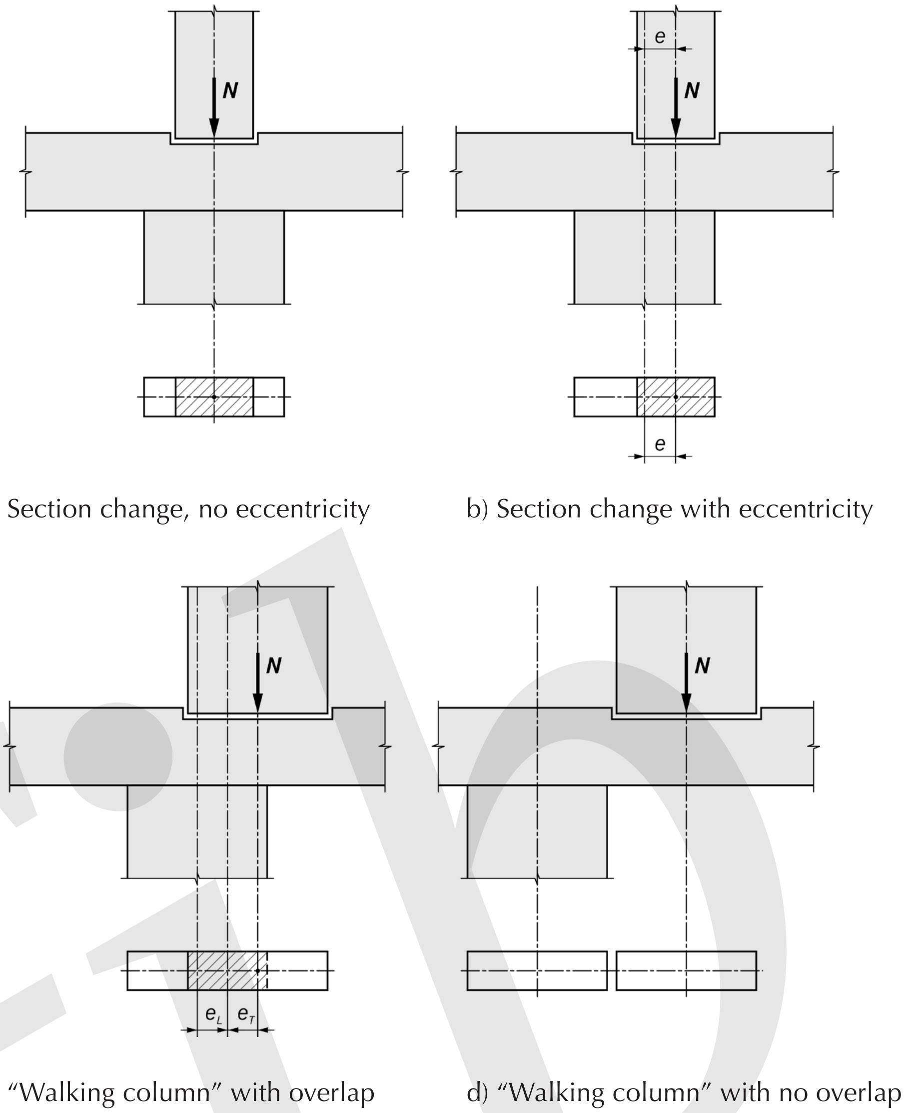


Fig. 6-11: Examples of column section changes at floor levels.

Examples a) and b) in Fig. 6-11 show simple cases where the upper column is within the plan area of the lower column, example b) indicates an eccentricity that must be accounted for in design of the lower column. Examples c) and d) show "walking columns", i.e. instances where either part or all the upper column is outside the area of the lower column. In example c) there is an overlap of upper and lower column sections. If the general slab thickness is maintained then the overlap is normally checked as a smaller column with top eccentricity e_T as shown. In circumstances where there is either minimal, or no overlap as shown in example d) then the transfer of force between columns should be achieved through a thicker slab, this could either be in the form of a localised column head or a general transfer slab if there are many situations of this type at one floor level. Careful attention to detailing, and close cooperation with the slab designer, is necessary to reflect the design approach adopted at changes in column section. Precast columns can still be used in these locations, but localised reinforcement congestion at connections at the insitu slab interface needs to be carefully considered.

6.2.6 Shortening due to axial load and shrinkage

Concrete columns in tall buildings shorten over time and as load is applied[35]. This effect is due to the following factors:

1. Initial axial strain: For each increment of load there is a corresponding initial elastic strain;

2. Creep: Increase in axial strain over time;

3. Shrinkage: Will start immediately after early thermal contraction and continues at a decreasing rate. Precast columns have an advantage over insitu columns in this regard as the first part of the shrinkage cycle may already be complete when delivered to site.

The amount and effects of axial shortening can be difficult to predict as they are sequence and time dependant. In addition, columns are normally more heavily loaded than walls and differential shortening between them can occur, particularly as insitu slip or jump formed cores would also be constructed in advance of columns and floors. Any differential shortening between connected vertical members will generate transfer of vertical load. This can be substantial if connecting horizontal structure is relatively stiff and needs to be checked in tall buildings.

The total deformation for a column at level n due to axial shortening can be calculated using the general formula in Eq. 6-8:

$$\Delta L_{tot}^{n} = \sum_{0}^{n}\left(\Delta L_{pp1} + \Delta L_{f} + \Delta L_{m}\right) - \Delta L_{pp3} \qquad (6\text{-}8)$$

Where ΔL_{pp1} is the deformation due to self-weight of the structure.

ΔL_f is the deformation due to superimposed dead loads.

ΔL_m is the deformation due to quasi permanent loads.

ΔL_{pp3} is the deformation that had already taken place in the structure beneath the column before it was installed, i.e., the column would have been installed at the correct level.

The deformation, ΔL, due to the application of any axial load, N, is calculated using the simple formula in Eq. 6-9 derived from the concrete stress / strain relationship:

$$\Delta L = k\left(\frac{N \cdot L}{A_c \cdot E_c}\right) \qquad (6\text{-}9)$$

k = Creep coefficient, and is a factor related to several variables such as relative humidity, age of loading, notional size of the member, concrete strength.

N = Axial load

L = The length of the compressed element, normally a storey height of column

A_c = The plan area of the compressed element

E_c = The elastic modulus of the compressed concrete

In the calculation of total deformation due to axial shortening at any specific level the deformations due to the design axial load from above (normally self-weight, superimposed dead load and quasi permanent loading effects are separated, as in Eq. 6-8, due to different times of application) are added together to arrive at the required value.

The total shrinkage typically consists of two parts: autogenous shrinkage and drying shrinkage. Autogenous shrinkage is a function of the concrete strength, and strains develop during hardening in the early days after casting. Thus, precast columns may not be affected by autogenous shrinkage if delivered to site several days after production. Drying shrinkage strain develops slowly due to water migration through hardened concrete and should be considered in design. Strain values due to drying shrinkage are related to the relative humidity of the column's environment and its strength, amongst other factors.

Rigid elements that may not deform with the main structure, such as non loadbearing external concrete facade panels, should also have their joints and connections designed to accommodate deformations due to column shortening.

6.2.7 Shear Resistance

Columns in tall buildings are generally subjected to large axial loads that enhance shear resistance. In circumstances where axial loads are relatively small (near top of building) or when horizontal loads are very large (seismic actions) then shear reinforcement may have to be designed using similar methods as used for beams. In general, transverse reinforcement or links should at least satisfy standard detailing rules[5],[17].

6.3 Methods of Production

6.3.1 General

Precast columns in tall buildings are generally produced in storey height lengths, particularly when combined with insitu concrete floor slabs. However, multi storey columns are becoming increasingly popular. At higher levels in tall buildings the section sizes can be smaller due to reduced axial load and consequently crane lifting weight restrictions can be satisfied, allowing multi storey lengths and further improving crane utilisation.

Fig. 6-12: Smaller section double storey column (courtesy of HSP Consulting).

Fig. 6-12 has a double storey column with a gap between concrete sections and continuous reinforcement connecting the upper and lower parts of the column. The horizontal reinforcement from the floor slab passes through the gap and the column and slab become monolithic once the floor is poured. Care is required in detailing and insitu concrete placement to ensure that there is adequate bearing at the column / slab interface.

Columns, such as the example above, can be produced in simple horizontal moulds, similar to those used for beams, provided the top floated surface satisfies architectural requirements. In circumstances where visual concrete is required on all vertical sides then the above detail may not be possible and the columns would probably have to be cast vertically with a moulded finish on all four faces.

It is generally the surface finish specification and uniformity of the concrete section that dictates the mould that is chosen and the production method adopted. Horizontal casting of reinforced sections in long uniform moulds, typically made from steel with repeated useage, would usually provide the most cost effective production solution. However, in many instances this approach is not possible and producers have developed materials, equipment and methods to satisfy specification demands.

Fig. 6-13: Horizontally cast double storey square columns with intermediate coupler connection for floo slab.

Fig. 6-14: Steel mould for non uniform shaped column.

6.3.2 Single storey columns

The simplest column is the square, or rectangular, uniform section that is produced in a horizontal mould with an open top surface, and has no projecting corbels, nibs or reinforcement. These would generally have cast in tubes to accept site placed connecting bars that are later grouted in place. Examples are shown in Fig. 6-15.

Fig. 6-15: Simple rectangular sections with cast in tubes for connection.

Note the element in the bottom right hand corner that has the tubes projecting a length equal to the depth of the floor slab to allow the slab concrete to be poured around them, with the connecting bar grouted in place later. The design detail reflected in Fig. 6-15 allows for fixity at the column / slab interface, both top and bottom. As mentioned earlier some producers prefer to provide a minimum of one or two connecting bars at the bottom of the column, which reduces the stiffness at that end, but, in turn would require reinforcement to be bent into the slab at the top for fixit . This approach creates additional difficulties when storing and transporting the elements.

A minor increase in complexity to the columns in Fig. 6-15 would be to have the connecting bars projecting from one end of the column with tubes at the other end. This may be attractive from the point of view of material savings, but it is likely there will be increased difficulties with accuracy, handling, storage and transport.

Circular columns generally have to be cast vertically. Short, large diameter columns are relatively straightforward, however, more slender columns may require the concrete to be pumped from the bottom of the mould to ensure adequate compaction. Fig. 6-16 shows a large number of circular single storey columns (probably required in basement locations) in the storage yard awaiting despatch.

Fig. 6-16: Circular columns in the storage yard awaiting despatch (note the steel mould in the foreground).

Expendable plastic lined formers are often used if there are only a relatively small number of circular columns. However, if there are a large number of columns of the same diameter and the demand from site is high then steel moulds may be required to give repeated rapid turnaround.

Short, single storey columns are generally associated with insitu flat slab construction and are therefore simple and uniform. However, they can also be used with precast beams and slabs, and can be cast with projecting corbels or nibs for their support. Typically, the corbel or nib is fixed at one end of the mould and the column length varied from the other end as necessary.

Irregular shaped elements are also possible, and would normally be produced in single storey lengths. However, the irregular shape, for instance a flared head, should be consistent in its location and size to minimise mouldage costs, particularly if a high quality finish is required. Cast in plates for connection to steelwork should be repetitively located to avoid excessive damage to expensive moulds.

6.3.3 Multi storey columns

Fig. 6-12 and 6-13 have previously shown examples of double storey columns, the former with continuous reinforcement and a gap left for the intermediate floor slab, the latter with a continuous concrete section and couplers cast into the column for connection to the floor reinforcement. However, more than one floor connection can be incorporated in a multi storey column provided the column is within the crane lifting limit. Fig. 6-17 shows a column with two floor connections at an interface with a mezzanine level.

Fig. 6-17: Precast column with allowance for two insitu floor connections.

Because multi storey elements are necessarily longer they need to be cast horizontally due to the practical difficulties of casting tall elements vertically. This is not a problem with square or rectangular shapes but presents challenges for circular elements, in particular if they also require corbels.

A building system has been developed in Belgium by Ergon Structural Concrete that uses double storey circular columns with supported precast beams and floor slabs. Fig. 6-18 and 6-19 show the family of moulds used to produce the columns and the finishing in the factory. Fig. 6-20 shows the installed double storey columns in the building during construction. A column / column connection is also achieved that avoids the issues of sandwiching normal strength floor concrete between high strength precast concrete columns.

Fig. 6-18: A family of moulds set up for horizontally casting double storey circular columns with intermediate corbels. Note the finished column alongside the mould in the background (courtesy of Ergon Structural Concrete).

Fig. 6-19: Finishing double storey columns at the factory.

Fig. 6-20: Double storey circular columns installed in the building frame.

6.3.4 Prestressed columns

Prestressed columns are rarely used in tall buildings as part of the section axial capacity is required to accommodate the prestress forces, thus reducing the overall axial capacity. They are generally used in situations to counteract tensile stresses that may develop where there is minimum applied axial force and relatively large bending stresses, and use similar casting technology and methods as long line prestressed beams, except that there would be a concentric strand arrangement to achieve centroidal prestress and avoid force eccentricity effects.

6.3.5 Spun columns

Precast concrete columns can also be produced by spinning concrete and using centrifugal force for internal compaction in a similar manner to concrete pipes. The technology allows most shapes to be formed; rectangular, elliptical and tapered, as well as the traditional circular / annular shape normally associated with spun concrete. The void inside the spun ring can be used for services if required. Fixings and connections similar to conventionally moulded precast columns are available.

Fig. 6-21: Spinning formwork / machinery for precast concrete columns.

Moulds have two halves that are separated to allow the reinforcement cage and built in components to be placed and fixed. The concrete is then poured into the lower mould, and the two halves of the mould closed and bolted together. The entire mould is then lifted onto the spinning machine. The mould is rotated with the concrete for about ten minutes at around 600 rpm. The fresh concrete is pressed against the inner face of the mould with a force with equivalent acceleration of 20 g. These centrifugal forces produce a cavity inside the concrete ring. A high strength, durable concrete is produced from the spinning process. Reference should be made to producers of spun concrete columns for axial capacities that are achievable.

6.4 Connections

6.4.1 General

Connection of vertical reinforcement at both ends of precast columns is generally the critical consideration when combining precast columns with other elements of the structural building frame. This is normally achieved by either grouting connecting bars into steel ribbed tubes that are cast into either the top or bottom of the element or through coupling reinforcement, or a combination of both methods. There are also other proprietary connections available that can perform the same function, such as column shoes. The vertical connection at the first base level, the “starter” level, is often different to those at the upper levels; pocket bases, cast in base plates, and projecting starter bars are commonly used. Tubes and couplers cast into the base are rarely used at starter level.

There are also other connections for support of horizontal members, that may be precast, insitu or steelwork. These connections could either be moulded corbels and nibs that are cast with the column, proprietary steel systems, or cast in steel plates for later welding of connecting steel members.

Then finally there are minor or non-structural connectors, such as steel slots and anchors for partitions, lightning protectors, sockets for temporary propping, and lifters.

6.4.2 Connections at starter level

Four examples of base connections commonly used with precast columns are illustrated in Fig. 6-22.

Connection a) is relatively robust and is the simplest in relation to factory production. The pocket foundation comprises a precast column with a simple plain end and no cast in items. The column is either concreted or grouted into a preformed pocket in the base. The column end is effectively "built in" and the flexu al resistance provided is proportional to the depth of the pocket. The disadvantage is that the base may have to be deepened or a concrete upstand collar provided to form the pocket.

Connection b) is often favoured by contractors as the requirements in the base are the same as those for an insitu column, except greater accuracy is required to match the tolerance provided in the tube cast into the column to receive the starter reinforcement. The cast in tubes are semi rigid metal ducts with corrugated side walls that are use extensively in both precast and post tensioning applications when bonding with both concrete and grout is required. A grout bed is provided between the bottom of the column and the base to transfer vertical loads, the connecting starter bars provide additional bending resistance.

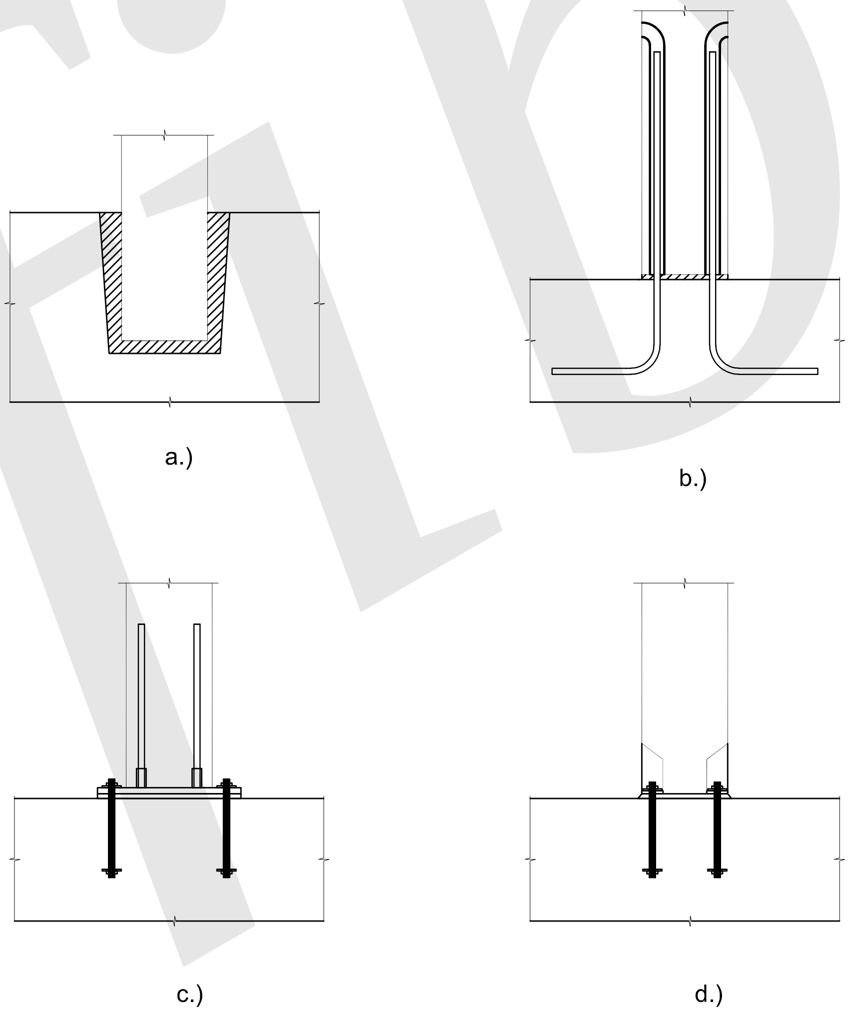


Fig. 6-22: Examples of starter connections.

Connections c) and d) are both like their steelwork counterparts. Prefabricated steel baseplates or shoes with connecting reinforcing bars welded to them are cast into the precast concrete column. The column is then landed onto anchor bolts projecting from the base, and the baseplate then bedded onto the base. Connection c) has an oversize baseplate that will provide more moment resistance than Connection d). Projecting bolts are designed to have adequate anchorage into the base to resist the design forces and moments. There is less flexibility in factory production with Connection c) as the baseplate extends outside the moulded column shape, and therefore limits the use of long line production methods. This limitation is overcome with Connection d) as the baseplate has the same section as the precast column.

Another possible refinement of Connection d) is to use proprietary column shoes in place of the steel baseplate, as shown below in Fig. 6-23.

Fig. 6-23: Proprietary column shoes cast into precast column, being lowered onto bolts cast into base, courtesy of ©Peikko Group Corporation.

In Fig. 6-23 a shoe is cast into each corner of the column, and each shoe has a reinforcing bar welded to it that provides connection to the main column section. Suppliers often have proprietary anchor bolts that are cast into the base and are compatible with the shoes. Generally specialist design software is also readily available from the supplier to verify the resistance of the system to applied forces and bending. Column shoes are often attractive to precast producers as they are versatile and can be adapted to any column shape, whereas the baseplate solution requires a special fabrication for each different section.

6.4.3 Connection of Vertical Reinforcement at Upper Levels

Currently, the most common vertical reinforcement connections at upper levels in tall buildings involve either connecting bars grouted into cast in tubes or coupled bars. Special conditions can arise that require the development of customised connections that may involve cast in steel plates, bolts and welding. Also, proprietary connections such as the column shoe system are used, but generally continuity of reinforcement across joints and floors is achieved through either grouting the bars into cast in tubes or through bar coupling.

If we firstly consider the cast in tube approach then there can be a number of variants to the typical detail depending on the priorities of the producer and contractor. An illustration of a typical thin walled corrugated metal tube is shown in Fig. 6-24.

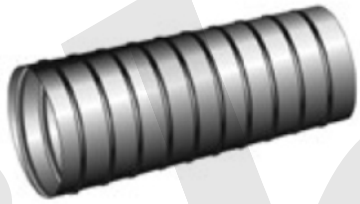


Fig. 6-24: Metal corrugated tube.

The tube diameter should be sized to allow for production and construction tolerances for placing of the tube, and then an allowance for the minimum amount of grout surrounding the bar. Tubes can then be cast either in the top or bottom of the column, or as a continuous length.

There are three main types of connection using vertical cast in tubes:

1. Tubes cast into the bottom part of the precast column: This is a similar arrangement to Fig. 6-22b. The tube length is at least an anchorage length of the connecting bar, and the top of the tube has an entry / exit point for the grout at the top of the tube. Grout could also be pumped from an inlet at the bottom of the tube with air expelled from a vent at the top. The connecting bar would project from the top of the column. The bar could either be one continuous length cast into the top of the column or attached to a coupler in the top of the column on site. The coupler solution is often favoured to make optimum use of long line production methods and for ease of storage and delivery.

2. Tubes cast into the top part of the precast column: This arrangement is illustrated in Fig. 6-25a. The connecting bars project from the bottom of the column in this case. The projecting bars can also be coupled as with the previous example.

3. Continuous tubes for full column height: Fig. 6-25b shows this configu ation. The connecting bars are placed on site. The resulting column suits production due to its simplicity, but the amount of grout required to fill the tubes may determine its commercial viability. This detail is often used as a transition from projecting bars at starter level to one of the other details with the tubes either cast into the top or the bottom of the column at the upper levels.

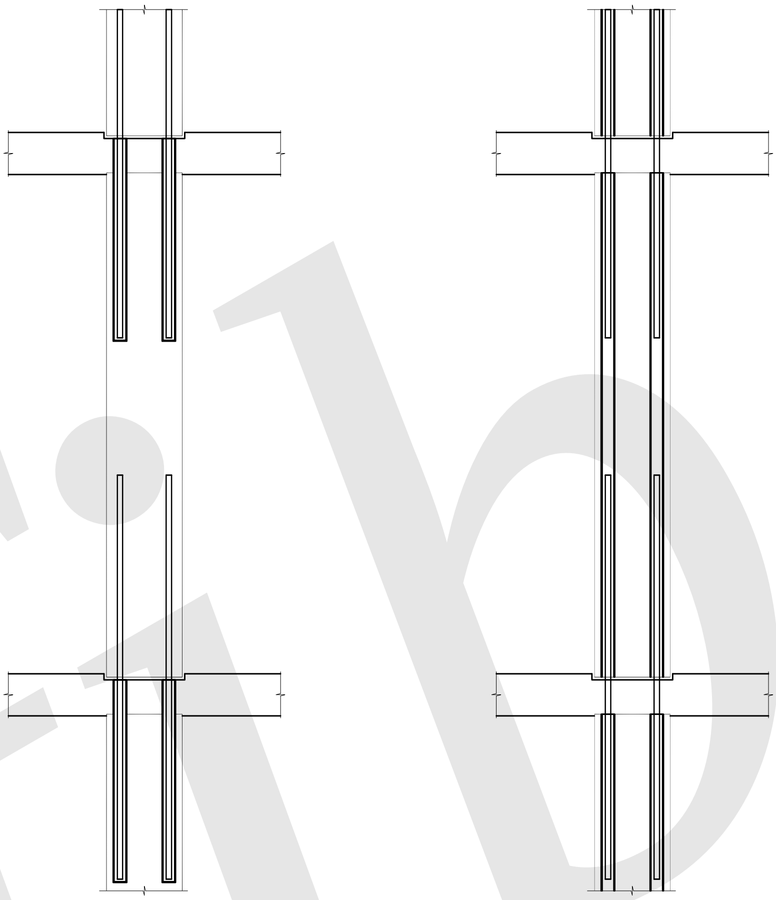


Fig. 6-25: Connection methods using cast in tubes; a) shows tubes cast into the tops of columns, b) has continuous tubes. In both cases the connecting bars lap with the main vertical reinforcement in the precast columns.

The other common way of connecting vertical reinforcement is through proprietary grouted couplers.

There is a variety of sleeves available, and they can be placed either in the top or bottom of the column, depending on the application and preference. One type of sleeve has the two connecting bars fully grouted inside it as shown in Fig. 6-26. Another option is to have a threaded connection at one end with the connecting bar grouted at the other end as shown in Fig. 6-27.

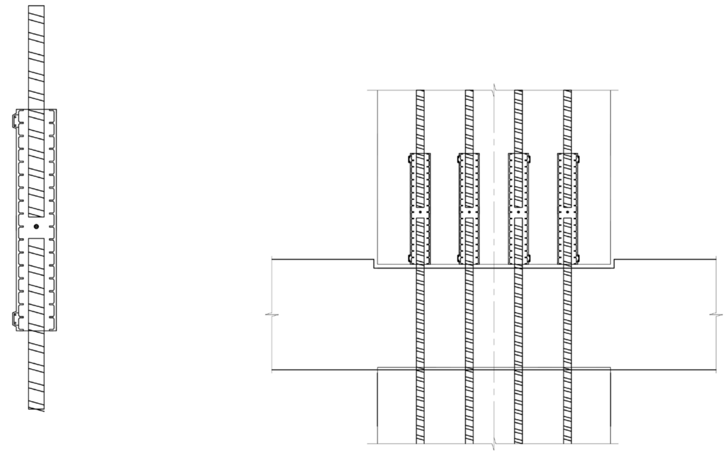


Fig. 6-26: Fully grouted sleeve, used at joint between columns and floo , in this case the coupler is cast into the bottom of the column.

The sleeve in Fig. 6-26 uses a cylindrical shaped ductile iron material and is fille with a cement based non shrink grout. Connecting reinforcement bars meet at the centre of the sleeve. In this application short connecting bars project from the top of the floo , and the column is lowered onto them. The connection is then completed by grouting the sleeve. This type of sleeve has been used extensively in earthquake zones.

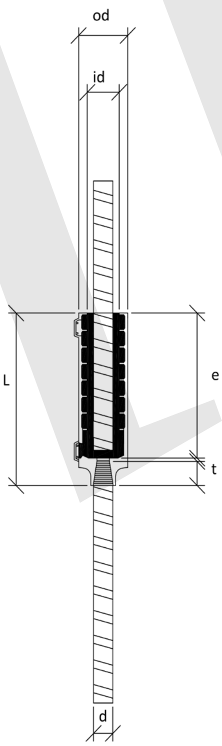


Fig. 6-27: Grouted sleeves with one end threaded.

In Fig. 6-27 the threaded length, t, would either project from the top of the column or be recessed in a pocket. The coupler length, L, would be attached after casting, either in the precast yard or at the construction site (see Fig. 6-29). Alternatively, the coupler could be cast in the bottom of the column, similar to the example in Fig. 6-26, and then installed onto projecting short bars from the column below. The coupler has allowance for pressure grouting in that case. Example dimensions for grouted couplers with one end threaded are given below for rebar diameter of 25 mm:

- *d* = rebar diameter 25 mm
- *t* = thread length 35 mm
- *L* = overall coupler length 220 mm
- *e* = rebar embedment 178 mm
- *id* = internal coupler diameter 51 mm
- *od* = outer coupler diameter 68 mm

Grouted couplers are often preferred by both producers and contractors as the length of projecting bar is relatively short, making the columns easier to handle and transport. If there is an insitu floor between the columns then the threaded end can be recessed in the column and the coupler fixed after casting.

Fig. 6-28: Short projecting bars at ends of precast columns with grouted couplers.

Fig. 6-29: Couplers fixed to top of precast column at construction site.

6.4.4 Connection of Horizontal Members

Precast beams and slabs are connected to columns through a variety of methods that can be generalised as follows:

- Type A: Concrete corbels and nibs where there is a connection between precast columns themselves;
- Type B: Concrete corbels and nibs where there is an insitu stitch connecting the beams across the column;
- Type C: Proprietary and non-proprietary steel connectors.

Fig. 6-30: Example of Type A connection at corner column.

Fig. 6-30 shows a circular corner column that supports precast beams in two orthogonal directions. Projecting concrete corbels, cast with the column, support the beams. The columns are joined together without any insitu stitching or precast layer between them. This is an example of Type A in the list above. Because there is no stitching between the columns slots have been formed in the top of each column to accommodate horizontal ties connecting the elements. This connection allows transfer of high axial compressions as there is no possible weaker insitu layer.

In Fig. 6-31 there is a view showing a number of corbel connections on an irregular building edge. Some of the supported beams arrive skew to the main axes, and careful detailing is required to ensure adequate bearing. In this example additional nibs are provided to support slabs as well as beams. In a lower rise building provision of connections of this type may appear expensive and complex, however, in taller buildings the irregular edge that is provided repeats over many floors and suits the factory methods of precast concrete.

Fig. 6-31: Example of Type A connection at irregular slab edge.

Fig. 6-32: Example of Type B connection. Insitu stitching to be provided between columns and supported beams at corner connection (Embassy 7B project, Bangalore, India, courtesy of KEF Infrastructure Pvt Ltd).

In Fig. 6-32 projecting corbels, again cast with the precast column, at a corner location support vertical loads from main beams. In this case the detail is similar to a monolithic insitu connection, with an area left clear, except for connecting reinforcement, between precast beams and columns. Once site placed reinforcement is fixed the connection is completed with insitu concrete. There are also proprietary steel connection systems (Type C) that can be cast into the precast columns, flush with the moulded face, and then have compatible fittings in the supported member.

Fig. 6-33a: Corbel

Fig. 6-33b: Beam shoe

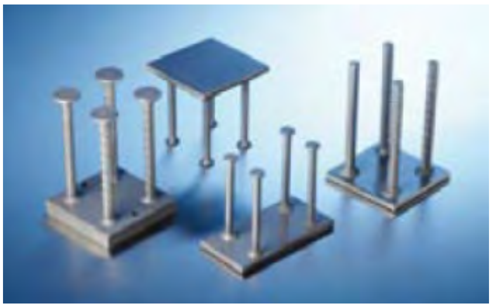


Fig. 6-33c: Fastening Plates

Fig. 6-33: a, b and c courtesy of ©Peikko Group Corporation.

Fig. 6-33a shows a steel corbel system for a precast column that is compatible with the beam shoe in Fig. 6-33b. The projecting part of the corbel is bolted to the cast in part after the precast column is demoulded. This system provides a hidden connection that may be necessary for architectural and building services requirements.

A range of steel fastening plates is shown in Fig. 6-33c. These are normally cast into the precast column for connection of steelwork, and sometimes precast elements. Connection to the supported element is generally completed by welding.

Fastening plates can also be provided with threaded sockets welded to the back of the plate so that bolted connections are also possible. The plates have either stud headed anchor bolts or reinforcing bars welded to the back of them to transfer the supported loads to the column. These plates can be obtained as proprietary items from suppliers or designed and fabricated for special cases. The larger suppliers are likely to have a wide range of fastening plates due to the many applications and loading requirements. However, where bolted steel connections are necessary then specific designs and non-proprietary fabrications may be required. Steel billets (rectangular solid or hollow sections) projecting from precast columns for connection into preformed recesses in supported precast members are commonly used. Steel connections are also possible where the connector projects from the beam and is engaged into a slot in the column as shown in Fig. 6-34.

Fig. 6-34: Beam with steel connector being installed onto precast columns.

Precast columns are also connected to insitu concrete beams and slabs. In circumstances where insitu beams connect into precast columns between floor levels, such as mezzanines, threaded couplers can be provided in the face of the column for connection to continuity reinforcement, and the interface area roughened to provide a key for insitu concrete. The finished connection is similar to a monolithic construction.

6.4.5 Non-structural connectors and fixings

In this instance, the term "non-structural" means an item that is not required to provide structural resistance as part of the primary building frame when the column is in its final position in the finished building. There is a wide range of non-structural connectors that are cast into and / or attached to precast columns. These include:

- Sockets to assist temporary stability during construction;
- Fixings for secondary items such as partitions and services;
- Lifters.

Fixing sockets (example shown in Fig. 6-35b) are often used for temporary bracing of columns before their permanent connections are complete. The sockets need to be sized and chosen to resist the design forces that can be expected in all circumstances prior to connection to an alternative stability system.

Structural design may be required for these items. In particular, lifters require careful consideration. Often different and separate lifters are required for demoulding / handling and installation. The type, number and size of lifters depend on the application, chosen work methods, load, weight and shape of element. Advice from suppliers of proprietary lifting systems should be sought when choosing the system for any large project with many columns. There are many types of lifter, a selection is shown in Fig. 6-35a. Often additional local reinforcement is advised by suppliers at lifter locations.

Fig. 6-35a: Various lifters.

Fig. 6-35b: Fixing socket example.

7. Precast Concrete Walls

7.1 Introduction

Walls are essential components of tall buildings as separators of circulation and service areas (cores and shafts) from the main floor areas and also as dividers of residences. Their application in residential buildings (apartments and hotels) is obvious for compartmentation of individual rooms or dwellings, but they are also applicable in open plan office applications where fire separation of vertical circulatory routes from the rest of the building is needed as part of fire strategy.

Walls can have many functions in a tall building. Some examples of the areas they can be used in are:

- Stair cores;
- Lift shafts;
- Service risers;
- Boundaries;
- Structural shear walls;
- Transfer structure;
- Non-structural partitions;
- Fire walls;
- Acoustic barriers;
- Façade support (see Chapter 9 for specific application to facades);
- Thermal insulators: sandwich panels (see also Chapter 9).

As walls are needed for the building to function then it makes sense to use them as part of the structural framework. Concrete is already acknowledged as having many important benefits for tall buildings, such as strength, fire resistance, durability, low maintenance, and good thermal and sound insulation properties. However, precast concrete also brings added benefits, including; budget, quality and time certainties, and less congestion on small sites.

Both columns and walls are vertical elements that generally transfer gravity loads, and in some cases lateral forces, through the building to the foundations. The difference between them relates to the ratio of the dimensions of the cross section.Walls are defined in Eurocodes as having a length to thickness ratio of 4 or more (value of 3 advised by ACI). Whilst the general design principles for walls and columns may be similar in many aspects there are also likely to be differences in design approach and details.

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Walls, through their section aspect ratio, have much greater in plane strength compared to out of plane strength. In the simple example in Fig. 7-1 the in plane flexu al stiffness is 64 times greater than the out of plane stiffness when the wall has a ratio of horizontal length to thickness of 8. Therefore, structural walls need to be braced, generally by other walls and floors, perpendicular to the plane of the wall being considered in order to reduce their susceptibility to out of plane or secondary effects that may arise due to gravity loading. Conversely, the respective stiffnesses of a column about its two main axes are comparitively closer. Unless a column has a very large section size, and consequently a large relative stiffness, it is unlikely that a significant lateral force will be attracted to it if the building is braced by walls. Therefore, in a tall building, walls required for other functional reasons can also be used as the primary means for providing lateral stability at a relatively small additional cost.

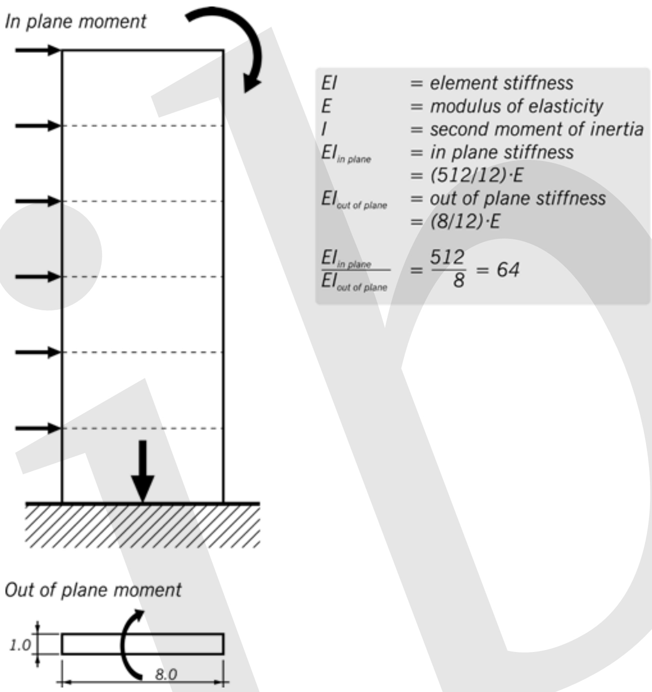

Fig. 7-1: Example of comparable wall stiffnesses for in plane and out of plane bending.

7.2 Design and Detailing

7.2.1 General

In tall buildings walls often provide the vertical bracing elements in the structural system (shear walls). Often the vertical circulation area comprising the stair cores, shafts and other risers are located in close proximity (see Fig. 2-2 from Chapter 2) and the layout may suit an insitu slipformed solution where rapid progress is possible. However, when the stabilising walls are more evenly dispersed around the building, and when the height of the building is not prohibitive, then precast walling could be more attractive.

The design of individual wall elements is similar in many respects to the design of insitu elements, however, it is the connections between the precast elements, and with other materials, that require a different approach.

Both horizontal and vertical connections need to be carefully considered so that the connection solutions chosen mirror the assumptions in the structural analysis and have sufficient capacity to withstand the design forces. In addition, the connection detail should have minimum impact on factory production (no changes to industrialised methodology) and be easily installed on site. Often details that are commonly used for insitu construction are not feasible in precast. For instance, vertical reinforcement is provided in two faces at floor joints for insitu construction, however, for relatively narrow walls, say 250mm thick and less, where the connection uses couplers or cast in tubes then there may not be enough space to incorporate a double layered reinforcement connection and a single central row of connectors may have to be used, then possibly returning to a conventional reinforcement arrangement inside the panel itself.

Walls can act as lateral stability elements if they are either continuous from their topmost level to the base level, or to an intermediate level that is suitably designed for redistribution of the horizontal forces to other lateral stability elements. There are, therefore, other situations where walls act as elements not providing lateral stability but still have to transmit gravity loads and provide fire resistance plus an acoustic barrier. In those circumstances walls may be predominantly subjected to out of plane bending and detailing rules for slabs may be applicable. Attention to detail at connections is always important to ensure structural integrity and the completeness of fire and acoustic sealing between rooms and / or compartments.

7.2.2 Structural shear walls

Shear walls are typically designed as vertical cantilevers, either as separate walls (for instance as crosswall or spinewall systems) or as integral cores / shafts. Horizontal forces are resisted by an axial diagonal force in the wall bracing as illustrated in Fig. 7-2.

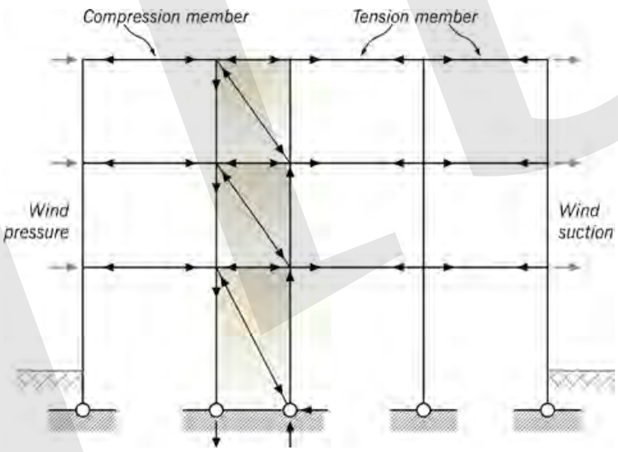

Fig. 7-2: Transfer of horizontal forces to wall bracing.

The distribution of horizontal forces to precast shear walls is calculated in a similar manner to that for other walling systems. The force is transferred through the floo diaphragm and distributed in proportion to the relative wall stiffnesses and their locations in the structure. In situations where the shear centre of the wall system is coincident with the line of the resultant of the applied horizontal forces in two orthogonal directions then the structure will only be subject to in plane deflection, also referred to as translation, i.e. there are no rotational effects.

The shear centre, also known as the global centre of stiffness, is the point through which a transverse force will develop normal shear without torsion, and therefore results in maximum structural efficien y in resisting lateral loading. At concept stage the wall layout and sizes should be developed so that the applied forces have the minimum possible eccentricity in relation to the shear centre. The position of the shear centre about one axis relative to an origin is given by:

$$\overline{X} = \frac{\sum E_i \cdot I_i \cdot x_i}{\sum E_i \cdot I_i} \tag{7-1}$$

Where $I = tL^3/12$ the wall has thickness t and length L in a simple rectangular wall.

The full plan section of precast cores and shafts can be used in the structural analysis provided there is sufficient resistance in the resulting box or other multi-walled shape (viewed in plan). When using simple plane wall panels completing joints between panels at corners in shafts and cores can often be problematic due to high force concentrations. In those cases it may be advantageous to form the corners as part of the wall element. An example of a panel with integrated corners is shown in Fig. 7-3.

Fig. 7-3: Precast element forming part of composite wall for integrated lift shaft and stair core. Corners with adjoining walls are provided in the precast element with projecting reinforcement for connection to other structure.

If the line of the applied force is not coincident with the shear centre then the structure will be subject to both translation and rotations. The centre of rotation is at the shear centre, where the deflection due to rotation is zero. Fig. 7-4a and 7-4b illustrate these principles.

In Fig. 7-4a the walls are sized and positioned so that the shear centre is coincident with the centre of the applied force, and consequently there are only translation effects. In 7-4b the shear centre is shifted further towards the left hand side due to a reduction in the length of Walls 2 and 3, resulting in the development of rotational effects and a resulting increase in the forces to be resisted by Wall 1.

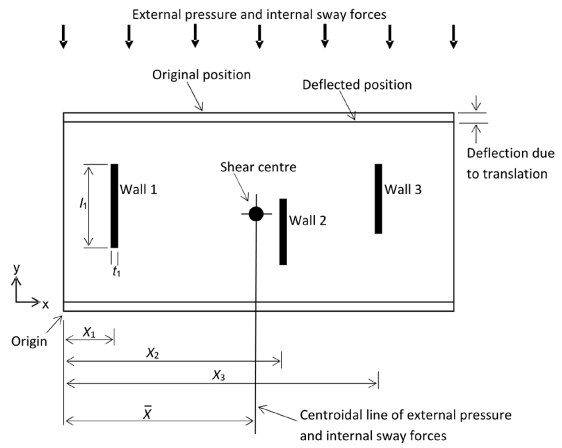

Fig. 7-4a: Balanced Shear Wall System, no rotation

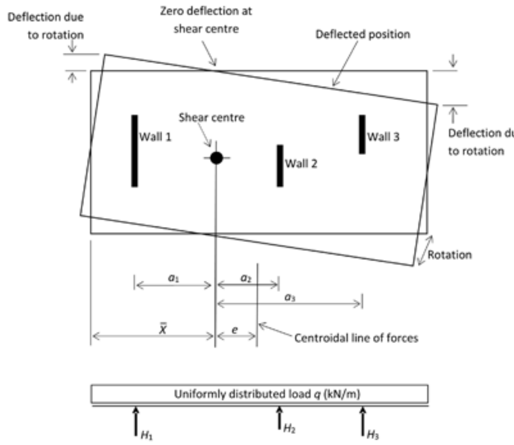

Fig. 7-4b: Assymetrical Shear Wall System with rotational effects

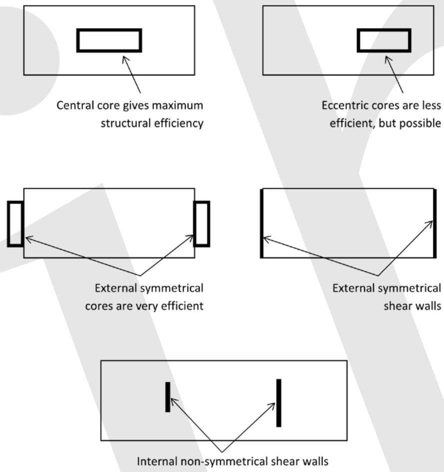

Fig. 7-5: Examples of shear wall and core locations.

Fig. 7-5 shows some examples of how the positioning of shear walls or cores can affect structural efficien y, i.e the location of the shear centre in relation to the point of application of the applied force (generally at the midpoint along the elevation in the case of wind pressures). For simplicity, stability in one direction only is considered with the external lateral force applied perpendicular to the longer side.

In the first two examples, (a) and (b), a single core is positioned in the building. In the first case (a) it is central, and consequently coincident with the applied lateral force. There will be no torsional effects and therefore there is maximum structural efficien y. In the second case (b) the core is shifted to the right and is eccentric to the applied force, resulting in the development of torsion.

The examples (c) and (d) have external symmetrical cores and walls respectively, that result in a central location for the shear centre, again achieving maximum efficien y with no torsional effects.

In the final example (e) there are two shear walls inside the building but of different lengths, and therefore stiffnesses. The shear centre will move away from the centroid of the building and towards the stiffer wall creating an eccentricity between the shear centre and the applied force. If the centre of the applied force is at the midlength of the building then additional reactions will result at the longer wall due to torsion.

If there are no rotational effects, i.e. the lateral force is applied at the shear centre, then:

$$\frac{H_n}{H} = \frac{E_n \cdot I_n}{\sum E_i \cdot I_i} \tag{7-2}$$

Where H_n is the reaction in an individual wall and H is the total applied force. In this case the sum of all the reactions in the individual shear walls will be equal to H. However, if the applied force is eccentric to the shear centre then additional effects are generated that are proportional to the eccentricity.

The eccentricity, e, of the applied force from the shear centre can give an increased element reaction as follows (the negative, or favourable, effects of rotation in Eq. 7-3 are often ignored and only the translational component used):

$$\frac{H_n}{H} = \frac{E_n \cdot I_n}{\sum E_i \cdot I_i} \pm \frac{e \cdot E_n \cdot I_n \cdot a_n}{\sum E_i \cdot I_i \cdot a_i^2} \tag{7-3}$$

Where a is the perpendicular distance from the shear centre to the centre line of the individual wall.

The ± sign in Eq. 7-3 means that the rotational component is additive if the shear wall is positioned on the opposite side of the shear centre to the applied force, i.e $x_i < \overline{X}$ for the example shown in Fig. 7-4a. Also, the term $\Sigma E_i \cdot I_i \cdot a_i^2$ is known as the torsional rigidity and comprises the rigidities from all stabilising walls.

The reaction H_n can be calculated for each individual wall.The design moments and horizontal shears along the height of each wall can then be calculated.

Torsional effects in non symmetrical systems may be balanced by walls perpendicular to the applied force as shown in Fig. 7-6. This arrangement may arise where the front of the building is open or glazed. At least three walls are required, with at least two of them, often called "balancing walls" at right angles to the applied force. If the shear centre is taken at the centre of the single wall parallel to the force, H, then:

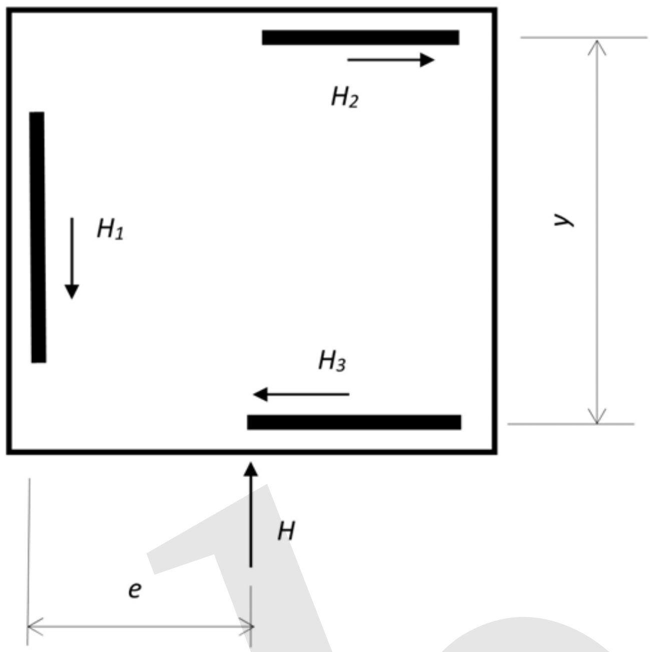

Fig. 7-6: Simple example of torsional effects in non symmetrical shear wall systems

For section design where the wall is subject to axial loads, and both in plane and out of plane moments, then the distribution of loading along the length of the wall due to in plane bending moments and axial loads should be firstly calculated. This will result in a maximum compressive load, and, possibly, a maximum uplift tensile load per unit length of wall, as illustrated in Fig. 7-7 for load combination giving critical uplift case. These values should then be used with any transverse moment to calculate the reinforcement area by treating a unit length of wall as a column subject to the calculated axial loads from the distribution and combined with the transverse moment in a similar manner to that described in Section 6.2.3 previously.

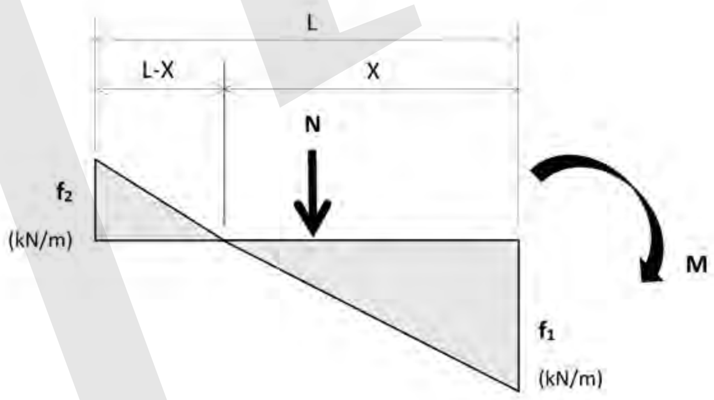

Fig. 7-7: Load distribution for combined axial load and in plane bending for critical uplift case when tension forces are developed.

$f_1 = N / L + 6M / L^2$ Maximum reaction in kN / m

$f_2 = N / L - 6M / L^2$ Minimum reaction in kN / m

N = Design axial load in kN

L = Length of wall in metres

M = Design moment in kNm

$X = f_1 \cdot L / (f_1 + f_2)$ Length of compressed zone, tension resisted over length L-X

$T = 0.5f_2 \ (L$-$X)$ Total tensile reaction developed

$C = N + T$ Total compressive reaction developed

When a cantilever shear wall is fully braced by other walls perpendicular to its main axis then transverse, or out of plane, moments in that cantilever shear wall should only be typically generated by one or more of; eccentricity of an applied axial load from the supported structure above, through framing effects of the supported horizontal members at the floor level under consideration or through secondary effects arising through slenderness of the wall section.

When considering secondary effects each floor to floor storey height should be assessed for slenderness. Lateral restraints provided by floors, perpendicular walls and integral columns have to be taken into account when calculating the effective height of the wall. Guidance is often given in National Codes in relation to the relative contribution towards the effective wall height of each type and / or combination of restraint (for instance Table 12.1 of EN1992-1-1[5]).

In cases, such as internal walls with a symmetrical arrangement of supported slabs, where the transverse moment is insignificant (eccentricity is no greater than nominal) then a simpler analysis of in plane bending combined with axial force only can be considered.

7.2.3 Types of precast structural walls

There are two main types of precast concrete structural walls. These are the solid wall and the twin wall (sometimes called double wall) types.

Solid walls are completely precast, normally in storey heights, and typically have cast in reinforcement or proprietary fittings along their edges for connection to adjoining elements. An example of a solid precast wall being installed inside an insitu core is shown in Fig. 7-8.

Fig. 7-8: Installation of a solid wall inside an insitu core.

Solid walls minimise on site work and can incorporate many other fixtures and fittings, such as windows, doors, service conduit etc. However, their use can be limited by the capacity of site cranage, particularly for thicker sections.

Twin walls have two skins (also referred to as wythes or biscuits) of precast concrete that are connected by steel trusses that hold the precast skins apart at a constant spacing to form a composite wall (the gap between the skins is filled with insitu concrete) of constant thickness. Twin walls rarely have cast in reinforcement or proprietary fitting for later connection to adjoining elements. Connections are generally achieved via site placed reinforcement in the insitu infills that cross the joints with other elements. A typical twin wall panel is illustrated in Fig. 7-9.

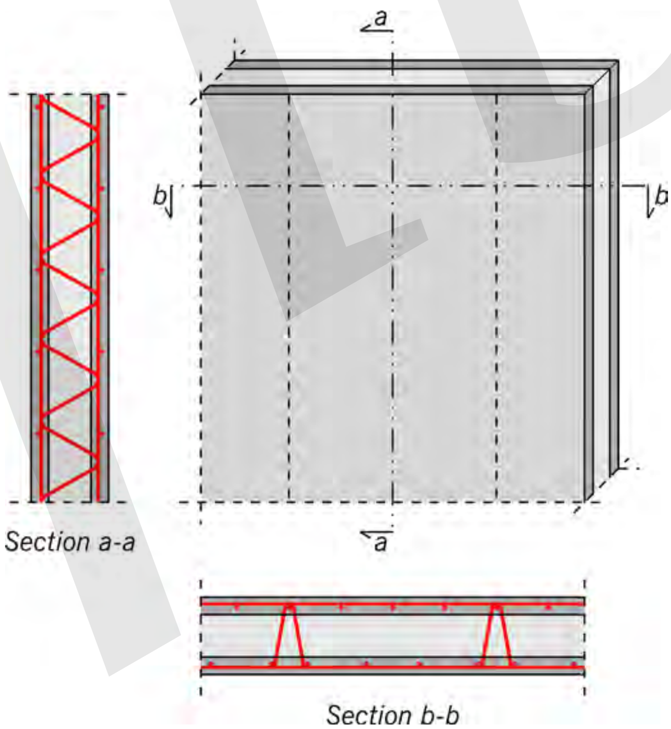

Fig. 7-9: Illustration of typical twin wall panel.

The main reinforcement for the composite wall is typically in the precast skins. The trusses provide a shear connection across the composite interfaces, and also have to withstand the temporary wet pressures from the infill site concrete. At base and floo levels site fixed vertical starters or connecting bars are provided that lap with the skin reinforcement in the composite section to achieve continuity.

Twin walls have a number of advantages. Whilst not fully precast they may overcome the problem of limited crane capacity for thick wall sections. The precast skins are relatively thin, normally about 60 mm each. Therefore, the weight to be lifted would only be the equivalent of 120 mm thick concrete that would then form a wall of much greater section thickness once infilled with site concrete . The infill can also be used to provide monolithic connections with adjoining insitu concrete elements and to accommodate service conduit. Holes in the precast skins can be provided for electric sockets and xings.

The twin wall system is often used with horizontal precast concrete shuttering slabs (also referred to as filigree or biscuit slabs). The combination of this wall and slab system minimises site formwork and results in monolithic connections. Both walls and slabs can be produced in the same factory using the same equipment.

The main drawback to the twin wall system is that the solution is not fully precast and wet trades are still evident. Therefore, the full benefits of precast are not realised.

7.2.4 Structural connections

7.2.4.1 General

Walls differ from columns in that they often require vertical joints and connections in addition to horizontal ones. This is of particular relevance at corners of cores and shafts if flat precast panels are being provided and a coupled wall solution is required. In those cases the connection has to be capable of resisting the design combinations of shear forces that can be developed.

The shear resistance, V_{Rdi}, at the interface (joint) between concrete cast at different times (insitu onto precast) is generally governed by the following equation and has three constituents:

$$V_{Rdi} = A + B + C \tag{7-4}$$

A is related to the tensile strength of the concrete and the roughness of the interface. In Eurocode 2[5] it is represented by $c \cdot f_{ctd}$, where c is a factor that varies between 0.025 for very smooth surfaces and 0.500 for indented surfaces, and f_{ctd}, is the design tensile strength of the concrete.

B relates to the frictional stress caused by the minimum external force applied perpendicular to the interface, σ_n, that can act at the same time as the shear force. If σ_n is tensile then *A* is taken as zero and *B* is negative. In Eurocode 2, *B* is given by $\mu \cdot \sigma_n$, where μ is a factor also related to the roughness of the interface and varies between 0.5 for very smooth surfaces and 0.9 for indented surfaces.

C is the contribution from the reinforcement crossing the interface, with adequate anchorage both sides of the interface. *C* is calculated in Eurocode 2 from $\rho \cdot f_{yd} \cdot \mu$, where ρ is the ratio of the area of reinforcement crossing the interface to the area of the interface itself, and f_{yd} is the design strength of the reinforcement.

For horizontal wall joints *A*, *B* and *C* typically apply over the compressed length of wall (length *X* in Fig. 7-7). In the tension zone (length *L-X* in Fig. 7-7) *A* would be zero and *B* negative. In vertical joints it is unlikely that there will be a contribution from *B*.

For horizontal joints in precast walls, both solid and twinwall, it is generally preferable to have a single central row of connecting vertical bars for wall thicknesses of 250 mm and less. This results from the reduced space for the connection in solid wall arising through the requirement for either cast in tubes or couplers, and in twin wall through the relatively narrow infill concrete width. This is different to conventional insitu walls that have a row of bars each side. Transverse bending should be checked to ensure that a single row solution is feasible.

7.2.4.2 Solid wall

Connections are typically achieved through projecting reinforcement or proprietary fixings around the edges of the solid wall and completed with relatively small amounts of grout or insitu concrete. Systems have been used that have vertical dry joints between walls and rely on a staggered or “masonry” jointing system for shear resistance (refer also to Chapter 4). Welded connections have also been used to connect panels, but have a number of drawbacks, particularly in relation to verification of the strength of site welding, health and safety issues and the introduction of another skilled trade.

There are a number of relatively simple proprietory connecting systems for vertical joints that use rope boxes as shown opposite in Fig. 7-10. Profiled concrete surfaces form the joint and house the metal rope boxes, which are placed flat against the former. Once removed from the mould the looped ropes are released, and overlap with the corresponding loops from the adjoining wall. A connecting vertical bar is then placed through the overlap, and the joint then either concreted or grouted to complete. In plane tension forces, and both in plane and out of plane shears can be resisted by this system. Capacities that have been verified by test can be obtained from suppliers. However, there may be limitations on the uses of rope boxes in certain design circumstances, particularly in relation to dynamic effects. Guidance should be sought from suppliers regarding their suitability for the design application being considered.

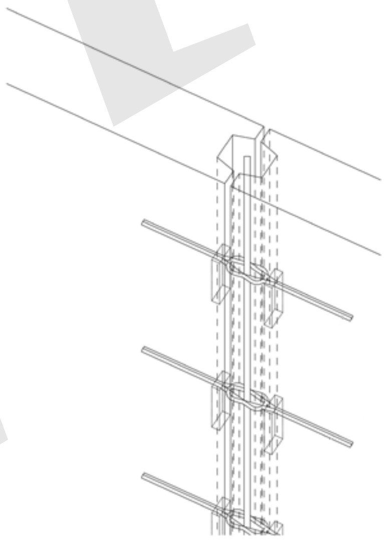

Fig. 7-10: Profiled vertical wall joint incorporating wire rope boxes.

Fig. 7-11 shows a typical rope box. The part of the rope that can be seen in the figure is cast in the concrete wall. The looped rope is contained in the box by a thin cover plate. Fig. 7-12 has completed walls that are stacked in the storage yard with the looped ropes now exposed.

This system has a number of practical advantages for both producer and installer. It is relatively easy to install both in the mould and inside the wall reinforcement cage. On site only a narrow strip of former is required to close the joint and a single reinforcing bar to be fixed per joint . The wet trades are limited to the filling of the joint with concrete or grout, choice of material is dependant on the depth of the joint.

Fig. 7-11: Typical wire rope box.

In instances where forces across the joint are insignificant a similar concrete profile is often still used to satisfactorily complete the joint with concrete or grout, and provide resistance to fire, sound and weather.

Fig. 7-12: Precast wall panels with profiled edge in stock yard.

Whilst the speed, and potential cost saving, advantages of this relatively simple system are obvious the disadvantage to the designer is that the structural capacity of the joint is limited. As the design joint forces increase beyond the capacity of the rope box system then stronger structural solutions have to be developed whilst trying to maintain as far as possible any time and cost advantages.

Once the limit of the rope box is reached then the next step is to replace with rigid reinforcement as the connector. This can be achieved in a number of ways but generally requires at least one side of the joint to be site shuttered. Fig. 7-13 indicates various insitu concrete stitched connections between flat precast wall panels that form a stair core. The vertical connections between back panel A-001 and side panels A-002 and A-004 have used lapping reinforcement inside an insitu stitch that is site shuttered on one side of the wall and has a narrower precast section on the other side. The corner connection between A-003 and A-004 has interlocking U bar reinforcement that covers the common area of the corner only. A-002 and A-003 have have a small vertical interface at the door, where the door head is detailed as a beam, and the end vertical U bars enclose continuity reinforcement from A-003 into the adjoining floor structure.

This example also illustrates the horizontal jointing system used for this core. All walls have a central row of connecting reinforcing bars along the horizontal joints. Wall A-001 has cast in metal ribbed tubes for grouting of connecting vertical bars, whereas the other three walls have coupled connections. Note the tongue and groove connection along top of walls A-002 and A-004 where there is no supported floor on at least one side. The groove is used to contain the bedding mortar during installation.

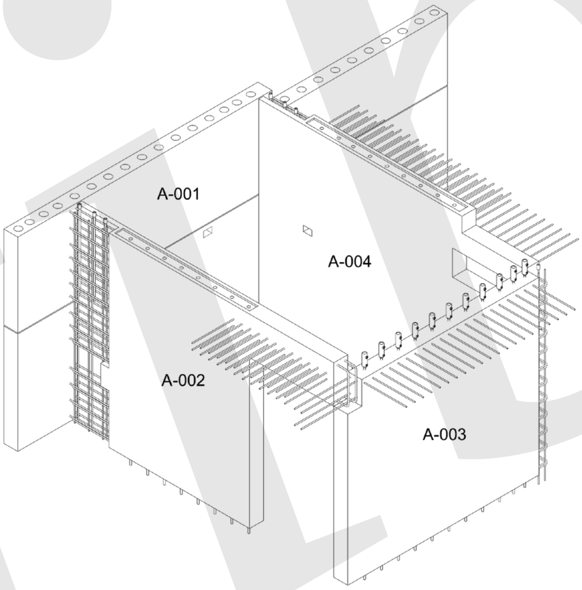

Fig. 7-13: Example of various horizontal and vertical connections between core walls.

A detail that is closer to the rope box system but uses bar reinforcement is shown in Fig. 7-14. An indented surface is formed in the precast wall edge on both sides of the joint. These joints are commonly known as keyed joints and Codes, such as Eurocode 2[5], allow shear strength increases for this type of joint compared to a surface that is only roughened. They have projecting and overlapping U bars with a continuous vertical bar, also known as the locking bar, that is placed inside the U bars to assist the tension transfer between the U bars. This detail is similar, but relatively stronger than the rope box solution.

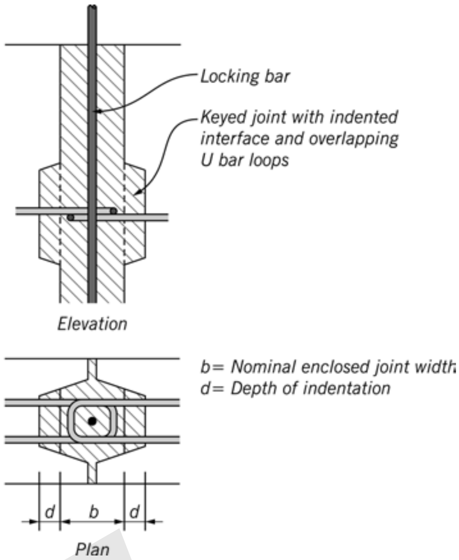

Fig. 7-14: Elevation and plan on profiled vertical keyed joint with overlapping projecting U bars.

Vertical reinforcement through horizontal joints between solid wall panels is generally connected using similar details to those previously described for columns in Chapter 6. These are full length bars grouted into cast in tubes, both mechanical and grouted couplers, and proprietory fixings, such as column shoes.

Fig. 7-15: Stored precast wall panels with mixture of top connectors.

In Fig. 7-15 there are a number of wall panels stored at the precast yard that have a mix of top connectors. Some have cast in tubes that will receive full length site fixe bars whilst others have couplers already fixed. The choice of connector is subject to a number of factors; application, price, speed, availability of suitable labour and materials. The preferred connector system can therefore vary from site to site, and even across the same site.

In tall buildings it is likely that vertical joints will require a relatively larger input from wet trades compared to horizontal joints due to the magnitude of design joint forces and the absence of the beneficial effects of normal compression (constituent *B* in Eq. 7-4). It is likely, therefore, particularly at lower levels, that insitu stitching of vertical joints will be required. To avoid the extra site work and time associated with wet trades, the precast wall elements are typically detailed to minimise vertical joints, with a likely corresponding increase in horizontal joints (the number of joints, and consequent panel size, is generally dictated by the site crane capacity). Liftshafts, service risers and isolated shear walls are often detailed as complete horizontal components with extra horizontal joints between floors. There is an example of this type of solution in Fig. 3-8 (Chapter 3). Vertical post tensioning can also be used to strengthen horizontal joints through enhancement of normal compression forces. Post tensioning of precast wall panels has added benefits in seismic areas (see also Chapter 10).

7.2.4.3 Twin wall

Connections between twin wall panels and other structural members are similar to those used in insitu construction, and proprietory fixings are rarely used.

Fig. 7-16 illustrates an intersection between twin wall panels and a floor slab. The reinforcement cast with the precast element is shown in red, the site fixed reinforcement is shown in blue. Careful detailing and fixing is necessary to ensure that there is no clash between the site fixed bars and the steel lattice connecting the two precast skins in the next wall to be installed over the connecting vertical bars.

A double row of vertical bars is provided across the horizontal joint between twin wall panels in this case. For narrower twin wall panels a single central row may only be possible due to reduced space available between precast wythes.

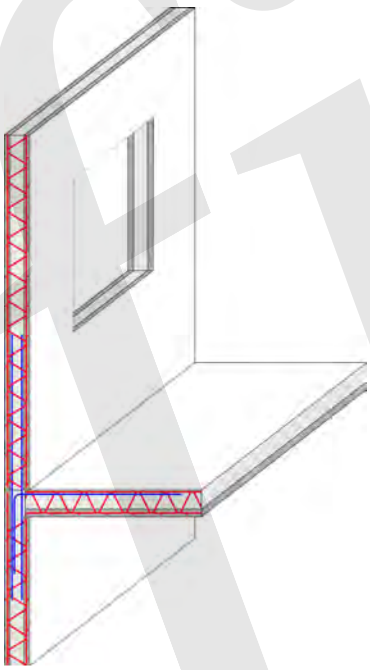

Fig. 7-16: Intersection between twin wall panels and composite floor slab (connecting site fixe bars coloured blue).

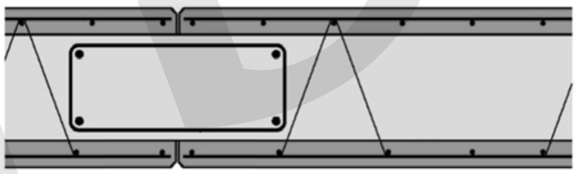

Fig. 7-17: Possible joint detail between adjoining twin wall panels with prefabricated rebar cage at joint connection.

Fig. 7-17 shows a possible simple connection between adjoining panels. A prefabricated reinforcement cage is prepared and is installed once the walls are in place. Again both detailing and the installation method need to take account of the proximity of the connecting steel lattice to the joint. Connection options using prefabricated cages at other intersections are shown in Fig. 7-18 and 7-19. Loose bars can also be used but may be more time consuming to install.

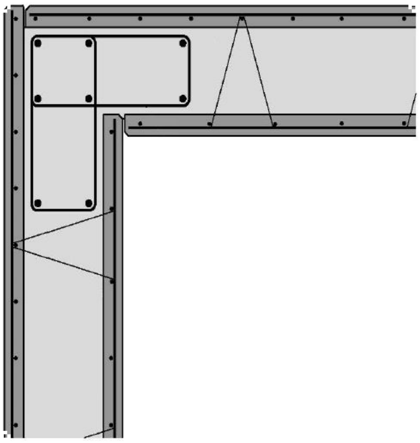

Fig. 7-18: Corner joint example with prefabricated rebar cage connecting panels.

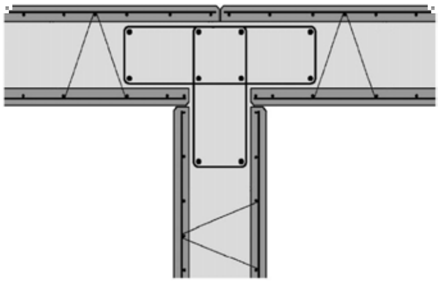

Fig. 7-19: "T" joint (three panel connection) example with prefabricated rebar cage connecting panels.

7.3 Methods of Production

7.3.1 General

Walls that are produced as flat, grey panels lend themselves to automation and can be mass produced. Where there is a visual aspect to the wall, particularly on the building exterior, then the production processes can become more specialised and skilled. Panels located in the external building façade and / or used as cladding are covered in Chapter 9, and often use different methods of production in comparison to internal walls. Even internal walls with projecting nibs, corbels, or reinforcement (see Fig. 7-3 as an example) may not be suited to automated processes. For the optimum utilisation of available precast systems in a region, and for best commercial and scheduling results, the designer should consult the local producer for limitations imposed by the system. For instance, if the production machinery incorporates magnet integrated continuous side formers then reinforcement projecting from the concrete edges needs to be avoided and replaced by an alternative detail such as rope or pull out bar boxes, shoes, couplers or cast in tubing.

Solid walls may be preferred to twin wall in tall building construction as there are likely to be fewer site activities in the construction process. However, site crane limitations may make twin wall a more attractive proposition due to its lighter unit weight. Solid wall production methods can vary from simple horizontal stationary casting tables to fully automated production. In the case of twin wall some level of mechanisation is required. Fig. 7-20 shows twin wall panels for a relatively thick wall of about 500 mm. The solid wall alternative would weigh significantly more than the twin wall. For instance, a precast twinwall panel weighs about 0.325 tonnes per m^2 for most overall wall thicknesses. A solid wall of 500 mm would weigh 1.250 tonnes per m^2. If, say, the crane capacity is limited to 8 tonnes then 24.6 m^2 of twin wall can be lifted each time compared to 6.4 m^2 of solid wall. The number of crane lifts per floor cycle would therefore increase considerably for the solid wall version in this example.

Most wall panels in tall buildings are cast as storey height elements. This means that the panel width as cast is close to 3 metres, which in turn means that for panels, with both length and width of 3 metres or more, the transported width is outside normal parameters for public highways if transported flat. Therefore, to overcome this difficult , and also to avoid having to turn the panel upright on site, the panels are often tilted vertically in the factory to suit their permanent position, and both stored and transported in purpose made frames.

Fig. 7-20: Horizontally stored twin wall panels.

The most cost effective way to manage the transport and logisitics of precast walling is through using the principle of "handling twice". The first handling is when the wall panel is moved from production and placed vertically into a purpose made frame or rack. The second handling is then into the panel's permanent position at the building site. This method limits the risk of damage and avoids unnecessary use of resources in the stockyard. In larger production plants "just in time" processes can be used that further minimise storage requirements. The number of purpose made frames required needs to be determined to meet production and delivery commitments. Fig. 7-21 shows a number of panels in steel framed platforms in the stockyard ready for delivery. Fig. 7-22 then shows a frame with its panels being loaded onto a delivery vehicle. The panels themselves will only have been handled twice once in place in the building as it is the steel rack that is handled once the panel is moved from production until it is installed in the building.

Fig. 7-21: Vertically stored wall panels on steel framed platforms.

Fig. 7-22: Wall panels in steel racks being loaded onto a delivery vehicle (courtesy of Progress Group).

7.3.2 Solid wall

There are several methods for producing solid walls that can be summarised as follows (in order of increasing automation and decreasing labour costs):

- Simple horizontal static casting tables;
- Vertical battery moulds;
- Tilting tables;
- Carousel system.

Horizontal static casting tables with minimal automation is the simplest way of producing wall panels. Relatively little financial investment is required, however labour costs can be high. The panel would be lifted as a flat panel from the table after casting and is likely to require extra reinforcement to resist temporary handling stresses due to bending about its weaker axis. Tilting would then be carried out in the storage yard or on site, with the potential for handling damage. One advantage is that complicated section shapes and details can be accommodated (see Fig. 7-3 as example). A moulded fair faced surface can only be produced on one face, with a manually floated finish on the other. This method is therefore not likely to be suitable in circumstances where a fair face is required on both sides of the wall.

To achieve a fair finish on both sides of the wall panel, and to avoid a tilting operation, vertical moulds can be used. However, individual vertical moulds are not normally cost effective. A more efficient process is to group the moulds together in a battery that is clamped by bulkheads. Fig. 7-23 shows a battery mould process from casting preparation through to demoulding. In the final frame one of the cast wall panels is being removed, and another cast panel is alongside awaiting removal.

Tilting tables are a mechanised version of the horizontal static casting table. The panels are cast horizontally and then tilted vertically. The elements can then be removed and stored upright. This can reduce the reinforcement content as sometimes horizontal handling can be the critical design case. Fig. 7-24 is inside a factory showing tilting tables partly prepared for casting.

The level of automation can be maximised when using a carousel (sometimes called circulation) system. This system can produce directly from a BIM model without any human interference, resulting in reduced risk of human error. The perception that precast concrete is more cost effective with increased shape repetition has little relevance to this method, allowing greater shape creativity for little extra cost. The concrete panel circulates around the factory and stops at several stations where a specific activity is carried out. The process is similar to that in the motor car industry, and a central master computer monitors and controls the entire carousel plant.


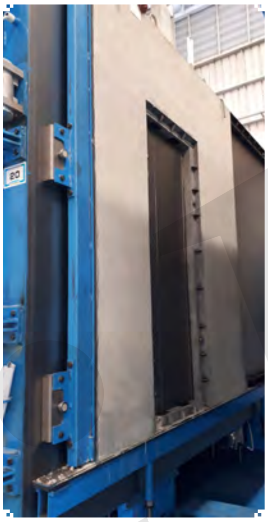


Fig. 7-23: : Battery mould process from preparation through to demoulding.

Fig. 7-24: Line of tilting tables with shuttering prepared.

Fig. 7-25: Circulation factory with tilting tables at end of process and panels stored in racks.

The main stages in the automated carousel process are:

1. Pallet cleaning and release agent spraying using automatic cleaning, brushing, and spraying equipment.

2. Placing of shuttering using robots (see Fig. 7-27 and 7-28) and integrated magnets with dimensions and setting out provided by the master computer. As an alternative to the shuttering robot a plotter or laser can be used to set out the panel outline on the pallet surface.

3. Reinforcing robots provide customised meshes with wires to suit various combinations of diameters and spacings with block outs if required for doors and windows. Bent mesh can also be provided (see Fig. 7-29) through automation to suit edge reinforcement details.

4. At the casting station a mechanised concrete distributor delivers the fresh concrete to the shuttered and reinforced pallets.

5. The concrete is then compacted using either low frequency horizontal oscillators or high frequency vibrating equipment.

6. After casting the fresh concrete, the top surface is smoothed using a vibrating levelling beam. Once there is initial hardening a finishing machine is used to provide a smooth, ready to paint concrete surface.

7. The freshly poured pallets are then stacked horizontally in a rack system. A computer-controlled stacker places the pallets above each other to optimise factory floor space.

8. The panels are removed from the rack when cured. The shuttering is partially released, and the panels then mechanically tilted for removal to the storage yard.

9. A stripping robot then fully removes the shuttering and returns the pallet to the start of the process.

Due to the high degree of automation used in the carousel process the amount of labour required is minimised. A high-quality product is possible, at mass produced rates of production. The stages described above are illustrated in Fig 7-26.

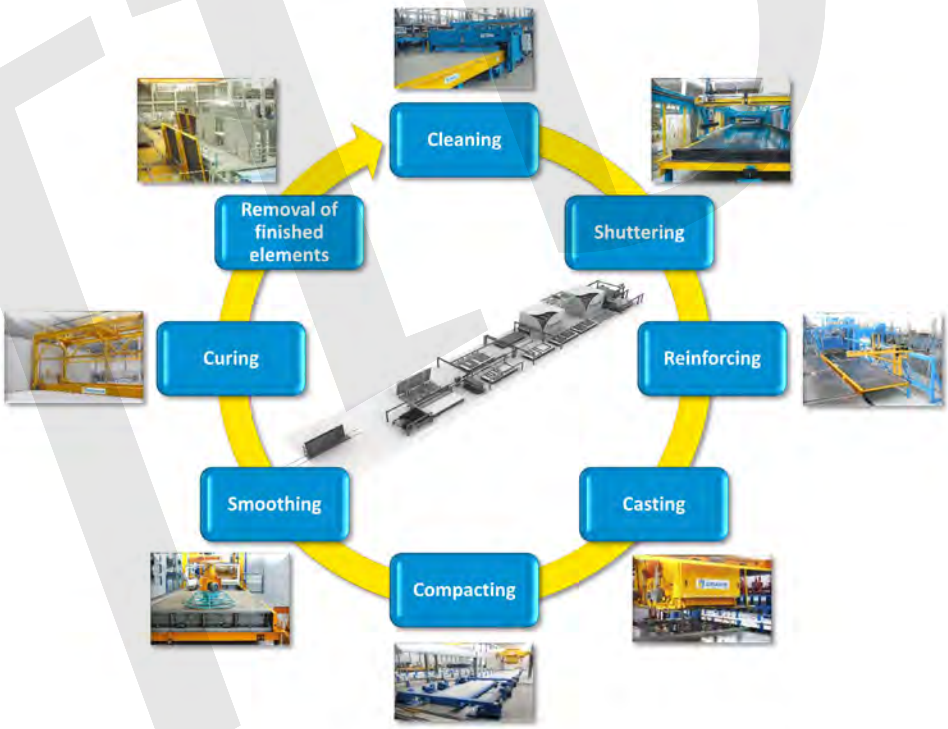

Fig. 7-26: Key stages in the carousel process (courtesy of Progress Group).

Fig. 7-27: Shuttering robot (courtesy of Progress Group).

Fig. 7-28: Robot placing shuttering at required locations.

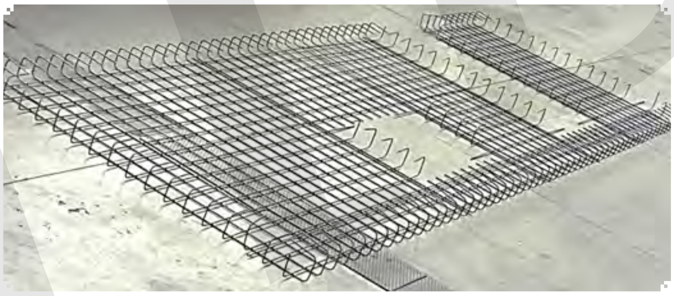

Fig. 7-29: Reinforcing mesh prepared using robot with edge bent bars to suit reinforcement details.

7.3.3 Twin wall

Twin wall is a hybrid solution that results in a composite wall section. It is generally used when economic solid wall sections are too heavy to lift on site, where monolithic connections are preferred, or when a moulded finish is required on both wall faces.

At least some mechanisation is normally required in the manufacturing process as firstly a filigree slab is produced as the initial concrete skin, and then once hardened, is mechanically turned through 180 degress and placed into a second skin of fresh concrete.

The further levels of automation that are available are dependant on the producer, but a fully automated and robotic carousel plant, similar to that described for solid wall is possible. Fig. 7-30 is an example of twin wall production, with the turning machine in the background and the initial filigree skin in production. This factory can give a complete slab and wall building solution (albeit insitu concrete would be required to provide composite action in both slab and wall).

Fig. 7-31 and 7-32 have a pallet turning machine with filigree biscuit (still on the casting pallet) ready for turning to the horizontal, and being turned, as part of a future twin wall.

Fig. 7-30: Twin wall production.

Fig. 7-31: Turning machine in twin wall production.

Fig. 7-32: Turning of the already cast wythe of twinwall.

A thermally insulated version of twin wall has been developed. Once finished on site it results in an insulated loadbearing sandwich panel with outer concrete wythe of about 70 mm thickness that forms the facade. Fig. 7-33 shows a turning device for an insulated twin wall panel. Fig. 7-34 is an example of buildings that have used insulated twin wall for their elevations.

Fig. 7-33: Turning device for insulated twin wall panels.

Fig. 7-34: Buildings with insulated twin wall panels.

8. Precast Concrete Stairs

8.1 Introduction

Stairs are common components of all tall buildings. They are generally enclosed by concrete walled cores but can also be supported by steel framing.

Stairs are required, and can be produced, in many shapes and sizes, from simple linear treads to curved, helical, winding, and other irregular profiles. Often the only practical option is to turn to precast concrete as the cast in place formwork can become extremely complex in congested areas inside tall walled cores.

In the example in Fig. 8-1 a three-dimensional model has been used to illustrate a stair arrangement that incorporates a central precast concrete spine wall that partly supports the precast stairs and their landings. Further support is provided by enclosing concrete walls. Conventional on-site formwork required for the stairs and spine wall would be complex, confined and time consuming to install and remove. In this case the core walls could be poured insitu, either slip formed, or jump formed, whilst the stairs and spine wall are being factory produced. The stairs could then be installed in a single operation once the core is made available. The benefits of consistent high quality, and budget and time certainty are therefore realised.

Stairs are the only pedestrian means of egress in the event of an emergency when lifts are disabled and play a fundamental role in the evacuation of tall buildings. They must be sized, designed, and protected to allow occupants time to safely leave the building, and enable emergency services access.

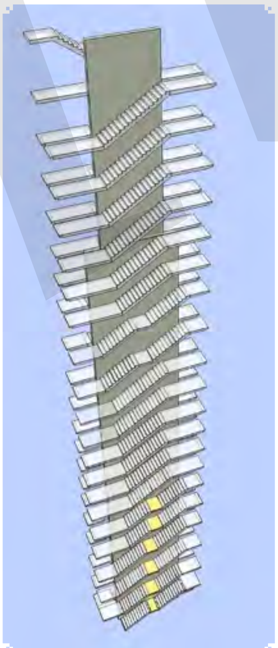

Fig. 8-1: Stair arrangement that incorporates a central precast concrete spine wall.

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The structural design of the stairs themselves is normally straightforward with elements acting as simply supported members. However, connections should be carefully considered. There should either be a continuous tying system through the flights and landings with adequately designed anchorage at supports or other alternative means of support. Where there is potential for large lateral forces and movements, such as in seismic zones, then connections and supports should be designed to accommodate them.

There are also architectural considerations to be accounted for. Often escape stairs do not have surface coverings and this can often dictate the mouldage and production methods that should be adopted as fair faced or visual concrete may be specified.

8.2 Design and Detailing

8.2.1 General

Rectilinear stair flights and landings are typically idealised as simply supported between supports. The flights can either be separate from the landings or integrated monolithic with them. In the first case, the flight waist (minimum cross section of the flight) can be minimised as the effective span is relatively small between the edges of the supporting landings. In the latter case, the waist, and consequently element weights, are increased due to the greater span, this may give benefits of fewer components, but on the other hand may require craneage of greater capacity.

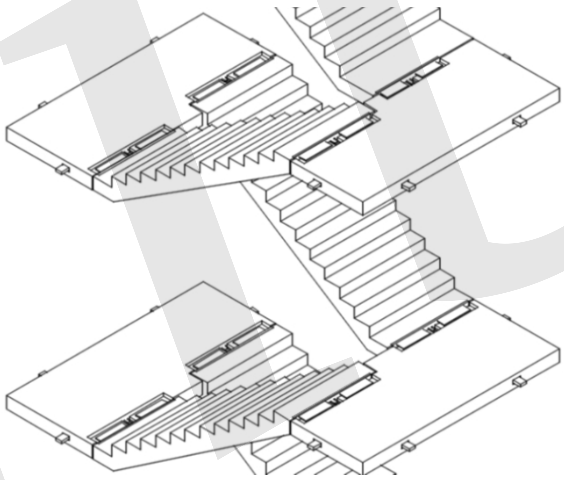

Fig. 8-2: Flights spanning simply supported between landings.

In Fig. 8-2 the precast flights span simply supported between landings that, in turn, span transverse to the flight span. In this example steel angles bolted to the ends of the flights bear into recesses provided along the edges of the landings. The recesses are then filled with structural mortar or grout to provide fire proofing and a level surface. Other types of end supports are possible and are covered later. The flights should be also either effectively tied or anchored to the landings to provide robustness. Sliding steel connectors are shown as supports for the landings onto the side walls. Note that the landings are finished precast elements in this case and the only site wet work required is at the flight bearing recesses.

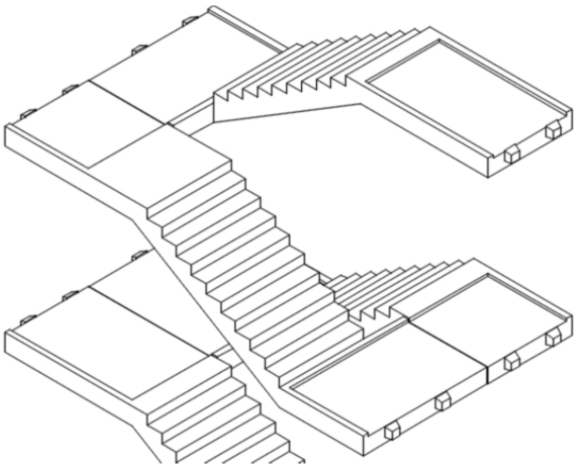

Fig. 8-3: Flights cast integral with landings.

Fig. 8-3 is an example of flights cast with landings integrated into both ends. In these circumstances, there is normally a joint across the landing. A finishing screed or topping is often used to cover the joint and provide an even surface. However, a finishing screed can be avoided if high levels of production and installation accuracy can be achieved so that there is minimal level deviation across the joint.

Helical, and sometimes winding linear stairs, generally require supports that cantilever from the core walls and may have increased analytical complexity due to their shape and geometry. The flights themselves can typically span between the cantilever supports, but particularly in the case of helical flights there should be fixity at those supports. Additional torsional moments, shearing forces and thrusts can be developed along the flight, but the governing design condition is usually the vertical flight bending moment at the support. The cantilever supports can be either structural steel sections or insitu landings.

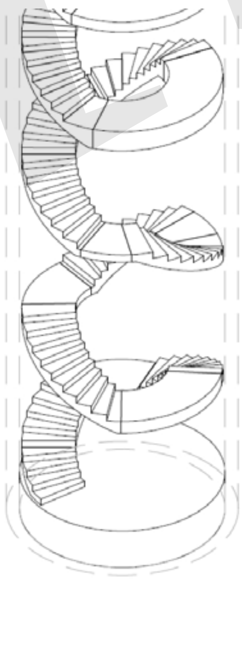

Fig. 8-4: Helical flights inside a circular core.

Fig. 8-4 shows helical flights inside a circular core. In this case the flights were precast, and the landings poured insitu.

The insitu landings cantilever from the walls and support the flights spanning between them. The landings at main floor levels are relatively long, however landings between floor are shorter. With helical flights, the largest bending moments and shear forces are at the landing supports and reinforcement at the precast / insitu mid storey interface can become congested. This solution requires the precast to be installed and temporarily supported inside the core in advance of the pouring of the insitu landings and therefore needs to be a non-critical scheduled activity. Projecting reinforcement from the walls required to resist the cantilever moments must be carefully coordinated with the reinforcement projecting from the ends of the flights.

8.2.2 Support systems

There are numerous support systems for both flights and landings. They can either be formed in concrete or be steel inserts that are either proprietary systems that have been developed by specialist suppliers or designed specifically for an application.

8.2.2.1 Flight supports

In cases where linear flights span simply supported between supporting landings the ends of the flights are typically supported by concrete nibs projecting from the longitudinal edges of the landings. It is often the preference of producers to have steel angles anchored to the ends of the flights to avoid complications to mouldage. A possible detail of this type is shown in Fig 8-5.

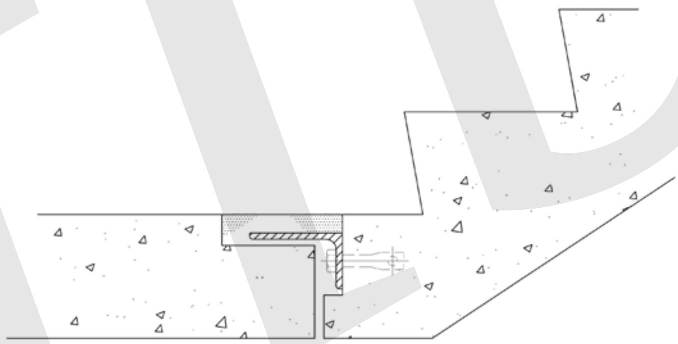

Fig. 8-5: Section through steel angle bearing connection at end of flight.

In this example, the angle is bolted to a cast in threaded insert. Careful attention should be paid to the local reinforcement required to anchor the insert; this is often essential to generate the required resistance. The angle can also be anchored by welded reinforcement or cast in channels. When sizing the angle, the reaction at the bearing should be applied at the tip of the angle to generate maximum cantilever action. The end of the flight, and the landing should also be recessed and detailed so that the angle is adequately fireproofed.

The steel angle can be replaced by a concrete nib that forms a halving joint with the landing support nib. The landing and flight nibs are typically equal depths with an allowance for shimming and bedding between them. Often the choice of concrete nibs at ends of flights can be limited by the mouldage available to the producer, particularly where there are varying numbers of risers between floor levels in a single core.

In tall buildings contractors and / or designers may prefer insitu concrete landings combined with precast flights. The detail at the end of the flight may have to mirror an insitu connection. A typical detail of this kind is shown in Fig. 8-6.

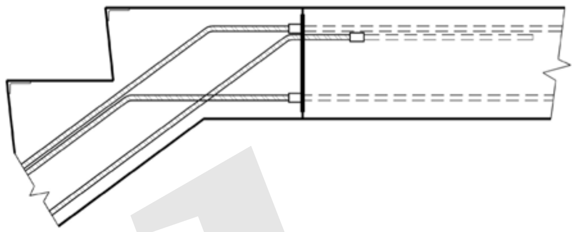

Fig. 8-6: Section through precast flight interface with insitu landing.

Reinforcement coupling is preferred in this case to avoid congestion of lapping reinforcement at the interface, and problems that could develop in handling and storage due to long projecting reinforcing bars. Note that the top tread has increased length to accommodate the reinforcement where it changes from sloping to horizontal. This detail affects mouldage and is likely to mean that special and more expensive moulds are required.

In regions where seismic actions are anticipated allowance should be made for possible large movements and the potential for the flight to slide from its bearing. Earthquake experience has shown that often cores can remain intact, but stairs become unusable and dangerous due to large movements, removing the main means of escape. The fligh / landing connection should be designed to either resist or accommodate such movements and allow continued pedestrian access.

Structural steel beams often provide support for precast concrete flights, and the bearing details can be like those used for precast supports. The simplest detail, however, is to bear the flight onto the top flange of the beam. Bearing levels below the top flange result in increased complexity for both steel and precast and should be avoided.

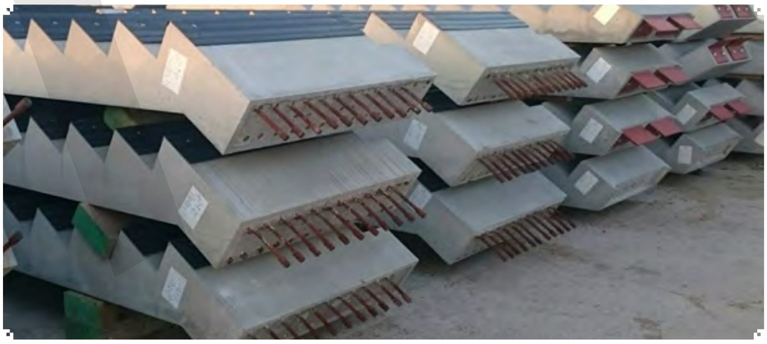

Fig. 8-7: Precast stair flights in storage with end details as shown in previous Fig. 8-5 and 8-6.

8.2.2.2 Landing supports

Landings are typically supported by core walls. In slip or jump formed insitu walls in tall buildings it is generally not possible to have projecting nibs or steel supports cast into the inside faces of the walls. However, allowance must still be made in the wall for the landing support.

The support type that has the minimum effect on insitu wall construction is the projecting steel angle. The angle can be anchored to the wall using expanding or resin anchors that are site fixed after wall construction. There are, however, a few drawbacks with this method. Firstly, the horizontal leg of the angle is exposed and will require additional finishing for both fireproofin and visual reasons. Then there are issues of tolerances, dimensional deviations, accuracy, workmanship, and obstructions (particularly clashes with wall reinforcement) when site fixing the anchors. These issues may be overcome by casting a steel channel insert for later anchorage into the wall during its construction, but this may, in turn, affect wall construction, positional accuracy may be difficult to achieve, and the channel size may be relatively large to provide the required capacity. Finally, support angle installation is an additional activity to add to the construction programme.

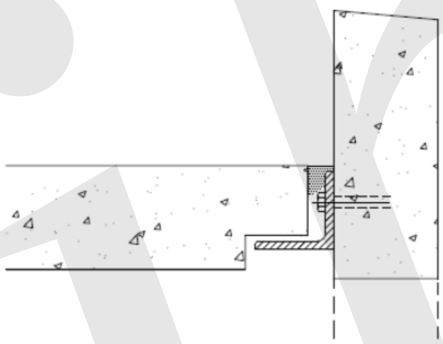

Fig. 8-8: Section through steel support angle recessed into precast landing.

Suppliers of precast concrete inserts have developed proprietary support systems that overcome the drawbacks of using steel support angles as shown in Fig. 8-8. The steel support is contained in the precast landing, which is then slid out onto a concrete bearing surface as the landing is installed. A simplified example is shown in Fig. 8-9.

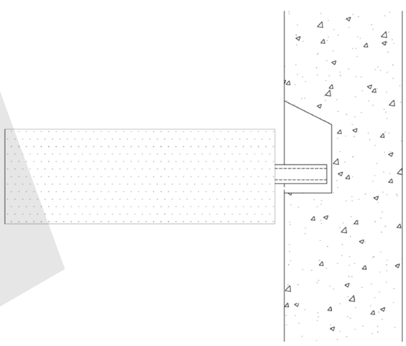

Fig. 8-9: Section through sliding steel support from precast landing into insitu wall.

A pocket must be formed in the wall to receive the sliding steel support. Once the steel support has been set at the correct level then the pocket is fully grouted, and the support encased. This solution overcomes the fireproofin and visual problems. It also is not susceptible to the accuracy related issues that affect the support angle option and is completed at the same time as the installation of the landing. Because the supports are now localised, and not continuous like the support angle, careful attention needs to be paid to the reinforcement around the support that is advised by the suppliers. An example of that reinforcement is shown in Fig. 8-10.

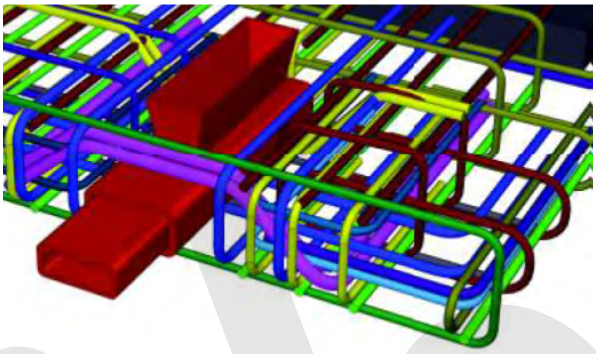

Fig. 8-10: Example of sliding steel support with localised reinforcement.

Fig. 8-11 shows the sliding steel support. This comprises a steel rectangular hollow section that sits inside a larger hollow section. Once the precast landing is in line with the bearing recess the smaller hollow section is slid out to complete the connection. Both the internal part of the connector and the wall pocket are later grouted.

Fig. 8-11: Sliding steel support for casting into precast landing.

Sliding steel supports can also be provided with soundproofing, wall anchorage and a modified version that avoids a grouting patch on the top surface for circumstances where the precast landing does not have applied top finishes.

The minimum number of sliding supports per landing is four. Two of those are located near the flight bearings and the other two on the back wall. The supports nearer the flight bearings transmit the flight loads (including an eccentric component of that load) plus a proportion of the load from the landing itself. The supports along the back wall resist the uplift component of the flight eccentricity plus a proportion of the landing load. A typical arrangement is shown in Fig. 8-12.

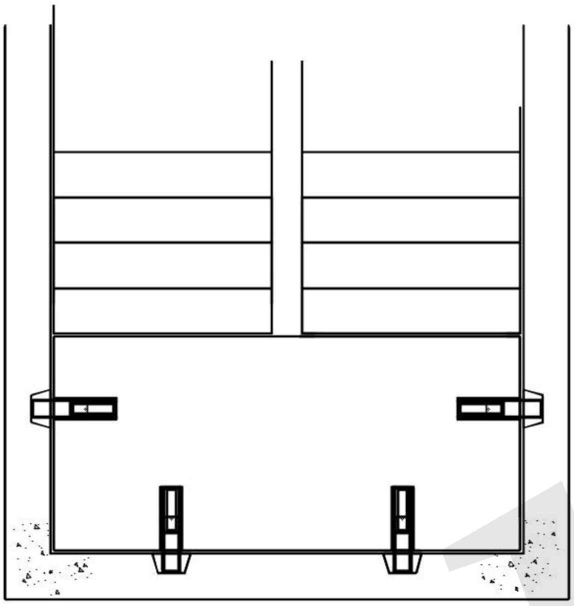

Fig. 8-12: Plan view of example of sliding support arrangement with four connectors; two in side walls and two in back wall.

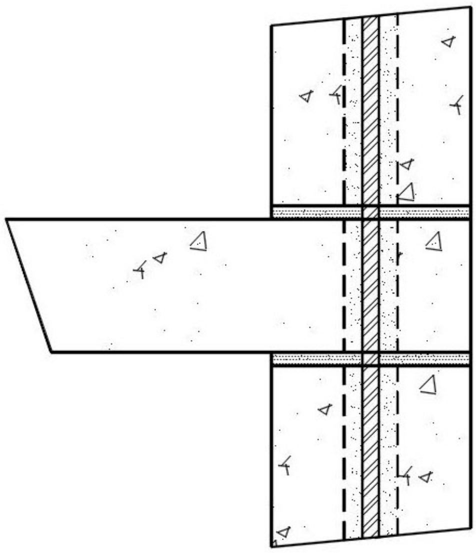

Fig. 8-13: Section through precast landing bearing directly onto precast core wall with vertical continuity reinforcement grouted into cast in tubes.

Landing support details already considered mainly relate to insitu core walls, where the walls are constructed in advance of the landing and flight installation. In instances where precast concrete core walls are provided then the support details can be simplified and the landings installed concurrent with the wall installation. In Fig. 8-13 the landings can bear directly onto the walls. The walls are notched to accommodate the landing and tubes provided through the landing to enable continuity of vertical wall reinforcement. The landings are consequently locked into the walls and act as diaphragms to further stiffen the core in relation to both construction and permanent stability cases.

Both steel angle and sliding support systems can also be used with precast cores if the interlocking landing system is not possible. A variant of the sliding support system has also been used where a steel sleeve or hole through the concrete wall is provided as shown in Fig. 8-14. A steel hollow section or billet is then slid through the hole from outside the core to project from the inside face of the wall, and then grouted in place. The precast landings would have pockets formed in them that would have the open face on the underside of the landing. They can then be lowered directly onto the projecting steel supports. The underside of the pocket would then have to be shuttered and the pocket fully grouted. Whilst access is required from outside the core, and some shuttering is required to complete the grouting, there is the benefit that the landing is immediately stable once landed onto the projecting steel supports plus the precast landing is simplified.

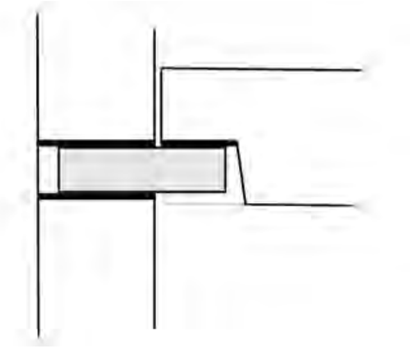

Fig. 8-14: Section through steel connector encased in wall and fully grouted into preformed pocket in precast landing.

8.2.3 Dynamic analysis and vibrations

There is sometimes a tendency for specification of stair flights with relatively thin waists. This, in turn, lowers the resonant frequency of the stair through a reduction in its stiffness making it susceptible to vibrations that may cause concerns and discomfort.

Vibrations in stairs and floors can make people uneasy and create unwarranted fears of structural failure. Whilst the displacements and stresses are relatively small, nevertheless it is desirable to avoid such vibrations developing. National Building Codes often specify a minimum natural frequency for a floor or stair, below which a dynamic analysis is required. It is, therefore, good practice to keep the natural frequency of the stair above this minimum, which is normally in the range of 5 to 6 Hertz. For stairs with relatively thin waists and longer spans it could be that achievement of the specified minimum natural frequency is the critical design requirement.

There is a simple formula in Eq. 8-1 that is used for the estimation of the fundamental natural frequency, f_o, for a simply supported member.

$$f_o = \frac{18}{\sqrt{\delta}} \tag{8-1}$$

δ is expressed in mm and is the maximum deflection due to the self-weight of the member and any other supported loads that may be considered permanent. It can be calculated from this equation that to achieve a value for f_o of 6 Hertz then the deflection due to permanent loading must be limited to 9 mm, which may be more onerous than normal deflection criteria.

Deflection, δ, is itself inversely proportional to the stiffness of the member, *EI*, where *E* is the modulus of elasticity of the concrete and I is the second moment of area of the section. In reinforced concrete design the value of I varies significantl , depending on the section being cracked or uncracked. Therefore, any analysis should firstly investigate the potential for cracking under service conditions, and thereafter determine the expected stiffness, before proceeding with the natural frequency calculation.

It is important that vibration calculations are carried out at concept stage before finalising section sizes for both stairs and floors.

8.2.4 Robustness and anchorage

Stairs should not only be designed to have enough strength to resist the design loadings but should also be sufficiently robust, and anchored in such a way, that an isolated failure does not lead to a disproportionate and / or progressive collapse. This requirement is particularly relevant to stairs in tall buildings as the dynamic forces initiated from collapse, or release, of a stair element at a high level could result in the further collapse of many more elements directly below.

Consequently, to avoid disproportionate and / or progressive collapse in tall buildings all precast stair members should be effectively anchored to that part of the structure that contains the building tying system (for instance, tying systems as advised in 9.10 of EN1992-1-1[5]), which is likely to be the core walls or the adjoining floors. Furthermore, this anchorage should be capable of carrying the dead weight of the member.

Flights and landings are often separate elements, with the landing supporting the flight. In that case the flight needs to be connected to the supporting landing, which in turn is anchored to the supporting wall, to satisfy the anchorage requirements mentioned previously. The simplest, and one of the more effective ways to achieve these connections is to provide a structural topping over the landing with reinforcement in the topping that connects the walls, landings, and flights. Fig. 8-15 is an example of a tie connection between a supported flight, landing and wall using a structural topping.

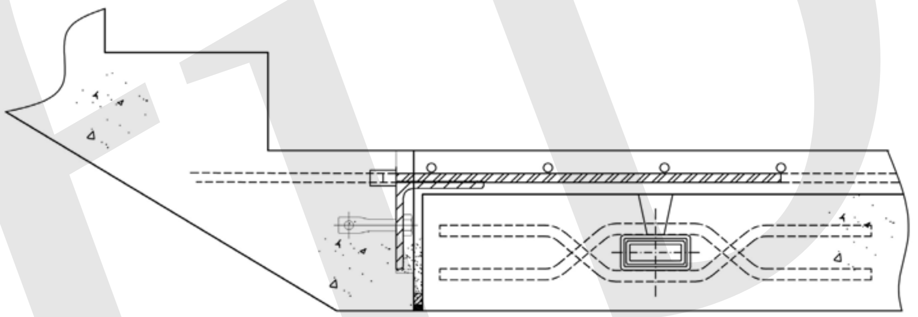

Fig. 8-15: Section through tie connection between flight, landing and wall using a structural topping. The transverse reinforcement in the topping is continuous into the side walls, the longitudinal reinforcement in the topping is coupled with the landing reinforcement.

However, structural toppings introduce an additional activity in stair construction, that is also in a location that may be difficult to access. There is, therefore, often a preference to avoid toppings and use fully precast landings if possible. This means that alternative ways of connecting the flights, landings and walls must be developed.

It is preferable that there is a continuous tie between the flight and the landing. When the landing is fully precast this can be achieved by casting tubes into the landing and grouting connecting reinforcement into the tubes in a similar manner to the vertical connection often used in precast wall panels, albeit in a horizontal plane in this application. Fig. 8-16 illustrates an example of a connection of this type in section and plan.

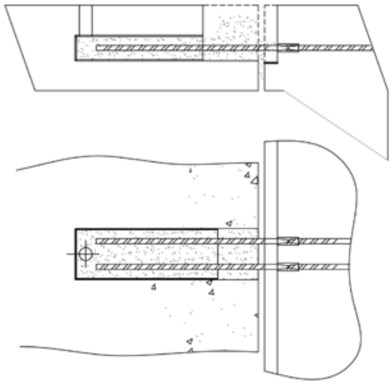

Fig. 8-16: Section and plan example of a horizontal tie connection between precast flight and landing; coupled tie bars grouted into tubes cast into the precast landing.

Where there is a precast halving joint between the flight and supporting landing a vertical shear dowel detail has been preferred by some producers to anchor the flight Care should be taken when using a detail of this type to ensure that enough resistance can be developed to withstand catenary forces that may be generated in the event of accidental failure of the supported flight. It is to be noted that development of bending stresses in the dowel can significantly reduce its shear resistance. When using this detail all vertical joints and bearings should be fully grouted so that there are no air gaps before the dowel can be considered fully embedded. Insitu stitching of fully precast flights and landings is also possible. These connections normally comprise vertical U bars projecting from the ends of both members, that overlap and have connecting horizontal reinforcement through the overlap that is site fixed.

Once the flight is tied or anchored to the landing then the anchorage of the landing to the support walls must be considered. The precast landing can be continuously supported along its edge in several ways. In cores with precast walls anchorage of the landing is often relatively straightforward, as shown in the example in Fig. 8-13, through attention to detailing of the end reinforcement in the precast landing itself. However, in circumstances where it is difficult to bear directly onto the wall, particularly with insitu construction where the landings are installed after the whole core may be complete, then doweled connections between the support and the precast landing may be needed. Again, careful attention must be paid to the capability of the dowel to realise its full embeded shear capacity, particularly if a vertical dowel is grouted into a preformed sleeve and an allowance has been made for shimming and grout in a horizontal bearing bed. Vertical dowels may also be fixed through holes in the horizontal leg of a steel support angle into threaded fixings cast into the underside of the precast landing. Vertically doweled and insitu stitch connections to landings are also possible when concrete support nibs project from the core walls.

Localised supports, such as the example in Fig. 8-9, are commonly used for precast landings. In these cases, the steel inserts are fully grouted into preformed pockets in the support walls and the connection is idealised as a pocket base for anchorage purposes. The combined effect of both vertical and horizontal actions for the accidental load case should be considered, and supports should be provided along two orthogonal axes, as shown in the example in Fig. 8-12, to withstand possible actions along both axes. When supports are only possible parallel to one axis then additional tie or dowel connections would be needed.

8.3 Methods of Production

8.3.1 General

The method of production, and particularly the mouldage chosen, for precast stairs largely depends on both the specified shape and surface finish of the flight.

Precast stairs are generally produced as individually moulded elements in reinforced concrete. They have also been produced as prestressed elements in self-stressing steel moulds. There are several different stair types, that can be broadly subdivided as follows:

- Simple linear flights;
- Helical or winding flights;
- Sawtooth flights.

Mouldage required to form precast flights is normally in either timber or steel. Timber would be used for more complex shapes where there is minimal repetition, steel would be chosen when there is greater repetition and good quality consistent finishing is required. Moulds can be subdivided into three categories:

- Face downwards with the treads and risers formed in the mould and the underside of the flight either hand or power floated as the top surface of the casting;
- Cast on edge with only a single narrow edge floated, all other faces are moulded;
- Ramp mouldage; where the stair is formed in an upright position, as it will be in its final position. The underside, risers and edges of the flight are moulded. The treads are floated.

Each mould type has its benefits and drawbacks, but often the type of mould chosen will be dictated by the finishes that have been specified. It is likely that the “cast on edge” mould will be the most expensive as it has the greatest proportion of moulded surface, but, consequently, is most likely to produce the desired result if the stair is left as fair faced concrete with no further applied finishes. In this case, attention to detail can result in all visible surfaces being formed if the floated face can be alongside the core walls.

Cast in items, such as nosings, anti-slip inserts and end connecting reinforcement and / or fittings, may also affect mouldage and the method of production.

8.3.2 Linear flights

These are the most common precast flights. Examples are shown in Fig. 8-2 and 8-3.

Fig. 8-17 is a simple linear flight cast face downwards. The underside of the flight is the top surface for casting and has been hand floated in this case, all other surfaces are moulded in steel forms.

Fig. 8-17: Linear flight cast "face downwards".

Fig. 8-18 to 8-21 are examples of linear flights moulded using the two other types of mouldage.

In Fig. 8-18 one side of the mould "on edge" is visible. In Fig. 8-19 the resulting precast stair has been demoulded and is stored at the production plant. The flight is integral with two landings and the example is like that in previous Fig. 8-3. Note that both the landings and treads are recessed to receive finishes, and that the landing is wider than the flight at one end.

In Fig. 8-20 a ramp mould is prepared, with formers and reinforcement placed to receive concrete. Fig. 8-21 shows the mould after concrete has been placed.

Fig. 8-18: Example of "on edge" mould.

Fig. 8-19: Example of "cast on edge" stair with integral landings.

Fig. 8-20: Ramp mould prepared to receive concrete.

Fig. 8-21: Ramp mould after concrete placed.

8.3.3 Helical or winding flights

Helical flights are curved in plan in their final position. They are often used in either feature stairs, that are visible through glazing in parts of the walls, or in circular cores. Fig. 8-22 shows a view down into a circular core with helical flights, like the previous example in Fig. 8-4.

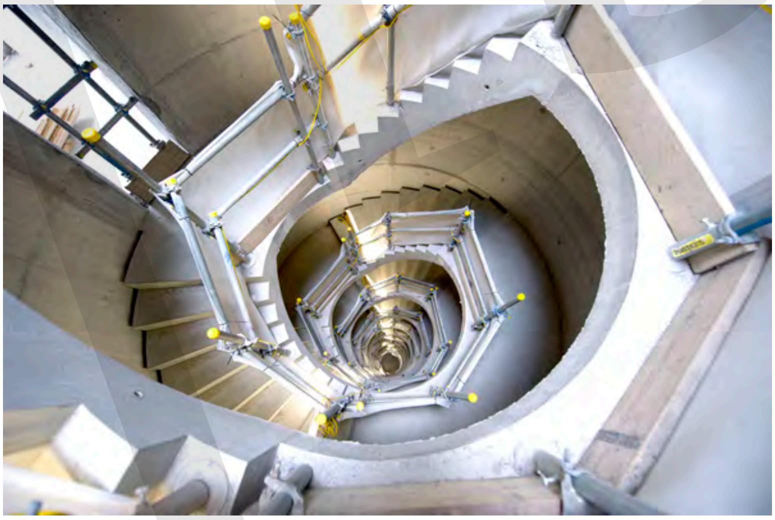

Fig. 8-22: View down into circular core with helical stairs.

The basic methods of mouldage for helical flights are like those for linear stairs, but obviously the geometrical shape is more complex. In Fig. 8-23 a steel "Face downwards" mould has been used for a helical flight, both the mould and the finished product can be compared. Steel was probably chosen in this example due to a repeating number of flights of the same basic shape. In Fig. 8-24 a ramp mould has been used to form the helical shape. A timber supporting structure and mould has been chosen in this case.

Winding linear stairs are often used in cores of larger plan area and / or where the stairs wrap around a central shaft. In these cases, there may be more than one intermediate landing between flights. Whilst the stair arrangement is slightly more complex than the examples in Fig. 8-2 and 8-3 the same basic production methods and mouldage as used for simple linear flights should still be applicable.

Fig. 8-23: "Face downwards" mould used for helical flight.

Fig. 8-24: Timber Ramp mould used for helical flight.

8.3.4 Sawtooth flights

Sawtooth profiles, that mirror the riser and tread on the underside of the flight, are sometimes preferred by architects for visual reasons. Fig. 8-25 illustrates an example of sawtooth flights with precast landings.

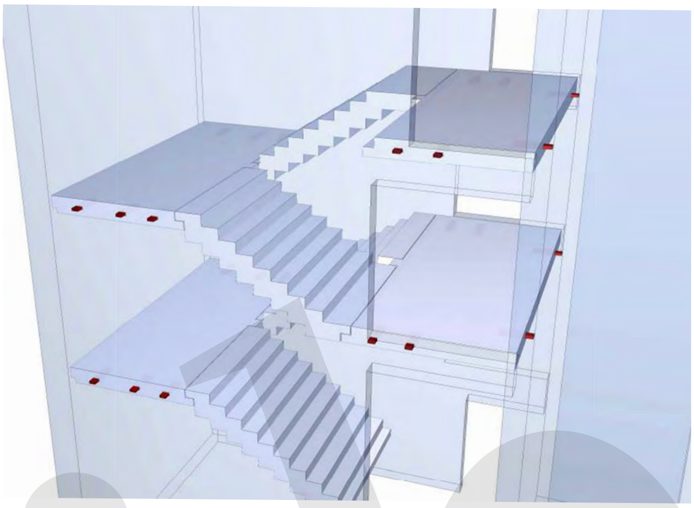

Fig. 8-25: Example of sawtooth flights with precast landings.

"On edge" mouldage is the likely choice for precast flights of this type due to the stair profile, particularly if there are no applied finishes to the top treads and risers.

Fig. 8-26 shows the installation of a precast sawtooth flight inside an insitu concrete core.

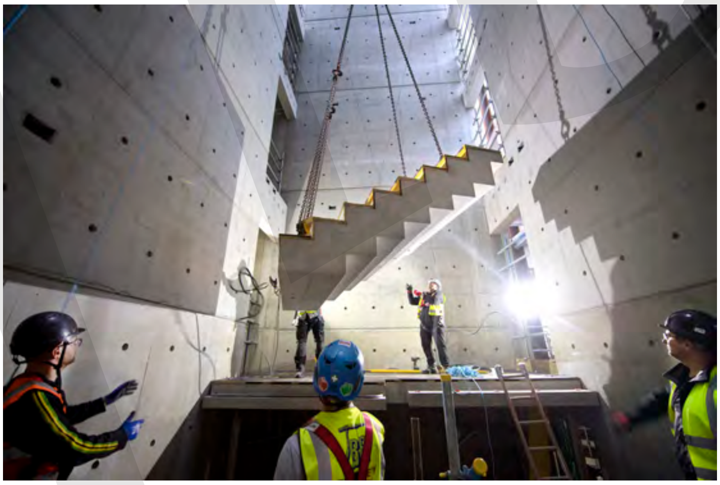

Fig. 8-26: Flight installation.

9. Facades and Claddings

9.1 Introduction

Every tall building requires a protective envelope around its perimeter to withstand the harmful effects of wind, rain, and excessive heat and cold, plus provide a safe barrier against the risks of fire and other hazards. It allows its residents to function in the comfort required for everyday living and protects their possessions.

The building envelope can take many forms, and in tall buildings there is a tendency to move towards lighter materials with large areas of glazing often the preferred option to reduce the building's self weight and provide more natural light. However, architectural precast concrete wall panels have also been successfully used in external building elevations since the early years of the twentieth century and have become increasingly popular as new finishing techniques and materials have become available. Precast panels can be mixed with glazing to produce attractive and elegant solutions.

Fig. 9-1: The Carlyle, Los Angeles, California.

In Fig. 9-1 large areas of glazing are framed by architectural precast panels that form vertical and horizontal bands and corners.

The versatility and durability properties of concrete are well known, but their benefits in the external building facade are accentuated. Plastic concrete can be moulded to express the visual intentions of the Architect whilst still providing its acknowledged resilience. Treatment of texture and colour can be rich using the best qualities of the constituent materials. Aggregates and cement are chosen to achieve the texture and colour required in the finished element.

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Architectural precast concrete initially relied on individual craftmanship to achieve the high quality standards required. Nowadays processes have progressed to highly controlled and coordinated factory production lines that give economic and physical improvements without sacrificing the expected high standards of the past and limiting the freedom of three dimensional design that displays its individuality.

Programme benefits in tall building construction resulting from factory production of the building envelope should be obvious to all. The external panels can be produced and stockpiled whilst the structural frame is being constructed. If the glazing is incorporated in each panel, as shown in Fig. 9-2, then the corresponding area of elevation is substantially closed in a single installation.

Fig. 9-2: External precast panel with windows (courtesy of Explore Manufacturing UK).

9.2 Types of External Precast Panels

Precast cladding panels can be subdivided into two main types:

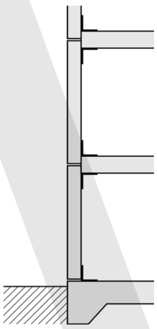

Fig. 9-3: a) Single leaf panels

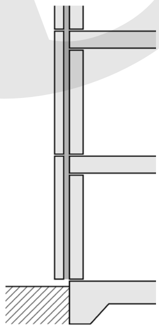

Fig. 9-3b: Sandwich panels

1. Single leaf panels: They provide a weather proof architectural envelope but perform no structural function. They are generally only used to enclose space and are designed to solely resist forces directly imposed on them, such as wind, and transfer these local forces and their own self weight to the nearby main supporting structure. Vertical loads can be transferred to the supporting slab at the bottom of the panel as shown opposite in Fig. 9-3a. Both top and bottom connections provide lateral restraint to the panel and horizontal load transfer to the frame. The insulation and internal finishes are installed after the panel has been erected.

2. Sandwich panels (see also *fib* Bulletin 84[16]): They comprise two concrete leaves, or wythes, that are separated by an insulating core. The outer wythe provides the weatherproof skin and architectural finish. The inner wythe is loadbearing, and as a minimum provides support for itself and the outer wythe. It can also be designed to support the perimeter of the building structure as shown in Fig. 9-3b, but there may be little benefit in utilising this function in tall building construction as the panels would most likely have to be installed as the structural frame is constructed. Therefore, the system in Fig. 9-3a would again most likely be utilised. The insulating core between the wythes provides the thermal insulation blanket around the building.

Single leaf panels may be more desirable in tall building construction as the element self weight is considerably reduced, which may result in savings in both crane and transport costs.

There are a number of ways to support precast concrete cladding panels, but some are really only applicable to short and medium rise buildings, e.g.stacked and load bearing systems . In tall buildings "floor by floor" systems can be more suitable (see Fig. 9-3a). The panels are independently supported at each floor level. There is no limit to the number of storeys that can be clad using this method, whereas stacking and load bearing systems are limited to a relatively small number of storeys. Slab deflections also need to be considered early in the design process, and it may be necessary to provide a means of accommodating the slab deflection in the top restraint connection and avoid redistributing vertical loading.

All panel types allow the designer to choose from a wide variety of external finishes, such as textured concrete, brick, stone, or any other suitable material integrated into the panel.

Fig. 9-4: Example of insulated sandwich panel from Finland.

The building envelope has to be treated as a complete assembly to ensure continuity of thermal insulation, air and fire barriers and vapour retarders. The precast panel will only be one part of this envelope that has to satisfy specified performance criteria, and in particular environmental separation requirements, i.e. control of heat, air, water vapour, precipitation and noise must also be considered.

The panel will interact at interfaces with other building components, such as glazing systems, the structural frame and floors, to satisfy these criteria. It is therefore important that the precast panel producer and the building's design team work together to develop suitable design details.

Precast concrete sandwich panels can provide a large part of the building envelope with thermal and acoustic insulation, fireproofing weatherproofing, external finishing, durability and strength in a single element. In Fig. 9-4 there is an example of a sandwich panel designed to be used in a cold climate with a relatively thick insulation core to provide a barrier to harsh climatic conditions. Single leaf panels require further attention to the interface on their internal face.

9.3 Precast Panel Layouts

External precast panel elements can typically either extend a storey height between floor levels or in narrower bands. Horizontal dimensions are normally defined by architectural requirements, but are generally between main building gridlines where columns may be located. Floor levels and column locations therefore often define the layout of panel joints.

In tall buildings there are three common facade patterns that suit precast panel layouts. These can be summarised as follows:

1. Cladding that covers the structural framing both vertically and horizontally with large openings then infilled with glass: An example is shown in Fig. 9-5.

Fig. 9-5: Conjunto Paragon, Santa Fe, Mexico.

2. Elimination of column cladding resulting in alternate horizontal bands of spandrel panels and glazing: An example is shown in Fig. 9-6. In this layout the spandrels and glazing are positioned in front of the columns. This gives the opportunity to provide relatively large, glazed areas that are divided by precast panels that can be architecturally finished.

Fig. 9-6: BMX Parque de Cidade, Sao Paulo, Brazil.

3. Rectangular window openings that can be punched through a plane panel: This pattern has been popular in lower and medium rise buildings where there was a requirement for loadbearing walls and an area between windows to carry the vertical loads. Therefore, windows tended to be relatively small. More recently this configu ation may have become more popular in taller buildings due to the perceived needs of energy conservation and the avoidance of large areas of poorly insulated glazing. However, the improvement in the energy efficien y of glazing products generally negates this argument and it is more likely that it is the demands of architectural fashion and the return to moulded facades and the visual results that can be obtained that is the main reason. An added advantage with this type of panel is that the window can be fixed into the panel at the precast factory and therefore remove another site operation. This is unlikely to be possible with the previous two patterns. Fig. 9-7 is an example of a punched window panel where three windows have been accommodated in a single panel. Fig. 9-8 shows how glazed winter garden boxes can be incorporated into a panel with minimal support concrete and installed on site in a single operation.

There is a fourth potential layout possibility, which is the solid panel configu ation, i.e. panels that are not impacted by glazing or other discontinuities. However, this is rarely used in tall buildings as a full solution, but may be required in localised areas where architectural treatements and false joints are likely to be needed to enhance the visual impact.

Fig. 9-7: Punched panel with three pre fixed windows (courtesy of Explore Manufacturing UK).

Fig. 9-8: Punched panel with two glazed "winter gardens" (courtesy of Explore Manufacturing UK).

9.4 Finishes and Textures.

9.4.1 General

Precast cladding offers a medium for designers and owners that allows unlimited scope for architectural expression of their ideas. The creative possibilities are endless and may only be limited by the practicalities of casting and cost.

Successful aesthetics can be achieved through attention to the following:

- Concrete colour;
- Surface texture;
- Facings;
- Minimising the detrimental effects of weathering.

Selecting the most suitable colour and texture for precast cladding is a subjective and iterative process. The colour choice can be influenced by many factors, such as the building's location, the local geology, historical use of material in the surroundings and contemporary materials already chosen for the façade. In turn, the texture may be influenced by the weathering strategy, and the scale of the building when compared to other nearby buildings.

9.4.2 Concrete colour

The concrete colour is directly related to the mix constituents, i.e. cement, fines, coarse aggregate and pigment. The finer the grading of any of the constituents the greater will be its influence on the colour of the hardened concrete. Consequently, cement tends to dominate the colour of the finished product as it is the finest material.

Two types of cement are generally available; grey and white. Because grey cement is widely used as a basic commodity and is produced by several manufacturers its colour can be variable. Also, grey cement dominates the mix colour and is normally only selected if the desired final concrete colour is a shade of grey or black. With grey cements and coloured pigments a duller version of the pigment colour is obtained.

Conversely, it is recognised that white cement is almost exclusively used for aesthetic purposes and suppliers would have more rigorous controls in place to maintain colour consistency. White cement does not dominate the final colour to the same degree as grey cement, and allows greater expression of the subtle colours of the constituent aggregates.

The colour of architectural concrete can be largely dependant on the availability of local aggregates. If there is a large variety of rock supplies, such as limestones, granites and other igneous rocks, then many colours are achievable; white, buff, green, grey and black for example. The fine aggregate can influence the concrete background colour to almost the same degree as cement, whereas the coarse aggregate has little effect unless exposed. Increasing the percentage of fine material in the mix is likely to improve colour consistency from batch to batch. However, as with most natural materials, colour variations do occur and this should be taken into account by the designer.

In circumstances where the required concrete colour is more intense than those achievable with natural sands then a pigment may need to be added. Reds, yellows, blacks and browns are possible through adding iron oxide pigments. White, green and blue colours are also achievable but require different types of pigment. The colour of the aggregate should complement the pigment. Examples are black basalt with a black pigment and grey cement, yellow pigment with buff coloured limestone and red pigment with river sands or pink granite.

The choice and achievement of the required concrete colour is an iterative process, and it is essential to conduct trials to understand and appreciate the effects of varying the the mix proportions to achieve the intended result. The water cement ratio can be critical as fluctuations in water content can significantly impact the colour intensity. A higher water content will result in a paler concrete.

Once the desired colour is achieved the concrete surface can be polished to further accentuate the finish. Examples of polished surfaces are shown in Fig 9-9.

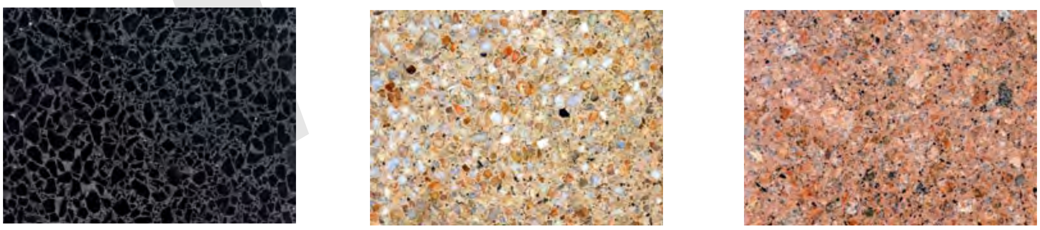

Fig. 9-9: Examples of polished coloured concrete surfaces.

Management of the impact of efflorescence on the concrete panel should be considered. Efflorescence is a white powdery deposit that can form on the surface of all concrete and masonry products after wetting and drying cycles at the beginning of the cladding's life. It diminishes with time, but can be either removed with a weak acid solution or avoided by application of a water repellant surface sealant.

9.4.3 Surface texture

Surface texturing of concrete panels is used to disguise the surface blemishes, colour variations and general surface imperfections that arise when using smooth formwork. Smooth exposed finishes are still sometimes used, but would most likely require painting and high maintenance if they are to avoid becoming unsightly.

Surface textures can be divided into two types; micro textures and macro textures.

Fig. 9-10a: Reconstituted stone - precast concrete with a fine texture.

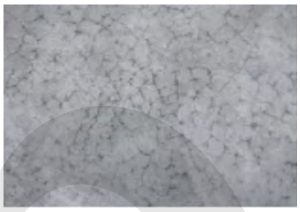

Fig. 9-10b: Evidence of cazing on fair faced concrete

A micro texture finish is applied after the concrete element is removed from the mould and involves removing the cement and fine aggregate that migrates to the concrete surface during the casting process. Concrete with this texture is often called reconstituted or reconstructed stone.

This texture visually improves the concrete surface considerably by not only removing blemishes and imperfections but also limiting the possibility of crazing, where differential contraction between the concrete surface and underlying layers causes interconnected surface cracking as shown in Fig. 9-10b. These cracks are fine and shallow and will generally only become apparent weeks or months after casting. A light grit blast will remove the surface layer and minimise the potential for crazing.

There are a number of ways of achieving a micro textured finish. These are:

- Exposing the aggregate using a chemical retarder: The surface layer of the matrix is removed shortly after stripping by brushing or power washing. This removes a portion of the cement paste between the coarse aggregate.
- Sandblasting (Abrasive blasting): Normally used for light to medium aggregate exposure, i.e. avoiding making the coarse aggregate the dominant surface texture and colour.
- Acid etching: Used for light exposure, revealing mainly the sand portion of the paste fraction and a small percentage of the coarse aggregate.
- Bush hammering: Uses power driven chisels to remove approximately 4mm of hardened surface concrete.

The degree of exposure of a micro textured surface can be subdivided into three categories:

- Light exposure: Removes only the surface film of the cement paste, exposing the edges of the coarse and fine aggregate closest to the surface. Small specks of colour show up on the surface.
- Medium exposure: Removes additional cement paste, exposing equal areas of fine and coarse aggregate and cement paste. The result is a more pronounced colour contribution from the aggregate.
- Heavy exposure: Exposes the coarse aggregate so that it dominates the surface texture and colour, as shown in the example in Fig. 9-11.

Fig. 9-11: Exposed coarse aggregate finish.

The lighter the degree of exposure the more skilled the operator / finisher needs to be. It can be difficult to achieve uniformity of appearance with a light exposure over large panels. Micro texture finishes can be obtained using a combination of the methods listed previously, for instance chemical retarders used with abrasive blasting.

A macro texture finish is formed in the mould rather than being applied after demoulding. It consists of a physical step in the surface that develops depth and shadow, creating a three dimensional physical feature on the panel face. Macro finishes can also be combined with micro finishes. Form liners are used to create the macro features that can be rebates, projections, false joints, chamfers or artistic impressions. Complex or random patterns are possible with rubber mould linings or mats that are available from manufacturers.

The introduction of a three dimensional pattern or feature can impose physical constraints on the cladding panel layout as it interacts with joints and windows. If the position of the joints and windows changes relative to the pattern then the pattern design may be cost prohibitive due to the number of panel liners required.

A wide variety of materials can be used as form liners to create a multitude of appearances, examples are shown in Fig. 9-12. Absorption of the concrete mix water into the liner needs to be considered together with the practical stripping of the panel from the liner. Liner joint lines need to be minimised to allow efficient panelisation of the facade. Form liners can be used on the whole panel, or only on part of the panel depending on the finished pattern required.

The pattern may also influence the panel's weathering characteristics. Dirt may be trapped or the fl w of water channeled across the facade causing staining. Grooves that run down the panel will direct water down specific routes, whilst horizontal features will interupt the water fl w and catch dirt and dust that may be randomly washed away leaving clean and dirty areas of facade.

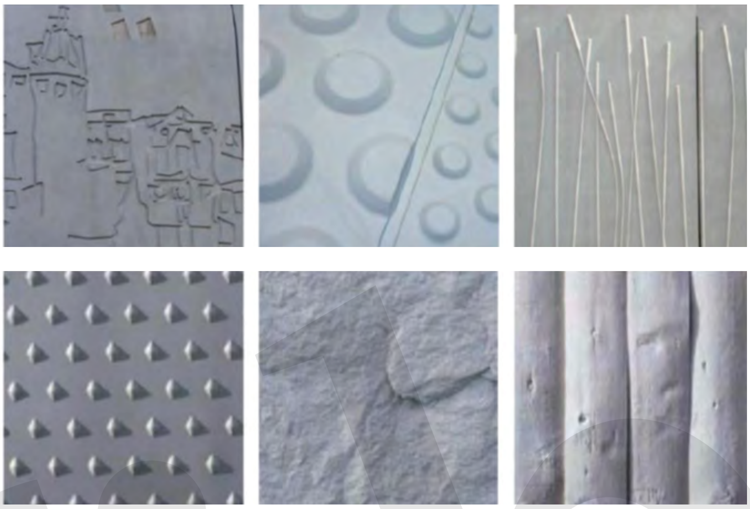

Fig. 9-12: Examples of macro texture finishes.

9.4.4 Facings

Precast concrete cladding panels can be faced with natural stone or clay products. This is often the case where the building facade is required to match adjacent buildings. Sometimes in the case of taller buildings the designer may choose more expensive stone faced panels for the first few storeys and then revert to micro textured panels for the upper storeys. Stone facings vary from 30 mm minimum for granite to 50 mm for marble and limestone.

The chosen material (normally stone, brick or terracotta) is placed face down in the mould in a set pattern with an anchorage in the rear face to ensure that each individual item is bonded to the concrete panel. Anchorages are normally stainless steel dowels or clips into stone facings. In the case of brick or terracotta slips are often used with precut keys (see Fig. 9-13a). Alternatively, bricks with pre formed holes can be cut along their centre to create a key (see Fig. 13b). Once the casting has been removed from the mould the joints between the facing elements can be finished with sealant or pointed with mortar.

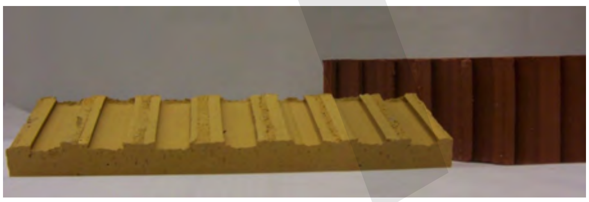

Fig. 9-13a: Brick slips with precut keys.

Fig. 9-13b: Cut brick with preformed holes.

Fig. 9-14a: Terracotta faced panel.

Fig. 9-14b: Polished red granite panels.

Fig. 9-14c: Brick faced panel with contrast central pier.

Fig. 9-14d: Faced panel with factory fitte window.

9.4.5 Minimising the detrimental effects of weathering

Weathering changes the appearance of a building over time. It affects all exposed surfaces and cannot be ignored, a weathering strategy needs to be devised that minimises any distortion to the designer's original intention for the building's external appearance.

The main concern is the control of rainwater fl w and how it redistributes dirt on the face of the building. Uneven concentrations of dirt can lead to unsightly marking patterns on external surfaces. At the initial design stage it is important to understand the climatic factors and local conditions affecting the building in its particular environment. Nearby buildings will influence rain patterns together with the direction of prevailing winds bearing rain clouds. Varying intensities of rain will be driven against different parts of the facade, with corners receiving more rain than centres of elevations.

Horizontal surfaces should generally be avoided as they act as dirt collectors. Any horizontal projection that is required should have a positive fall and project well forward of the building line so that rain fl wing onto it is thrown clear of the cladding below. Consequently, projections and rebates used with cornices, copings and sills can play an important part in controlling water fl w and reducing dirt deposition. Properly detailed cornices and sills can create an undisturbed rain shadow below, with rainwater clearing the facade and prevented from running back to the vertical face because of drip features on the underside of the projections.There are some cleaning effects due to rain, but these can depend on the volume of water running down the facade and the porosity and texture of its surface. Porous materials and coarse textures slow down the fl w, and can reduce the cleaning effect and increase the risk of dirty deposits (although iregularities and shadows on coarse aggregate surfaces can mask discoloration and disperse fl w). Runoff can be improved by using sealers or polishing the concrete surface. Fig. 9-15a and 9.15b are examples of finished polished panels.

Exterior details affect the weathering pattern of the building. Therefore, in order to minimise the detrimental visual effects of water fl w (weathering) the following detailing principles should be considered (also refer to Chapter 9 of *fib* Bulletin 74[14] for details):

- Provide steeply sloping surfaces to allow self-cleaning and limit the distance of water flow;
- Detail surface finishes or features that disperse rainwater flow;
- Avoid concentrating water flow;
- Prevent water fl w over sheltered positions by providing drip grooves that throw the water clear.

Fig. 9-15a: Polished facade panel.

Fig. 9-15b: Polished spandrel.

9.5 Design and Detailing

9.5.1 General

The design strategy adopted for architectural cladding panels in tall buildings, and the details developed from that design, has to contribute towards satisfying a number of important performance criteria that are required from external building envelopes:

- Control of heat and air fl w, water vapour transmission, rain and snow penetration, light and solar radiation, noise, and fire;
- Provision of strength and rigidity;
- Durability;
- Aesthetic appeal.

Aesthetic appeal, and the design and layout of panels in the facade itself, has already been covered in previous sections. In this section individual panel design is mainly considered together with the jointing and connection between themselves and the main structure.

9.5.2 Heat transfer control

Building shape, size, orientation of prevailing winds and sun, glazing to solid wall area ratio, air tightness and general thermal resistance of the external envelope all combine to affect a building's energy consumption. Minimising heat transfer between the building interior and the external environment, and vice versa, is a critical consideration in the design of the external façade and conservation of energy.

The movement of heat through dense building envelope elements, such as concrete cladding panels, in taller buildings is driven by conduction. Insulation materials prevent air movement between inner and outer wall leaves and restrict conductive heat transfer.

Insulation can be provided between two wythes in a sandwich wall panel or as a later on-site application to a single wythe cladding panel. Choice of type and thickness of insulation is dependent on local Code requirements for energy conservation and transmission that are related to climate zone and intended use of the enclosed space. The thermal performance of the system should be based on the effective wall thermal conductivity, typically represented by a U value (w / m² k), which includes losses due to thermal bridges (penetration of the insulation by conductive materials) or local material changes of type and thickness, particularly at panel edges.

In addition, sealants between panels should be compatible with the required heat transmission requirements, and proper detailing of the moisture management system is critical to avoid degradation of insulation.

9.5.3 Air leakage control

Air leakage control through a building envelope affects several building performance criteria that include; heating and cooling requirements, condensation management, and rain, smoke and sound transmission.

Air leakage into (infilt ation) and out of (exfilt ation) a building can typically take place through cracks or joints between components, narrow gaps at windows and openings for building services. Exfilt ating air carries away heating and cooling energy whilst infilt ating air can bring moisture and pollution into the building interior and reduce the effectiveness of the rain screen.

Control of air leakage generally results from provision of a properly designed and detailed air barrier assembly, which is a continuous barrier preventing the movement of air across it. For the proper design of an air barrier the following must be considered:

- The airtightness of the materials themselves in the barrier: There is no measurable air leakage through concrete.
- The continuity of those materials in the envelope, and interfaces and joints with other different materials: Allowance should be made for the relative movement differences due to thermal and moisture variation, creep, and structural deflection. Careful attention is needed when transferring the air barrier seal from precast panels to adjoining assemblies such as other cladding materials, glazing and roofing Sealants must be compatible with both materials they connect.
- Their durability and performance for the life of the building.
- Their structural resilience to withstand the design forces imposed on them: Each material in the assembly should be capable of supporting the air pressure design load. Concrete is an ideal material for an air barrier due to its inherent strength, but sealants between panels and at joints must also be capable of resisting air pressure loads.

In single leaf architectural precast panel assemblies the plane of air tight continuity is typically located in the precast concrete, and continues from panel to panel through the sealants in the joints, generally the inner seal of a two part sealant.

In double leaf insulated precast concrete sandwich wall assemblies the inner structural wythe is normally used as the air barrier plane, again with continuity provided through the sealant at joints. The inner wythe undergoes little thermal expansion and contraction as it has the same relatively constant temperature as the building interior. This results in reducing stresses on the internal part of the sealant and increases its service life.

In concrete clad buildings there is generally less air leakage than in buildings clad with other materials. Adequate ventiliation needs to be ensured with sufficiently designed air intake systems to provide fresh air. Without an adequate intake source concrete buildings have negative pressure, with poor intake air quality possibly resulting.

9.5.4 Condensation and moisture control

Moisture condensation on the interior surfaces of a building is both unsightly and can cause damage to the building fabric and its contents through the generation of fungal growths.

Fungi and biological growth can flourish in the presence of moisture or at relatively high humidities of 70 % and above on wall surfaces. Concrete itself does not provide nutrients for fungal growth but dirt and dust particles on its surfaces allow germination with sufficient available water. It is almost impossible to control most of the contributors to fungal growth except for the water related factors; high humidity (RH), surface condensation and water leakage.

Condensation occurs on surfaces inside buildings when the surface temperature is less than the dew point of the indoor air. The dew point of air is related to its relative humidity (The ratio of the amount of moisture in the air to the amount the air can hold). As an example; an indoors temperature of 20^{o}C at a 50 % RH would have a dewpoint of about 9^{o}C. One of the main aims therefore of any moisture / condensation control system is to prevent air reaching its dew point. Ways of preventing condensation are:

- Maintain the internal surface temperature of the wall assembly above the dew point;
- Reduce the moisture level in the air;
- Use vapour retarder or resistant materials with the insulation selected depending on external climatic conditions;
- Prevent air infiltration;
- Prevent rainwater leakage;
- Use internal pressure control depending on the climate.

Precast concrete wall panels can act with the insulation as part of a strategy to control condensation. In the case of single wythe panels the insulation can be designed so that it acts as a vapour control layer to maintain the internal surface above the dew point. For double wythe sandwich panels the inner concrete structural layer may have enough vapour diffusion resistance to maintain the inner surface above the dew point and therefore acts as the vapour management layer.

9.5.5 Water leakage control

Water migration across a wall assembly requires the presence of a number of conditions acting together:

- A water source i.e. rain or melting snow;
- An opening or crack to pass through;
- A force to drive the water through the opening.

Whilst preventative measures should be taken in the design process to minimise the effects of the above conditions, and designed air barriers previously discussed will assist in this aim, water leakage controls should still be developed for vertical building envelopes. Examples of strategies for controlling water leakage are:

- Mass assemblies;
- Drained assemblies;
- Perfect barriers.

Mass assemblies function on the basis that rainwater striking the external building face will either drain off the surface or be absorbed. In the case of absorption the material should have sufficient water storage capacity so that it can be released later without detrimental effect. There is a reliance on evaporation when drying out, and the rate of drying between rain showers so that the surface materials are not permanently saturated. The use of this method could affect other aspects of the design, such as control of heat transfer and condensation, and needs to be carefully considered. Examples of mass wall assemblies include multi layered stone, masonry, porous concrete, composite layered construction (such as facing brickwork onto precast sandwich panel).

In drained assemblies the external water passes through the external surface and is then directed back to the exterior before damaging water sensitive materials. Often a watertight space is provided behind exterior cladding panels and the water is guided through the wall system via flashings and weepholes. Careful attention to detail is required to avoid compromising other barriers, particularly fire barriers. Examples of drained assemblies are shown in Fig. 9-16.

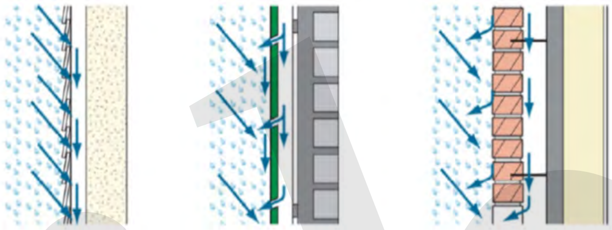

Fig. 9-16: Examples of drained assemblies.

The perfect barrier controls water leakage at one plane within the wall assembly. This could either be at the most exterior face or at an inner surface. The weak point in the system would normally be at the joint between panels. However, if a two stage joint system is used the exterior seal acts as the barrier to rain whilst the concealed interior seal is protected from the damaging effects of the environment. This is a commonly used approach for precast concrete wall assemblies and is known as "the perfect barrier drained joint system"[23].

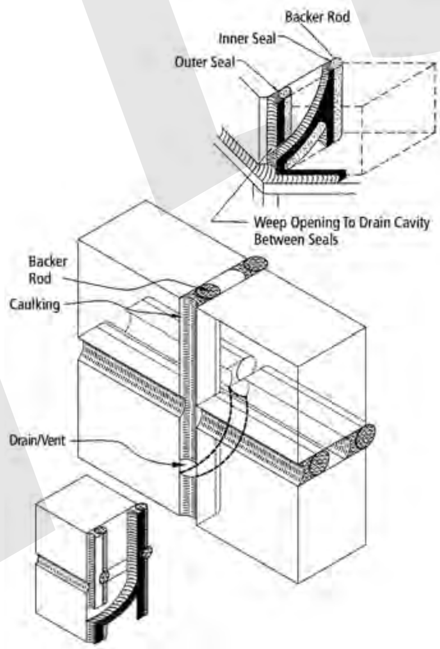

Fig. 9-17: Example of two stage joint system.

9.5.6 Solar and ultraviolet radiation

Unlike most other cladding materials precast concrete itself has an inherent resistance to UV degradation. However, the jointing system between panels and other materials is likely to be susceptible to UV damage depending on its exposure and the sealant product used. In a two stage joint system the outer sealant will be affected by UV, but the concealed inner sealant is likely to be unaffected. In that case the outer sealant would require periodic maintenance and replacement.

9.5.7 Noise insulation

The aim with any acoustic design should be to provide a satisfactory environment so that any desired sounds are clearly heard by the listener and any unwanted sounds (noise) are satisfactorily blocked. In tall buildings it is normally the insulation of external noise from the interior of the building that has to be considered in the design.

Large reductions in external sound levels can often only be achieved by continuous impervious barriers. Airborne sound transmission loss in wall assemblies is a function of their weight, stiffness and vibration damping characteristics. Weight is concrete's greatest asset when used as a sound insulator. National Codes and Regulations often specify minimum concrete wall thicknesses to achieve their necessary sound insulation requirement. If the minimum thickness is provided then precast concrete walls do not require additional treatments in order to achieve adequate sound insulation. In cases where lesser thicknesses than those specified in the Code are used further sound insulation can be obtained by using attached layers of gypsum board or other suitable materials.

The performance of previously adequate insulation barriers can be significantly reduced by flanking noise, i.e. noise that reaches the interior via paths other than the main barrier. Common flanking paths are at windows and service outlets. There can also be leakage at the junction between exterior cladding panels and floor slabs. It is critical that this gap is sealed to ensure acoustic integrity and provide the fire stop between floors.

9.5.8 Fire resistance and control

Precast concrete members are inherently non combustible and can be designed to meet fire resistance requirements specified by the national Code. However, both fire resistance and containment must be considered in the fire strategy. Fire resistive construction should contain the fire in the room or compartment where it starts. Floors, walls and roof surrounding the compartment must act as fire barriers.

It is normally the transmission of fire and smoke between floors that is the main issue to consider with precast concrete cladding. Since precast cladding panels are hung on the outside of the building frame then fire and smoke seals must be provided at slab level between the inside edge of the panel and the slab edge. Any steel connections providing panel support should also be suitably fireproofed.

9.5.9 Structural design

Most precast cladding panels in tall buildings are non loadbearing and transfer their own self weight and any imposed dead loads, such as windows, to the main structural frame. They are also designed to transfer lateral forces, such as wind and seismic, applied directly onto the panel surface. Loadbearing panels should only be considered in the lower few storeys where the transfer path to the foundation is relatively short.

Non loadbearing panels should also be designed to accommodate expected movements. Potential differential movements between panel and supporting frame must not be allowed to generate unintended restraint to the panel and overstress both connections and the panel itself. Relatively thin panels should be checked for excessive bow or deflection that may distress joints and cause leakages into the building. As the height of a building increases the cumulative movements at the top of the structure also increase. Non structural components at the building interior must be detailed to allow for such volume change movements of the exterior panels.

Panels should also be designed to allow for expected deformations. These can be itemised as follows:

- Deflection
- Bowing;
- Frame shortening.

Deflection of a panel support is a function of its stiffness. Several panels supported on a slab edge between columns will follow the deformed profile of that edge resulting in possible unintended restraint developing or compressions and tensions in the joints as shown in Fig. 9-18. This problem can be overcome by spanning between columns with a single panel or stiffening the slab edge to minimise deflection.

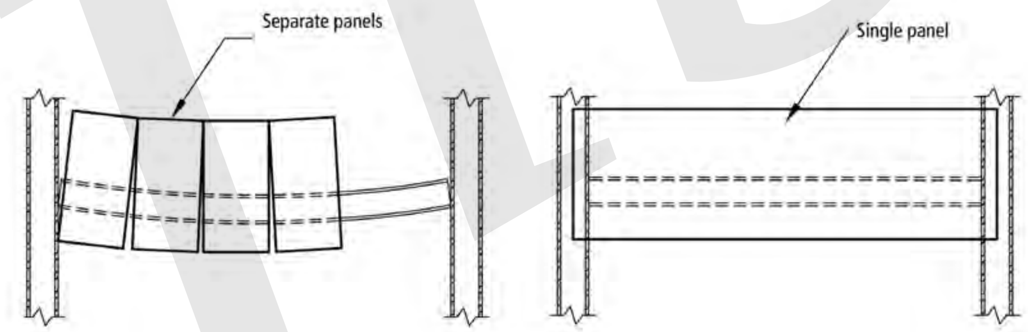

Fig. 9-18: Deflected position of supporting member due to weight of panels.

Allowance should be made in the top restraint connection to accommodate any vertical movement of the panel due to deflection of its support.

Bowing generally develops because of temperature differences between the external and internal faces of a panel after installation. Panels generally bow outwards and if supported in a manner that permits bowing will not be subject to additional stresses due to bowing. However, if restrained laterally along its length then bowing restraint stresses will develop. Non loadbearing panels that have windows may develop stress concentrations around the opening through restrained bowing as shown in Fig. 9-19. Additional reinforcement around openings should be provided.

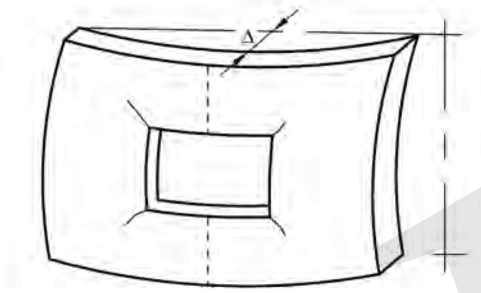

Fig. 9-19: Panel bowing.

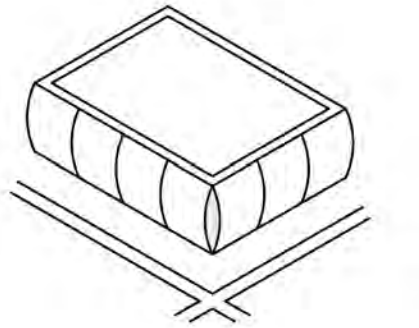

Fig. 9-20: Corner bowing.

Bowing of panels in a single plane may prove not to be significant, however at changes in direction (corners) unacceptable separation may occur (Fig. 9-20). Damage to joint sealants is possible at these loactions and it may be advisable to restrain bowing at corners with connectors between the adjoining panels.

Non loadbearing panels should be designed and detailed so that they do not restrain frames from lateral translation. If this occurs then the panels are likely to act as unintended shear walls causing significant diagonal compression and may become overstressed. To avoid this panels should have connections that allow frame distortion (top and bottom connections only, no connections along the vertical edges).

Vertical shortening of concrete columns is important in tall buildings. Shortening of concrete columns can be as much as 150 mm for a 40 storey concrete building (refer to Chapter 6 for axial shortening of columns). Each of the storeys undergoes a proportional shortening that is cumulative for the height of the structure. At each level the differential shortening between that level and the floor above is a small amount, but nevertheless frame shortening must be considered in the development of the fabrication elevations. If this is not done then there is a risk that there will be misalignment at connections and the panels may not match the frame. Panel connections must line up at time of installation and there must be sufficient space at the joints to accommodate future movement.

Differential shortening occurs when two adjacent columns have different loads but are similar sizes. This is particularly relevant to corner columns, which are likely to carry a smaller load than the columns nearby along the general elevation.

In situations where adequate clearance is not provided between the cladding panels and the support structure, or if the top connectors are not slotted to allow vertical movement, loads from the nearby floor can be imposed on non loadbearing panels. Moments and shears can develop in beams across window openings as unintended support is provided to the floor as shown in Fig. 9-21.

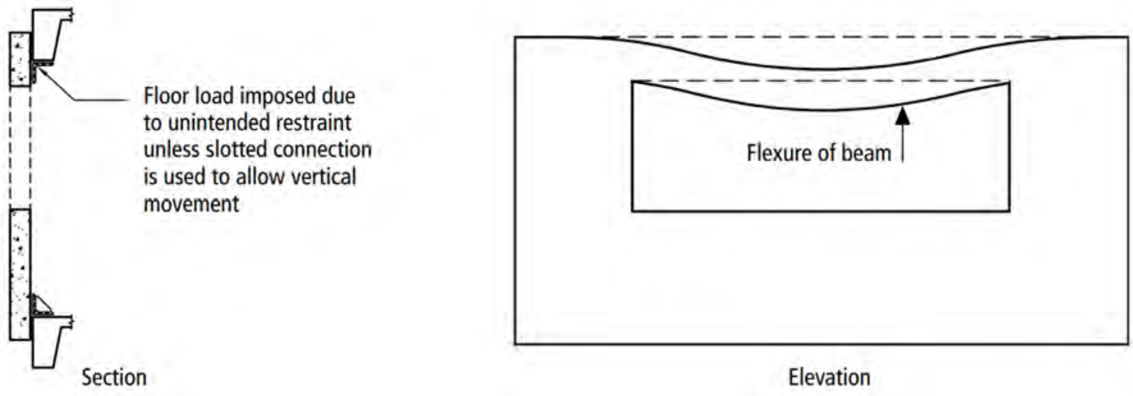

Fig. 9-21: Unintended floor loading onto cladding panel.

Precast concrete panels are often used as covers over columns and beams to achieve architectural expression, special shapes or fire rating. These panels are generally supported by the column or beam they are covering in a similar manner to non load bearing wall panels but in a more localised manner. The cover panels are relatively narrow and can extend both vertically and horizontally a greater distance than a single storey or bay respectively depending on site and logistical constraints.

Mullions are also relatively narrow vertical elements that separate glazed areas and resist wind loads applied to them from the supported glass. They must be sufficiently stiff to satisfy deflection limits set by the window supplier.

9.5.10 Durability

Precast concrete wall systems have been proven to be one of the most durable building envelope components available. To enable the concrete panel to reach, and exceed, its design service life the concrete mix must be developed to suit the exposure conditions and take account of severe weather, including freeze-thaw cycles, and chloride exposure. National Codes will give guidance on concrete mix constituents, proportions, and reinforcement cover to suit exposure conditions and design service life.

Sealant joints have a shorter service life than concrete, but careful detailing should allow easy access and regular maintenance.

10. Precast Concrete in Seismic Zones

10.1 Introduction

Designers are familiar with structural analysis and details associated with gravity and wind loads. However, forces arising from earthquakes are different as they result directly from the shaking motion of the ground that supports the building. The response of a building to these motions is dependent on the characteristics of the building itself, its foundation, and the earthquake ground motion[15,21].

Earthquakes generate large magnitude horizontal and vertical ground movements of short duration that must be resisted by the building structure without causing collapse, and preferably without causing significant damage. The target performance (minimum by law) for a "design level earthquake" refers to Life Safety, which implies a substantial amount of damage that would possibly be difficult to repair. When a seismic motion shakes beneath a structure its base will move and accelerate cyclically (back and forth) with the ground whilst the superstructure's inertia will resist the motion, consequently generating equivalent inertial forces, and cause the building to distort. This distortion wave travels along the height of the structure. The continued shaking of the base during the earthquake causes the building to undergo a complex series of oscillations.

The frequencies of the structure, associated with first and higher modes of vibration, can cause it to resonate with the frequencies of the ground shaking to amplify the effect. The increased height of tall buildings magnifies these effects due to resonance at higher modal frequencies.

As ground shaking during an earthquake is randomly directed then a structure shaped and designed so that it has equal resistance (both in terms of strength and displacement / ductility) in any direction is an optimal solution. Buildings that are symmetrical in plan with minimum torsional eccentricity (the shear centre is close to the centroid of mass) behave better in earthquakes than those that are asymmetrical with centres of mass and rigidity far apart. This latter case would amplify distortion through consequent torsional effects. Closed sections such as boxes and tubes also demonstrate improved behaviour when compared with open sections.

Codes generally require that structures that could be susceptible to earthquake loading should have simple and regular forms both in plan and elevation, and if necessary this should be achieved by subdividing the structure by joints into dynamically independent units.

The earthquake ground motion parameter most used in structural analysis is the design spectral acceleration, related to the period of oscillation of the equivalent single degree of freedom structural system oscillator, that is then modified by the soil characteristics in the immediate locality of the building. At any point, the ground acceleration may be defined by horizontal components along two orthogonal axes and a vertical component. There may also be rocking and twisting components.

This chapter was mainly authored by Cleland, Jones, Pampanin

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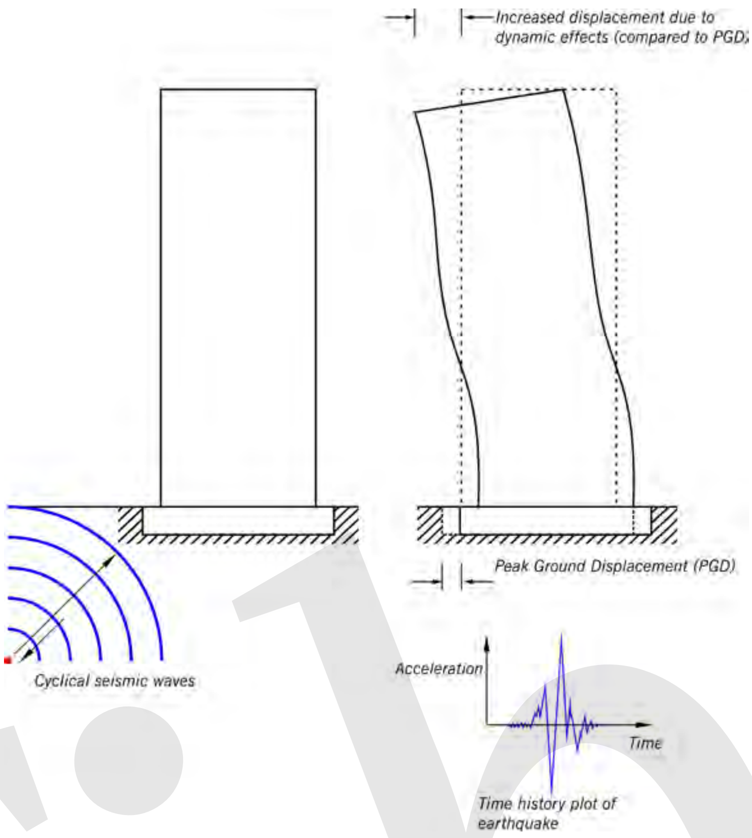


Fig. 10-1: Seismic waves (that act back and forth)-Simplistic distortion of superstructure during earthquake.

The structural properties of a building, such as its mass, strength, stiffness and damping and their distribution through the building, together with the characteristics of the ground motion and the foundation material govern the building's seismic response. Once the earthquake actions applied to the structure have been derived most Codes typically give a choice of four methods of analysis. These are:

- Equivalent lateral force procedure (linear static);
- Modal response spectral analysis (linear dynamic);
- Push over (Nonlinear static);
- Nonlinear response history analysis (Nonlinear dynamic).

The method chosen generally depends on the type and complexity of structure, as well as the objectives of the analysis. A modal response spectrum analysis is commonly used to predict structural response to strong earthquakes for tall buildings. For structures that exceed the limits of permitted structural systems for the simpler analyses, it may be necessary to use nonlinear response history analysis which subjects a nonlinear model of the structure to the application of earthquake records derived for the site. However, for purposes of illustration an equivalent linear method, the Equivalent Lateral Force Method, is used to show, as a general example, the relative effect of height in tall buildings on the forces and moments generated by applied seismic horizontal force.

The seismic force distributed to each level is proportional to the distance from the base and the mass at the level considered. The seismic force F_i applied to each storey can be estimated from the following relationship:

$$F_i = F_b \left[\frac{Z_i \cdot m_i}{\sum Z_j \cdot m_j} \right] \tag{10-1}$$

Where Z_i is the height of the mass mi above the level associated with the base shear F_b. The set of seismic forces F_i derived in this manner in each horizontal direction results in a static load case, each force being the resultant of the inertia loads applied to a floo .

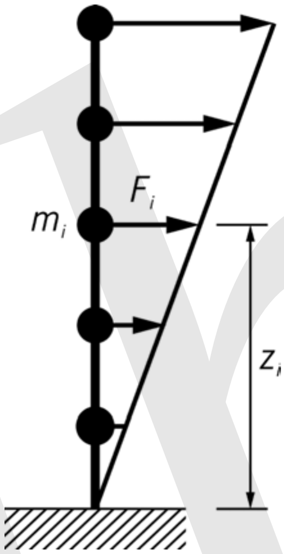


Fig. 10-2: Example of variation of seismic force with height.

From this relationship the magnitude of the applied horizontal seismic force distributed to each level increases with height, i.e. if there is a rigid connection at the base then the cantilever moment generated will increase disproportionately when compared to wind generated moments. Wind and seismic forces are often mistakenly thought as being similar because Codes often specify both force types as equivalent static forces. Whilst both wind and earthquake derived forces are characteristically dynamic, i.e. they vary with time, there is a substantial difference in the way they are transferred into the structure. Wind loads are externally applied directly and are proportional to the external surface of the structure. As shown earlier, seismic forces result from distortion / shaking from the earthquake motion itself and the inertial resistance of the structure. The magnitude of the seismic force is a function of the mass and frequencies of the structure rather than its exposed surface in the case of wind loading.

Eq. 10-1 distributes force F_b, the horizontal seismic base shear force in each principal analysis direction. F_b is a global force quantity and is simplified in Eq. 10-2.

$$f_b = K_s \frac{m}{R} \tag{10-2}$$

k_s is a factor that relates to the design spectral response of the ground and will vary for each building and its location. Methods for derivation of this factor, which may have several constituent parts, are given in National Codes.

m is the total seismic mass of the building above the top of foundations or rigid raft or basement. It largely consists of the dead load plus a portion of the live load in a seismic load combination as defined by Code requirements.

R is a reduction factor (referred to as behaviour factor *q* in Eurocode[6]) known as the response modification factor in US Codes[7,8,9] and relates to the type of seismic force resisting system (SFRS), or Lateral Force Resisting System (LFRS), and material chosen for the building, e.g. frames, walls, braces, concrete, steel, timber, as well as the dissipation capacity and ductility detailing provided. It can vary between 1 and 8 and should be taken as a maximum value and not as a target.

For materials such as concrete that are inherently brittle, and lack inelastic deformation capacity or ductility, it is critical that special reinforcement and connection detailing is provided to assure a ductile response to lateral forces, with adequate mechanisms at both local (sections and connections) and global (structural system and building layout) levels. Inelastic displacement / ductility is the ability of a structure to sustain gravity loads as it deforms laterally in the plastic domain beyond the yielding point. Notably, ductility is associated with material and structural element damage, considered an inevitable compromise to resist strong earthquake ground motion without having to uneconomically build a "bunker" for more frequent response in the elastic range. In addition to material damage associated with the maximum displacement of the building (typically termed as inter-storey drift, i.e. distortion of each level, measured as the relative displacement of two adjacent floors divided by the storey height), in the past two decades attention has been given to the residual or permanent (irrecoverable) deformations which could remain once the structural shaking subsides.

As a general principle, if inelastic deformation can be restricted to discrete ductile connections in precast concrete acting as seismic "fuses", then it may be possible to limit the amount of damage and possibly repair the connection rather than the precast element itself.

The energy dissipation characteristics of precast frames (beams and columns) and walls depend mainly on the behaviour of the connections. Properly designed precast walls with large openings, such as windows, can still behave in a flexu al and ductile manner like frame systems or coupled walls, if they have coupling beams above and below the opening.

Architectural precast concrete facades can also consist of earthquake resistant structural elements. This solution can be achieved through shear walls or spandrel beams acting with closely spaced columns or mullions which provide both the exterior skin and the structural tube. These elements can be used to provide a seismically efficient structure that is symmetrical in plan. However, governing codes may limit the height or even the application of the system itself in regions of higher seismic risk.

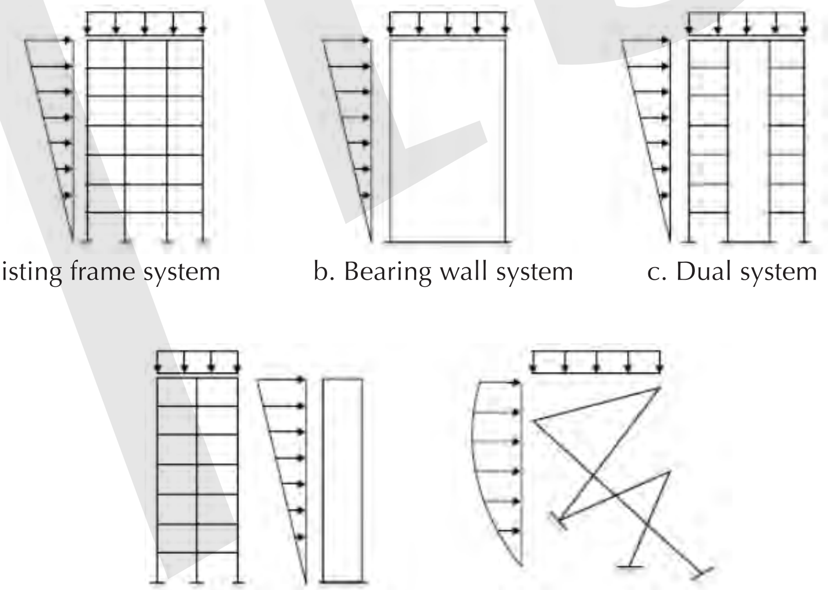
It is advisable to limit the damage to non-structural elements in a flexible building by uncoupling them from the structural system so that they are not subjected to as much deformation as the supporting structure. Therefore, connections between individual elements and their supports will have to be designed and detailed to accommodate large distortions without fracture.

The structural analysis of a tall building subjected to earthquake motions will inevitably involve complex computer based numerical models to derive the internal design actions, moments, shears, axial loads, and deformation demands (curvatures, rotations, displacements) related to its structural components and connections. However, following simple guidelines at the start of the design process and ensuring that the connection details reflect the analysis assumptions can mitigate the damaging effects of the earthquake. In some International Codes Performance Based Seismic Design (PBSD) approach combined with non-linear time history analyses can also be used for structural systems outside prescribed Code limits and to demonstrate higher performance levels[45].

10.2 Seismic Force Resisting Systems

In seismic conditions reinforced concrete buildings are designed to provide energy dissipation capacity and an overall ductile behaviour. This is ensured if the ductility demand globally involves a large volume of the structure and is distributed to different elements and locations at all storeys. Capacity design (hierarchy of strength) principles should be followed to provide protection against brittle failure mechanisms at both local and global levels, e.g. beam plastic hinges developing rather than column plastic hinges to ensure a beam-sway mechanism in a framed structural system.

The term "Seismic Force Resisting System" (SFRS) is used in US Codes to describe basic structural systems that may be used to resist earthquake forces. Five basic types of SFRS for structural concrete buildings are commonly used and are illustrated in Fig. 10-3:



a. Moment-resisting frame system b. Bearing wall system c. Dual system

d. Building frame system e. Undefined systems

Fig. 10-3: Seismic Force Resisting Structural Systems.

Each system consists of either beam and column frames, shear walls or a combination of both. There are further subdivisions within each type of varying strength and ductility. Tables are provided that indicate the application of each SFRS to each US Seismic Design Category (SDC), the detailing requirements, system limitations, response modificatio factor *R* and other factors relating to element design forces and design storey drift.

R is intended to account for differences in the inelastic deformability or energy dissipation capacity of the various structural systems. It reflects the reduction in structural response caused by damping, overstrength and inelasticity. In determination of values for *R* for each system consideration was given to the following characteristics:

- The capability of the system to go beyond the elastic range and the related energy dissipated, together with the stability of the vertical load carrying system during inelastic response due to maximum expected ground motion.
- The effect of full or partial failure of vertical elements in the SFRS on the global vertical and lateral stability of the structure.
- The inherent redundancy of the SFRS that would allow inelastic redistribution without overall failure, i.e. localised failure would not lead to a progressive collapse.
- In dual systems, the capability of the secondary system to maintain vertical support, redistribute lateral loads and stabilise the system when the primary system suffers significant damage at maximum deformation response.

In the Eurocode 8 approach to seismic building design the general methodology is similar but with different terminology and formulae. The structural systems identified in Eurocode that could be applicable to tall buildings are:

- Frame system: Lateral load system in which resistance to horizontal and vertical loads is provided mainly by space frames whose shear resistance at the base of the building is more than 65 % of the shear resistance of the total structural system.
- Wall system: Lateral load system in which resistance to horizontal and vertical loads is provided mainly by vertical structural walls whose shear resistance at the base of the building is more than 65 % of the shear resistance of the total structural system.
- Ductile wall system: A system where walls are fixed at the base that is designed and detailed to dissipate energy in a flexu al plastic hinge zone just above base level that is free of openings.
- Coupled wall system: A system composed of two or more single walls connected in a regular pattern by coupling beams able to reduce by at least 25 % the sum of base bending moments of the individual walls if working separately.
- Frame equivalent dual system: Dual system of frame and walls in which the shear resistance of the frame at the base of the building is greater than 50 % of the total shear resistance of the structural system.
- Wall equivalent dual system: Dual system of frame and walls in which the shear resistance of the walls at the base of the building is greater than 50 % of the total shear resistance of the structural system.

In Eurocode 8, these structural systems influence the behaviour factor q, which in turn affects the design base shear F_b. The behaviour factor q is a structure dependent factor used to reduce seismic design forces in a similar manner to R in the US Codes. It is a function of the ductility of the structure and the ratio of ultimate lateral strength to lateral strength at yield. It is also affected by the regularity of the structure.

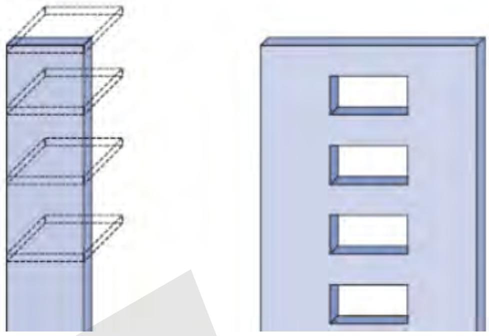


Fig. 10-4: Example of Ductile and Coupled Walls.

Structural systems for tall buildings in seismic regions can often be outside the prescriptive limits imposed by National Codes. Performance Based Seismic Design (PBSD) is a methodology that has been developed to overcome this challenge. This method focuses on inelastic behaviour and predicts, through non-linear time history analyses, member strains and member rotations and ductility demand at the anticipated level of building drift. Verification of analysis and component behaviour is achieved through testing of large scale models and sub-assemblies.

PBSD allows for design flexibility and offers opportunities to improve building performance and encourage innovation. More effort is generally required at analysis and design stages, with verification of building performance required at multiple seismic demand levels using linear and advanced non-linear analysis. PBSD would generally be used for the following reasons:

- Where the tall building structure is outside prescribed Code limits, e.g. the height of the building is greater than the prescribed limit.
- The SFRS is an innovative system not prescribed by the Code.
- High strength materials and mechanical devices are not prescribed by the Code.
- Reduction of structural and non-structural damage is targeted through improved seismic performance.

Design inputs required by PBSD are extensive and require substantial knowledge of non-linear seismic design, building performance and numerical modelling. This has not inhibited the design of tall building structures but on the contrary has led to highly efficien tall building designs that would not be possible using traditional Code prescriptive design methods.

10.3 Precast Concrete Elements and their Connections

10.3.1 General

To dissipate energy resulting from seismic actions through inelastic deformations of the structural members critical regions are chosen and arranged with the potential to behave as plastic hinges. A flexu al ductility is therefore assured in those critical regions that provides enough rotational capacity in the post elastic phase when undergoing repeated reversed loading. This is achieved through the application of specific design criteria and structural details. Through hierarchy of strength and capacity design principles designs should ensure that ductile modes of failure (e.g. flexure) should precede brittle failure modes (shear).

Precast concrete seismic systems have been broadly divided between emulative and non-emulative types. An emulative precast system can be defined as the equivalent of its cast insitu monolithic version. Generally, precast elements can be produced with the same dimensions and reinforcement as their cast insitu counterparts. The difference is at the wet connections between precast concrete elements. Plastic hinges can, however, be formed at these precast connections through suitable details.

Non emulative precast systems use the unique properties of precast concrete elements interconnected predominantly by dry connections (jointed precast). The Precast Seismic Structural Systems (PRESSS) research program was initiated in 1991 as part of a joint US-Japan effort to achieve broader acceptance of precast concrete construction and develop new concepts and technologies for precast concrete in seismic regions. Their research has concentrated on non-emulative precast systems and has led to many advances in the understanding of the seismic behaviour of non-emulative precast concrete framed and walled structures. Case study 12.12 is an example of a non-emulative precast structural system based on jointed ductile connections (PRESSS technology).

10.3.2 Emulative systems

Emulation of monolithic behaviour typically relies on the formation of plastic hinges to dissipate energy during an earthquake. These are often located so that there is a strong column-weak beam mechanism under seismic loading. Wet connections are commonly used that rely on reinforcement splicing methods to connect precast or precast to cast insitu members. The splicing closure is normally filled with insitu concrete or high strength grout.

Because emulative systems reproduce monolithic solutions the benefits of precast concrete are unlikely to be fully realised. Emulative solutions often give comfort to structural engineers as they are familiar with monolithic design and detailing. However, there are also disadvantages:

- The advantages of precast are not fully exploited.
- The erection speed is slowed due to wet jointing and splicing.
- The system is only partially precast due to joint complexity.
- Traditional ductile systems lead to unavoidable damage, with high costs for repair and possible demolition.

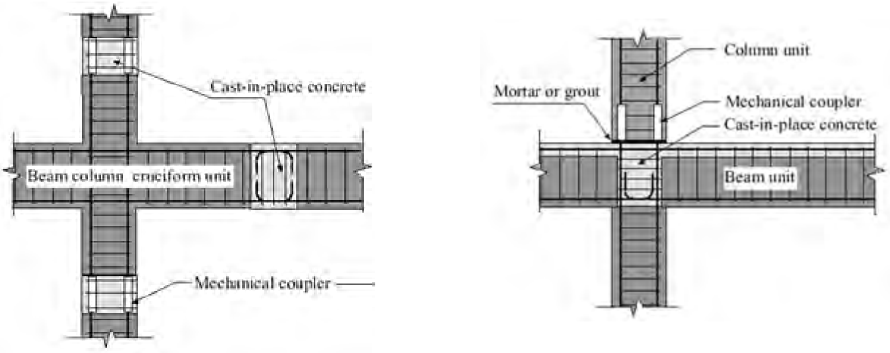


Fig. 10-5: Traditional emulative details at column-beam interface, from *fib* Bulletin 27.

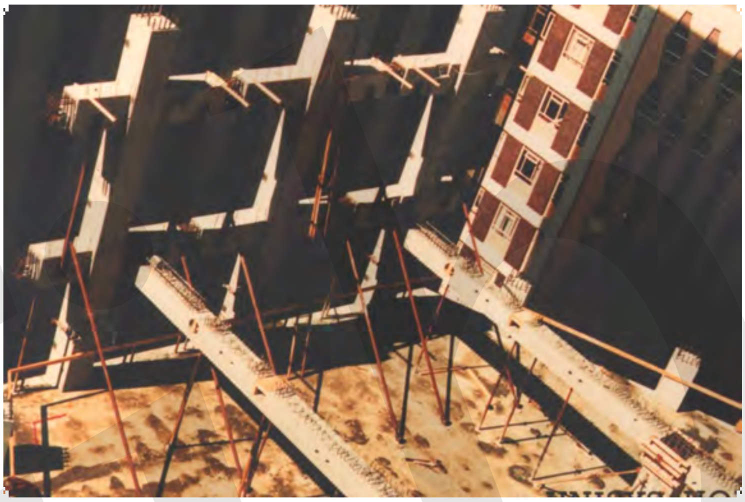


Fig. 10-6: Multi storey columns and beams in emulative system, Unisys House, Wellington, New Zealand, courtesy of A.O'Leary, from *fib* Bulletin 27

Fig. 10-5 illustrates traditional emulative details for a beam-column sub-assembly. Couplers are used to maintain reinforcement continuity in the columns whilst reinforcement splices and insitu concrete closures are used in the beams. In one example the stitches are located outside the beam-column intersection at points of approximate zero moment under lateral loading (also shown in Fig. 10-6 but with multi storey columns). A precast cruciform element is used to achieve this. In the other example the insitu stitch is at the beam-column intersection.

10.3.3 Non emulative systems

The PRESSS research has led to the development of advanced (dry) jointed ductile connections for both framed and walled systems that use the principles of unbonded post tensioning (PT) in precast concrete elements. The hybrid frame connection is one example that relies on PT technology and controlled rocking to dissipate energy.

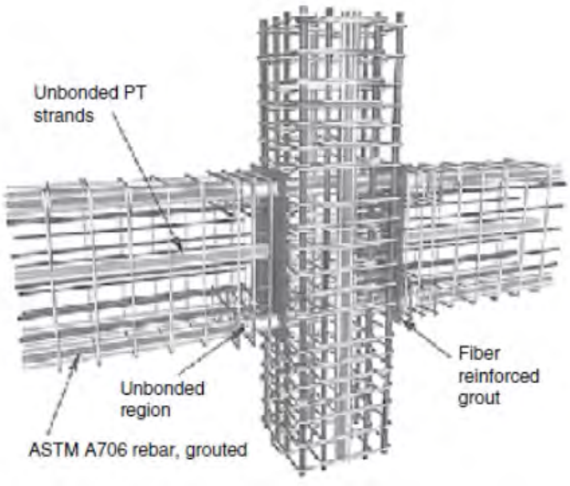


Fig. 10-7: Hybrid frame connection, US Example (from the PRESSS Research Program, modified after Suzanne Nakaki).

Unbonded PT strands pass through a duct in the centre of the beam and then through the column. The top and bottom mild steel reinforcing bars must be debonded for short lengths in the beams close to the column interface to reduce the high cyclic strains that would otherwise occur. In the "hybrid" system the amount of mild steel reinforcement and PT tendons are proportioned so that the frame recentres itself after a major seismic event. The resulting hysteretic response, named "Flag-shape", shows a peculiar recentring-dissipating behaviour.

In addition to the design moment capacity at the connection level, PT provides through friction the beam seismic shear resistance, potentially eliminating the need for corbels (in US Codes, but European and New Zealand Codes require a dedicated shear key for factored gravity loading), and top and bottom mild steel reinforcement provide ductility and energy dissipation in the connection region by yielding. The PT strands behave elastically, acting as restoring springs, throughout the seismic event whilst the mild steel is yielding. The self-centring capabilities of the PT allows the structure to return to its original position with negligible residual displacements.

There are a few variances to the hybrid frame connection that use the same self-centring principle. One such example is the use of storey height columns supporting pretensioned continuous beams with lengths of the prestressing strand debonded (Pretensioned Frame Connection) so that they remain elastic as the frame displaces laterally under seismic action.

Considerable research has also been carried out in the use of PT in precast concrete walls. An unbonded PT precast coupled shear wall system has been developed by the PRESSS Program and subsequent research developments that comprises two or more precast wall panels, connected along the vertical joints by energy dissipating shear connectors, e.g. in the form of U shaped Flexural Plates (UFP), and anchored to their base with unbonded PT tendons along the vertical centreline of each panel as shown in Fig. 10-8.

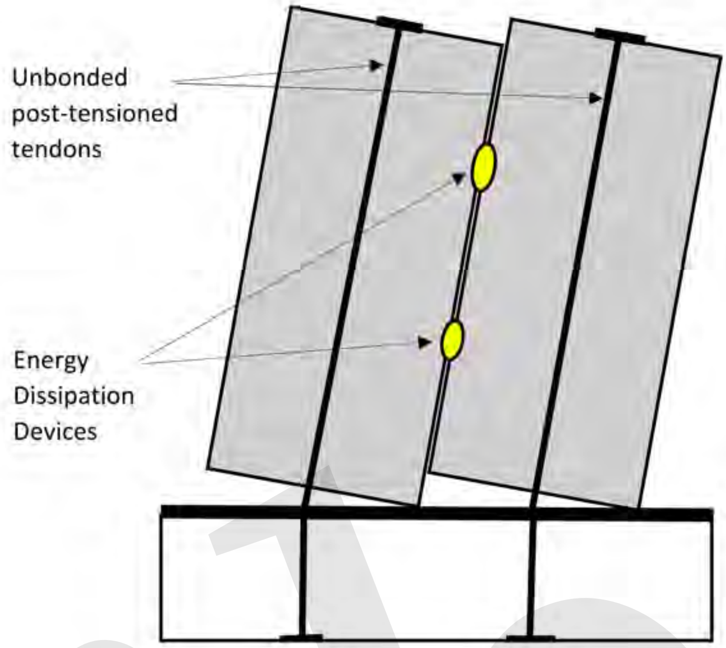


Fig. 10-8: Illustration of PT coupled shear wall system, from the PRESSS Research Program, courtesy of Suzanne Nakaki.

Each wall has PT tendons at its centre that act as an elastic restoring spring. There can also be non-linear inelastic yielding reinforcing bars or alternatively dissipative devices at the base of the wall where the rocking motion occurs. Both the PT and the conventional reinforcement contribute to the overall flexu al strength of the panel. When adopted, the mild steel reinforcement yields to dissipate energy whilst the unbonded tendons remain elastic to provide a positive restoring force and centre the panel after inelastic response. The elastic restoring component is proportioned to be larger than the yielding component so that more than half the total resistance is derived from the PT. This ensures a favourable rocking response and a consistent tendency to re-centre.

The level of damage suffered by a conventional concrete wall system compared to a single or coupled PT shear wall system after an earthquake is significant. Fig. 10-9 illustrates the trademark plastic hinge development in an emulative system with the expected likely damage around the base hinge. The PT wall system will still have reinforcement that has yielded but with little visual damage. Further refinement of the PT rocking-dissipating frame and wall systems has led to the implementation of external replaceable dissipaters, referred to as "plug and play" dissipaters as shown in Fig. 10-10, consisting of "fuse" shaped yielding bars inserted in anti-buckling restraint tubes that allow easy maintenance, repair and dismantling of the structure.

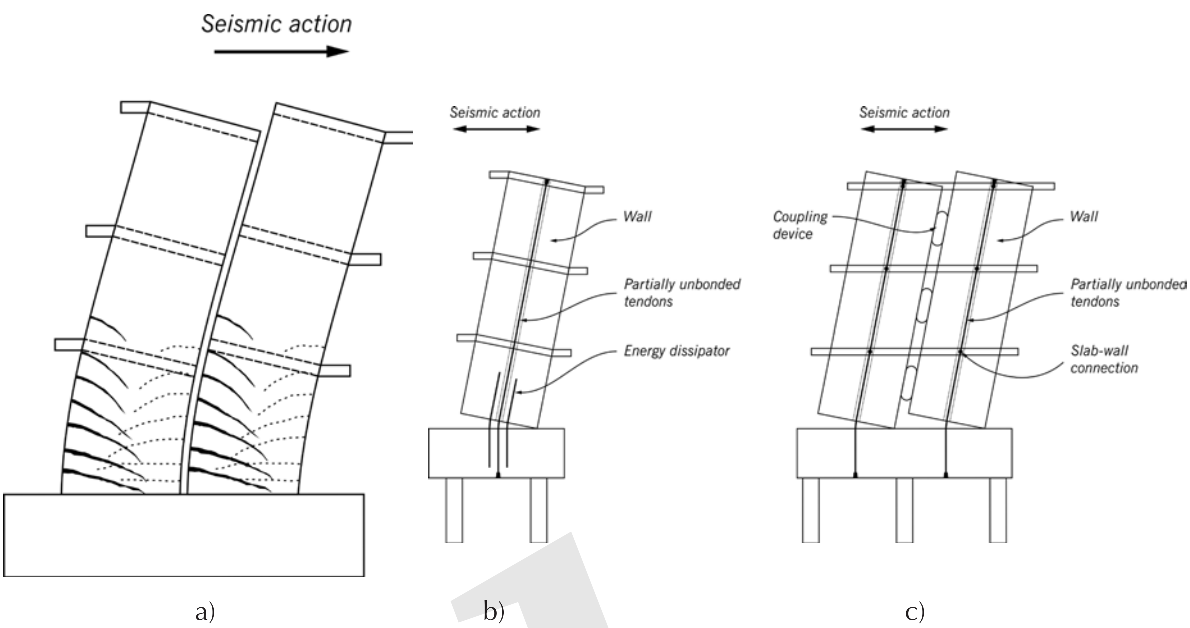


Fig. 10-9: Comparison between emulative (a) and non-emulative rocking-dissipative unbonded post tensioned (b & c) wall systems, cantilever and coupled wall examples, from *fib* Bulletin 27.

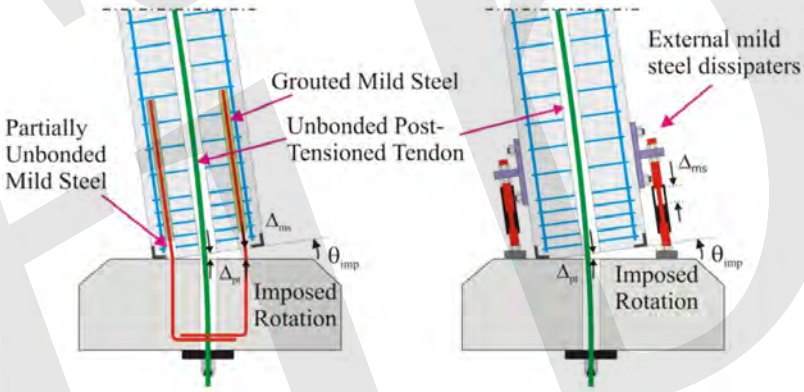


Fig. 10-10: Progression from internal mild steel bars to external "Plug and Play" dissipaters (courtesy of Marriott, S. Pampanin, A. Palermo, 2007, "Seismic Design, Experimental Response, and Numerical Modelling of Rocking Bridge Piers with Hybrid Post Tensioned Connections". University of Canterbury Report, ISSN-0110-3326 (2007-01) NZ.

The self-centring properties and near absence of residual deformations after an earthquake that are achieved through the elastic performance of PT, and associated precast concrete systems, have major implications for seismic design and, in turn, sustainable construction.

A typical Code compliant building, designed using force methods and emulative systems, would be considered to have performed well if it remained standing and allowed safe evacuation of its occupants after an earthquake (as per the definition of Life Safety Limit State), even if substantial damage had been incurred. After such events, the SFRS would generally have enough remaining strength to stay in service, however, the residual drift and damage to both structural and non-structural components would most likely make it unfit for continued use, requiring substantial and expensive (when feasible) interventions. Permanent deformations can also seriously interfere with the functioning of doors, windows, elevators, and other fixtures. In such circumstances, although both drastic and undesirable, the extent of damage may suggest demolition and rebuilding a more cost effective option. Obviously, the continued long-term functioning of the building after a large earthquake is the desired sustainable outcome for the future, particularly in the case of expensive tall buildings. Low-Damage technologies based on non-emulative design and self-centring structures can achieve this target. This would mean that precast concrete has evolved to become the preferred choice as an integral part of a Seismic Force Resisting System.

It is worth noting that, along with the development of innovative low damage solutions for precast concrete systems in seismic regions, the design of PRESSS technology buildings tends to adopt the innovative displacement based design approach (Priestley 1998: Priestley et al 2007) instead of the traditional force-based system widely used in National Codes and described previously.

PRESSS researchers realised that force-based design methods, in addition to their intrinsic and emerging limitations that were being revealed from the late 1990s onwards, did not adequately represent the behaviour of jointed precast systems. Force methods rely on an initial elastic period that is difficult to compute in a jointed system that largely relies on its connections for its flexibilit , but also has little influence on the post-elastic behaviour of the structure. It was also felt that the *R* values in the Codes were not intended for use in non-emulative systems. A direct displacement-based procedure was therefore chosen for their research. This method is based directly on an inelastic target displacement and effective stiffness. The target structural displacement is a more realistic indicator of resultant damage than the forces derived from traditional methods and is determined from an allowable inter-storey drift and an effective stiffness approximated to the building secant stiffness corresponding to its expected response mode.

This approach achieved a design base shear lower than a comparable force-based design, but more importantly is a more reliable control of the drift and damage resulting in the structure.

10.4 Lateral Diaphragms

10.4.1 General

The seismic force resisting system (SFRS) of a building consists of a three dimensional assembly of elements that transmit loads and forces from their point of origin to the bearing strata beneath the foundation. This system consists of horizontal and vertical structural elements. When the building is subject to seismic actions the horizontal elements (roof and floor slabs) bend in their own plane and act as diaphragms to transmit the forces horizontally to the vertical elements of the SFRS, and thereafter to the foundations. Together the horizontal and vertical elements combine as a system to provide a complete load path for seismic forces to fl w through the building. The diaphragms are an integral part of the SFRS and deserve significant attention during the design process. Should the diaphragm fail to transfer the inertia forces to the vertical seismic resisting structural systems (frames, walls, dual system) then the whole earthquake resistance would be impaired leading to catastrophic effects, as shown in recent earthquake events.

In most cases diaphragms utilise the floo -slab system. Concrete diaphragms can be conventionally reinforced or prestressed. In tall buildings in seismic zones diaphragms are likely to be either cast insitu concrete slabs (often PT) or insitu toppings overlaying precast slabs. Previous section 5.4.2 covers the general approach to diaphragms in non-seismic areas. However, conservatively, it is common practice to consider the continuous insitu topping alone as the lateral diaphragm in seismic design even though it may act compositely with underlying precast slabs to resist gravity forces for conventional strength checks. In seismic design for such cases the precast slabs would provide out of plane restraint only for the insitu topping diaphragm.

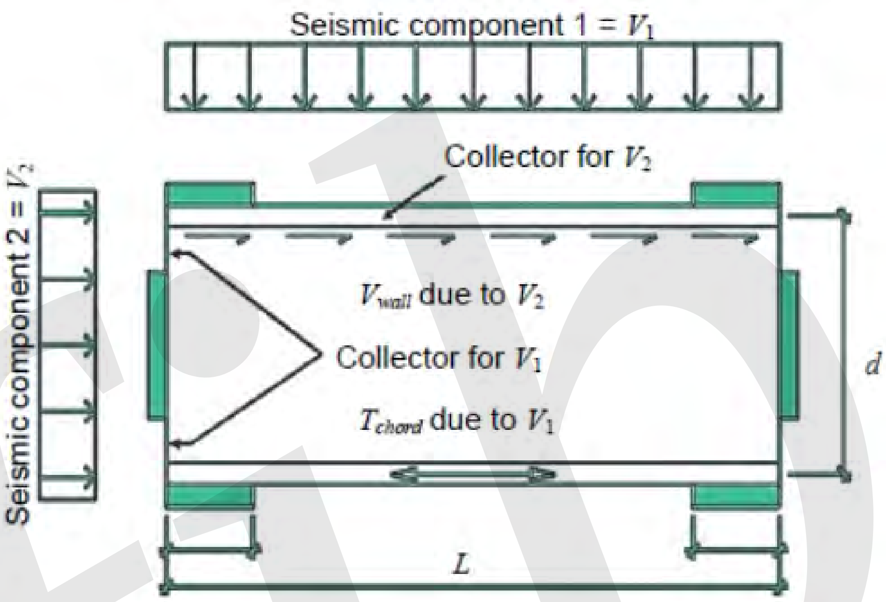


Fig. 10-11: Diaphragm chords and collectors.

Flexural strength is determined using the same assumptions applied to beams. When the reinforcement required for these areas forms a couple with corresponding reinforcement on the opposite side of the diaphragm to resist flexure it is commonly called chord reinforcement. When the axial forces develop a mechanism for transfer of the in-plane shear from a diaphragm to the vertical elements of the SFRS the reinforcement acts as a collector. Therefore, a chord resisting lateral forces in one direction may act as a collector for forces in the orthogonal direction as illustrated in Fig. 10-11.

10.4.2 Developments in diaphragm design in the United States[33,40]

In many seismic design standards for equivalent force design for diaphragms, the diaphragm design force has been derived from the base shear used to design the vertical elements of the SFRS. In the US, that base shear is taken as 2/3 of the inertial force from the Maximum Considered Earthquake (MCE) for the site with the further reduction by a response modification factor, *R,* that reflects the post-yield energy dissipation and structural resiliency of the vertical system. The implicit assumption of the reduction is that the SFRS need not be designed to behave elastically for the full energy of the earthquake because yielding will occur that will dissipate energy and reduce forces. However, designing the diaphragms to comparable levels of force does not assure that the diaphragm will not yield before the SFRS yields.

Under the historic code provisions in the US, the lateral forces for diaphragm design are determined by the following formula:

$$F_{px} = \left[\frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} W_i} \right] \tag{10-3}$$

where

F_{px} = the diaphragm design force at level x

F_i = the design force applied at level i

w_i = the weight tributary to level i

w_{px} = the weight tributary to the diaphragm at level x

The lower bound force is $F_{px} = 0.2S_{DS}I_e w_{ps}$ and the upper bound force is $F_{px} = 0.4S_{DS}I_e w_{ps}$.

The coefficient S_{DS} is the 5 % damped spectral response acceleration parameter at short period (0.2 sec.), adjusted by 2/3 of the mapped acceleration and modified by soil effect coefficients.

Based on a research program called Diaphragm Seismic Design Methodology (DSDM), new diaphragm design provisions have been developed in the US. The design force is determined considering the overstrength of the vertical SFRS and the energy from higher mode effects. The design force at level x is calculated using the following formula:

$$F_{px} = \frac{C_{px}}{R_s} w_{px} \tag{10-4}$$

where

w_{px} = the weight tributary to the diaphragm at level x. This weight need not include the weight of the elements of the SFRS at that level in the direction of the diaphragm force.

R_s = diaphragm design force reduction factor.

C_{px} = diaphragm design acceleration coefficient at level x which includes first mode with a modal contribution factor and an overstrength factor and higher mode effects with a model contribution factor.

For the use of this force approach with precast concrete, the diaphragm response modification factor, R_s, is a separate factor based on the deformation capacity of the diaphragm details. The values are assigned for three separate levels of detailing to achieve response targets.

The elastic design option (EDO) and the basic design (BDO) option are included to permit use of mechanical or welded connections at joints in diaphragms assigned to low and moderate seismic design categories. The performance target of the EDO is for the diaphragm to remain elastic during the maximum considered earthquake (MCE). Since the connections will remain elastic, there is no deformation capacity requirement for the connections used. This option is limited to low seismic design category or, with an additional force penalty, to the moderate seismic design category. The target of the BDO is for the diaphragm to remain elastic for the design basis earthquake (DBE), but not necessarily for the MCE. Connections used in this option must have a tested deformation capacity of 15 mm while carrying cyclic loading. The BDO is permitted only in low and moderate seismic design categories, unless used with an additional force penalty in high seismic design categories.

The reduced design option (RDO) considers a performance target that permits some inelastic behavior below the DBE. RDO requires mechanical connections with tested deformation capacity of 30 mm while carrying cyclic loading. Mild steel reinforcement used in cast-in-place concrete topping is deemed to satisfy the high-deformability requirements for RDO, and it is expected that floor systems in precast concrete tall buildings using current conventional precast concrete floor systems will have a cast-in-place concrete topping to complete the floor system and the diaphragm The benefit of the lower lateral force coefficient may encourage its use in regions of lower seismic risk that would otherwise permit the EDO or BDO options to be applied.

The effects of the updated diaphragm design provisions will vary depending on the type of SFRS selected, the building configu ation, and on the height of the building. To make these comparisons, three different building heights are compared. The prototype building is simplified for this comparison.

The building plan is selected as approximately 30 m by 60 m. Each floor (and roof) is assumed to weigh 12,000 kN. The floo -to-floor height is taken as 3.5 m. Comparisons between the traditional diaphragm forces and forces determined using the new provisions can be made for 3-storey, 14-storey and 30-storey buildings. This gives a range for low-rise, to threshold high-rise, and then moderately tall structures. The seismic hazard was chosen to be moderate (Seismic Design Category (SDC) C) and high (Seismic Design Category D). To be able to compare similar systems that can be applied at the different heights, dual systems of combined walls and frames are chosen for the SFRS.

For low-rise buildings, the first mode contribution factor is 1.283 and the higher mode contribution factor is 0.34. The diaphragm forces for this example for moderate seismic risk increase between 1 ½ and 2. For the high seismic risk case, the first level increases by a factor of 2, but the top-level increase is less than 1 ½ times the traditional method.

When tall buildings are considered, the higher mode effects are more significant. At 14-storeys, the first mode contribution factor is 1.395 and the higher mode factor is 0.66. The comparison for the SDC C case with intermediate frames and ordinary shear walls, the 1st level diaphragm force is larger than the force at the 0.8H level. At the base, the diaphragm force is increased by more than a factor of three. At the 11th floo , the increase is about by a factor 1 ½. At the top level, the increase is by a factor of 3.

For the high-seismic case, the increase at the base is by a factor of more than 4. At the 11th floo , the increase is by a factor of 2. At the roof level, the increase is by a factor more than 3 ½.

For the 30-storey comparisons, the first mode contribution factor is 1.411 and the higher mode contribution factor is 0.715. For these cases, the diaphragm force at the first level governs the design force for the full height of the structure and the increase at the first floor is a factor of about 6 for the moderate seismic cases and 7 ½ for the high seismic case.

For floors with relatively short spans between the vertical elements of the lateral force-resisting system and lower aspect ratio of length to width, the increase in diaphragm design forces may not actually result in much greater reinforcement for designs that use beam analogy for chord reinforcement. It is important, however, to recognize that the design shear forces increase by another factor of two to maintain elastic response in shear of the diaphragm for the RDO. The higher forces also increase the design force for the collector connections that tie the floors to the walls and frames. The research showed that the traditional methodology for determination of diaphragm design forces does not assure that the delivery of forces to the vertical elements of the SFRS will occur before there is a failure in the diaphragm.

11 Construction

11.1 Introduction

Precast concrete has its own unique characteristics in construction and when combined with tall buildings there are certain key factors that must be addressed early in the planning and design cycle if a successful construction project is to be achieved.

The main factors to be considered in the early design phase that affect construction of a tall building using a relatively large proportion of precast concrete elements are:

1. The site configu ation and layout in relation to delivery, handling, and storage of precast elements; pick up point from centre of lifting equipment, available short and long term storage areas;
2. Precast element shapes and weights;
3. Simplicity, speed, and repetition of connection details;
4. Temporary building and element stability;
5. Method of vertical and horizontal transport of precast elements;
6. Consideration of finishes and access to complete them.

If the above factors are embodied in the design process then the probability of achieving the benefits of precast concrete are enhanced. The construction time and, in turn, the financial risk should be reduced.

With precast concrete the construction is largely moved from the building site to a centralised factory resulting in greatly reduced weather dependency, higher quality, fewer delays, and reductions in the transport of materials and equipment. Site construction is largely replaced by an assembly process requiring less space and personnel. In a comparison between traditional cast in place concrete construction and prefabricated concrete construction the ratio between the costs of materials and the costs of labour changes dramatically. For traditional construction it is estimated to be 2:1, whilst for prefabricated construction it is closer to 20:1. In traditional construction it is therefore labour management that is critical, in prefabrication it is the logistical movement of materials that takes over.

Shorter and more controlled building times, less site labour and waste, and earlier delivery of rental incomes lead to the inevitable conclusion that building high rise structures using prefabricated elements is the future. The remainder of this chapter will consider the factors already listed above and assesses their impact on the use of precast concrete in tall buildings, and how design and planning decisions can affect the success of the project.

This chapter was mainly authored by Jones

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11.2 Site Configuration and Layout

The demand for tall buildings normally arises because of relatively high population densities and a scarcity of available land for building in those areas. They are most common in densely populated urban areas where land values are high. It is therefore often essential that the footprint of the building itself covers the full available site area. This can give rise to congestion on the building site with minimal space for storage, parking and movement of delivery vehicles and site amenities. Spaces for the various activities must be carefully considered and allocated at an early stage.

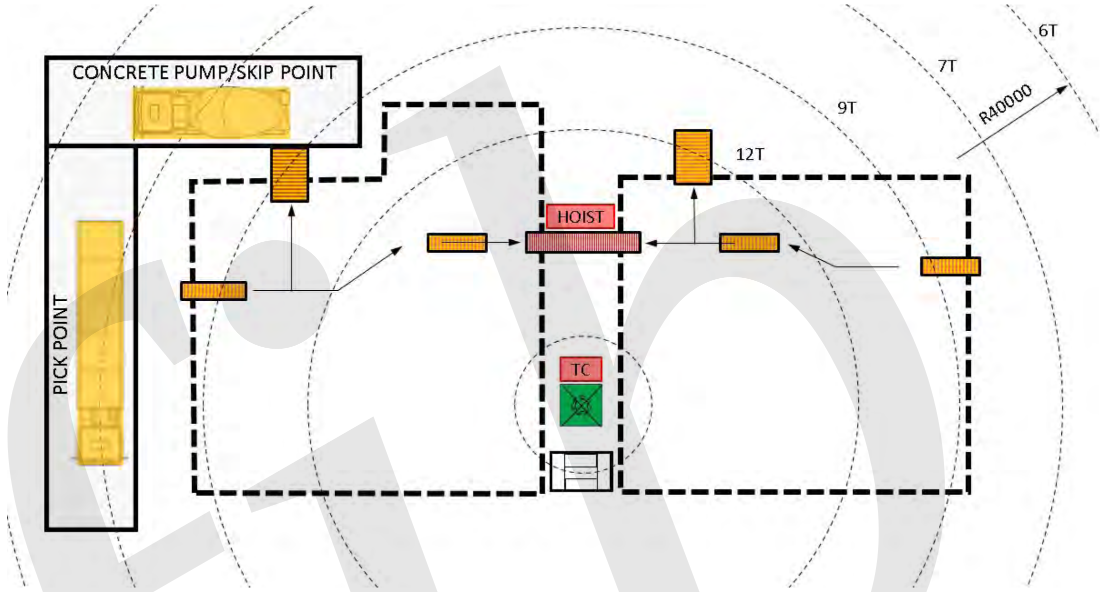

Fig. 11-1: Example of Site Logistical Arrangement.

Fig. 11-1 is an example of a site logistical arrangement that allows movement of materials and personnel inside the building site. In this case the new building is surrounded by either the external walls of existing structures or public highways. There is minimal space for parking construction vehicles. One space has been allocated to a delivery truck and another for a concrete pump and ready mix truck. A single tower crane, TC, and personnel / light materials hoist is in the central area of the building.

The crane pick point in the above example is typically the furthest horizontal distance from the crane that the precast concrete elements will have to be transported on the construction site and therefore will determine the maximum weight of precast concrete element to be used in the building. In this case it is seven tonnes. If vertical and horizontal site transport of the precast elements could be separated, and the vertical lifting resource moved nearer to the pick point, then there is the potential for more efficient use of crane capacity, heavier precast and time savings.

Our example has allowance for parking only one delivery vehicle. There is no space for storing materials around the building boundary, and any precast lifted from the truck must either be immediately installed or stored inside the building footprint. This situation is common in tall building construction and presents its own planning and logistical challenges. Ideally, single delivery vehicles should arrive in a predetermined sequence with the precast available for immediate installation. If this is not possible there should be a small, allocated storage area in the building capable of holding an agreed number of elements for a short time. A holding site should also be close to the building, say no more than 30 minutes' drive, where vehicles can wait for their turn to be called to the building site. In some cases, the holding site may have the capability for precast storage, particularly if the precast factory is a relatively large distance from the building, and the elements can then be shuttled to the site when needed.

11.3 Precast Shape and Weight

Generally, the greatest benefit from precast is achieved through simple rectangular modular shapes of maximum element weight, i.e. the maximum building area (elevation or plan) is covered by a single installation. However, the possibility of achieving this ideal is often constrained by building shape, location, maximum width, and weight that can be hauled on the highway and crane capacity (refer to example in Fig. 11-1). Initial design and planning should take account of these factors and the precast designer should divide the precast part of the building into elements that maximise all potential benefits whilst working within those constraints.

Precast production methods, particularly for large panels, commonly involve horizontal casting. In tall building construction, when delivery vehicle waiting times at the building site have to be minimised, the elements should arrive in the orientation that is required in their permanent location (walls vertical, slabs horizontal) in order to avoid rotating pieces that, at the least will take additional time, and may not even be possible in the restricted space available. Modern automated production facilities can rotate panels from a horizontal to a vertical position so that they are stored and delivered in the same orientation as they will be installed.

Odd-shaped elements are often unavoidable and special handling devices may be required. Shaft and wall construction often involves the use of "L", "U" and box shaped segments where special lifting beams should be designed to lift the element over its centre of gravity and avoid the generation of temporary twisting and bending effects during handling. Methods used at the precast factory for demoulding and lifting odd-shaped elements should be replicated on site, and similar lifting apparatus provided.

Fig. 11-2 is an example of a lifting arrangement developed for a segmental double lift shaft where vertical jointing was intentionally eliminated, and four horizontally jointed segments provided for each storey. Temporary reinforcement was provided across the door openings to provide resistance to twisting and bending. Crane capacity dictated the depth of the segments.

Fig. 11-2: Lifting Arrangement for Double Shaft Segments.

Lifting beams are also needed for simple horizontal slabs when the slab is part of a composite construction and the precast part is relatively thin and requires significan temporary site propping. This is particularly relevant to longer span reinforced filigre slabs. The lifting points connecting the precast slab to the lifting beam should be clearly indicated to avoid overstressing the slab during handling.

There are also circumstances when planar elements may have to be installed in locations that are inaccessible for a traditional crane lift. Often a counterbalancing arrangement must be designed to enable installation. Fig. 11-3 is an example of a vertical precast panel being lifted using a counterweight system where the panel centroid is offset from the crane hook.

Fig. 11-3: Counterbalanced Lifting System.

Challenges arising from shape, size and weight of precast elements can be overcome provided the correct crane capacity and lifting arrangements are chosen. However, simplification of the precast solution in the first place will accrue maximum benefit.

11.4 Connection Details

Simple and repetitive connections completed in minimum time greatly assist in the success of a tall building project using precast concrete elements. The different types of connection on each project should also be minimised to increase familiarity and efficiencies, plus reduce the risk of mistakes.

Connecting horizontal elements, i.e. beams and slabs, is relatively straightforward. In most circumstances bedded bearings, using high strength mortars or grouts, are favoured. Where bearing stresses are relatively low then padded bedding materials, such as neoprene or rubber are possible. For high stresses steel bearing plates may be necessary. It is important that the amount of "hook time" (the crane time required for installation) per element is minimised as this is likely to affect the floor cycle time. Therefore, bearing surfaces should be prepared in advance of precast installation. In the case of floor slabs, the elements are generally landed directly onto their bearings or temporary props and the crane released. For more heavily loaded beams it is often necessary to provide structural restraints, such as tightened bolts, to avoid potential instability before crane release. Permanent horizontal tying systems are generally completed in insitu concrete toppings or strips after precast installation. Where precast beams are required to provide moment resistance at their supports then they should be designed as simply supported in their temporary case and the connection completed after release of the crane.

For vertical elements, i.e. columns and walls, the connections can be more complicated.

For columns there would typically be a horizontal bedded bearing at bottom level. The connection would most likely have to resist combinations of substantial axial and shear forces together with bending moments in service. Significant areas of reinforcement (or possibly proprietary steel bracketry) would therefore connect columns and provide the necessary resistance. In the temporary case the column would be landed onto levelling shims and capture any upward projecting reinforcement / bolts from the slab or have projecting reinforcement from its end that would connect to tubes or couplers below. Temporary props connected to threaded sockets cast into the column on at least two orthogonal faces can be used to provide lateral stability. Proprietary steel bracketry, such as column shoes, can also be used in that way. The temporary restraint system for each column should be designed to allow the safe release of the crane in the minimum possible time.

Wall connections can be subdivided into two general types, i.e. horizontal and vertical.

Horizontal connections in walls of tall buildings are generally governed by the magnitude of axial compression applied to the joint. Precast solid panels are typically bedded onto high strength mortars. The horizontal joint is also subjected to both in plane and transverse shear forces and bending moments. In plane moments will both redistribute the applied axial load and increase its maximum value per unit length of wall at its extremities. When there are high compressive forces within the wall capacity the transverse moment can often be resisted by the axial force alone. Similarly, the normal stress from the compressive force assists in resisting the applied shear forces. Minimum joint reinforcement must at least be provided to satisfy robustness provisions and reinforced wall design criteria. Where designed vertical reinforcement is required across the horizontal joint to resist applied tensions and shears then it can be provided as either coupled bars or as reinforcement grouted into cast in tubes.

Fig. 11-4: Connecting Vertical Bars at Horizontal Wall Joint.

In Fig. 11-4 central connecting bars at a horizontal solid wall joint are in cast in corrugated steel tubes. They will be grouted before installation of the next wall segment over the projecting bars. The bedding mortar will be contained by short precast upstands along the joint as shown in Fig. 11-4. These joints tend to be simple and repetitive, but careful preparation and accuracy is essential for their success.

Whereas large compressive forces can tend to dominate horizontal joints and can, in turn, provide beneficial effects, vertical joints can largely be dominated by unfavourable shear and tensile forces.

The vertical joint detail provided needs to replicate the intentions of the structural analysis. If the walls have been analysed as individual members and there is no composite action with the adjoining panel then connecting reinforcement across the joint is likely to be unnecessary. However, the joint would still need to be closed for other purposes, e.g. heat and noise transfer, fire resistance etc. The joint profile in solid walls should allow for the adequate placement of the fill material (grout or small aggregate concrete) to provide a similar barrier to the wall itself. An example of a possible joint profile for a solid wall thickness of 200 mm is shown in Fig. 11-5. This type of joint would not have to be filled until after panel installation and with correct planning the filling operation should neither affect floor cycle time nor be critical to overall programme.

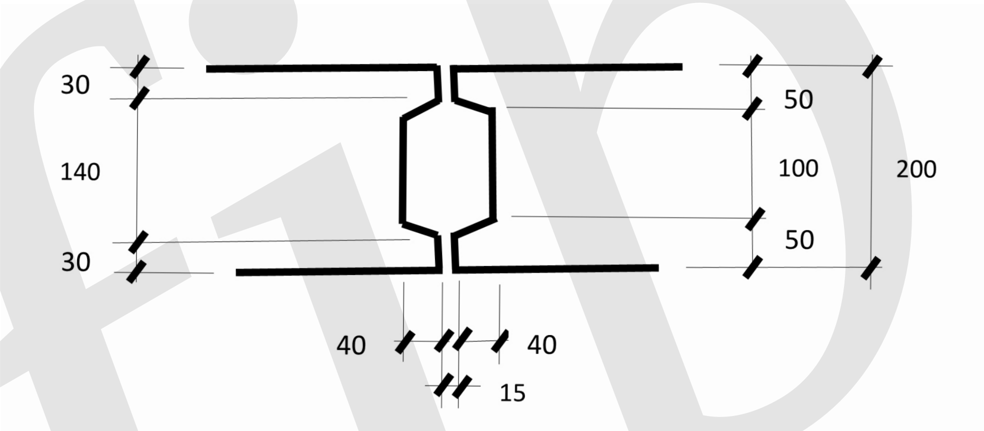

Fig. 11-5: Example of Vertical Solid Wall Joint Profile.

As shear and tensile forces acting on the vertical joint become more significant the complexity of the connection across the joint also increases. Relatively small forces in solid wall joints can be resisted by loop connectors, but as the forces increase the connection moves towards emulation of a monolithic insitu concrete solution (see also Chapter 7). In situ concrete stitching of panels can be a slow process and some of the speed benefits of precast construction can be lost. A twin wall solution that naturally utilises insitu connections may be applicable in those circumstances.

An option in core and shaft construction is to use relatively shallow precast segments where all vertical joints can be avoided (see Fig. 11-2).

11.5 Temporary Element and Building Stability

All precast elements and the overall building, that may wholly or partly consist of precast elements, must always be structurally stable in withstanding the effects of the applied design forces.

In this regard two important aspects must be checked at every stage of the building construction:

- The stability of individual elements before their connection to the building's structural frame;
- The overall stability of the partly completed building; with due regard given to the philosophy of the global structural analysis and the means of transferring the applied forces to the building's foundations.

The load path of each precast element must be assured at each stage of its installation and connection to the building frame. In a partially completed structure the load path may be different to that finally intended. It may be necessary to strengthen the element itself to withstand temporary forces that can only arise during construction or provide a temporary supporting system that avoids the development of increased internal stresses. For instance, a precast slab landed onto the projecting support nib of an internal beam could cause transverse rotation of the beam due to loading eccentricity. Once the beam is loaded on both sides the forces are balanced. Solutions to the temporary problem could be to either provide extra torsional resistance in the beam and its end connection or install temporary propping beneath the loaded beam nib.

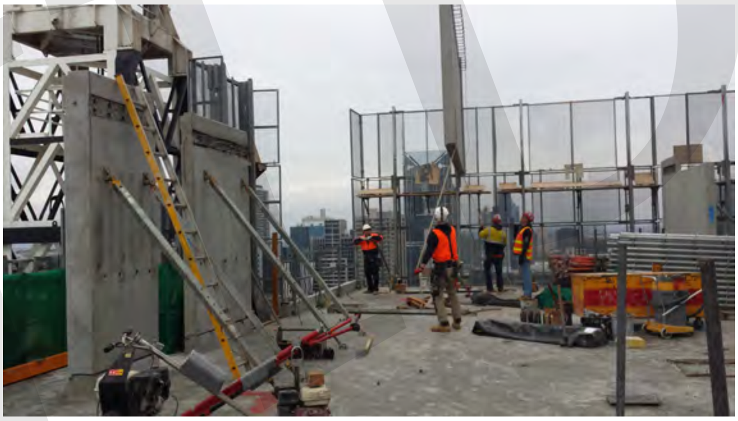

Fig. 11-6: Example of Temporary Propping and Installation of Wall Panels.

In tall building construction when efficien y of crane usage is critical to control floor cycle times it is often the case that connection of precast elements to other parts of the structure is not fully complete at the time of crane release. To stabilise the elements quickly and safely in those circumstances it is necessary to provide temporary support systems These can be in the form of metal props or brackets.

Fig. 11-6 is a typical example of metal props used to stabilise wall panels before completion of the top wall connection. In this case the top end of the prop is connected to a threaded anchor cast into the face of the precast wall, the foot of the prop is connected to an anchor site fixed into the floor slab. The panels are now stable, and the erection crew can move onto installation of the next panel.

The global stability system chosen in the structural analysis of a tall building would most likely need to be followed as the building is constructed. There can be exceptions to this general rule, for instance in a "tube in tube system" (see Chapter 4 also) the inner tube could be designed to withstand the temporary forces during construction but then must rely on the outer and inner tubes in service. This would then allow the inner tube to be completed in advance of at least part of the building façade and give more flexibility to the construction programme.

If a building relies primarily on its core containing the stairs and lift shafts for its structural stability in service then the core can be constructed well in advance of the rest of the building and the overall temporary stability is then ensured provided the lateral forces acting on the peripheral elements can be satisfactorily transferred to the core. In buildings that use composite floor slabs of precast and insitu components as a lateral diaphragm for transfer of vertical loads or as a bracing for the outer tube then the sequential pouring of insitu toppings and strips must be strictly controlled so that satisfactory transfer and distribution of forces is achieved through the partially completed diaphragm.

There are many overlaps between the precast design and its installation in relation to structural stability. It is essential that the design approach adopted by the precast designer is conveyed to the installer or contractor in a clear and concise manner so that both element and building stability are accommodated and assured in the site method statement. The precast designer should therefore prepare a design statement that is issued to the contractor that sets out the design approach, general stability principles, design standards, assumed construction sequence, temporary propping requirements, typical connections, and material characteristics. The design statement and site method statement should then be linked and consistent in their approach to structural stability.

11.6 Vertical and Horizontal Transport of Precast Elements

Precast elements must be transported both vertically and horizontally at the building site. In tall building construction vertical transport can become the dominant factor in construction timing and floor cycle time as the building height increases, together with lifting distances and wind forces. Horizontal transport itself has only limited effects, but when combined with the vertical transport time exacerbates the negative effect on the floor cycle time.

Fig. 11-7: Example of Construction using Tower Cranes.

Tower cranes are the traditional site transport solution for tall buildings. Where there is enough space on the site it is generally beneficial to mount the cranes alongside the building as shown in Fig. 11-7. This generally means that more than one crane can be used. Also, the crane can be sited near the pick point (see 11.2 also) and therefore it is likely that heavier precast elements will be possible compared to siting the crane inside the building footprint as the distance to the pick point is no longer critical.

There are now examples of hoisting sheds being used for construction of tall buildings in precast concrete. This advance can remove the need for a permanent tower crane on the site.

A hoisting shed is an assembly that is mounted on top of the building as it is constructed, i.e. it rises with the building floor by floor and encloses the construction site. It is a safe, dry, and windless environment on top of the building as it progresses.

Fig. 11-8: Example of Hoisting Shed, courtesy of BAM.

The hoisting shed can enclose the top floor plus an area outside the façade as shown in Fig. 11-8 , providing a working platform for joint sealing, and finishing.

The transport fl ws can be separated by use of gantry cranes inside the shed. In Fig 11-9 a gantry system has been assembled. The crane can be sited directly above the pick point and vertically lift the precast elements to internal cradles. This direct vertical lift means that heavier precast elements can be handled compared to tower crane options. The crane then distributes the elements horizontally inside the shed. There is also enough room in the shed for welfare facilities for site personnel, avoiding lengthy trips to ground level.

The effects of wind are minimised at top level inside the shed; however, the vertical lift can be exposed. As the vertical transport is relatively close to the building a guide system projecting from the facade could be introduced to control these effects.

The use of hoisting sheds can be limited by the building shape and the capacity of the partially complete structure to support the weight of the shed. In our example load bearing façade walls in a rectangular building were ideal for this application. Hoisting sheds can also be designed to climb back down the building for dismantling at ground level.

Fig. 11-9: Hoisting Shed Assembly, courtesy of BAM.

One of the drawbacks with this system is that currently building specific assemblies need to be designed for each application, unlike tower cranes that are readily available. However, if the system gains popularity then modularisation is possible in a similar way to other temporary supporting and lifting systems.

11.7 External Finishes

Application of external finishes to tall buildings can become difficult and affect completion dates if not carefully considered at an early stage in the design process.

There are three main factors related to external finishes that affect tall building construction:

1. The type of finish;
2. The method of application / installation of the finish;
3. Completion of the finished façade.

Surface finish, texture and shape is normally part of a larger building element, e.g. architectural precast concrete, GRC (glassfibre reinforced concrete) panels and glazing.

The type of external finish applied to a tall building is influenced by several factors but is mainly governed by location and function. In Chapter 9 the various possibilities for precast concrete cladding finishes are explored, however precast concrete cladding is only one means of finishing the façade, typically for residential buildings. In circumstances where a lighter finishing medium is required then relatively thin GRC factory produced panels may be appropriate where a concrete moulded finish is required but architectural precast panels are considered too heavy. GRC panels can vary between 10mm and 50mm thick depending on the structural requirements and are normally fixed to a supporting structure, e.g. precast backing panel, steel or insitu frame. They also have the advantage of being easily replaceable if the façade appearance requires updating in the future. Glazing is another popular solution for office facades. Each of these options present their own challenges in relation to weight (craneage and handling issues), area of façade covered by individual elements, durability, resilience, accuracy, and tolerance.

In tall building construction it is important to minimise the amount of external work required. Where possible, protected walkways should be provided around the building perimeter to enable the façade to be safely installed as construction progresses vertically. Alternatively, fixings for façade panels should be located inside the building if the façade is installed after the structural frame. If there are large areas of glazing then windows fitted in the factory in relatively slender precast panels may be an attractive option to minimise site work and assure water and air tightness.

Precast concrete panels are produced and installed as individual elements with gaps between each adjoining element. These gaps or joints must be designed to accommodate all foreseeable movements and dimensional deviations but remain waterproof and airtight. In tall buildings axial shortening of columns will have a more dominant effect on the joint size required compared to other buildings.

Flexible sealants are normally provided to close these gaps and accommodate the expected movements without distress during their working lives. Installation of external sealants can be difficult and time consuming if carried out after the main working platform is no longer available. In addition, sealants will have a shorter working life than the main façade elements. Joints should therefore be routinely inspected for potential degradation and failure. Methods for specification, installation and maintenance of joint sealants need to be carefully considered at design stage to avoid large costs at a later stage.

12 Case Studies

12.1 Introduction

Case studies selected for this chapter will illustrate the benefits, applications and principles relating to precast concrete in tall buildings that have been described in the previous chapters. These are global examples that show innovation, modern techniques and the versatility related to this form of construction.

The case studies that have been chosen are:

12.2 Bella Sky Hotel, Copenhagen, Denmark[47]

12.3 Dexia Tower, Brussels, Belgium

12.4 Breaker Tower, Bahrain

12.5 Urban Dock Park City, Toyosu, Japan

12.6 Deux Tours, Tokyo, Japan

12.7 Tampere Tower Hotel, Helsinki, Finland

12.8 BMX Parque de Cidade, Sao Paulo, Brazil

12.9 Conjunto Paragon, Mexico

12.10 Northwestern Memorial Hospital, Chicago, USA

12.11 Erasmus Medical Centre, Rotterdam, The Netherlands

12.12 The Paramount, San Francisco, USA[44]

12.13 Premier Tower, Melbourne, Australia

12.14 Australia 108, Melbourne, Australia

12.15 Seismic Resistant Office Structure, China

12.16 Torre de Cristal, Madrid, Spain

This chapter was mainly authored by Corres Peiretti, Dahl, Falger, Graziano, Jones, Hughes, Rajala, Sugaya, van der Zee, van Keulen, Zhao

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12.2 Bella Sky Hotel, Copenhagen, Denmark[47]

Fig. 12-2a: Bella Sky Hotel, Copenhagen, Denmark.

Precast Concrete: Full building structure, mainly wall panels and hollowcore floors.

Occupancy: Hotel accommodation, 812 rooms

Storeys / Height: 23 storeys, 76 m high

Floor Area: 44'000 m²

Completion Date: 2011

This was the winner of the *fib* Award for Outstanding Concrete Structures, 2014.

Bella Sky hotel provides accommodation for the nearby Bella Conference Centre and was subject to several design constraints that led to the building shape adopted. These included the limited size of the building plot, height restrictions from the nearby airport, requirements for light and views, and a road bisecting the plot.

The finalised building consists of two towers that are each inclined at an angle of 15 degrees away from each other and are connected at a high level. In addition to the general vertical inclination there is an added twist part way up each tower that locally increases the subvertical angle to 20.4 degrees. This elegant structure is one of the most leaning buildings in the world, and despite the challenges provides a precast solution that appears to defy the laws of gravity.

Fig. 12-2b: Bella Sky Hotel, Copenhagen, Denmark.

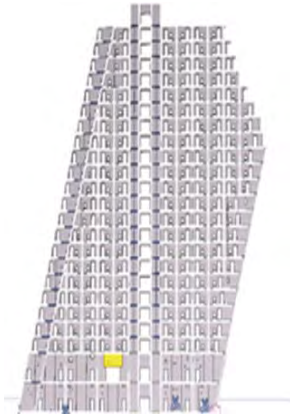

Fig. 12-2c: Bella Sky Hotel, Copenhagen, Denmark.

The original structural concept used steelwork. However, concrete was eventually chosen due to the benefits of stiffness, acoustic insulation, and fire protection. Precast was then used instead of insitu concrete because of the extra benefits of construction speed, plus a cost benefit as there is a well-established precast concrete industry in Denmark.

In the structural design load paths were simplified wherever possible. Precast concrete walls divide the rooms and form the corridors. They provide the loadbearing structure but are only inclined in the outer bays of each tower as shown in the illustrations above and opposite. Prestressed precast hollowcore slabs were used for the floors.

Large forces are developed due to the building inclination and are compounded by the large number of openings for doors and services in the loadbearing panels. This meant that standard connection details often used for precast concrete had to be redesigned and adjusted to suit the increased structural demand.

Overall equilibrium is achieved through the centre of mass being within the footprint of each tower, and ground anchors were not required to assist building stability.

Transfer of horizontal forces developed through both wind and the building's inclination, to the foundation is achieved through the precast walls. In the direction of the building longitudinal axis the corridor walls provide the horizontal resistance. In the transverse direction resistance is provided by a loadbearing crosswall every 7.8 m. Due to the magnitude of the design forces the wall stability system had to be analysed as a series of Vierendeel trusses. Therefore, beam elements across doors and corridors are required to transfer moments and shear forces to maximise shear wall lengths. Consequently, large forces are developed at the joints.

Project Team:

- Client: Bella Centre A/S
- Architect: 3XN Architects
- Structural Engineer: Ramboll
- Main Contractor: NCC Construction, Denmark
- Precast Producer: Contiga Tinglev

12.3 Dexia Tower, Brussels, Belgium

Fig. 12-3a: Dexia Tower, Brussels, Belgium.

Precast Concrete: Columns, beams, and floor slabs.

Occupancy: Offices

Storeys / Height: 37 storeys

Floor Area: 60'000 m^2

Completion Date: 2004

The building is located at Place Rogier in Central Brussels.

The intention from the start was to use as much precast concrete as possible. The stability cores remained as insitu concrete, but the remaining structural elements were generally precast concrete with an insitu topping to assist robustness and diaphragm action.

Columns had to be limited in cross section to avoid encroaching into valuable floor space but were also heavily loaded. The column diameter for the first three storeys was limited to 800 mm for a design axial load of 50'000 KN. Initially it was thought that the solution for the lower storeys would have to be steelwork due to the resultant high material stresses. However, a precast concrete column solution was achieved using C80/95 concrete and high densities of reinforcement up to 7 % in places. With this method, the number of steel columns at the lower levels was substantially reduced.

In addition, above the taller lowest storey circular columns were produced in double storey heights to reduce the number of installation operations. They also had integral corbels for beam supports. The total number of double storey columns in the superstructure amounted to 591.The floor construction consisted of prestressed ribbed floor units supported by prestressed beams acting compositely with insitu toppings. Inverted precast U sections were used near columns to aid the distribution of services in the void above the false ceiling. There were 1'259 prestressed beams, 52'000 m^2 of hollowcore floo slabs with spans up to 13.5 m, and 5'300 m^2 of U sections. Beams and columns were welded together via embedded steel plates to provide tie connections as an alternative load path in the event of the accidental loss of a column.

Fig. 12-3b: Dexia Tower, Brussels, Belgium.

Fig. 12-3c: Dexia Tower, Brussels, Belgium.

Special heavy plunge columns as shown opposite, weighing up to 42 tonnes each, were provided in the four storeys deep basement to support the 37 storeys superstructure in a "top down" construction that allowed 20 storeys of superstructure to progress at the same time as basement construction, saving 18 months of construction programme.

Project Team:

- Client: Brussels Business Centre SA
- Architects: Samyn & Partners, Jaspers- Eyers Architects
- Structural Engineer: Setesco SA
- Main Contractor: BPC SA, Vinci-Construction & CIT Blaton SA
- Precast Producer: Ergon nv Belgium

12.4 Breaker Tower, Bahrain

Fig. 12-4a: Breaker Tower, Bahrain.

Precast Concrete: Full building structure; wall panels, columns, beams and hollowcore floor

Occupancy: Apartments, car park, swimming pool, show room

Storeys / Height: 35 storeys, 165 m high

Floor Area: 29'800 m^2

Completion Date: 2014

The building has two basic volumes. The high-rise has the shape of a vertical "cylinder lock". The apartments are situated in the round part of the high-rise. This building has 28 storeys with apartments. Each storey has a free height of 4.2 m. The residents of the apartments have an exceptional view of the surroundings. The rectangular part of the high-rise functions as the "backbone" of the building and houses the elevators and staircases. The low-rise of the building has a rectangular volume. The 5-storey car park and the show room are housed in this volume.

The precast concrete structure in the round part of the high-rise is framed with columns and beams (see figure "a" overleaf). Columns are positioned in the round peripheral to facilitate placing internal partition walls. The shear walls at the back of the building form the lateral stability structure. Vertical joints connect the individual shear walls at the intersections. The shear walls act together structurally as stabilizing 3D structures. However, the contribution of the vertical joints at the intersections is ignored in the calculation models. These models consider the precast concrete superstructure as a shear wall assembly with 2D individual walls.

The wind force $F_{wind;y}$ (Fig. 12-4b, a) is very eccentric (e_y) relative to the centre of gravity of the precast concrete shear wall assembly. Eccentricity (e_x) is present in the other direction because of the absence of shear walls in the west façade. The third eccentricity is that of the vertical loads relative to the centre of gravity of the vertical stability structure. The superstructure rotates because of these three eccentricities (Fig. 12-4b, b). The precast concrete stability structure will be loaded in bending and torsion. The forces caused by torsion dominate in the force distribution of the superstructure.

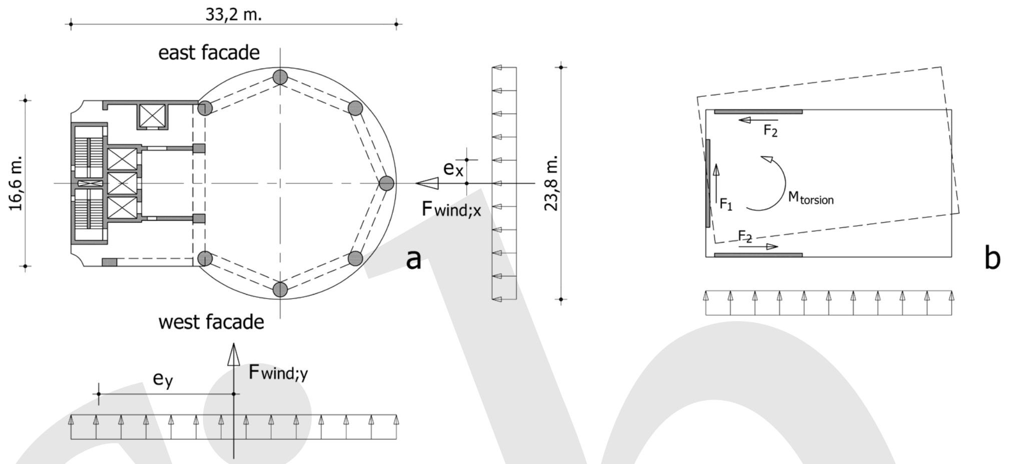

Fig. 12-4b: Breaker Tower, Bahrain.

Hollowcore slabs, 500 mm deep, in the round part of the superstructure span 18 m onto beams spanning between 1.5 m deep columns with support corbels. In the structural analysis the columns are modelled as pin jointed and do not contribute towards lateral stability. An overview of the slab structure is shown in Fig. 12-4c. The perimeter of the round area is cast insitu and stitched to the support beams and hollowcore slabs to provide diaphragm action.

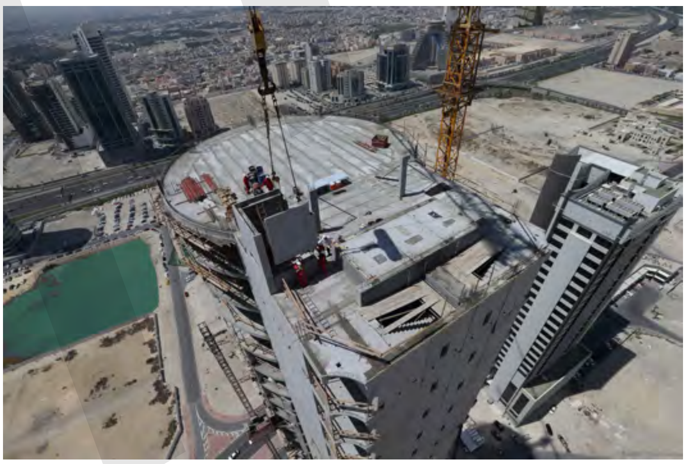

Fig. 12-4c: Breaker Tower, Bahrain.

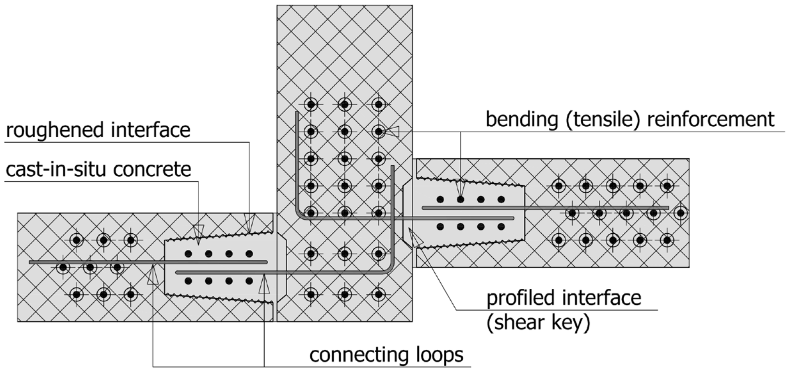

Fig. 12-4d: Breaker Tower, Bahrain.

Individual wall panels are connected via vertical reinforced insitu strips, as shown in the plan section in Fig. 12-4d, to ensure structural integrity between panels.

Production of the precast elements was always at least two floors ahead of construction. Installation progressed at an average speed of 2.5 floors per month, i.e. 14 months to complete the concrete superstructure.

Project Team:

- Client: Bin Faqeeh Real Estate Investment
- Architect & Structural engineer: Arab Architects
- Main contractor: Al Taitoon Contracting Co. S.P.C.
- Precast concrete design, production, and construction: Bahrain Precast Concrete Co. W.L.L.

12.5 Urban Dock Park City Toyosu, Japan

Fig. 12-5a: Urban Dock Park City Toyosu, Japan.

Precast Concrete: Columns, beams and slabs

Occupancy: Residential, apartments

Storeys / Height: 52 storeys, 180 m high. Also includes a second building of 32 storeys on same site

Floor Area: 60'000 m^2

The building was designed and constructed using the Sumitomo Mitsui Quick RC Integration Method (SQRIM). This method was developed with the aim of converting all main structural members to fully precast concrete. The method utilises all frame elements to resist lateral forces. This is a seismic damping device equipped structure and uses concrete of 120 N / mm2 compressive strength for some structural members.

The construction period was 33 months with a SQRIM applied floor structural framing cycle of 3-4 days. Seven tower cranes were set up with a capacity of 15 tonnes each. The total number of precast elements was 24'035, which were produced in twelve precast factories. The percentage of insitu concrete elements converted to precast were 100 % columns, 95 % beams and 74 % of floor slabs. This resulted in the on-site labour demand being reduced by 95 % for formwork and 97 % for rebar fixing.

Fig. 12-5b: Urban Dock Park City Toyosu, Japan.

Project Team:

- Client: IHI Corporation and Mitsui Fuducan Residential Co. Ltd. (Joint Venture)
- Architect: Jun Mitsui and Associates
- Structural Engineer: Sumitomo Mitsui Construction Co. Ltd
- Main Contractor: Sumitomo Mitsui Construction Co. Ltd and Kajima Corporation (Joint Venture)
- Precast Producer: SMC Preconcrete, Japan (Main producer), one other in China.

12.6 Deux Tours, Tokyo, Japan

Fig. 12-6a: Deux Tours, Tokyo, Japan.

Precast Concrete: Columns (SQRIM), beams (SQRIM-H) and hollowcore slabs

Occupancy: Residential, apartments

Storeys / Height: Two buildings that are each 52 storeys, 180 m high

Floor Area: 60'000 m^2 each building

Completion Date: 2015

The SQRIM method was improved in 2008 and SQRIM-H (H stands for horizontal) was introduced. This made the floor by floor construction of fully precast members of complex shape, if necessary, that can be skeletal (beam and column) framed or wall framed, or a mixture of both possible. This development lead to further shortening of construction periods, reduction of the number of workers and less temporary supports and bracings.

This building uses the SQRIM and SQRIM-H method and is also a seismically isolated structure. It was built after the Tohuku earthquake (Great East Japan Earthquake) of 2011, and is one of the largest seismically isolated super high-rise buildings in Japan

Fig. 12-6b: Deux Tours, Tokyo, Japan.

Fig. 12-6c: Deux Tours, Tokyo, Japan.

Project Team:

- Client: Sumitomo Realty and Development Co. Ltd.
- Architect: Sumitomo Mitsui Construction Co. Ltd
- Structural Engineer: Sumitomo Mitsui Construction Co. Ltd
- Main Contractor: Sumitomo Mitsui Construction Co. Ltd
- Precast Producer: SMC Preconcrete, Japan (main producer).

12.7 Tampere Tower Hotel, Helsinki, Finland

Fig. 12-7a: Tampere Tower Hotel, Helsinki, Finland.

Precast Concrete: External architectural sandwich panels, internal load bearing partition walls

Occupancy: Hotel accommodation, 305 rooms

Storeys / Height: 25 storeys, 88 m tall

Floor Area: 30'000 m^2

Completion Date: 2016

Tampere Tower hotel was the tallest building in Finland when completed. The building façade presents a lively interaction of black concrete, accent panels made from acid resistant steel, and irregularly placed windows.

The surface is a deeper black than normal, created by using a deep pigment with an exposed aggregate surface. The roughness of concrete combined with the gloss of the steel panel suits the spirit of the old industrial town. The black concrete façade has different live appearances depending on the light and viewing direction.

The loadbearing internal panels are 250 mm thick at lower levels, and reduce to 200 mm thick from ninth floor upwards, and provide the building's lateral stability system.

Fig. 12-7b: Tampere Tower Hotel, Helsinki, Finland.

Project Team:

- Client: SOK Real Estate Operations Ltd
- Architect: Seppo Valjus Architects Ltd
- Structural Engineer: Sweco Structures Ltd
- Main Contractor: Sweco Structures Ltd
- Precast Producer: Parma Ltd (Consilis)

12.8 BMX, Parcel A, Parque de Cidade, Sao Paulo, Brazil

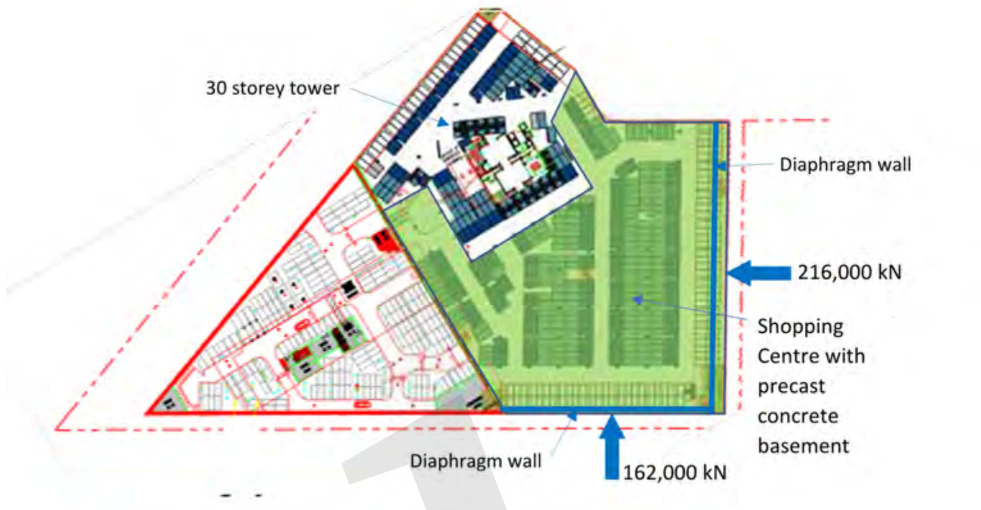

Fig. 12-8a: BMX, Parcel A, Parque de Cidade, Sao Paulo, Brazil.

Precast Concrete: Six storey basement structure and four storeys of shopping.

Occupancy: Car parking, shopping centre, offices, hotel, apartments

Storeys / Height: 30 storey tower, 4 storeys of shopping centre and 6 storeys of car parking

Floor Area: 17'300 m^2 ground floor footprint, 190'000 m^2 finished floor area.

Completion Date: 2018

The major use of precast concrete in this project was in the four storeys of shopping and six-storey basement, where a structural system was required that could resist significant horizontal forces acting on the two primary diaphragm perimeter walls and avoid those forces affecting the tower.

Precast concrete was chosen for speed of construction as the insitu concrete alternatives had a far longer timeline. However, precast concrete also presented load transfer challenges as the large unbalanced earth forces acting on the structure created substantial moment transfers at the structural frame nodes. Shear walls had been initially considered but large section thicknesses would have disrupted the parking arrangement and circulation. Precast beam and column framing connected by a composite floor diaphragm was the structural system eventually chosen.

Hollowcore slabs, acting compositely with structural toppings, provide the lateral diaphragm. The large moments developed at the beam / column junctures were resisted through emulation of monolithic insitu construction. Mechanical couplers were used extensively to connect main reinforcement in the beams and columns with localised areas of insitu concrete at the interface.

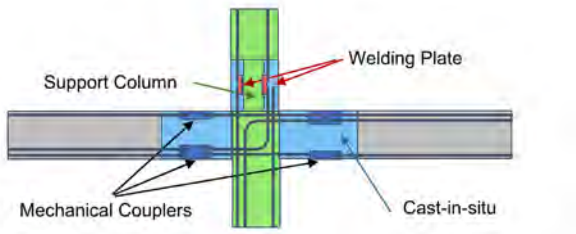

Fig. 12-8b: BMX, Parcel A, Parque de Cidade, Sao Paulo, Brazil.

An indicative sketch above illustrates a typical beam / column moment connection.

The perimeter diaphragm walls could not carry the building façade due to several reasons: construction alignment accuracy, differential settlements, local byelaw restrictions, poorer quality subsoil near the diaphragm. It was therefore decided to have a sloping column connecting the façade structure to the main perimeter building column as illustrated below.

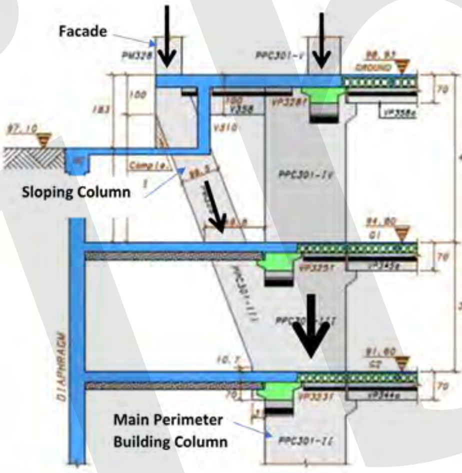

Fig. 12-8c: BMX, Parcel A, Parque de Cidade, Sao Paulo, Brazil

The main perimeter building columns are precast concrete, but parts of the sloping columns connecting with the facade were cast insitu to reduce the weight of the precast element for lifting and handling purposes.

The basement car park, shopping centre and tower were owned by the same client, and it was a design requirement that the tower should be constructed first. The tower would also not be required to resist the horizontal diaphragm forces transferred through the basement structure beneath the shopping centre. Therefore, precast beams in the basement that interface with the tower are supported by the tower columns but have special connections that allow transfer of vertical load only. Allowance was made in the design to account for settlement of the completed structure at the junction between the shopping centre beams and tower columns after release of the anchors in the diaphragm wall.

Fig. 12-8d: BMX, Parcel A, Parque de Cidade, Sao Paulo, Brazil.

Project Team:

- Client: PREVI – Fundo de Previdencia dos Funcionarios do Banco do Brasil
- Architect: Aflafo e Gasperini Arquitetos
- Structural Engineer: Pasqua and Graziano Associates
- Main Contractor: Odebrecht-Realizacoes Imobilarias
- Precast Producer: CPI Engenharia

12.9 Conjunto Paragon, Santa Fe, Mexico

Fig. 12-9a: Conjunto Paragon, Santa Fe, Mexico.

Precast Concrete: Building façade, comprising 520 curved and straight pieces, both concave and convex.

Occupancy: Hotel

Storeys / Height: 29 storeys plus 7 underground storeys

Floor Area: 40'000 m^2

Completion Date: 2011

The building is sited on the highest ground in a recently developed area, making it a prominent landmark. Its relatively massive size was slimmed visually with the undulating design which blends the precast panels with large windows that give expansive views of the landscape.

This building has a winding "S" shaped profile, and consequently precise fabrication of the precast panels was the key to defining the unique shapes needed. High-quality off-site manufacturing ensured achievement of complicated geometry, curved panels, intricate medallions, cubic protruding shapes and balconies.

Panels were erected in a horizontal sequence around the building perimeter. This allowed early phased shell completion at each floor level and release of large sections of the façade for fixing of glazing and protected crews working on interior finishing. The early close-in also meant that hotel owners were able to adapt interiors to specific needs.

Wind, site restraints, building height, the wavy design, protruding windows at the top floors and the construction schedule all created challenges that could only be met with precast architectural panels.

Project Team:

- Client: Condiminio Paragon
- Architect: IDEA Associates of Mexico
- Structural Engineer: DYS SA, Mexico
- Main Contractor: DEZ Construcciones, Mexico
- Precast Producer: PRETECSA, Mexico

12.10 Northwestern Memorial Hospital, Chicago, Illinois, USA

Fig. 12-10a: Northwestern Memorial Hospital, Chicago, Illinois, USA.

Precast Concrete: Building façade; Prestressed wall panels, insulated sandwich panels for levels above parking garage, non-insulated for lower garage floors.

Occupancy: Health care

Storeys / Height: 25 storeys

Floor Area: 60'000 m^2

Completion Date: 2014

The Outpatient Care Pavilion at Northwestern Memorial Hospital comprises two floors of reception area, seven floors of enclosed parking, fourteen floors of medical space and two floors for mechanical plant. The lower floors with the parking garage are insitu concrete framed and the medical floors are steel framed. The parking areas are reached by sky bridges consisting of precast concrete components.

The building is clad with architectural precast concrete panels that have an acid etched limestone appearance. There are also limestone-clad panels that form a “tablet” feature on the front elevation. There are 5'000 m^2 of non-insulated precast panels, 3'500 m^2 of insulated sandwich panels and 540 m^2 of limestone clad panels. There are also precast column covers, element sizes up to 8 m high x 2 m wide.

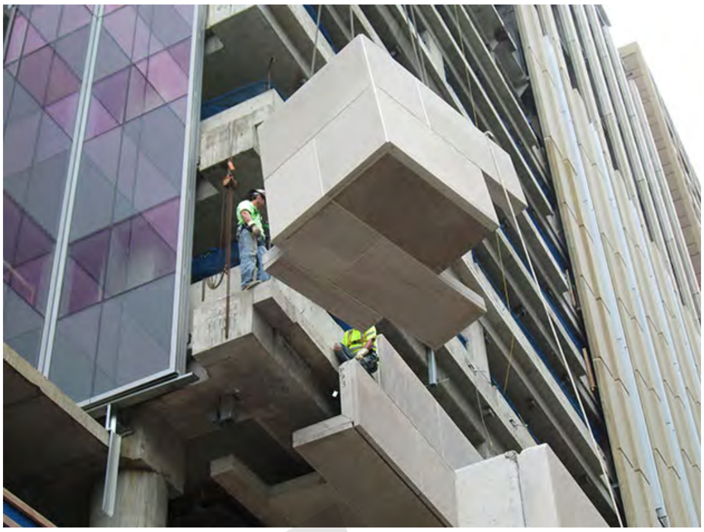

Fig. 12-10b: Northwestern Memorial Hospital, Chicago, Illinois, USA.

Precast concrete was chosen for the building façade to help overcome the logistic problems of a downtown site, allow the building to be closed more quickly, achieve the required quality demanded through factory manufacture, and eliminate the need for internal insulation through use of sandwich panels.

Project Team:

- Client: North-western Memorial Hospital
- Architect: Cannon Design Inc.
- Structural Engineer: Thornton Tomasetti
- Main Contractor: Lend Lease Construction Inc., Pepper Construction Group
- Precast Producer: Lombard Architectural Precast Products Company

12.11 Erasmus Medical Centre, Rotterdam, The Netherlands

Fig. 12-11a: Erasmus Medical Centre, Rotterdam, The Netherlands.

Precast Concrete: Fully precast building; Insulated external sandwich panels, internal solid walls and hollowcore slabs.

Occupancy: Health care

Storeys / Height: 35 storeys / 120 m

Floor Area: 30'000 m^2

Completion Date: 2016

The building structure consists of a loadbearing façade and internal service core. The walls act together as a tube inside a tube to provide lateral stability.

The contractor decided to use a climbing shed for handling and installation of precast elements instead of the conventional tower crane approach. The elements are installed one floor level at a time and the shed jacked to the next level as the storey is completed.

Using the shed it was possible to split vertical and horizontal transport of elements, whereas they are generally combined when using a tower crane. By splitting vertical and horizontal transport a creditable floor cycle time of fi e days was achieved on a floor area of 43 m x 19 m. Moreover, the roof of the shed provides cover with consequent health and safety benefits in the closed working environment. Weather, particularly wind, has little effect and there can also be an early start to non-structural work.

Fig. 12-11b: Vertical lift of precast panel from delivery vehicle.

Fig. 12-11c: Horizontal distribution of precast panel inside climbing shed.

Project Team:

- Client: Erasmus MC
- Architect: EGM Architects bv
- Structural Engineer: ABT Consulting Engineers
- Main Contractor: BAM-Ballast Nedam
- Precast Producer: Byldis, Westo Beton, Hoco Beton

12.12 The Paramount, San Francisco, California, USA[44]

Fig. 12-12a: The Paramount, San Francisco, California, USA.

Precast Concrete: Architectural façade cladding and seismic bracing system

Occupancy: Retail, offices and parking, Levels 1 to 8; Apartments from Level 8 upwards

Storeys / Height: 39 storeys / 128 m

Floor Area: 65'000 m^2

Completion: 2001

This award-winning building incorporates a precast hybrid moment resisting frame in the most severe earthquake region of the United States. It was the first major high rise building in that region to be braced by a precast architecturally finished exposed aggregate ductile frame. Post tensioning and high strength mild steel reinforcement were combined in the beam / column frames to provide a structural solution that accommodates severe design seismic movements.

The building façade consists of 732 sandblasted precast beams and 478 two storey precast moment frame columns. There are also 641 architectural precast panels, 68 precast gravity columns and 312 precast prestressed beams. The extensive use of precast significantly improved the construction schedule.

The combination of the architectural façade and seismic bracing system also had the advantage of a more water-resistant exterior. In traditional construction in a seismically active area slip joints must accommodate significant differential displacements between adjoining façade panels, this leads to long term maintenance issues. The Paramount façade solution avoids this problem as the precast components are rigidly attached to each other and designed so that the pieces move together.

Two types of structural lateral force resisting systems are used in the building. Below eighth floor the type of occupancies created a natural choice of a shear wall system. Above the eighth floor a perimeter precast moment frame uses both the Precast Hybrid Moment Resistance Frame (PHMRF) System at multi bay locations and the Dywidag Ductile Connector (DDC) System at single bay frames that occur at re-entrant corners. The PHMRF system was successfully tested at the University of California, San Diego as part of the design development process. The DDC systems had been tested and used previously. The test results showed that the PHMRF system could sustain drifts of 3.5 to 4 % with a loss of less than 30 % of its ultimate strength. Codes normally require designs for drifts of between 2 and 2.5 %.

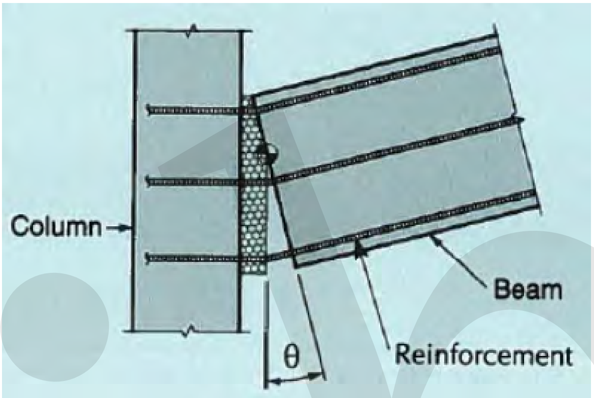

Fig. 12-12b: The Paramount, San Francisco, California, USA.

The ilustration above shows how the precast beam and column interact during an earthquake. The beam rotates in relation to the column opening a gap θ. This rotation is achieved through top and bottom ductile reinforcement and central posttensioning. Mild steel reinforcing in grouted ducts above and below the posttensioning tendons deforms plastically during the earthquake but the central tendons behave elastically and pull the frame back to its original position. A typical beam / column interface detail is shown below.

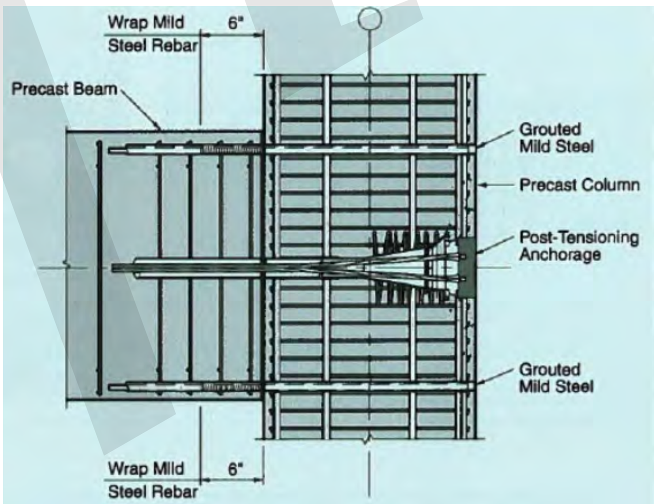

Fig. 12-12c: The Paramount, San Francisco, California, USA.

In comparison, the cast in place system would distribute the equivalent gap over part of the beam length causing extensive cracking and spalling, often leading to substantial repairs and sometimes replacement. The improved structural behaviour in seismic conditions of the precast system compared to the cast in place alternative gave another inherent benefit.

Fig. 12-12d: The Paramount, San Francisco, California, USA.

The visual aspects of the precast solution are especially important as it formed the finished building façade. A mock-up of a typical assembly was produced (above left) at the precast producer's yard to agree detailing and alignment issues before production. The photo above right shows a view of the architectural precast façade installed on the building.

Project Team:

- Client: Third and Mission Associates Inc., Irvine, California, USA
- Architect: Elkus / Manfredi, Boston, Massachusetts, USA
- Structural Engineer: Robert Englekirk Consultant Engineers, Los Angeles, California, USA
- Main Contractor: Pankow Residential Builders II, Ltd, Altadena, California, USA
- Precast Producer: Mid-State Precast, L.P., Corcoran, California, USA

12.13 Premier Tower, Melbourne, Australia

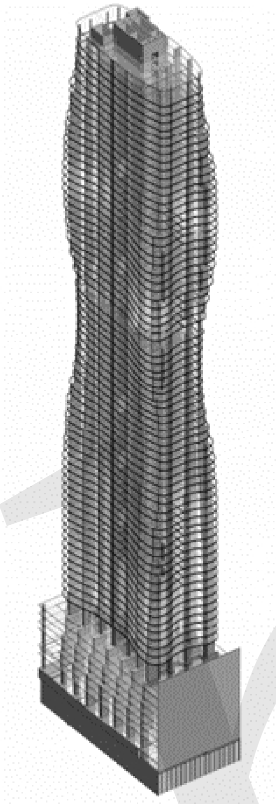

Fig. 12-13a: Premier Tower, Melbourne, Australia.

Precast Concrete: Panels, columns, and mega shell columns

Occupancy: Mixed Used (ground retail tenancy space

5 level podium hotel, and 67 levels of apartments (780 apartments, 180 hotel rooms)

Storeys / Height: 78 storeys, 249 m high

Project Status: Scheduled completion late 2020

As one of Melbourne's tallest and most prestigious developments, this project is best known for its inspiration: Beyoncé's video 'Ghost', which features writhing dancers tightly shrouded in fabric.

The result is an elegant, amorphic form, designed by Elenberg Fraser, that sits on an island site opposite Melbourne's main train terminal, Southern Cross Station. When completed it is expected to rise to 78 storeys, comprising 780 one and two-bedroom apartments and 180 hotel suites, as well as a range of leisure facilities.

Recreating those sinuous curves in glass, concrete and steel was no mean feat for the structural engineering team, especially while minimising intrusion on the usable floo area and internal layouts.

Melbourne's construction industry is predisposed to concrete, with precast vertical elements (columns and panels) with post-tensioned flat slabs a common method of construction.

Fig. 12-13b: Premier Tower, Melbourne, Australia.

At Premier Tower, the flat plates are typically 200 mm bonded post-tensioned slabs, allowing floo -to-floor heights of 3 m, with ceiling heights of about 2.7 m. The shape of the floorplates varies throughout the building, so walking columns were used to transfer loads to different parts of the structure. At the deepest curves, the slab edge on the line of the corner columns varies by 5 m. Typically, the internal edge of the column remains aligned, while the external edge is stepped to support the longer cantilevers. The result is that the corner columns change in section from 800 x 800 mm to 3,950 mm x 300 mm in six steps. Along the shallower curves, the internal columns walk in and out to fit around internal layouts and doorways.

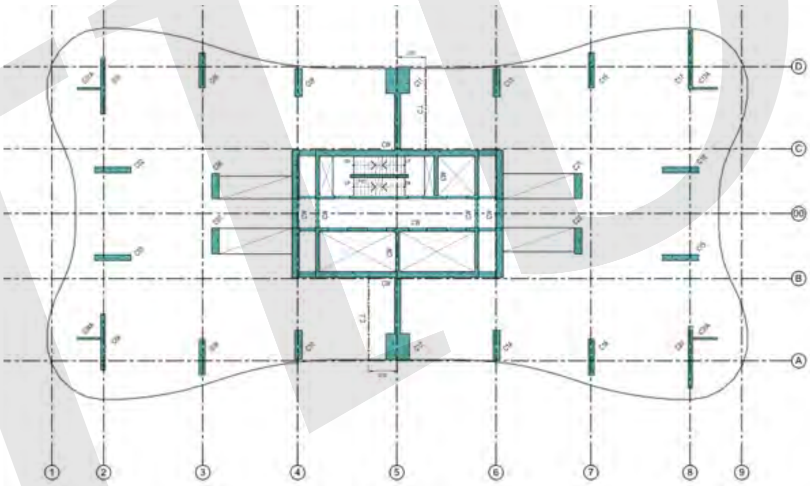

Fig. 12-13c: Premier Tower, Melbourne, Australia.

Below ground, there are four basement levels incorporating car parking. These are broader than the tower, so the columns transition horizontally as they reach the podium, by approximately 4.5 m over a height of 18 m, or six storeys.

Premier Tower is a slender structure – the ratio of its height to structural width is 8.3 from the ground up, and a much more challenging 10.8 above the podium.

To maintain the building's movement, due to wind, to within acceptable levels, precast mega-columns on the façade maximise the width of the stabilising structure. These are tied to the core by two- or three-storey outriggers concealed in party walls, and secondary outriggers at the mid-height plant floor.

Controlling the acceleration is the most important thing for residential buildings, so extensive wind tunnel testing was undertaken. There is provision for a tuned mass liquid damper at the top of the building, to further slow the movement, and this will also double as the fire tank.

The precast mega columns are sized to carry both gravity and wind loads. The forces generated by the wind loads can be equal to the weight supported by the column.

Due to the overall weight of the mega columns, Westkon Precast and WSP worked closely to create a precast option to form the projects 'mega-columns' within the structure, that could be safely lifted by the site tower cranes.

Fig. 12-13d: Premier Tower, Melbourne, Australia.

The result is a composite 'shell' column that not only has the required vertical capacity, but also easily accommodates the outrigger connections through the building.

Premier Tower will become Melbourne's new home of Architectural ingenuity. The artistic vision of this building has been brought to life with impressive technical knowledge. It is clever, contemporary design that maximises floorspace and enhances sweeping views, it is a stunning new addition to Melbourne's cityscape.

Project Team:

- Client: Fragrance Group
- Architect: Elenberg Frazer
- Structural Engineer: WSP Structures
- Main Contractor: Multiplex
- Precast Producer: Westkon Precast

12.14 Australia 108, Melbourne, Australia

Fig. 12-14a: Australia 108, Melbourne, Australia.

Precast Concrete: Panels and columns

Occupancy: Residential (1'105 Apartments)

Floor Area : 138'000 m^2

Storeys / Height: 100 storeys, 320 m high

Project Status: Scheduled completion 2020

Australia 108 (previously 70 Southbank Boulevard) is a residential supertall skyscraper currently under construction in the Southbank precinct of Melbourne, Victoria, Australia. When completed, it will become the tallest building in Australia by roof height, surpassing its neighbour – Eureka Tower, and the second tallest building in Australia by full height, surpassed by Q1.

Australia 108 is an exhilarating new development perfectly positioned within Melbourne's most desirable arts, dining and entertainment precinct. Rising 320 meters tall, making it the tallest residential building in the Southern Hemisphere at time of completion. It has been created by the team that successfully delivered the iconic Eureka Tower. Australia 108 will comprise over 1050 apartments with associated amenities and car parking. The iconic "Star Burst" will house premium facilities including two pools, cinema and private dining facilities. It is anticipated that the top level will be ultrahigh penthouse apartments with breathtaking views of Melbourne and Port Phillip.

Structural Features:

- The key structural system used in this 320 m tower is a concrete core and outriggers to mega column and mega frame.
- Floors are flat plate two-way post-tensioned plates on a ring pattern of columns and mega columns. Lateral stability and acceleration control are a key design element, thus provision for liquid column damping is provided on the 97th Level.
- Complex 'Star Burst' construction sees amenity zones cantilevering over 8 m.
- Building façade access is by way of a main crane housed at the roof with supplementary systems housed under the 'Star Burst' zone.
- Carparks are concealed behind a pattern of cantilevered planters
- Large diameter piles are used to support the structure; these are founded more than 40 m below ground level.

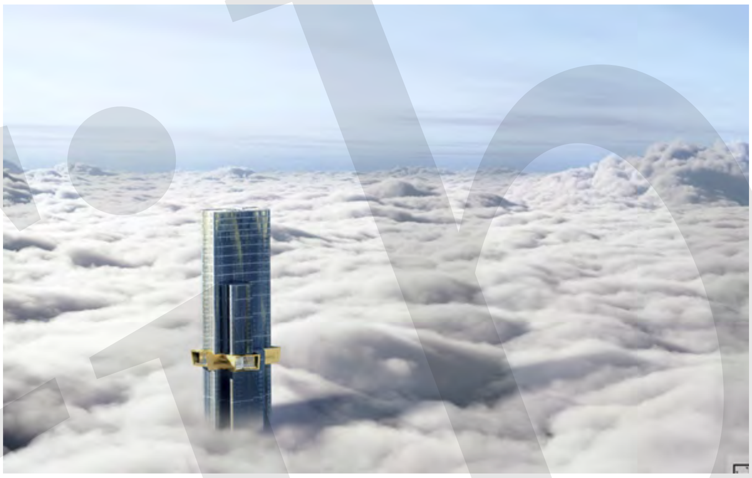

Fig. 12-14b: Australia 108, Melbourne, Australia.

Australia 108 is a highly sculptural residential tower unlike any other in Australia. Its slender form is highlighted at the Cloud Residences levels by a golden starburst expression and then morphs into a curvaceous profile against the sky. The starburst which contains the resident facilities is inspired by the Commonwealth Star on the Australian flag and is an obvious celebration of the sense of community within the building.'

Project Team:

- Client: World Class Land
- Architect: Fender Katsalidis Architects
- Structural Engineer: Robert Bird Group
- Main Contractor: Multiplex

12.15 Seismic Resistant Office Structure, Shanghai, China

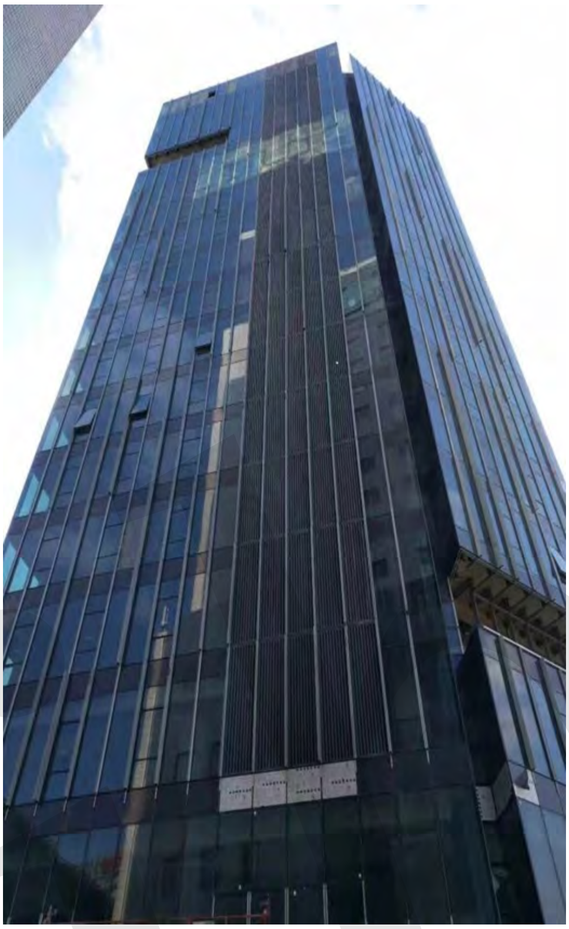

Fig. 12-15a: Seismic Resistant Office Structure, Shanghai, China

Precast Concrete: Beam and column frame, precast floors, with insitu concrete core

Occupancy: Offices

Floor Area: 20'000 m^2

Storeys / Height: 18 storeys, 80 m high

Completion: 2018

One of fi e office blocks in a commercial area of Shanghai, it is the first office building of this height to be framed in precast concrete in the city.

Double tee slabs were possible in a storey height of 4.5 m. They were supported on beam and column moment resisting frames. Connections emulate insitu concrete monolithic construction.

The structure was modelled to resist high seismic activity with a resultant design maximum drift angle of 1/837 with a design base shear force of 13'000 kN.

Fig. 12-15b: Seismic Resistant Office Structure, Shanghai, China.

Project Team:

- Client: Greenland Group
- Architect: Yinglu Xu, ZHONGSEN Design
- Structural Engineer: Xinhua Li, ZHONGSEN Design
- Main Contractor: Jun Wang, XINGBANG Engineer

12.16 Torre de Cristal, Madrid, Spain

Fig. 12-16a: Torre de Cristal, Madrid, Spain.

Precast Concrete: Hollowcore floors

Occupancy: Offices

Floor Area: 60'000 m^2

Storeys / Height: 53 storeys / 250 m high

Completion: 2008

The Torre de Cristal (Spanish for Glass Tower) is in the Cuatro Torres Business Area (CTBA) in Madrid, Spain, and is one of a group of four towers completed in 2008. It ranked as the tallest building structure in Spain when completed, and the fourth in the EU.

The building frame is generally insitu concrete with heavy concrete / steel composite columns and beams around the perimeter. The composite columns had encased large steel sections working with high strength concrete that minimised the effect of differential vertical displacement between the perimeter structure and the stiff central core. This, in turn, would allow more flexibility in the choice of flooring solution.

The main framing elements are horizontally connected by precast prestressed hollowcore slabs topped with insitu concrete. Careful attention to detail was required to ensure effective connections between the floor slab and framing elements to achieve the idealisation of the structural analysis.

The hollowcore solution was chosen because the slabs were readily available from local producers at reasonable price, its general versatility, and its compatibility with the elevations. There is also the huge benefit of improving construction speeds. This choice allowed completion of one floor inside a cycle of one week for an individual floor area of 51 m x 33 m.

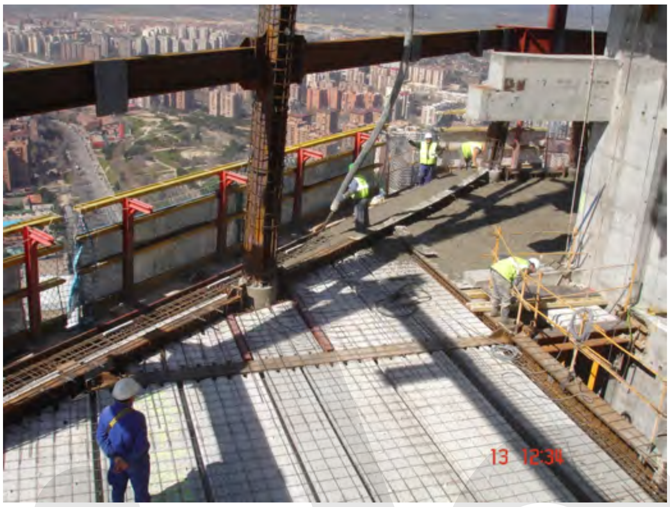

Fig. 12-16b: Torre de Cristal, Madrid, Spain.

Project Team:

- Client: Mutua Madrilena
- Architect: Cesar Pelli and Associates
- Structural Engineer: OTEP International (initial design) / FHECOR Ingenieros Consultores (finalised design)
- Main Contractor: Dragados

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