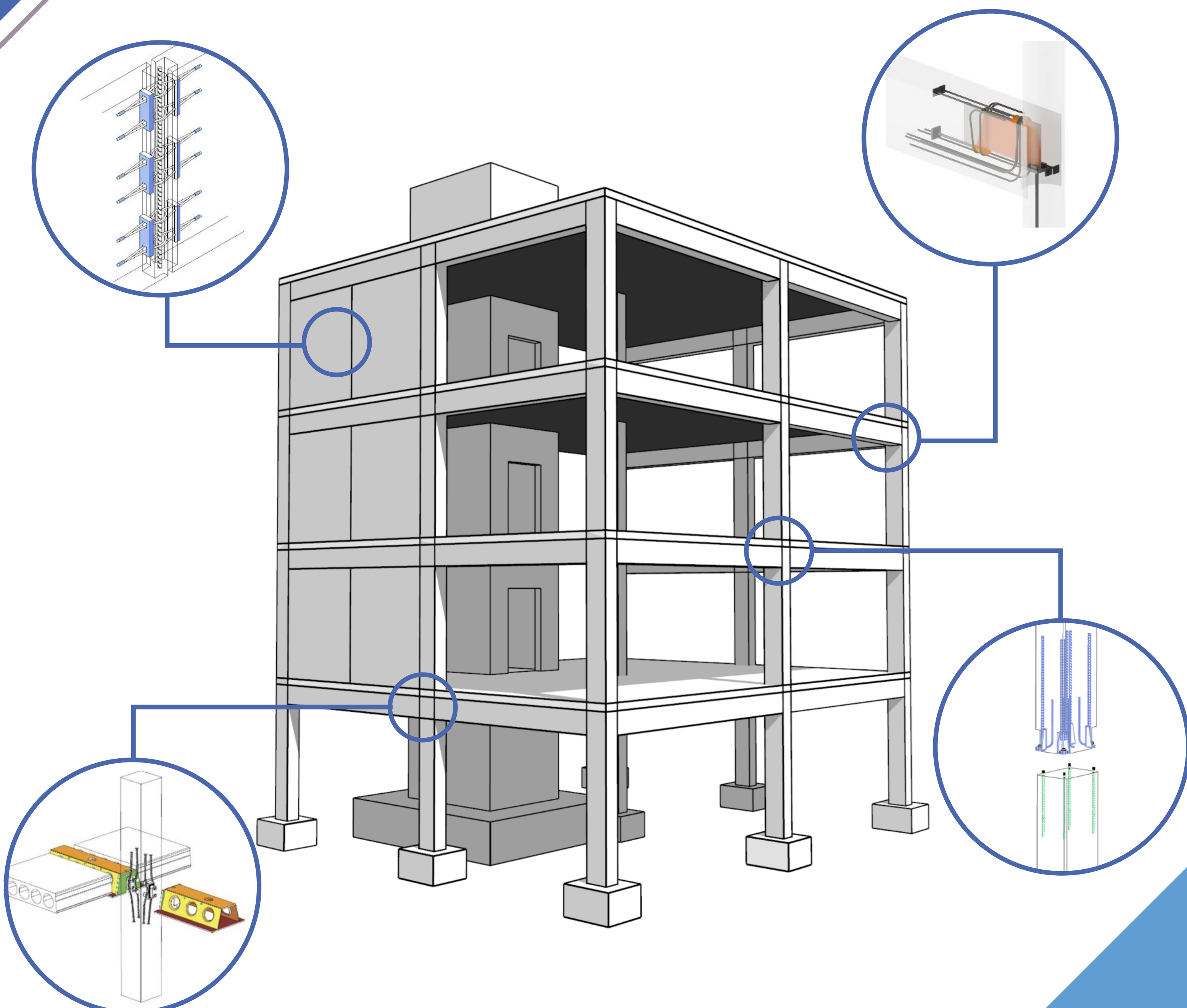
Building and Construction Authority

Design for Manufacturing and Assembly (DfMA)

CONNECTIONS FOR ADVANCED PRECAST CONCRETE SYSTEM



DISCLAIMER

Structural Engineers and Qualified Persons shall exercise due diligence and shall be fully responsible when adopting the information in the guide. The adoption of any types of connectors shall be in accordance with the manufacturer's guidelines and recommendations.

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FOREWORD

A key focus of the Construction Industry Transformation Map (ITM) is to champion widespread adoption of Design for Manufacturing and Assembly (DfMA) technologies. By moving construction activities from worksites to a controlled factory environment, projects can reap the benefits of higher productivity and quality.

Advanced Precast Concrete System (APCS) is a key technology under the DfMA continuum. Building on the experience gained from adopting precast concrete in more than three decades, we are working closely with the industry to advance precast concrete designs and technologies for higher productivity and quality.

This guidebook represents a joint collaboration between BCA and the industry, and serves to provide guidance on incorporating DfMA principles in the design, fabrication, and installation of APCS. Good practices on connection design and detailing, efficient automated production, and productive site installation are provided to practitioners adopting precast concrete systems. The guidebook also features projects in Singapore adopting precast concrete systems that reduce in-situ wet works.

This guidebook is not meant to be a definitive publication on how connections for APCS must be designed, fabricated and installed. Practitioners are encouraged to use this guide to innovate and continually improve the system, so as to achieve higher productivity and quality. To obtain more comprehensive information and guidance, readers should seek professional advice from designers and suppliers of such connection systems. We gratefully acknowledge the contributions of key technical agencies and industry practitioners in the production of this guidebook and trust that the industry will find this publication useful. We welcome any contributions from readers to improve subsequent editions of this guide.

Neo Choon Keong

Deputy Chief Executive Officer
Industry Development
Building and Construction Authority

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A Technical Committee and Working Committee were formed to review the contents and good practices identified. We wish to thank the co-authors and members of both committees for their valuable contributions and feedback in the review of this guide.

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CONTENTS

INTRODUCTION

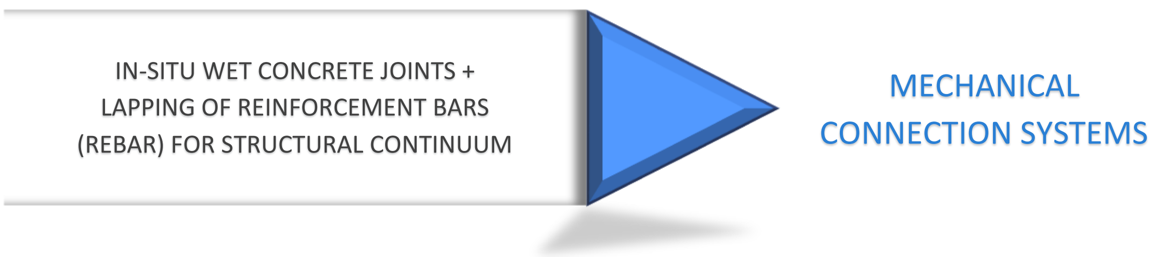
Precast Concrete (PC) construction was introduced in Singapore in the early 1980s for the public housing developments. This construction method was made prominent through the 1990s with increasingly advanced design and on-site practices, eventually becoming the preferred construction method in Singapore's construction industry.

However, apart from embracing a more modular and systematic design in both structural and architectural works to improve site productivity and cost efficiency, the design techniques used in structural connection systems in local PC construction have seen little changes since the late 1990s.

The concept of Design for Manufacturing and Assembly (DfMA) which emphasises the use of precast and prefabricated building components have been strongly advocated in the construction industry in recent years. The underlying principles of DfMA call for more efficient building design and construction detailing, especially in the area of PC connection designs, so as to minimise the amount of wet works on-site.

PRECAST CONCRETE CONNECTIONS

This guidebook will focus on the DfMA approach in PC connection designs. To achieve higher site productivity, the guidebook will focus on the following shift:



The adoption of mechanical connection systems to connect structural components have been extensively adopted in Europe for some time and is envisaged to substantially reduce the amount of in-situ wet joints commonly adopted in local construction sites at present. This shift could potentially help to improve the site productivity by an average 2-3% per annum by 2020.

Advanced Precast Concrete System (APCS) requires major concept modifications in structural connection design and detailing. It entails the adoption of mechanical connection systems to realise the maximum advantage of DfMA design principles in both offsite precast manufacturing and on-site assembly works. Figure 1 illustrates the various types of PC connections applicable to our industry's context.

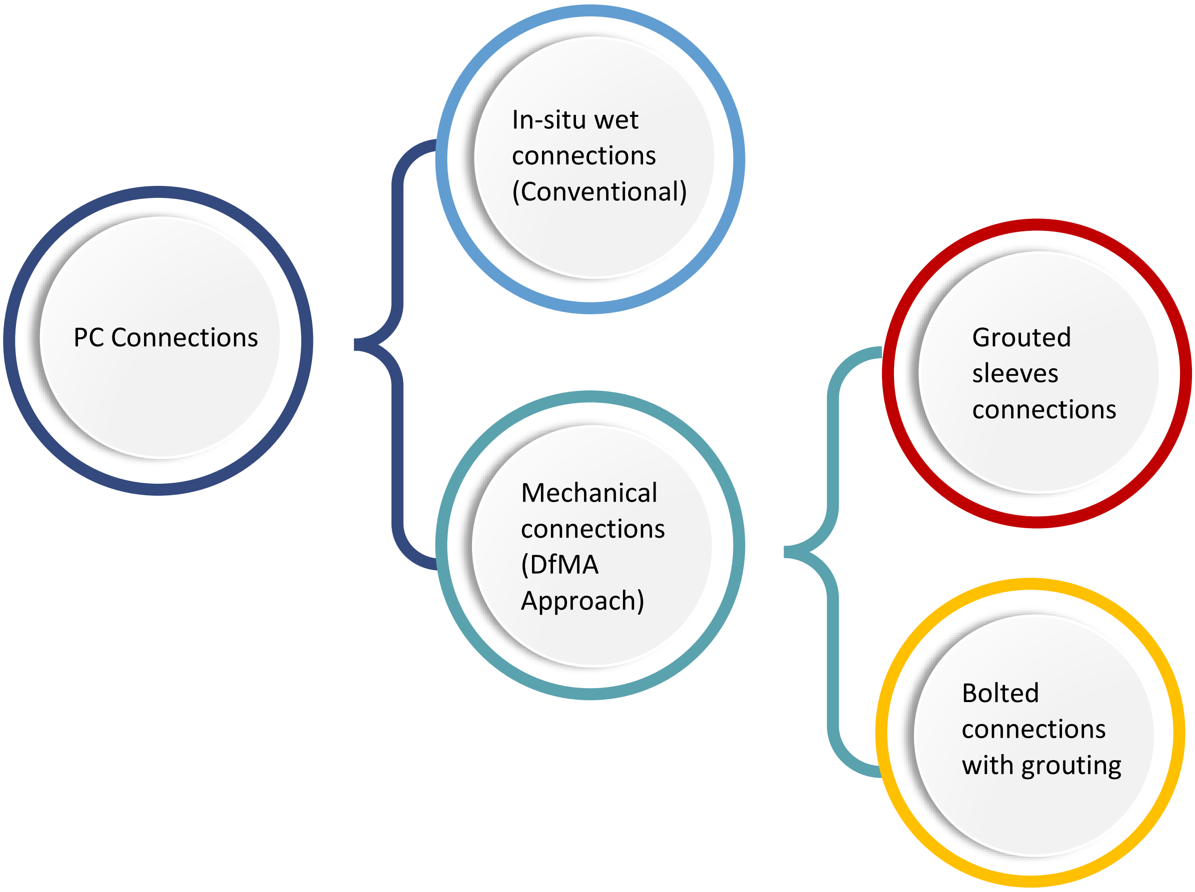


Figure I: Various types of PC connections with highlight of DfMA alternatives

The design approach incorporating DfMA principles in APCS for architectural and structural works will be shared in the subsequent chapters. The design approaches are based on various case studies of successfully completed projects in recent years.

To facilitate the adoption of more efficient PC connection systems, the design and features of proprietary products[1] by various suppliers are also introduced in this guidebook to provide better understanding. It does not implicit endorsement of any product.

Apart from sharing the basic fundamentals and considerations in PC connection designs (with reference to Eurocodes requirements), practical design examples and application considerations of proprietary connection systems are also included.

Structural designers can further examine and evaluate the various proprietary connection systems by seeking suppliers' technical support and guidance for relevant product application.

[1] The inclusion of proprietary products by specific suppliers in this guide is purely for illustration purpose. This guide is not an endorsement of these products by BCA nor does BCA give any warranties, explicit or implied, regarding the availability of the products, information or services, efficacy of the products or services or warranties of merchantability, fitness for a particular purpose, title or non-infringement. The supplier/user must ensure that their products are safe for public use and comply with all relevant regulations and statutory requirements at all times.

KEY FEATURES IN ADVANCED PRECAST CONCRETE SYSTEM (APCS)

OUTCOME BASED APPROACH

EASE OF ON-SITE ASSEMBLY

IMPROVED SITE PRODUCTIVITY

SHORTER FLOOR CYCLE
no increase in the number of workers/mandays compared to conventional pc construction

BETTER QUALITY AND IMPROVED SAFETY during site execution

ACHIEVABLE (BUT NOT LIMITED TO)

- Less wet joints and cast in-situ pour strip between PC components
- Minimal or no temporary work / falsework
- Integrated PC components (e.g. integrated with architectural or mechanical, electrical and plumbing finishes)
- Larger PC components that would optimise crane usage
- Higher standardisation in structural design and connection detailing
- Simplified structural design and connection
- Wider adoption of automation in PC manufacturing

CHAPTER 1

DESIGN BASIS FOR ADVANCED PRECAST CONCRETE SYSTEM (APCS)

1.1 GENERAL

1.1.1 In developing an APCS, the functional considerations, building aesthetics and design approaches must first be understood by the various building disciplines, before conceptualising and commencing design development to meet the project specifications.

1.1.2 The APCS design must be coordinated with the specification for components manufacturing, construction planning and site work.

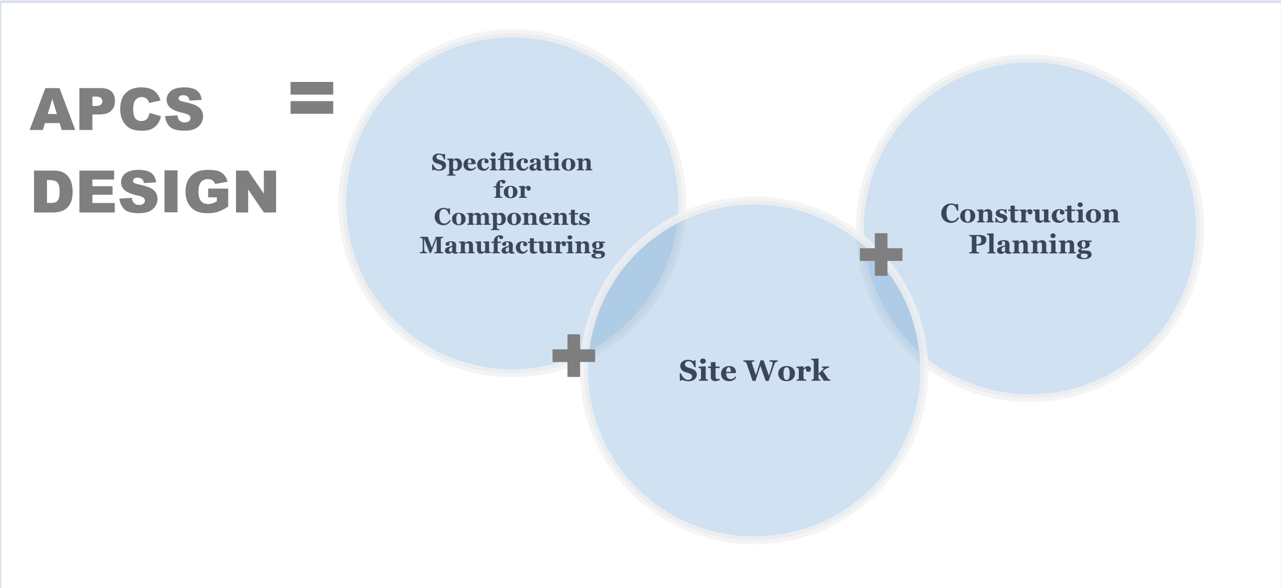


Figure 1.1: **Key factors for APCS Design**

1.2 THE UNDERLYING DESIGN APPROACH AND OBJECTIVES

1.2.1 An efficient Precast Concrete (PC) construction design:

- Commences on a modular architectural layout,
- Adopt high degree of the PC components repetition with optimal standardisation in connection design,
- Achieve economy in mass production and
- Ease site assembly work.

1.2.2 ***Figure 1.2*** below shows a total design approach in PC components manufacturing and site work, to achieve high productivity.

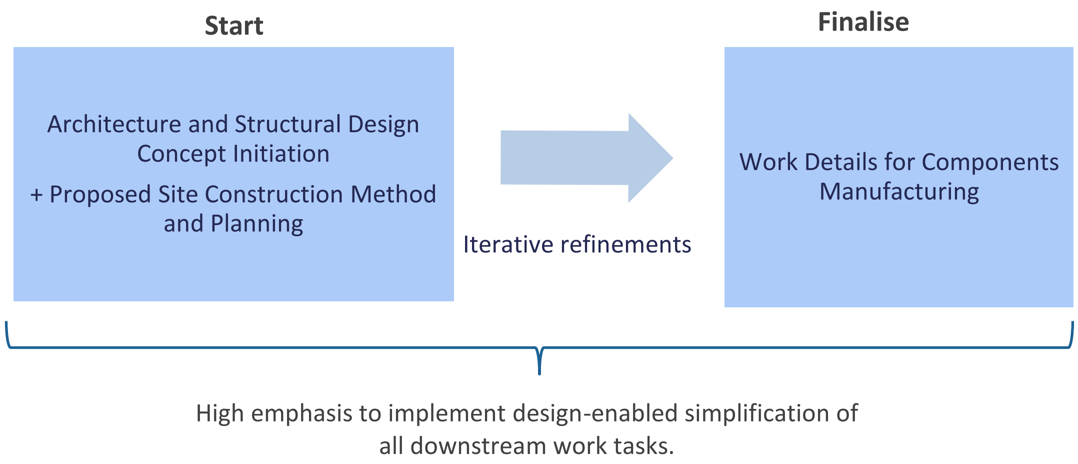


Figure 1.2: **Total design approach in PC components manufacturing and site work to achieve high productivity**

1.2.3 Structural connection design and construction plays an important complementary role in the PC construction system. It influences the PC construction methodology at the manufacturing plant and site work. It is also a factor to facilitate higher automation and resources optimisation in manufacturing and efficient erection of PC components.

1.2.4 Higher productivity can be achieved through design standardisation in component type and profile design in PC manufacturing, as compared to traditional PC manufacturing with low automation in the production handling and dependence on semi-skilled labour. Hence, the developments in work details through innovative connection design is effective in maximising the advantages of automation in the manufacturing plant.

1.2.5 Manufacturing automation using robotic technology with no manual intervention requires the following:

- Work processes to be streamlined to enable the fully automated assembly of prefabricated components
- Avoid complex detailing in connection design *(for example in current in-situ joint design)*

The disadvantages of manual interventions are:

- Extra time and labour input in the preparatory process
- Holding up other downstream processes
- Total automation become ineffective

DESIGN

a) Easier standardisation in the connection detailing.
b) Simplification and faster in detailed calculation check process.
c) Faster completion in selection of connection products to satisfy design force requirement. This can be done through the proprietary system suppliers' design software in which the designers can carry out quick design

MANUFACTURING

a) Faster preparatory process. For example, the quick fixing of connectors into a standard mould
b) Minimal manual work in customised moulding.
c) Time-saving in automated production lines.

In contrast, the current in-situ connection details are not only complex, it also involves difficult customised moulding work and block-outs. These also include the manual fixing of many loose rebars at the connection.

ASSEMBLY

a) Quicker and simpler works at the site through reduction or elimination of temporary propping, scaffolding, and stability bracing works.
b) Easier grouting work.
c) Faster site erection with an unbraced tall column or wall installation and unpropped floor construction. In such design, the connectors also function as temporary erection seating support and leveller thus saving crane time and temporary shoring. Nevertheless, it is important to note that total system stability at temporary stage must not be neglected in all situations.

1.2.6 The potential enhancements and advantages offered in various work processes are strong reasons for using proprietary mechanical connection systems. The key in APCS design is applying the appropriate PC connection system.

1.3 ARCHITECTURAL DESIGN ASPECT

Architectural design is a process to create a built environment with functionality for human comfort and sustainability through art and engineering design. In an effective PC construction, the external facade design is generally achieved through optimal standardisation in the following:

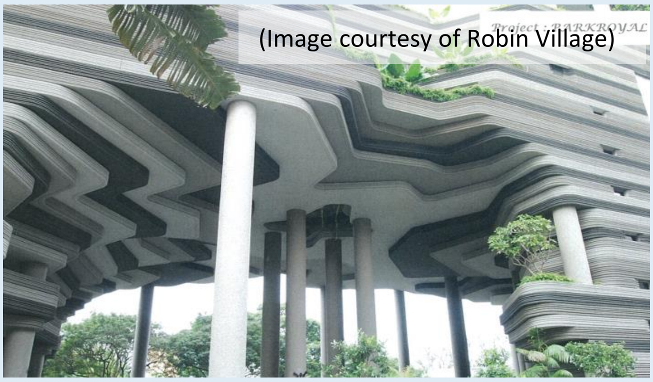
Shapes, Forms and Patterns	a) Base steel mould with fibre-reinforced plastic (FRP) mould, rubber mould lining, cast-in tile and granite slab. b) They can come with an intentional break or non-continuity in groove line or pattern at the panel connection to avoid the issue of misalignment during erection.
Textures	Surface polished, off-form, sand blasted, rope and hammered, reconstituted stone, exposed aggregate, revealed, acid etched.
Colours	a) Coloured cement or natural aggregates, pigments. b) There could be inconsistency in colour due to varied base material supply sourced locally.
Integration of Features	Elevation design variations created by integrating add-on features to a standard flat or simple facade module base panel.
Repetitions in Elements Profile and Section	Curves and special features design with built-in repetitive characteristics, such as: a) Limiting types of common radii in curvature b) Simple and consistent recess for panel section standardisation These factors help to enable economical mould fabrication for practical precasting and construction.
Modular Design	a) Plan and elevation design using repetitive multi-bay, cluster and multi-floor sub-modular arrangements. b) It is to create variation with interesting combination and standardisation of key dimensions.
Rotational *(instead of mirrored)* Modular Layout Design	a) Repetitive clusters in a floor plan with asymmetric rotation is preferred instead of the mirrored layout design. b) The asymmetric rotation method can create more identical PC components. c) Mirrored PC components generally do not support mould sharing, resulting in higher costs.

Examples of successful applications of the above design approaches in some Singapore projects are compiled in the subsequent sections.

EXAMPLES OF PRECAST CONCRETE CREATING ARCHITECTURAL UNIQUENESS IN SINGAPORE PROJECTS

EXAMPLE 1 – USE OF ARCHITURAL FINISHES ON PC FACADE


(Image courtesy of Greyform)



(Image courtesy of Robin Village)


(Image courtesy of Advan-TIS)

EXAMPLE 2 – PROJECTION FEATURES USING ADD-ON INTEGRATION TO FAÇADE COMPONENT DESIGN

(Image courtesy of Advan-TIS)

Trellis and sunshade features are separate add-on components integrated to a base standard façade panel

EXAMPLE 3 – PROJECTION FEATURES USING REPEATED ADD-ON FEATURES MODULE IN ELEVATION DESIGN

(Image courtesy of Advan-TIS)

(Image courtesy of Advan-TIS)

EXAMPLE 4 – RESIDENTIAL PROJECT USING REPETITIONS IN PC COMPONENT IN CHALLENGING CURVE PROFILE AND SECTION


(Image courtesy of Advan-TIS)

- Standardisation of curve radii in roof trellis and balconies to only a few types
- Adopt common section profiles to increase in mould re-use


(Image courtesy of Excel Precast)

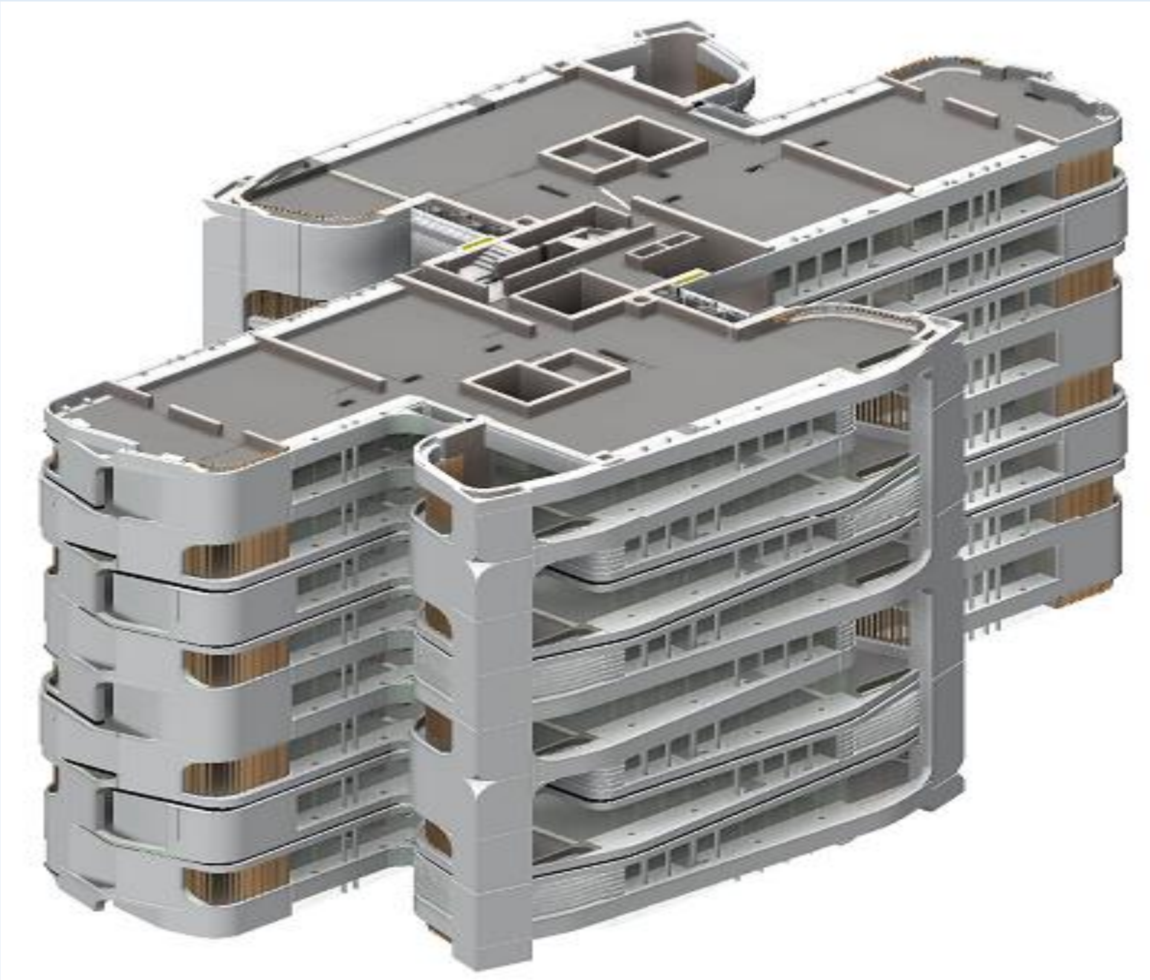

(Image courtesy of Excel Precast)

EXAMPLE 5 – RESIDENTIAL PROJECT USING STANDARDISED PC ROOF COMPONENT 3-D PROFILE AND SECTION APPLYING TO MULTI BLOCKS

PC curved roof canopy feature beams are economised with use of standardised common radii and section in varying length combination

(All images above courtesy of Advan-Tis)

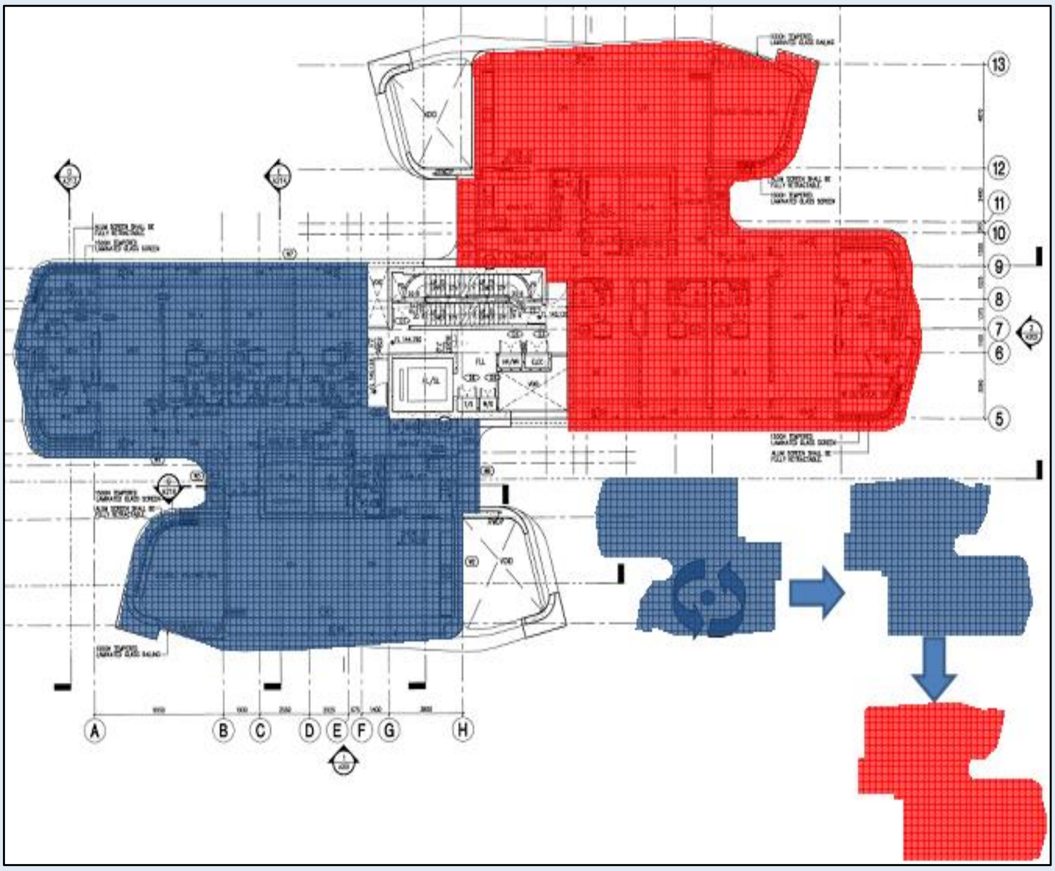
EXAMPLE 6 – INTERESTING MODULAR DESIGN IN ADMORE 7 PROJECT USING REPEATED MULTI-FLOOR MODULES IN ELEVATION DESIGN



Complex design features which are friendly to PC construction

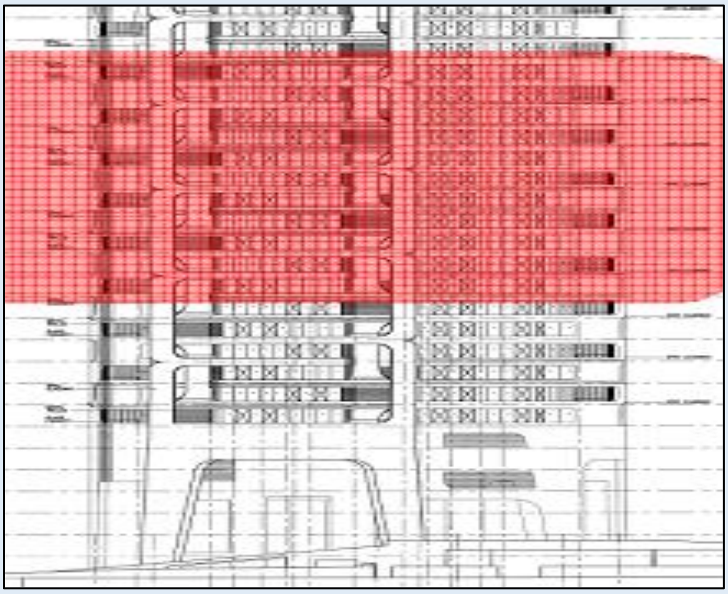
(All images above courtesy of Shimizu Corporation)

EXAMPLE 6 (CONTINUED)



a) Typical floor layout is not mirrored. Instead, it uses the rotation of a common floor segment arrangement (from the front to the rear) to create repetition in its curvy design features.

b) Similarly, the floor elevation modules are repeated after every fourth floor with mirror and protruding design components to create an interesting 3-D effect.



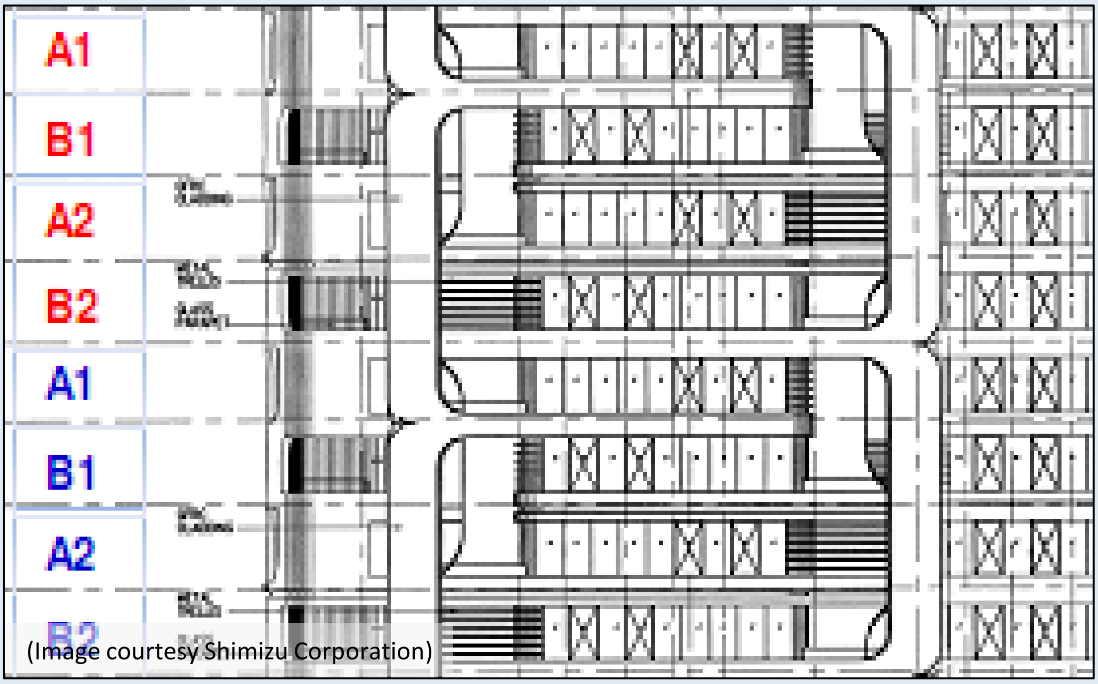
a) Multi-floor modules in a 4-storey stack in two main elevations design.

b) Identical design features which are sometimes mirrored.

c) Increased repetition enables economical use of PC technology.

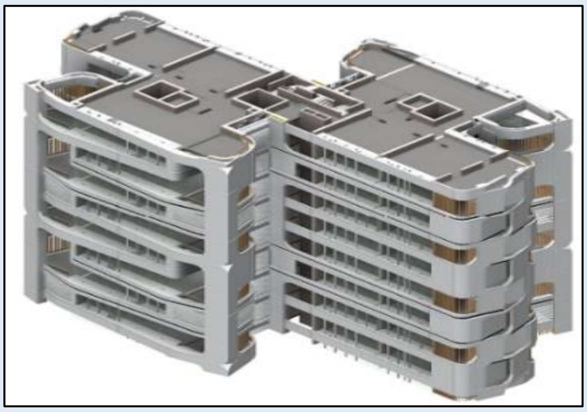
(All images above courtesy of Shimizu Corporation)

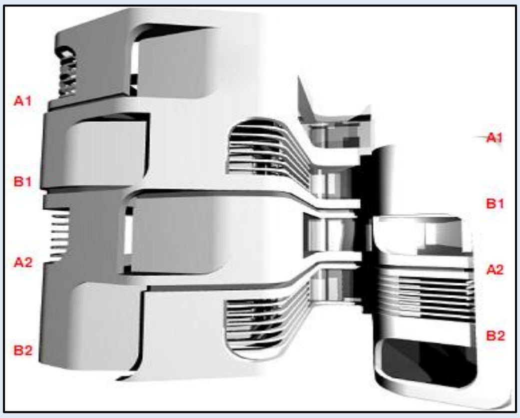
EXAMPLE 6 (CONTINUED)

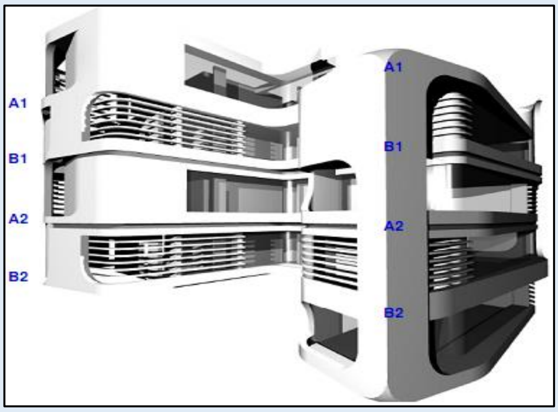


(Image courtesy Shimizu Corporation)

3-D forms and curved features can also be PC friendly when standardisation and modular concept are in-built into design.



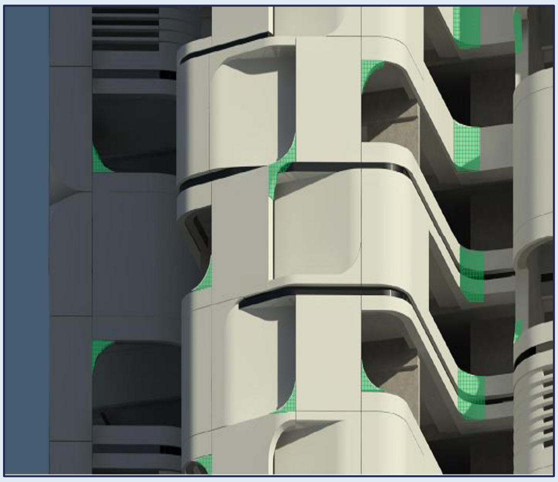


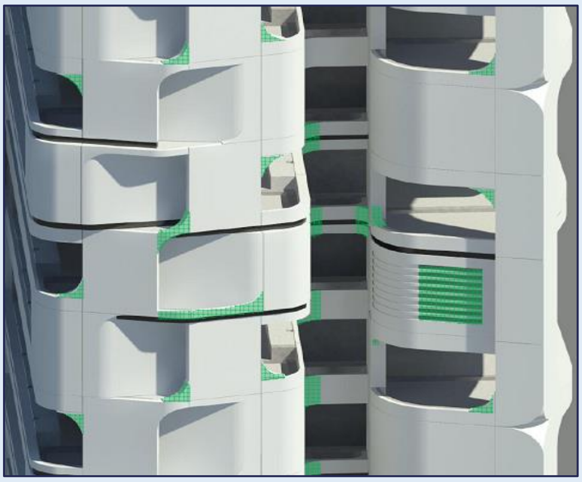


(All images above courtesy of Shimizu Corporation)

EXAMPLE 6 (CONTINUED)

a) Additional glass fibre reinforced concrete (GRC) small panels (highlighted in green) are used at locations to help complete the smooth transition between PC components.
b) Small feature elements such as louvres, are also done in GRC instead of PC for practical and overall effective cost reasons.
c) Limitations in moulding and handling damages to delicate edge profiles in PC components can thus be overcome in such difficult areas.





Note: Small GRC add-on is much more economical as compared to adding complex moulding requirement into the large PC panel

(All images above courtesy of Shimizu Corporation)

EXAMPLE 7 – SUPER TREE STRUCTURES CONSTRUCTION AT GARDEN BY THE BAY

Super Tree Structures with segmented curved 3-D PC components.

(All images above courtesy of Expand Corporation)

CHECKLIST - GOOD PRACTICE FOR ARCHITECTS TO ACHIEVE EFFICIENT PRECAST DESIGN

- Can the majority of main column gridlines be standardized?
- Can the floor layouts be designed using modular arrangements?
  Or can systematic clustering of largely similar units be achieved?
- Can add-on features to the base façade design or sub-modular arrangements be used to create interesting elevations?
- Can repetition in components' profiles, sections and surface features design in building components be achieved?
- Can shapes, textures or colors be incorporated into PC façade with sufficient standardisation?
- Can the design adopt highly repetitive characteristics? (Especially for curves and special 3-D features)

1.4 STRUCTURAL DESIGN ASPECT

1.4.1 Structural System and Design

Structural design has a direct impact on the ease of PC manufacturing and site erection works. The key consideration for structural system design solutions are:

a) The architectural design basic layout arrangement and modular features should be in a gridline system, including the construction execution conditions.

b) Simple structural system contributes to simpler design, work details and quicker construction with lesser components types and on-site connections.

SUITABLE STRUCTURAL SYSTEM FOR VARIOUS BUILDING TYPES

Commercial, institutional and industrial buildings (typical usage and loadings)	Long span design
High-rise residential building	Shear or core walls and flat plates
Medium/low-rise building	A combination of shear or core walls and beam-column system with/without precast floor system

1.4.2 Structural Model and Element Layout Design Considerations

1.4.2.1 Structural analysis with the correct model assumption is important in establishing:

a) PC components design
b) Panelisation layout
c) Site connection works details

It may also dictate the construction sequence and methodology of structural frame erection.

1.4.2.2 The following are the considerations for a practical PC structural system and design:

a) Clear identification of key stability vertical components. For example, shear or core walls to resist lateral forces, especially for medium / high-rise buildings.

b) Correct modelling of connection boundary conditions for structural components. In some cases, modelling is also carried out with the intentional release of connection bending moment to facilitate simple connection design without moment transfer at selected non-critical locations, and yet maintaining adequate overall system stability resistance.

c) Reduce the requirement of moment connection at component joints as much as possible. This can also help in connection design to reduce constraints such as rebar congestion due to lapping requirement within the PC components or inside in-situ wet connection works. It is important to note that hinge joint is generally not to be located at the building external frame due to the high probability of crack development which can contribute to water tightness problem.

d) Suitable division of structural member into PC components and target for high standardisation in component types, section dimensions and profiles for ease of manufacturing and erection handling.

e) Frame structure having simple floor structural system using long spanning one-way slab and main beam supported directly onto column with independent stability structural elements.

1.4.3 <u>Construction Considerations</u>

1.4.3.1 Structural PC component design may sometimes need to consider the temporary loads that act on the structure during site erection, lifting handling and transportation.

1.4.3.2 The following are the PC design considerations for construction stage:

a) Size and weight of PC components, and the corresponding available crane capacity at the site. This is to facilitate ease of delivery and erection lifting.

b) Sequence of floor and superstructure erection, and its associated temporary stage stability.

c) Forces acting at PC connection during erection before permanent connection.

d) Design of self-supporting PC component to minimize or avoid temporary propping, shoring or bracing of tall vertical or long horizontal components at erection.

e) Construction equipment weight on a partially erected structure.

f) Other transient forces during erection, such as the operation of special heavy lift jack, tower crane tie-back or mobile crane movement on an incomplete structure.

TYPES OF STRUCTURAL SYSTEMS

All semi or full PC structural systems need clear consideration for stability. It may take one of the following forms:

a) Skeletal frames, i.e. Beam/Column/Slab (braced and unbraced) – ***Refer to Figure 1.3***

b) Load-bearing walls, gable end and party walls with flat plate slab – ***Refer to Figure 1.4***

c) External façade load bearing walls with precast or in-situ floor system – ***Refer to Figure 1.5***

d) Cells or 3-D Box elements with column/beam frames – ***Refer to Figure 1.6***. The installation sequence is illustrated in ***Figure 1.7.***

e) A mixed system of the above

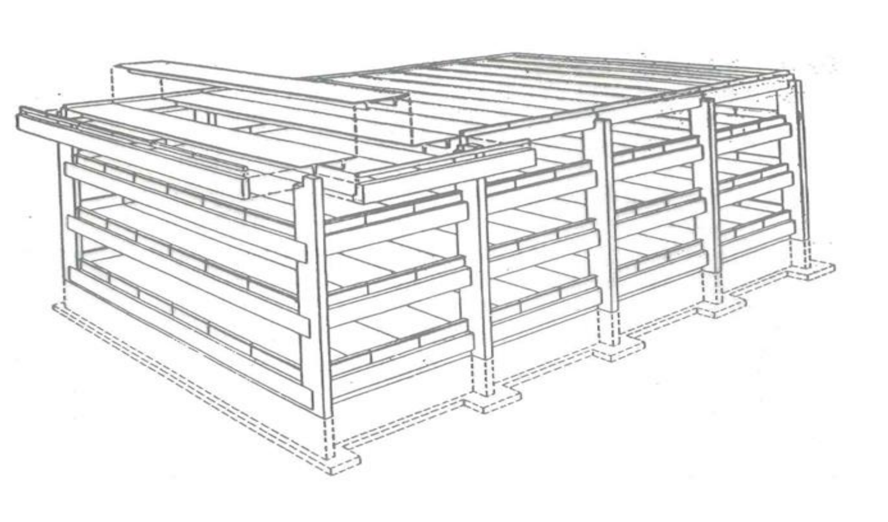
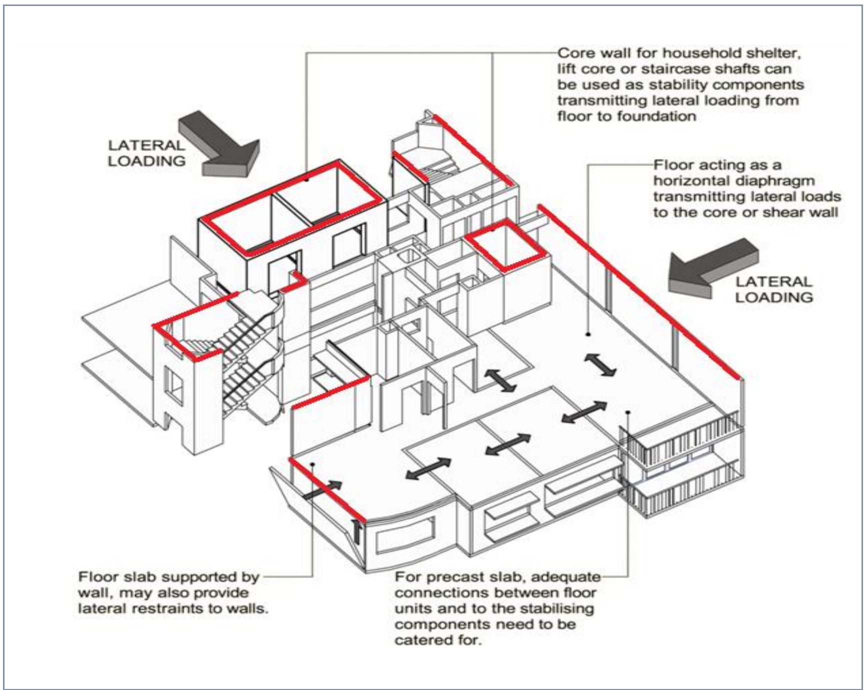


Figure 1.3: **Skeletal column-beam frame structure without stability provided by shear walls/core walls. This demands full moment connection design or cantilevered columns with fixed base/foundation Long spanning multi-storey car parking structure**

Figure 1.4: **Lateral stability provided by shear walls or core walls in lift and stair areas so as to minimise moment in columns for simple connection design**



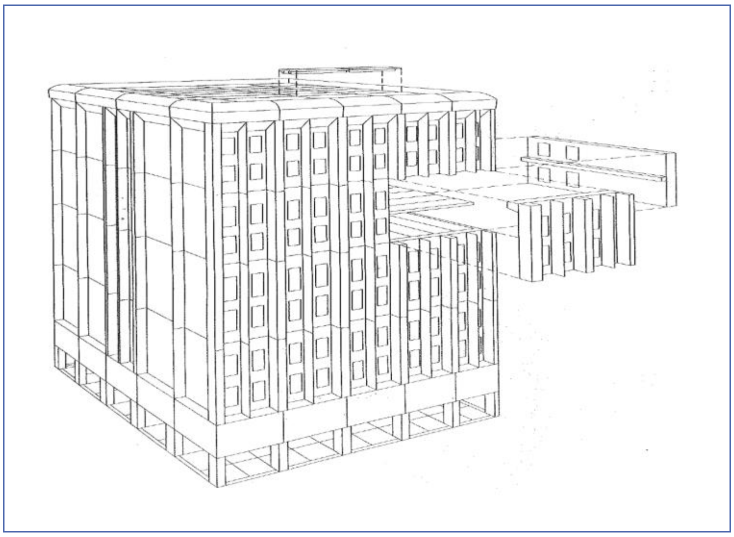


Figure 1.5: **Precast external façade load bearing large panels with internal core-wall and precast floor construction**

Figure 1.6: **Cells or 3-D Box elements with column / beam frames**

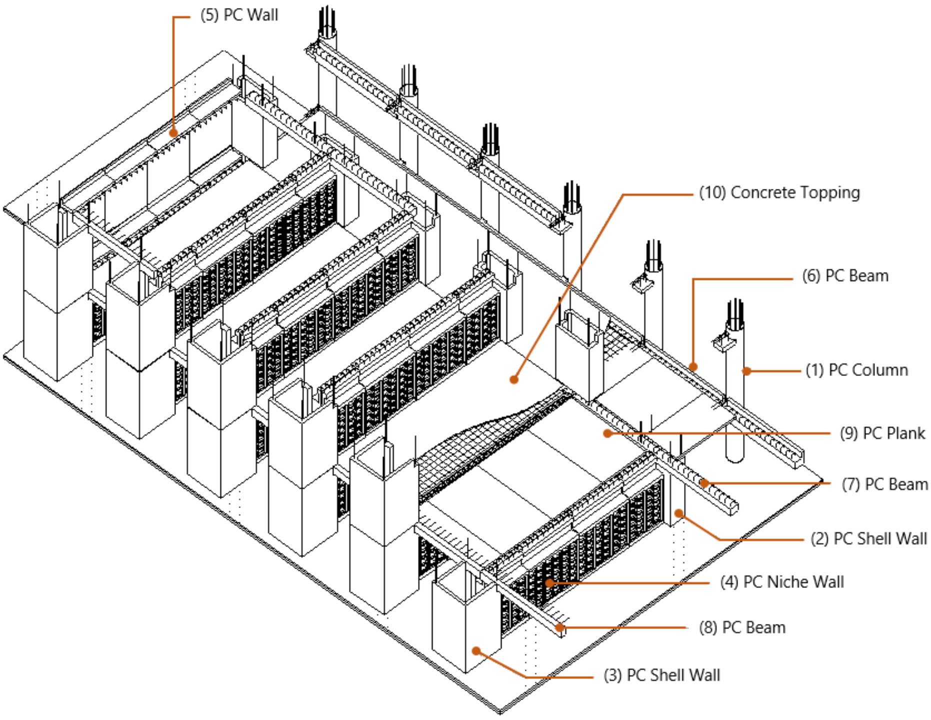


Figure 1.7: **Proposed sequence of erection for PC components**

CHECKLIST - GOOD PRACTICE FOR ENGINEERS TO ACHIEVE EFFICIENT PC SYSTEM DESIGN

- [ ] Can overall building stability be catered for with selected vertical components in fewer number?

- [ ] Can the number of floor components be minimised by employing longer spanning design to eliminate secondary beams?

- [ ] For simple connection design, can most minor column / wall components be designed with purely axial load without high bending moment transmission at the base?

- [ ] Can repetition in component types, profiles and sections be optimally achieved by high standardisation?

- [ ] Are the shape, size and weight of components sufficiently friendly for manufacturing, handling, transportation and erection?

- [ ] Can the components design enable unpropped / unbraced erection at site?

- [ ] Is connection design considered for easy fixing during PC manufacturing and fast assembly at site?

1.5 STRUCTURAL CONNECTIONS DESIGN ASPECT

1.5.1 Approach in Connection Design

Connection in PC construction must be designed to take the forces derived in the structural modelling. To achieve work efficiency, the design approach should consider the following aspects.

Simplicity, standardisation in design and durability in service	Construction methodology of PC components during erection at site
Sufficient standardisation in connection types, configuration and dimension details Ease of sub-component fixing during manufacturing Explore the use of suitable proprietary connection systems to simplify design process Fire resistance	Ease and speed of components assembly Good hand access for fixing work at the connection points Reduce and avoid wet connections

1.5.2 Types of Connection Design

1.5.2.1 Connections in PC construction shall perform the following two key functions:

a) To ensure the final structural performance of the building
b) To provide temporary support during component erection

1.5.2.2 The following are the main types of components connection design. In general, the construction is via corbel / bracket seating, wet connection casting with starter rebar, grouted connector sleeve, plate welding and mechanical bolting (or proprietary system).

a) Horizontal floor structural components
   i) Precast slab / Double-T / minor beam to main framing beam
   ii) Precast slab / stair flight / main beam to bearing wall/column

b) Vertical structural components
   i) Base connection of wall or column
   ii) Column to column at splicing connection

c) Cladding element connections in non-critical erection
   i) External façade to perimeter framing beam
   ii) Parapet or spandrel wall

Special attention is required to ensure good accessibility at the connection points during erection. This is to enable quick securing of the PC components and ease of alignment.

1.5.3 Structural Connection Design Technical Considerations

1.5.3.1 Performance-based design criteria of connection are to accomplish the prerequisite of strength, ductility, geometry tolerance, durability and fire-resistance.

1.5.3.2 Connection type can be broadly classified by structural performance into the followings:

a) Compressive connection
b) Tensile connection
c) Shear connection
d) Coupling moment connection

1.5.3.3 Basic understanding of the following is important for engineer to design and develop correct work details for the PC connection.

a) Internal forces at the connection
b) Structural response action in transmitting or distributing them beyond the connection area
c) This is to ensure quality and final satisfactory performance of the in-service structure.

CHECKLIST – DESIGN PRECAST CONNECTIONS

SIMPLE CONNECTIONS

a) Can PC beam connection design be a simple shear connection without temporary seating?

b) Can the connections enable quick vertical site installation with a simple vertical slot-in or bearing / seating?

REPETITION

a) Can vertical and horizontal connections be standardised to just a few types?

b) Is the position / layout of starter rebars or couplers standardised?

c) Can the connection profile dimensions be standardised into limited types?

d) Can the cost of mould be kept low without customisation in side form at connection areas?

FIXING DURING PRODUCTION

a) Are connectors and starter rebars friendly to install and easy to secure into the moulds without any cutting of slot openings or additional side forms to facilitate work automation?

REDUCE NUMBER OF PROTRUDING STARTER REBARS

a) Can the number of connecting rebars be standardised in layout configuration and be reduced with the use of larger size rebars to avoid congestion?

b) Can a proprietary connection system be used without using protruding starter or lapping rebars?

c) Can couplers or threaded inserts be used for site bolting or connecting starter rebars at connection?

d) Can connection rebars which are laid on-site be minimised in PC components?

PC CONNECTION WORKS ARE DIFFERENT FROM IN-SITU

a) Will the connection details allow for easy works execution on-site?

b) Is it feasible to omit nominal redundant rebar continuity?

c) Can a lesser number of rebar, but with larger size be used so as to avoid congestion and simplify installation work?

EFFECT OF CONNECTION DESIGN ON ERECTION

a) Can the proprietary connection systems be employed to enable quick site erection and to eliminate wet work?

b) Can minimal falsework and self-supporting (or un-propped) erection be made possible by suitable usage of proprietary connection systems?

USE OF NON-CRITICAL EXTERNAL PC CLADDING ERECTION

a) Can external cladding façade be designed for non-critical installation after the completion of the main building frame to speed up construction?

b) Can mechanical connectors be employed to facilitate fast fixing of cladding façade?

ON-SITE CONNECTION WORKS

a) Depending on the size of gap and level of stress, has a suitable connection design been chosen for practical work?

b) Are there too many starter rebars to be inserted into the PC connection sleeves simultaneously which resulted in slow erection?

c) Can quicker bolting or threaded coupler be used instead of site welding?

d) Can long starter rebars from PC components with lapping rebars laid on-site be replaced with couplers or to use grouted sleeves for connection?

e) Are there too many starter bars or rebars which are laid on-site inside in the connections causing congestion especially with small connections dimension?

f) Can connection grouting use highly flow-able pre-mix grout for quick filing under low gravity pressure head, to replace the high pressure grouting that requires strong formwork? (except for proprietary sleeve splice)

g) Can final connection work details prevent ingress of water into joint and ensure long term durability?

CONNECTION DESIGN USING THE PROPRIETARY CONNECTOR SYSTEM

a) Has the joint design and detailing taken care of all the forces and in accordance to the analysis assumption?

b) Are the buildability and required construction tolerances considered in the design?

SUMMARY OF DESIGN RECOMMENDATION FOR PC CONSTRUCTION STRUCTURAL CONNECTION

SIMPLICITY

- Easy connection design in structural framing system.
- Adopt an efficient connection system, e.g. connector from the proprietary system

REPETITION

- Connection design with few types and each involving standardised configurations and dimensions

MINIMISE

- Reduce protruding starter rebars from PC components into connections
- Avoid in-situ wet casting connection

1.6 OTHER INNOVATIVE CONNECTION DESIGN AND CASE STUDY

In recent years, builders are also embracing hybrid PC construction as an alternative structural system to achieve higher productivity. Examples include the adoption of Precast Column and Steel Structures (PCSS) at Yishun Community Hospital as well as the Lotus Root PC column connection system adopted in Skyline@Orchard (Residential Project) and CapitaGreen (Commercial Project). These methods use innovative modular component connections at the column and beam joints that simplify installation works on-site.

CASE STUDY 1 – PRECAST COLUMN AND STEEL STRUCTURES (PCSS) AT YISHUN COMMUNITY HOSPITAL

Design Description:

A hybrid PC column and steel structural system was adopted for typical floor construction for this project. In this hybrid system, PC columns are merged with steel beams to form the building's structural frame, while the floor is constructed with concrete over steel decking.

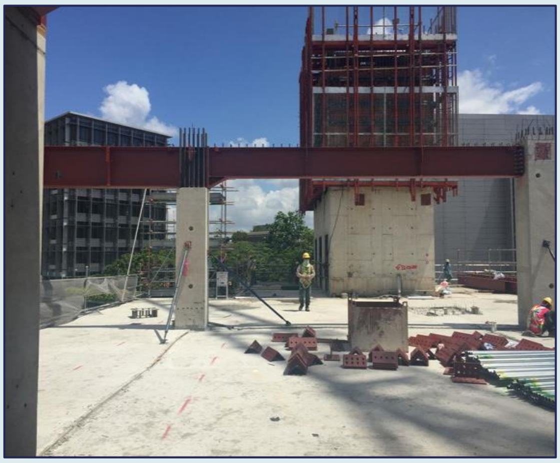


Figure 1.8: Installation of Steel Beam

Figure 1.9: Installation of Bondek Floor

(All images above courtesy of Kimly-Shimizu Joint Venture)

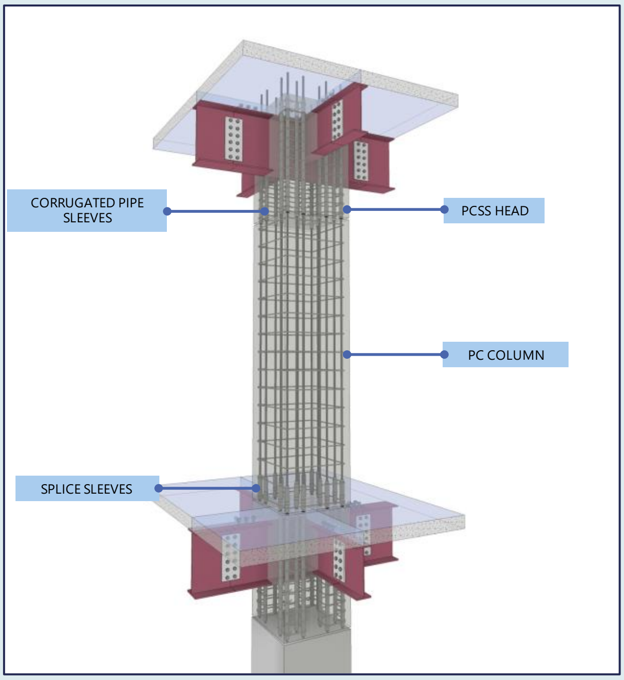


Figure 1.10: PCSS Column Detail

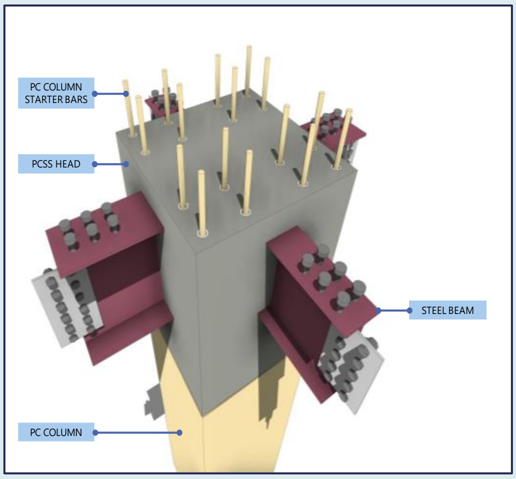


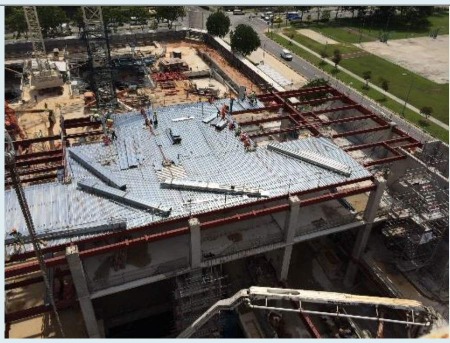
Figure 1.11: PCSS Head Detail

By utilising such hybrid system, builders may potentially achieve as high as 25% productivity improvement when compared to conventional cast in-situ construction. The typical floor cycle schedule was reduced to just 9 days for each zone. The gross floor area for each floor was approximately 3624m^2, and this was divided into four zones at about 906m^2 per zone.

Illustrations of the floor cycle work sequence:

Day 1 Setting out and PC columns installation	
Day 2 Steel beam and staircase PC walls installation	

(All images above courtesy of Kimly-Shimizu Joint Venture)

Day 3 Continue steel beam installation and commencement of metal decking floor installation	
Day 4 and 5 Continue steel beam and metal decking floor installation	
Day 6 Metal decking floor installation	
Day 7 Metal decking floor installation and commencement of rebar installation	

(All images above courtesy of Kimly-Shimizu Joint Venture)

Day 8 Continue rebar installation	
Day 9 Inspection and concrete casting	

(All images above courtesy of Kimly-Shimizu Joint Venture)

CASE STUDY 2 – LOTUS ROOT PRECAST COLUMN AND BEAM CONNECTION AT SKYLINE@ORCHARD BOULEVARD RESIDENTIAL PROJECT

Design Description

The PC column and beam system was adopted for typical floor construction for this project. A PC column with protruding starter rebars at the top end was used to receive crosshead precast main beam with cast-in splice connectors to form the building's structural frame.

Figure 1.12 (above): **Lotus Root connection installation in progress.**

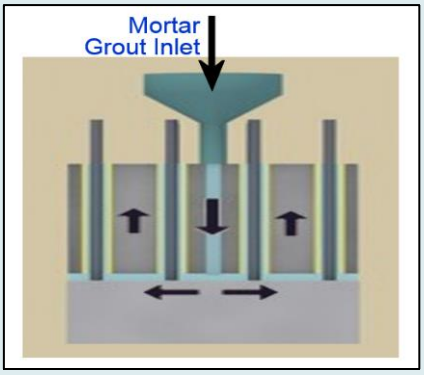


Figure 1.13 (left): **Grouting of beam-column joint of Lotus Root**

(All images above courtesy of Kajima Overseas Asia Pte Ltd.)

Installation of Precast Concrete ‘Lotus Root’ System in a Project in Japan

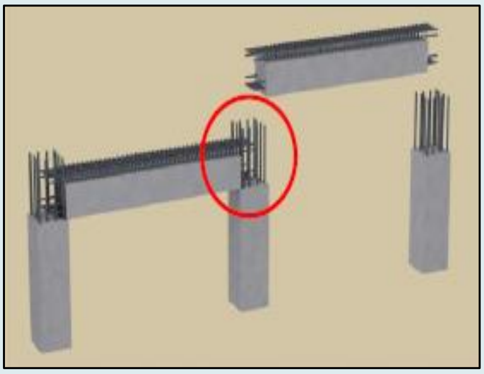
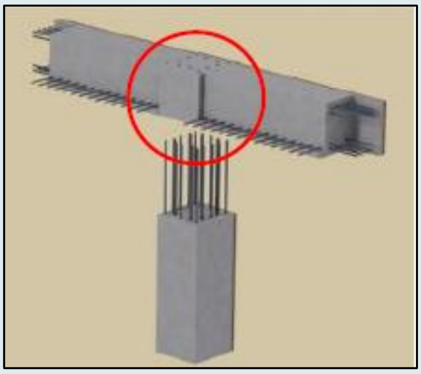


Figure 1.14 (left): **A conventional precast joint with in-situ casting at column head**

Figure 1.15 (right): **A Lotus Root precast joint without in-situ casting at column head**



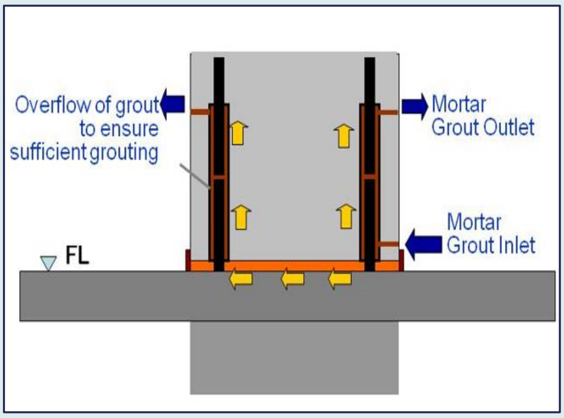


Figure 1.16: **Grouting of a column base with splice connector after completion of Lotus Root precast joint**

Figure 1.17: Site erection of a Lotus Root precast joint

Figure 1.18: **Site erection of a Lotus Root precast joint**

(All images above courtesy of Kajima Overseas Asia Pte Ltd.)

Kajima Overseas Asia Pte Ltd, in collaboration with Kajima Corporation (Japan), precaster, builder and consultants adopted the Precast Concrete 'Lotus Root' System for the Skyline@Orchard Boulevard residential project. Floor cycle time was reduced from 12 days (if conventional PC construction was adopted) to 8 days. The general illustration of the activities during the 8-day floor cycle, are appended as below. The gross floor area for each floor was about 312 m^2.

Illustrations of the floor cycle work sequence:

Day 1 Site preparation – survey set-out (after slab casting) Quality check and ensure the PC panels delivered to site are according to delivery order and approved shop drawings	 
Day 2 PC components (vertical) installation and grouting works	

(All images above courtesy of Kajima Overseas Asia Pte Ltd.)

Day 3 Safety screen and loading platform jack-up, slab table system form transfer	
Day 4 Slab table system form setting out and formwork installation	
Day 5 Perimeter PC beam installation, grouting works and slab formwork installation	    

(All images above courtesy of Kajima Overseas Asia Pte Ltd.)

Day 6 and 7 Slab reinforcement placing and fixing	 
Day 8 Slab concreting works	

(All images above courtesy of Kajima Overseas Asia Pte Ltd.)

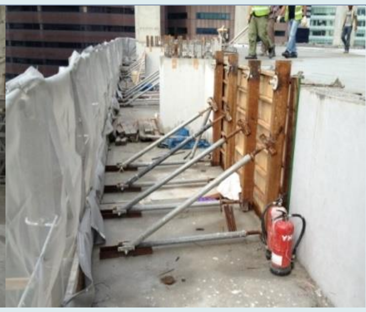
CASE STUDY 3 – LOTUS ROOT PRECAST COLUMN AND BEAM CONNECTION AT CAPITAGREEN

Figure 1.19: **Erection of the Lotus Root system**

This system was adopted by Takenaka Corporation in the CapitaGreen commercial project for its mid-to-high level structural system. By adopting the Lotus Root System, the project was able to achieve 6-day floor cycle for each zone. The floor cycle is illustrated as below.

Day 1 PC beam installation	 
Day 2 Structural steel beam installation	 

(All images above courtesy of Takenaka Corporation)

Day 3, 4 Structural steel deck / BRC installation	
Day 4 Grouting of PC connection (beam and top of column)	 
Day 5 Grouting of PC connection (bottom of column) and PC column installation	 
Day 6 Slab casting	 

(All images above courtesy of Takenaka Corporation)

CHAPTER 2

DESIGN OF CONNECTIONS IN PRECAST CONCRETE SYSTEM

2.1 GENERAL

2.1.1 Design methods for joints in precast construction have been covered by various international technical journals and published academic research papers. Their recommendations are mostly derived from the following

a) The load stress path distribution
b) Structural mechanics
c) Response behaviour of the connection systems

2.1.2 For proprietary connection systems, the design theory used is often verified by product principal using dedicated laboratory tests or research programmes. This is carried out to assess the failure modes and safe structural capacity in the simulated application of the mechanical connector devices.

2.1.3 This chapter aims to:

1 Provide basic understanding on the general fundamental principles used in various precast connection designs.

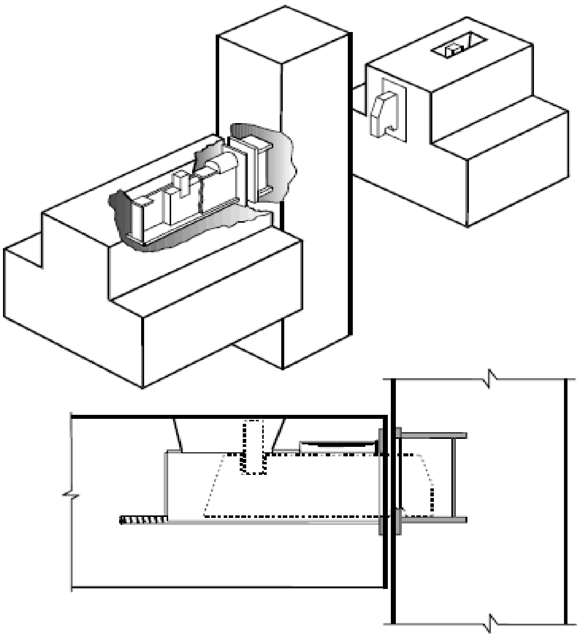
2 Include discussion on locally available and proprietary mechanical connection systems.

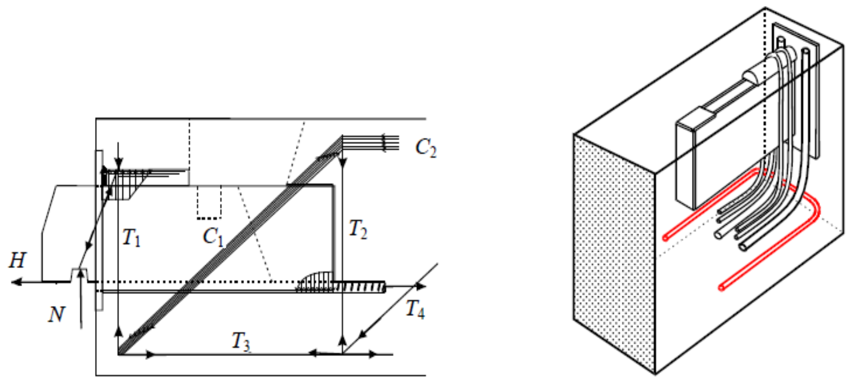
These principles and discussions are meant for the structural designers' consideration of adoption.

2.1.4 Some examples using proprietary mechanical connectors are illustrated as follows:

A) CORBEL-CUM-BEAM END JOINT DETAILS USING CONCEALED MECHANICAL CONNECTORS

Note: Designer shall include provisions to address any possible torsional rotational effect induced at joint caused by imbalanced or eccentric loads on the beams

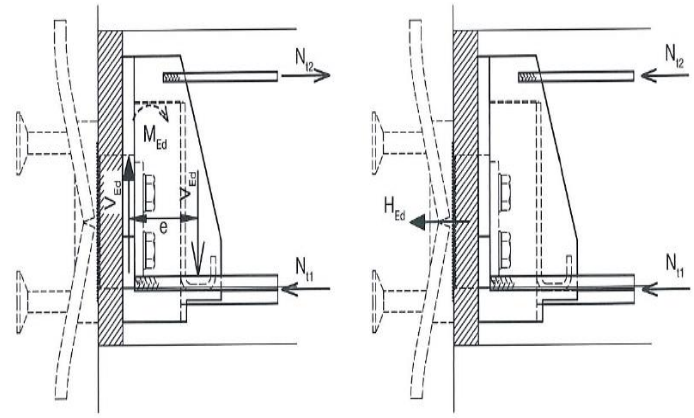


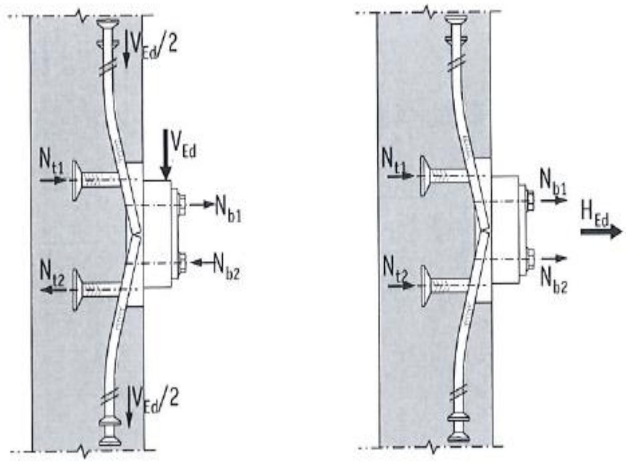


a) b)

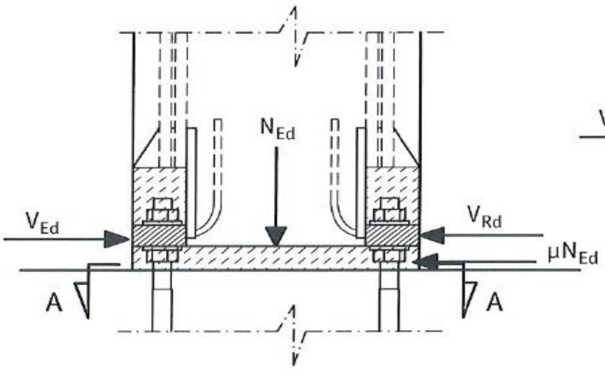
N = Vertical support load
H = Horizontal support load
T_1 = Resulting tensile force at the front of the beam unit
T_2 = Resulting tensile force at the rear of the beam unit

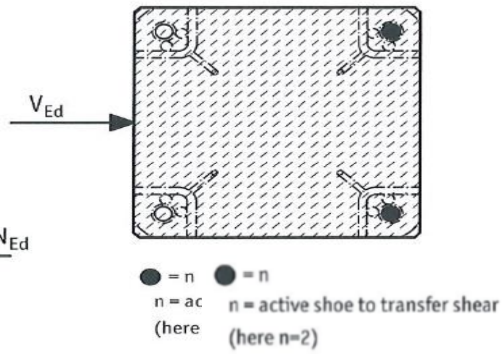
T_3 = Resulting tensile force at the bottom of the beam
T_4 = Tensile force caused by restraint from the column unit
C_1 = Resulting compressive struts on each side of the beam unit
C_2 = Resulting compressive force in the top of the beam

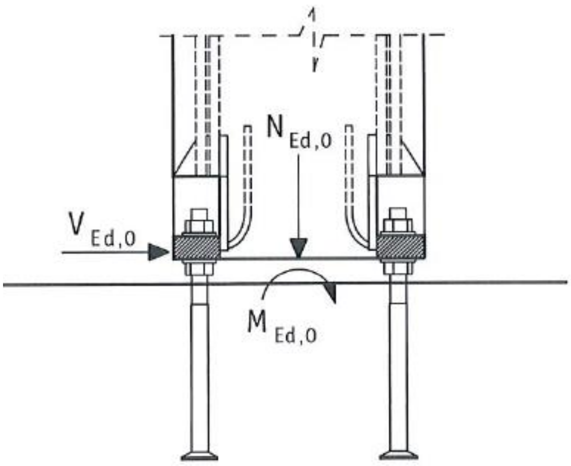

N_t2
M_Ed
V_Ed
V_Ed
e
N_t1
H_Ed
N_t2
N_t1

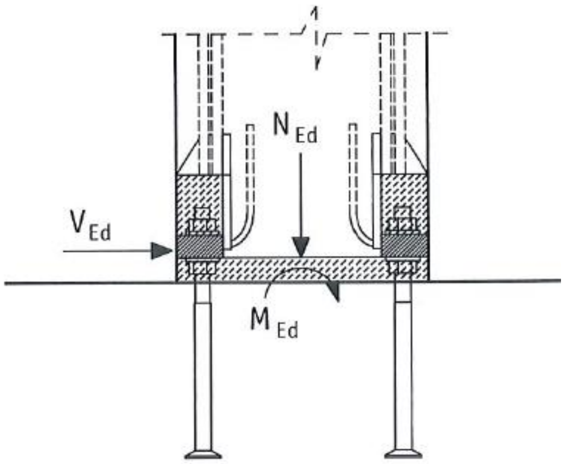

V_Ed/2
V_Ed
N_t1
N_b1
N_t2
N_b2
V_Ed/2
N_t1
N_b1
H_Ed
N_t2
N_b2

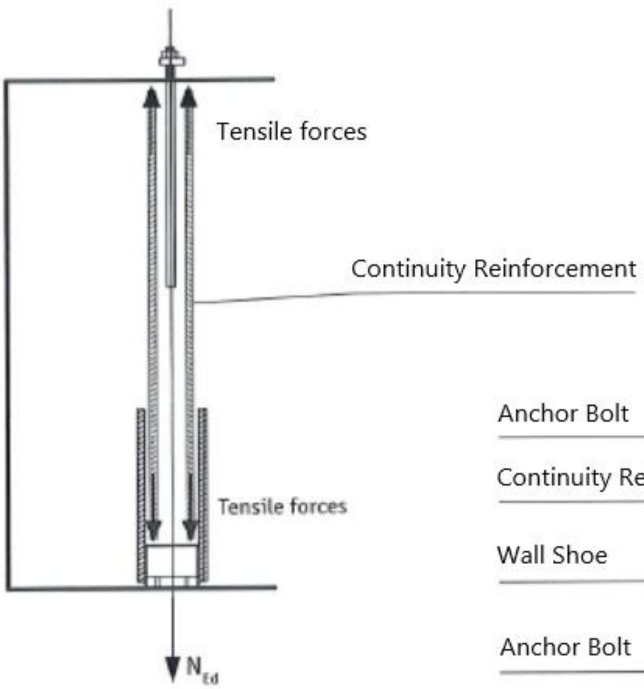
B) COLUMN AND WALL CASE CONNECTIONS USING MECHANICAL STEEL CONNECTORS

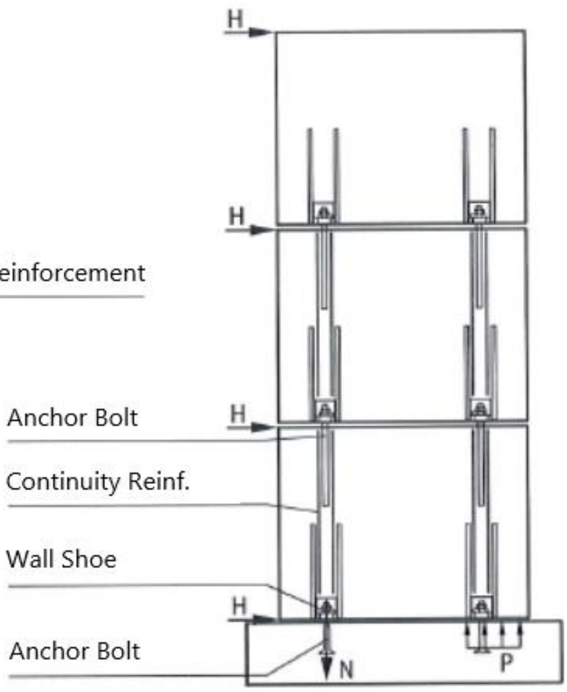




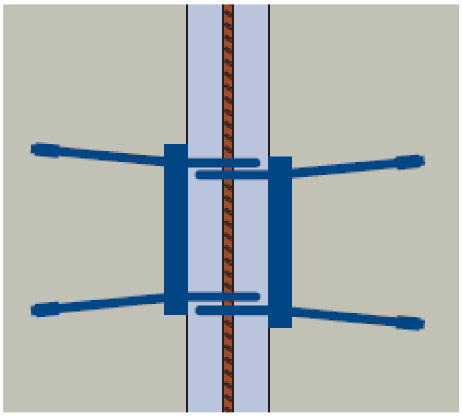


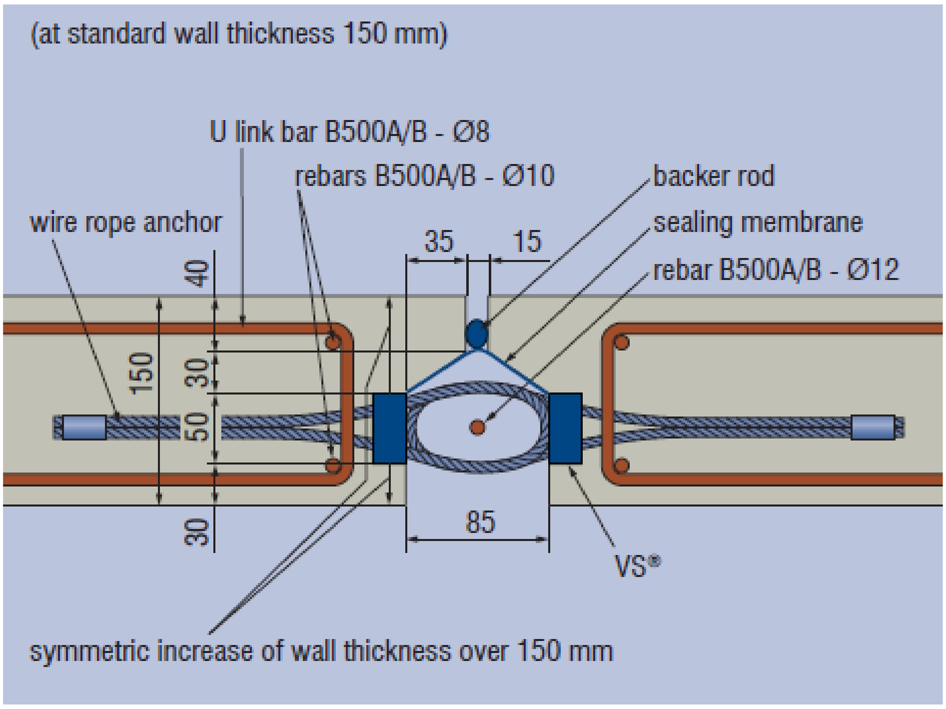




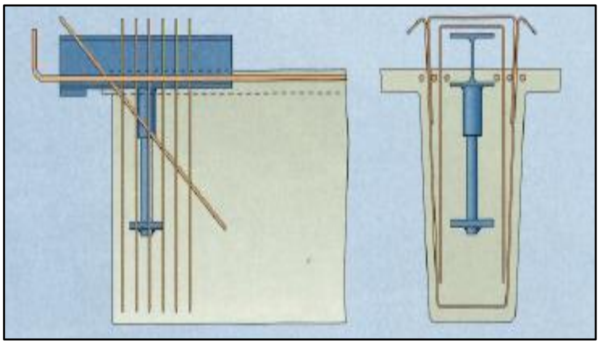


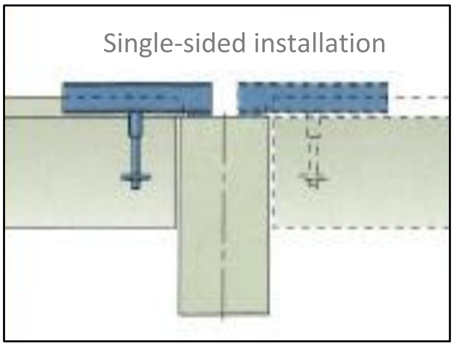
C) PRECAST WALLS USING WIRE LOOP CONNECTOR



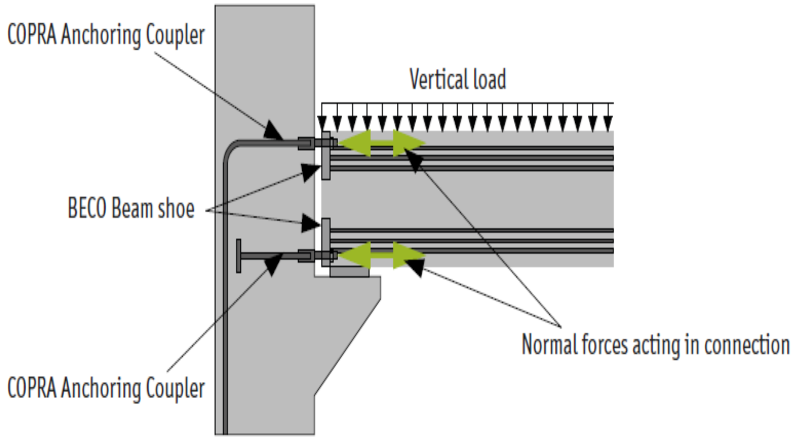


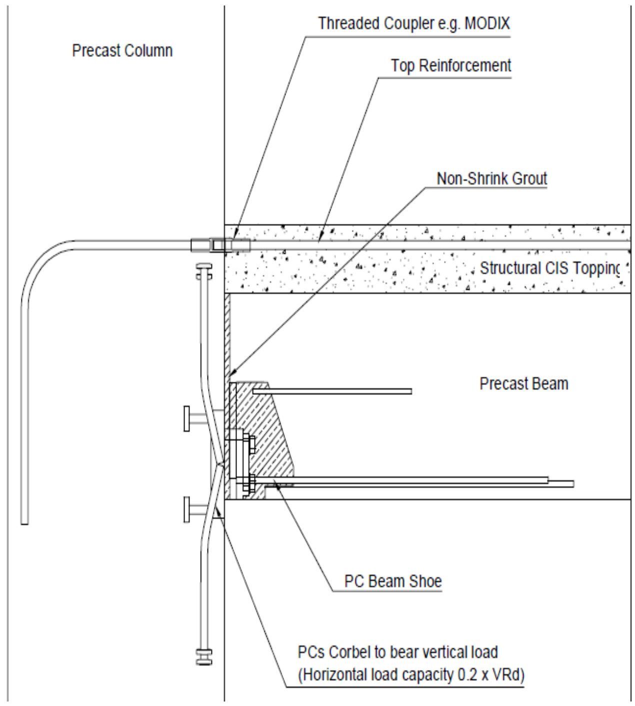
D) STEEL HANGERS IN BEAMS





E) RIGID BEAM-COLUMN CONNECTION





2.1.5 <u>Other design considerations in using mechanical connections</u>

As compared to the transfer of compression forces, the transfer of tension is inherently effective for bolted connection. In cases where column shoe connector is used as compression reinforcement, the bottom leveling nuts for erection and the top locking nuts with appropriate tightening will be required to ensure the effectiveness.

Some connection design may need to consider torsional effect which often occurs during PC installation due to unbalanced erection loads. Mechanical connectors may be checked as a group similar to steel bolting design. Connector which acts singly will have limited torsional capacity and can be critical at PC erection stage. As such, supplier's advice should always be sought.

2.2 STRUCTURAL SYSTEM AND CONNECTION DESIGN CONCEPT

2.2.1 Basic Considerations

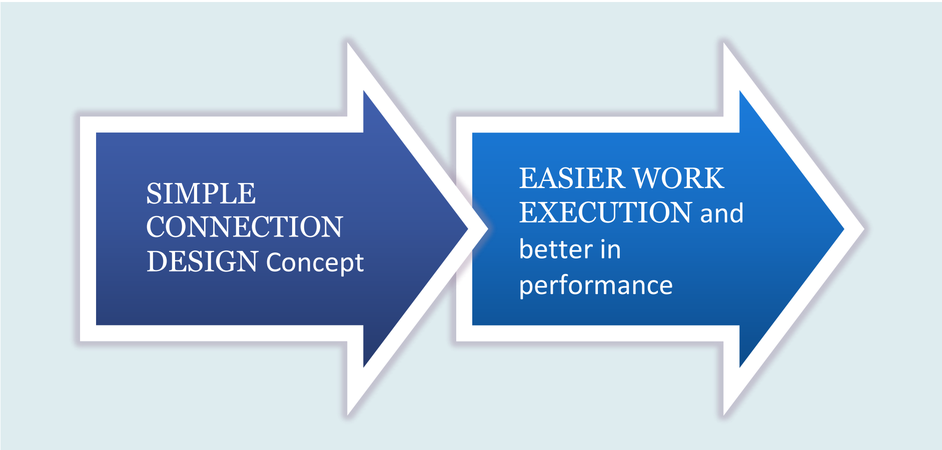
For PC buildings, structural designers are to ensure that global structural continuity is created by joints and connections between the assembled components. The connections which act as bridging links at the joint interface of the PC can affect the behaviour, structural integrity and robustness of the precast concrete buildings.

PC joints are regions of high stress concentrations and are the weakest links in the structural system. The connections of these joints must be able to:

a) Provide resistance to the design joint forces
b) Remain ductile to withstand the joint deformations.

The design of the connections must also be:

a) Simple in fabrication
b) Easy for erection
c) Able to carry out final jointing on site



The material used for the connections must be:

a) Stable and durable for the structure design life
b) Chemically and physically compatible with the environment
c) Able to offer good protection against adverse chemical and physical influences and of similar fire resistance

2.2.2 Load Paths Descriptions

What are load paths?	Continuous internal force direction in which each consecutive load passes through the interconnected PC members Start from the highest point of the structure and all the way to the foundation system The magnitude, direction and nature of loads (axial, bending, torsion) and sequence that the loads are being transferred from one member to another are also affected by the hierarchical nature of the structural members
What should the designers check?	Vertical and horizontal loads separately Superimposed the solutions in the development of the structural system
What transfers the vertical loads?	Floor slab is designed to support the imposed gravity loads (self-weight, finishes, partitions, services, imposed live loads) The load travels from the floor slab to the immediate beam that supports it The beam transmits the slab loads to its end which may connect to the main beam The main beam transmits the accumulated loads from the floor slab and beams to a connecting column/wall. The load then travels down to column/wall to the foundation that ultimately distributed into the ground.
What about transfer of horizontal loads?	It may not be so obvious as the load path/load transfer for vertical loads Wind, notional and earthquake loads act in a horizontal direction and are transferred through the floor diaphragms. It is an essential part of the stabilising system.

Figure 2.1 below illustrates the transfer of vertical and horizontal loads in a structure.

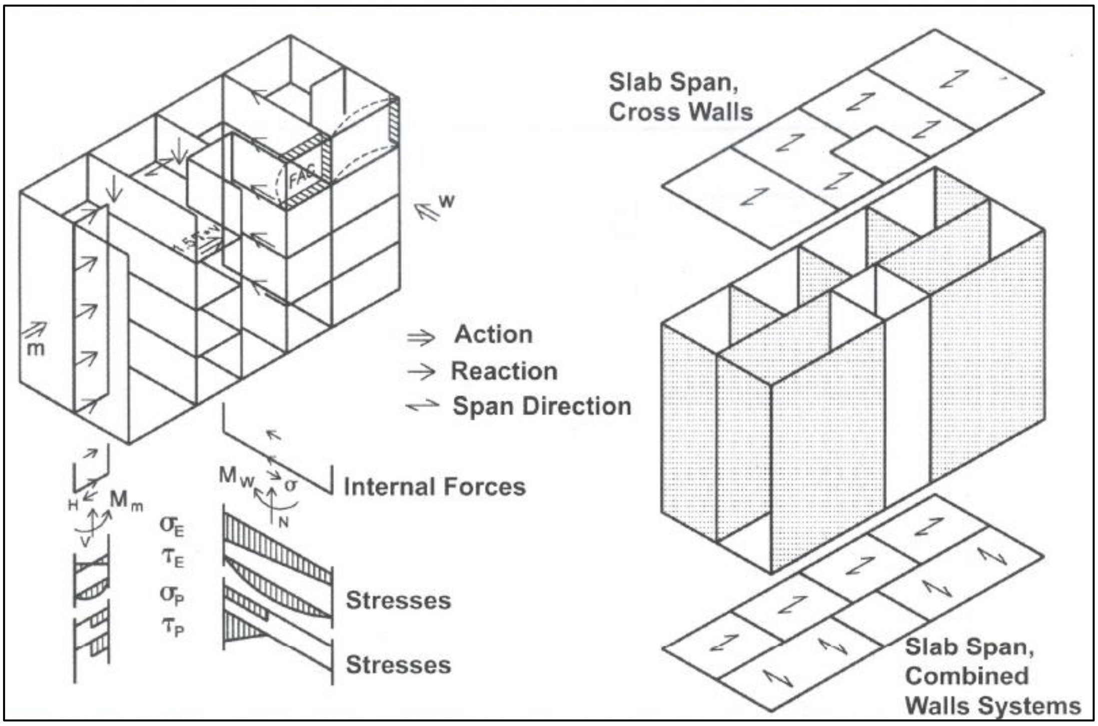


Figure 2.1: **Load Paths and Transfer of Horizontal and Vertical Loads**

PC floors or roofs have considerable in plane stiffness once the PC members are connected. They are normally considered as being rigid diaphragm in design as shown in the ***Figure 2.2***. The transfer of horizontal loads through the floor sub systems is shown in ***Figure 2.3***.

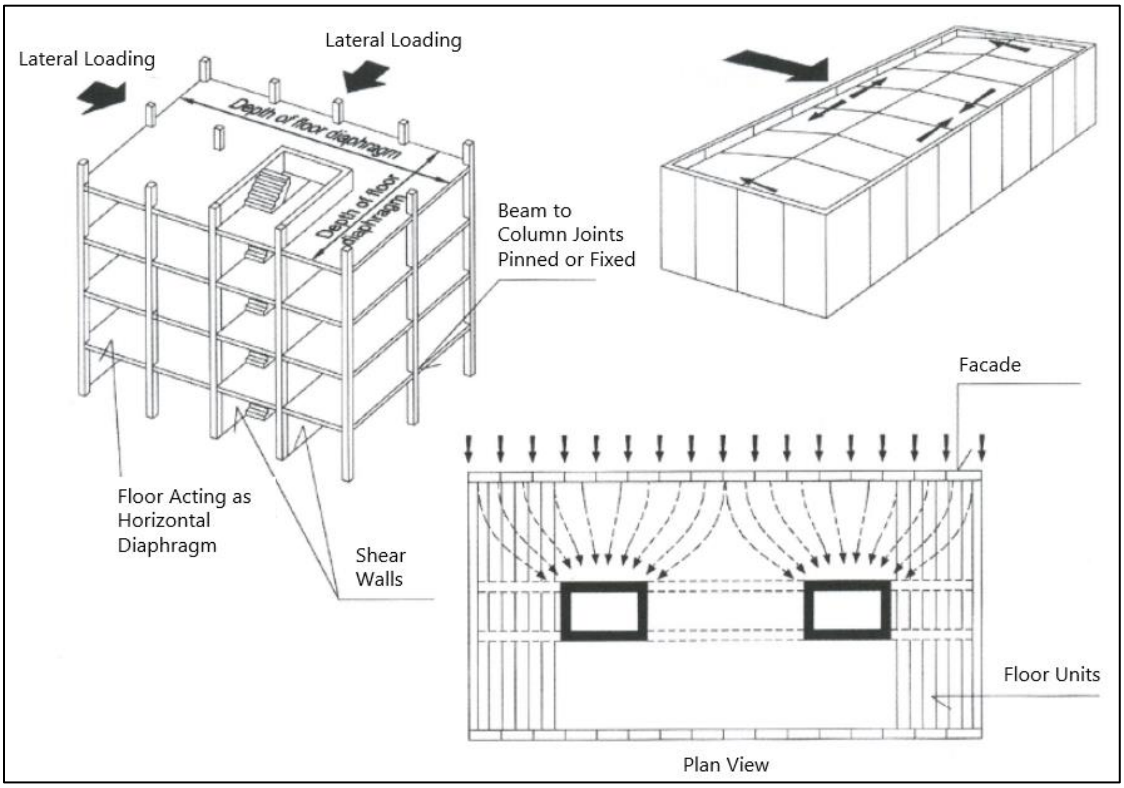


Figure 2.2: **Floor Diaphragm Action for Transfer of Horizontal Loads**

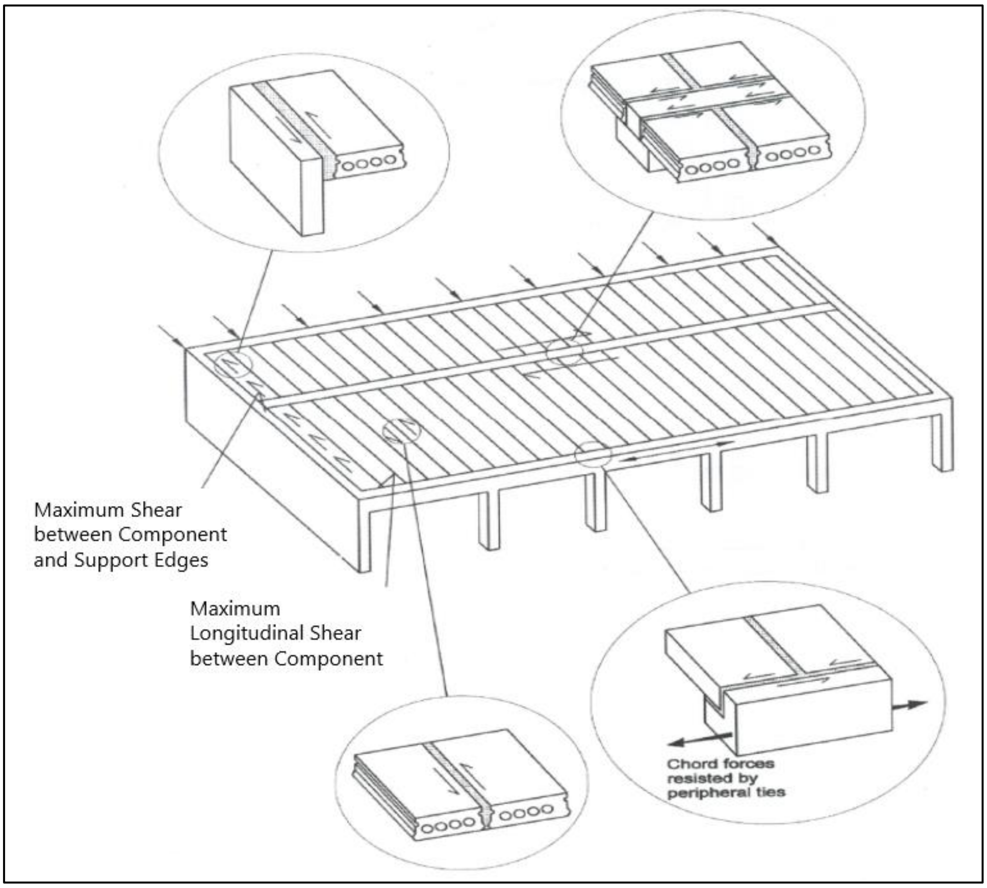


Figure 2.3: **Transfer of Horizontal Loads in Floor Sub Systems**

The stabilising elements of precast buildings may comprise of the followings (***Figure 2.4*** to ***2.6***):

a) Rigid moment frame of columns and beams (unbraced frames)
b) Cantilever columns or walls
c) Shear walls
d) Lift and/or staircase cores
e) Cross bracings of strategic column and beam frames
f) Or a combination of the above

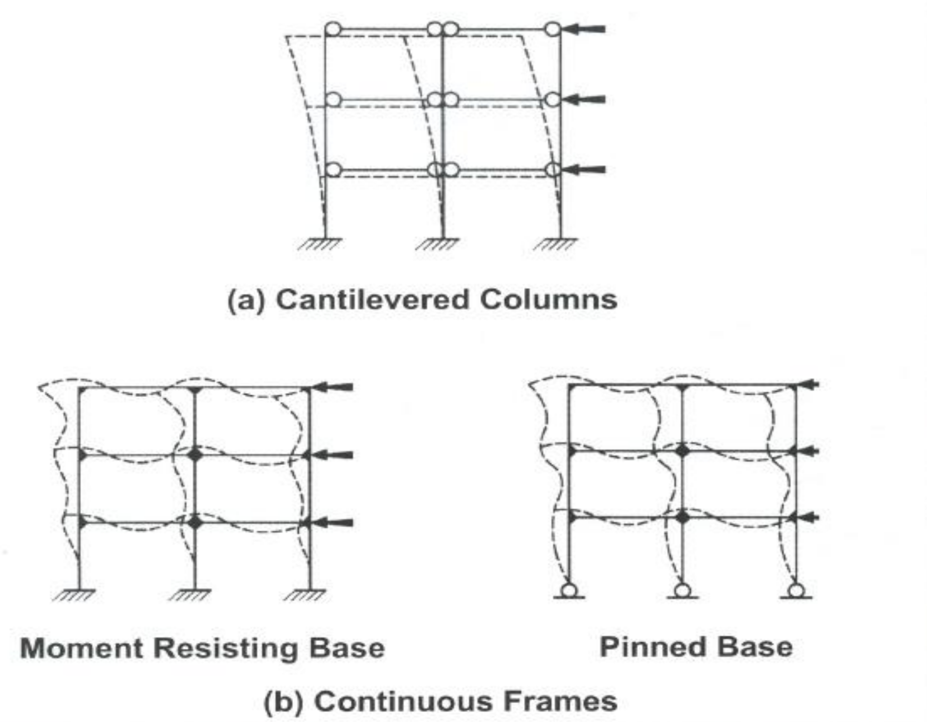


Figure 2.4: Unbraced Frames

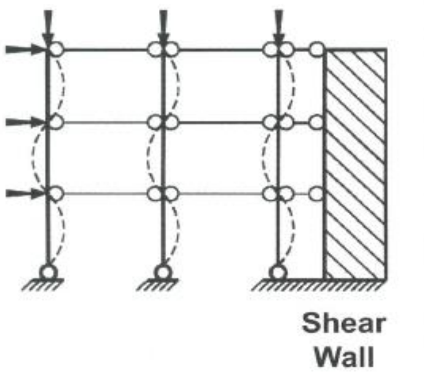


Figure 2.5: Braced Frame

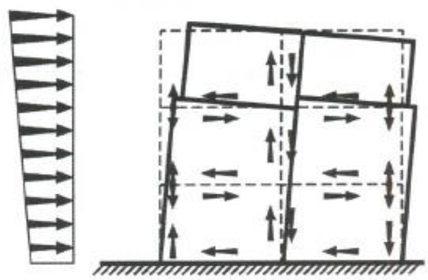


Figure 2.6: Shear Wall

2.2.3 Accidental Loads

A poorly designed and detailed PC building is susceptible to structural instability and possibly major collapse when subjected to accidental loads.

What are accidental loads?	These are loads which are not considered in the design. For example, explosion, collision impacts by vehicles or failing objects, local overloading, intense localised fire, errors in design or construction, unexpected localised foundation settlements and slight earth tremor.
Why is it important to know?	This is to ensure that the entire structure will not be jeopardised by the chain reaction progressive failure as a result of localised damage to a small portion of the structure.
How to prevent progressive collapse?	a) Design and remove every conceivable hazard expected to happen during the service life of the structure. b) It is not realistic to expect structural designers to have foreknowledge and to design the structure against the accidental loadings arising from these hazards. c) It should be guarded by strengthening or erecting barriers to protect vulnerable columns and walls against high probability of vehicular impact.
What is the common method to enhance the structural stability and robustness?	By providing reinforcement as structural ties. The structural ties are placed in the building structure in the following manners: a) Vertical direction from foundation to roof b) Horizontally across and around each floor c) All external and perimeter load bearing columns and walls are required to be anchored into the floors and beams. These are shown in ***Figure 2.7***. Reinforcement provided in the columns, walls, beams and floors can form part of or whole of these ties as the ties are intended as a minimum and not as additional reinforcement to that required by design and analysis. If a building is divided by expansion joints into separate independent sections, each section is to have an independent typing system.
What is the reference for design approaches and provisions of structural ties to prevent progressive collapse for buildings which are not designed to withstand accidental loads?	SS EN 1992-1-2: 2008 clause 9.10. Singapore Annex NA to SS EN 1922-1-1:2008 (to be referred as EC2 and Singapore NA subsequently). Clause NA.4.2 in Singapore NA states that the provisions of EC2 are not sufficient in some respects. Structural designers should further refer to the details and design approaches stated in the non-contradictory complementary information in PD6687.

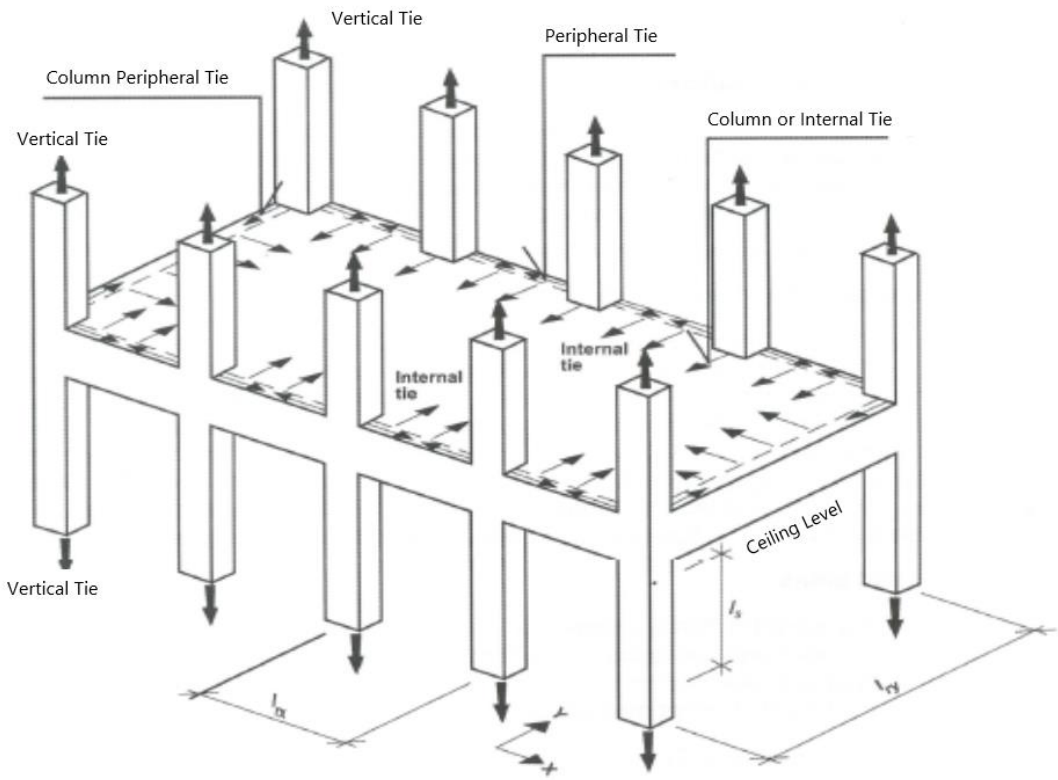


Figure 2.7: Structural Ties

2.2.4 Design of Ties

The ties designed for the prevention of progressive collapse should be provided and detailed in PC structure. It is either wholly within the in-situ concrete topping or at connections of PC members. Also, mechanical connectors should be used in narrow PC joints where normal lapping of reinforcement is not feasible.

From EC2 and the decisions in Singapore NA, the designs of horizontal and vertical ties are as follows:

1. Horizontal Ties

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

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

Horizontal ties should be provided at every floor regardless the height of a building in the form of (a) peripheral (b) internal and (c) column and wall ties. The resistance of the ties is stressed to the characteristic strength i.e. material partial safety factor γ_s=1 for reinforcement.

----

a) Peripheral Ties

Design tie force = F_t. Peripheral ties should be located within 1.2m from the edge of a building or within the perimeter walls and beams. Structures with internal edges such as atrium, courtyard, L- or U-shaped floor layout should have the peripheral ties detailed as shown in Fig 2.8. Ties should be anchored straight inwards on both sides at the re-entrant corner of the perimeter.

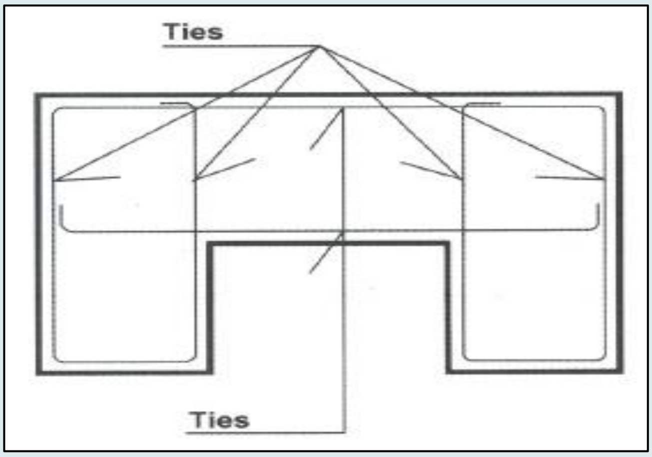


Figure 2.8: **Peripheral Ties**

b) <u>Internal Ties</u>

Internal ties force should be the greater of (in kN/m)

F_t or $[(g_k + q_k)/7.5]\ (l_r/5)\ (F_t)$

Where,

F_t is the basic tie force

$(g_k + q_k)$ (in kN/m^2) is the sum of the average permanent and variable floor loads

l_r is the greater distance between centres of vertical load bearing elements in the direction of the ties being considered

The ties are to be placed in orthogonal directions and may be spaced evenly across the floor or grouped within beams or walls as convenient with a maximum spacing of $1.5l_r$.

----

c) <u>Column and Wall Ties</u>

Edge or perimeter column and wall should be tied horizontally to the floor structure at each floor and roof. The horizontal tie force for the column and wall ($F_{tie,col}$ and $F_{tie,fac}$ respectively) should be the greater of:

$2F_t \leq (l_s/2.5)F_t$ or

3% of the total design ultimate vertical load carried by the column or wall at that level

Where,

column tie force $F_{tie,col}$ in kN

wall tie force $F_{tie,fac}$ in kN/m

l_s the floor to ceiling height in metre

2. Vertical Ties

Each load bearing column and wall should be tied continuously from foundation to roof for buildings of 5 storeys or more. The design vertical tie force, considered as tension force, is equal to the maximum design ultimate load carried by the column or wall from any one storey or the roof. The ultimate vertical load is computed from expression 6.11b under clause 6.4.3.3 of SS EN 1990:2002 Eurocode - Basis of structural design. The variable live load should be the realistic or quasi-permanent loading with appropriate Ψ_2 factor for the building.

The purpose of the vertical ties in wall and column is to:

- Contribute and form part of a bridging system to span over damaged floor area
- Limit the collapse of a floor in case of accidental loss of the column or wall below

When effective vertical tie requirements are not complied or the horizontal ties on every floor and roof do not have sufficient anchorage in the direction of the span, a "damaged" structure analysis may need to be carried out.

Such an analysis includes:

a) <u>Key Elements</u>

"Non-removable" elements – the failure of which would cause the collapse of more than a limited portion of the structure close to the failed element.

All other structural components and their connections that are vital to the stability of the key elements are also considered as key elements.

To ensure non-removability of the elements, the element and its connections are designed to be able to withstand an ultimate design load or pressure of 34 kN/m^2 applied from any direction to the projected area of the member.

Key elements should be avoided as much as possible by revising the building layout within the architectural constraints.

----

b) <u>Alternative Load Paths</u>

In this analysis, the vertical load bearing element other than key elements is removed one at a time at each floor.

The loads carried by the failed vertical element are transferred via catenary action in alternative load paths to other load bearing members.
The loads carried by the failed vertical element should be computed as described previously.

Continuity of Ties

Continuity of tie reinforcement can be achieved by the following:

a) Lapping in PC components using enclosing links may be adopted as shown in Fig 2.9.

b) Welding

c) Mechanical devices such as couplers and anchors

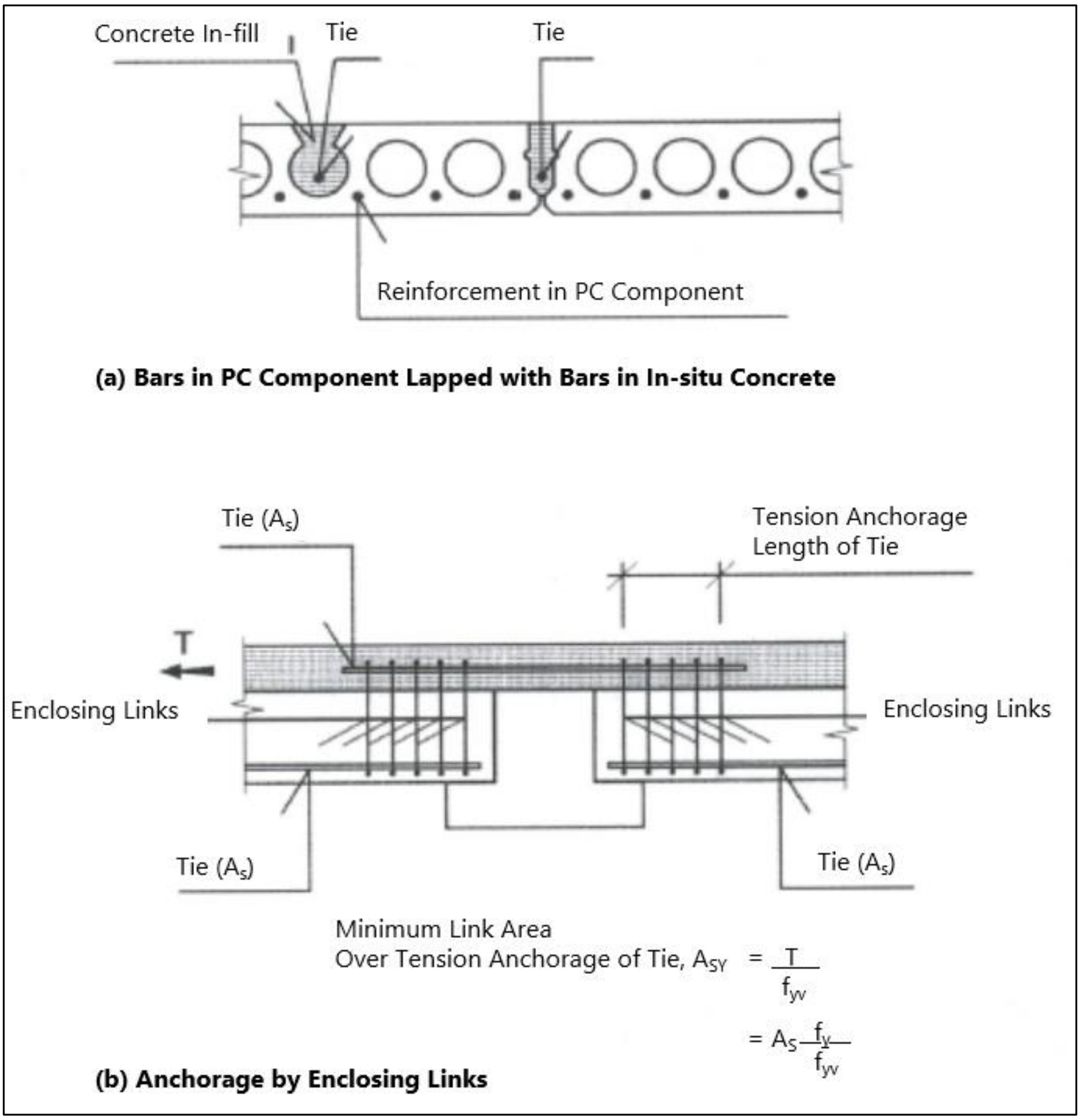


Figure 2.9: **Tie Continuity by Lapping and Enclosing Links**

Anchorage of Ties

Internal floor ties are to be anchored to the peripheral ties as shown in Fig 2.10.

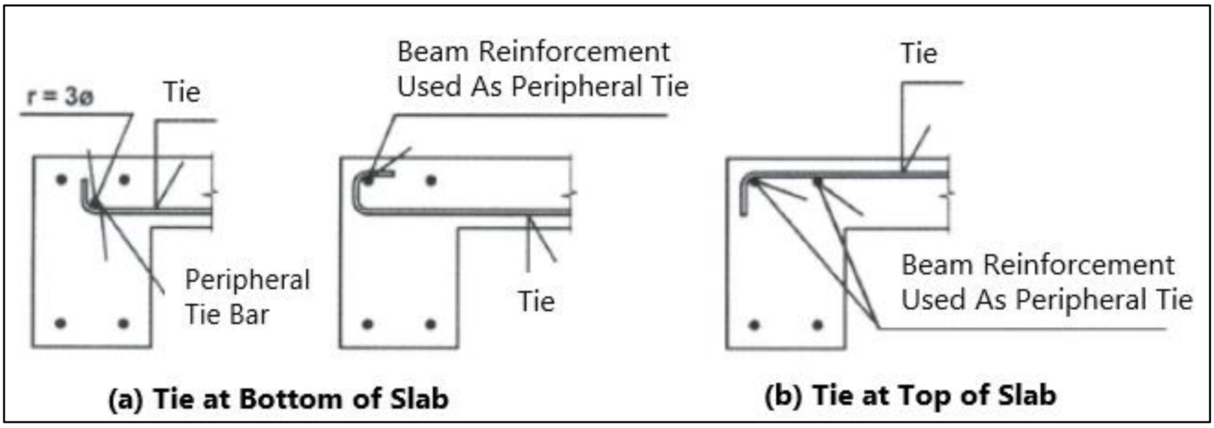


Figure 2.10: **Anchorage of Ties at Peripheral Ties**

Figure 2.11 illustrates the floor tie backs of corner column in both directions. The tie backs can be part of the main reinforcement in the perimeter beams that have been framed into the column.

Because of compatibility between B and D regions, primary designs of the member must be performed first before using STM in the design of the D regions.

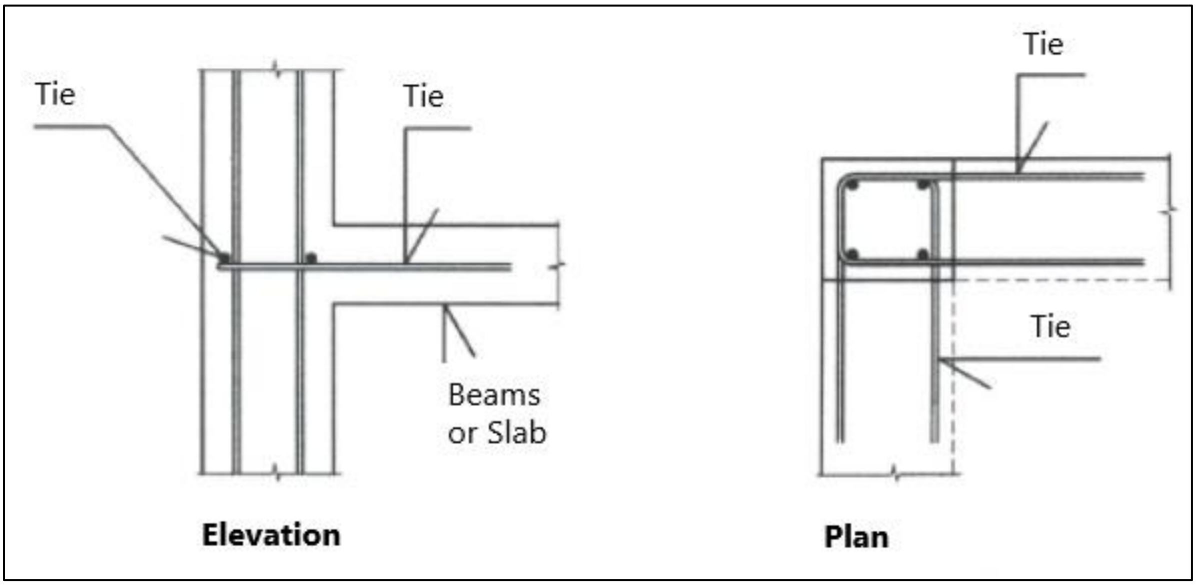


Figure 2.11: **Tie Backs for Edge Columns**

Continuity and anchorage of tie reinforcement in in-situ construction seldom poses any serious problems. Likewise, PC construction presents a more serious problem as continuity of the ties is to be maintained across the joint connection especially in simple and easily constructed joint. In usual circumstances, ties should not be lapped in narrow joints between PC components and mechanical anchorage should be used in these cases.

JOINT AND CONNECTION

The performance and behaviour of PC connections depend on the interaction of several components at the joint. The various joint components as illustrated in the ***Figure 2.12*** consist of:

a) Joint opening or gap between adjacent precast members
b) Joint filler
c) Connecting element across the joint opening and
d) Connection zones

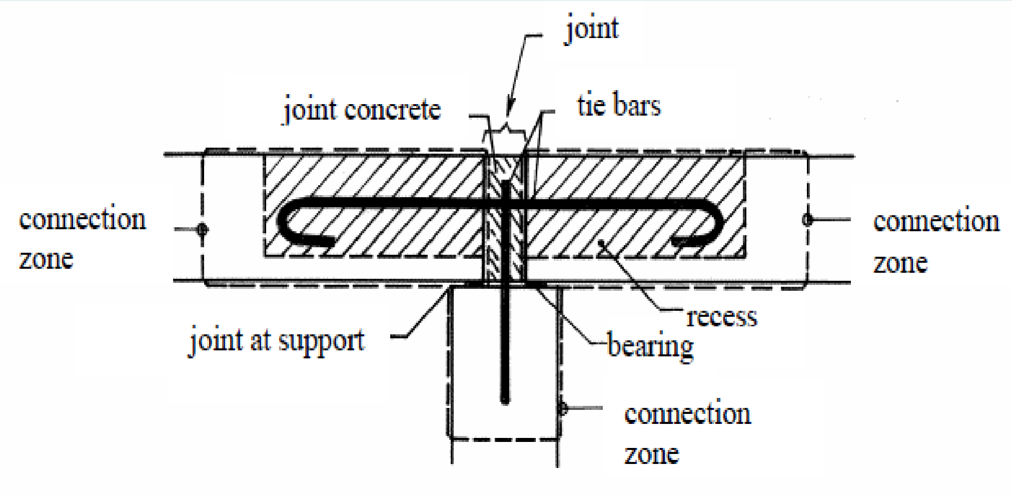


Figure 2.12: **Components of Structural Connection**

What is the size of joint opening?	Size of joint opening is determined by the following: a) Type of connection. b) Tolerance studies of PC components manufacturing, erection, method and working accessibility of connection. c) Aesthetic, fire and water tightness.
What fills up the joint opening?	a) Grout, mortar and concrete. b) "Joint filler" – various type of soft and hard bearing pads. c) Non-structural "joint filler" – silicone, polyurethane, rubber strips, foams, rock wools, resins or mastic.
What is the difference between joint and connection?	a) Joint is subjected to high concentration of forces when loaded. b) Connection is an integral part of the structural behaviour of connected members. It is an essential part in the connection design so that the forces originated at the joint can be safely transferred in the connection zones to the principle structural members.
What are connection zones?	a) Connection zones are joint surfaces which are influenced by the type of joint filling works and the force transfer ability across the joint.
What are the connecting elements that transfer the forces across the joint?	a) Tie bars, anchors, coupling devices, bolts or welded steel plates. b) These are anchored in the connection zones found at the end regions where the PC components meet.

2.3 STRUT AND TIE MODELLING AT CONNECTION

2.3.1 B and D Regions

The RC member can be classified into two distinct regions, which are namely the B and D regions as shown in ***Figure 2.13.***

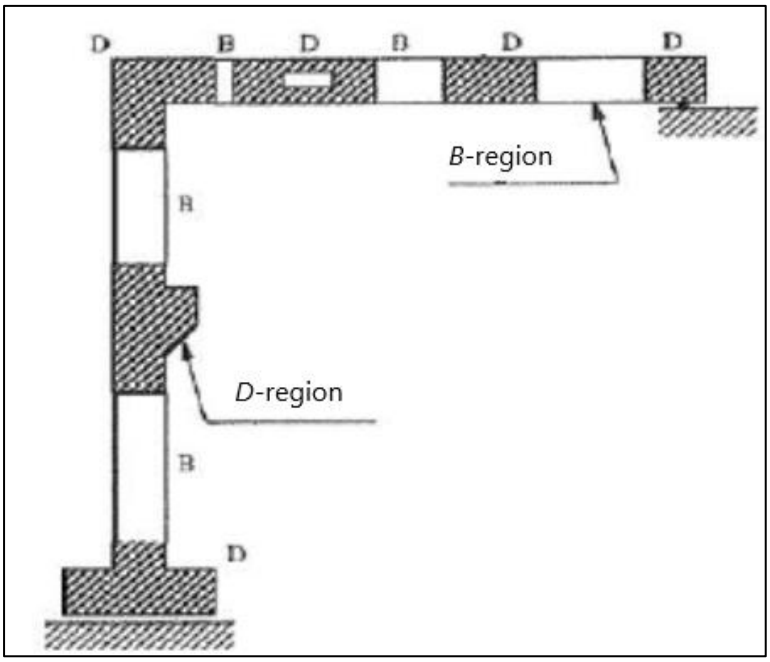


Figure 2.13: **B and D Regions in Structural Member**

B-regions	a) Parts of the structure where the Bernoilli hypothesis "Plane section remains plane after bending…" applies. b) Such a hypothesis allows for the flexural design of RC member, including ultimate flexural capacity, in the B-region to be based on linear strain variation over the depth of the member.
D-regions	Regions outside the B-regions where the uniform stress distribution is "disturbed" or "discontinued" due to: a) Presence of concentrated forces (loads and reactions) b) An abrupt change in the member geometry such as cross-section, presence of openings, bends, corners or other discontinuities.

Based on St. Vernant's Principle where *"The localised effects caused by any load acting on the body will dissipate or smooth out within regions that are sufficiently away from the location of the load…"*, the transition of a plane section stress field in the B regions to a local stress field in the D regions could occur in a span of about 1.0 times the depth of the member on either side of the point of concentrated force or member geometry changes as illustrated in ***Figure 2.14***.

Figure 2.14 also shows the imposed loads and boundary forces which must be in equilibrium in the Strut and Tie Modelling (STM modelling).

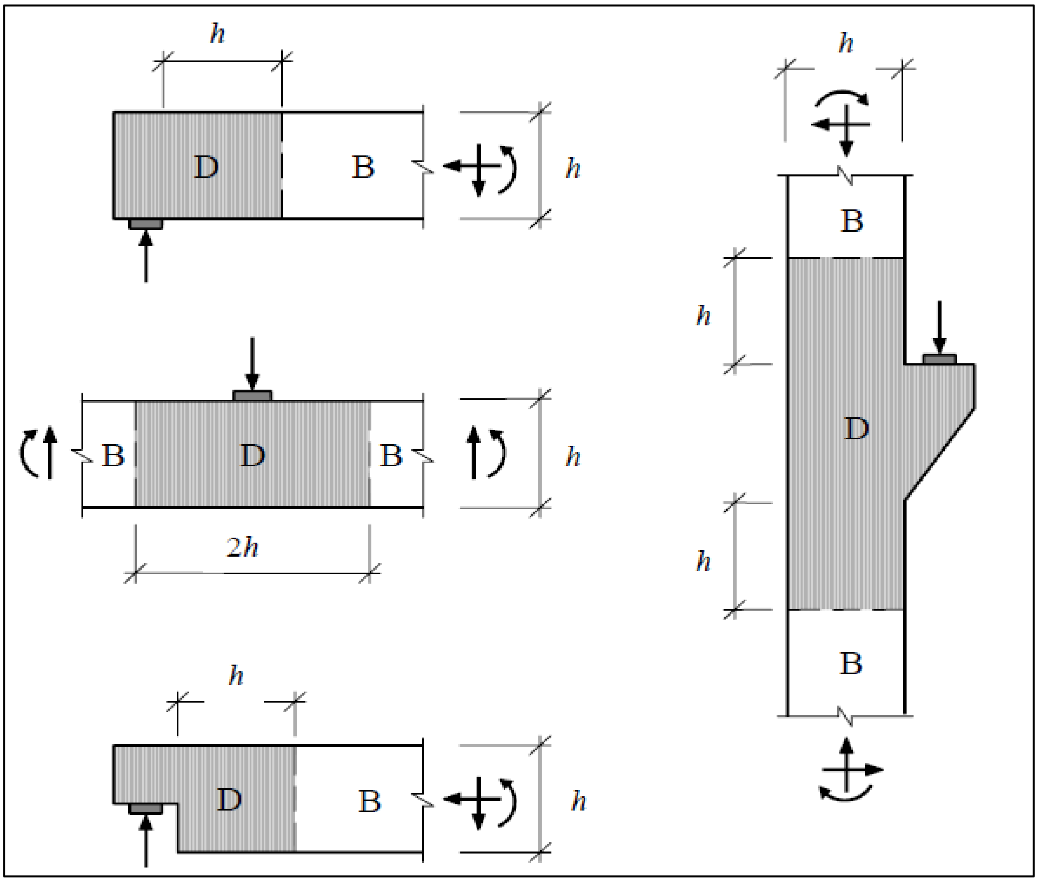


Figure 2.14: **Boundary and Boundary Forces in D region**

Designs of D-regions, which may be inaccurate, are conventionally based on:
a) Rules of thumb
b) Designer experience
c) Empirical code provisions or guidelines

The Strut and Tie Modelling (STM) would be the appropriate design for D region.

STM is a unified approach that considers all load effects of moment, shear, compression and tension simultaneously. It is one of the most useful methods in the design of:
a) Deep beams
b) Beam-column joints
c) Supporting brackets
d) Corbels
e) Beam half-joints
f) Anchorage zones of post tensioned members and pile caps

2.3.2 The STM Model

STM is a conceptual framework, as shown in ***Figure 2.15***.

The stress distribution in the D region at the ultimate limit state are idealised as an analogous truss system and comprises the following:

a) Uniaxial compression struts, and

b) Tension tie elements with transfer of the forces between struts and ties at intersecting zones or nodes

In a real physical truss, the truss mechanism in STM model must be stable and in balance with external applied loads. The imposed load is then carried through the region to the supports.

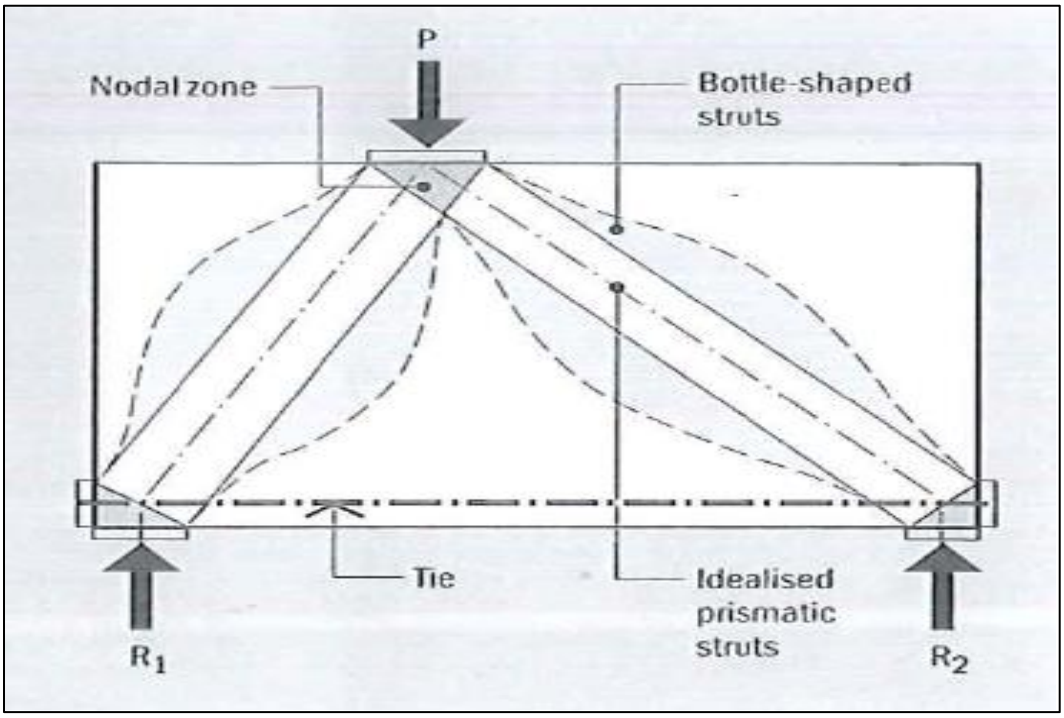


Figure 2.15: **Description of Strut and Tie Model**

The components of STM model consists of (a) Ties, (b) Struts and (c) Nodes.

a) Ties	• Tension elements which include links, longitudinal reinforcements, pre-stressing steel or both, plus a portion of the surrounding concrete concentric with the axis of the tie. • The surrounding concrete is not considered in resisting axial tension forces even though it reduces the tie elongation by tension stiffening effect, especially under service load.

b) Struts

- Compression chords which represent the resultants of the concrete compression fields.
- These serve as the compression chord of a truss mechanism to resist moment and as diagonal struts to transfer shear to the supports.
- Orientated parallel to the direction of initial cracking of the concrete.
- May be reinforced by reinforcing steel to increase their compression capacity.

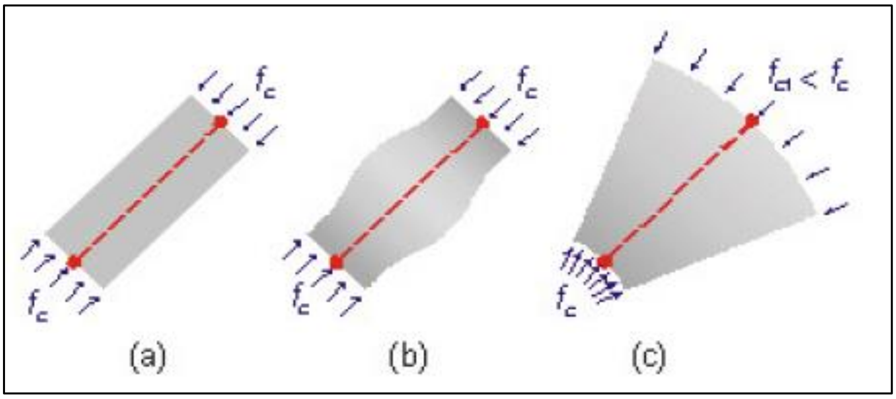


***Figure 2.16*: Basic Type of Struts in a 2-D Member: (a) Prismatic, (b) Bottle-shaped, (c) Fan-shaped**

- "Bottle-shaped" Strut

  The typical shape formed when large surrounding concrete allows the struts to spread laterally at mid length. It may be idealised as an equivalent truss for a better appreciation of the flow of forces.

  The spreading and expansion of this strut produces tension stresses which may require transverse reinforcement as shown in the more elaborated strut model in EC2 as shown in ***Figure 2.17***.

  For practical design purposes, the strut is often assumed to be prismatic with constant width along the length.

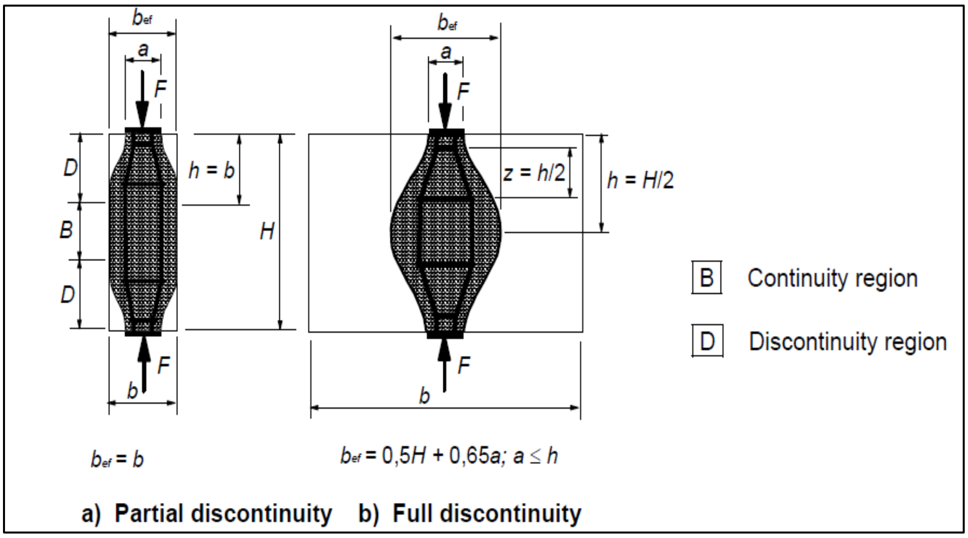


***Figure 2.17:* Bottled-shaped Strut Model in EC2**

- “Fan-shaped” Strut

  It is likely to occur in deep beam where an array of struts with varying inclination meet at or radiate from a single node. The flow of forces in this type of strut may be reduced to a simple truss system, as shown in the ***Figure 2.18***.

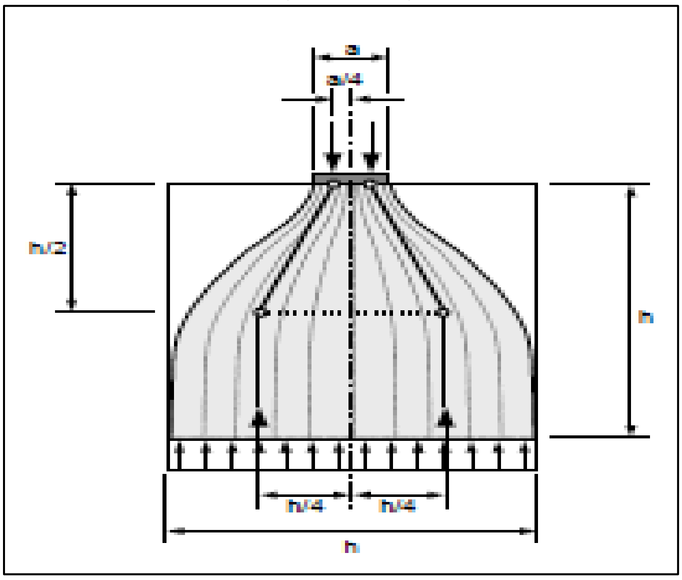


Figure 2.18: **Fan-shaped Strut Model**

c) Nodes

- As analogous to joints in a truss, are localised zones where the axes of the struts, ties and concentrated forces intersect as well as forces are transferred between struts and ties.
- Nodes essentially define the zones in which the forces in the struts and ties are to be anchored.
- They can be classified by the type of forces being connected as shown in ***Figure 2.19*** where **C** and **T** are the **c**ompression and **t**ensile forces respectively.

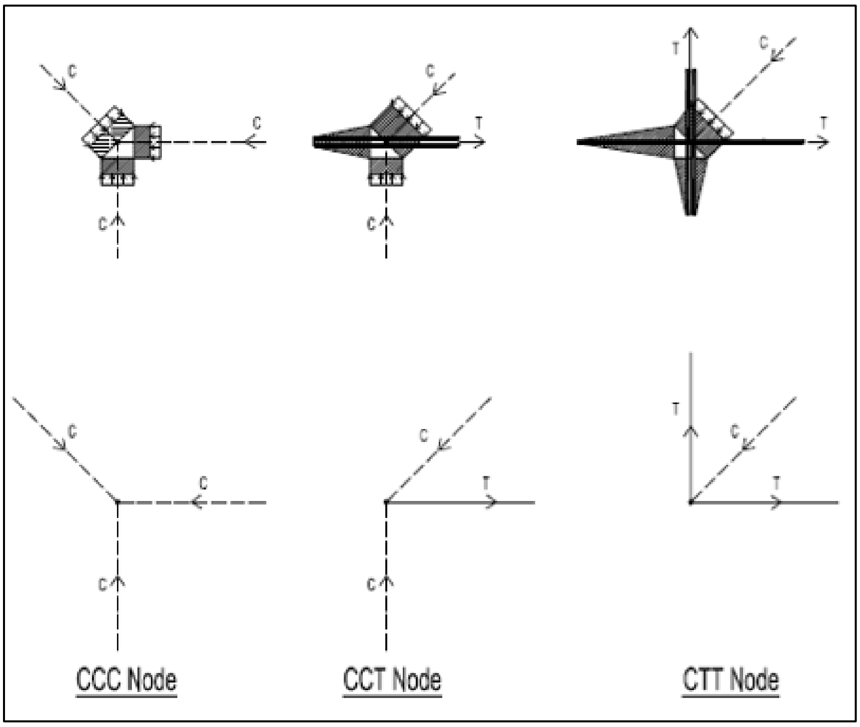


Fig. 2.19: **Basic Node Types**

2.3.3 Design Strengths

A. Design of Ties

The ultimate tie capacity is given as:

$T \leq A_s f_{yd}$ $f_{yd} = f_{yk}/\gamma_s$

where A_s is the area of reinforcement
f_{yk} the ultimate strength of reinforcement and
γ_s material safety factor for reinforcement (= 1.15)

For the tie force to be effective, the reinforcement must be adequately anchored in the node by providing sufficient development length from the point beyond the extended nodal zone.

B. Design of Struts

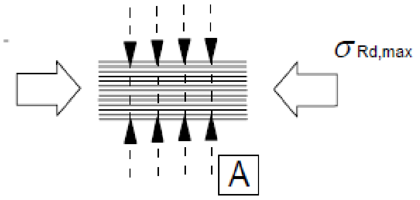
The ultimate compression capacity of a concrete strut should be as follow:

$F_c \leq A_c\, \sigma_{Rd,max}$

where $\sigma_{Rd,max}$ is the design strength of the concrete strut.

$\sigma_{Rd,max}$ is given in **EC2** for the following two conditions:

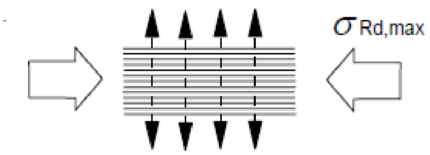
i. Under Transverse Compressive Stress/No Transverse Stress:



$\sigma_{Rd,max} = f_c$

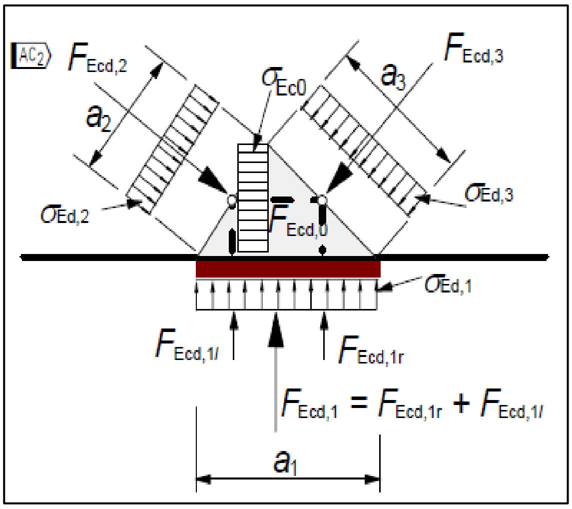
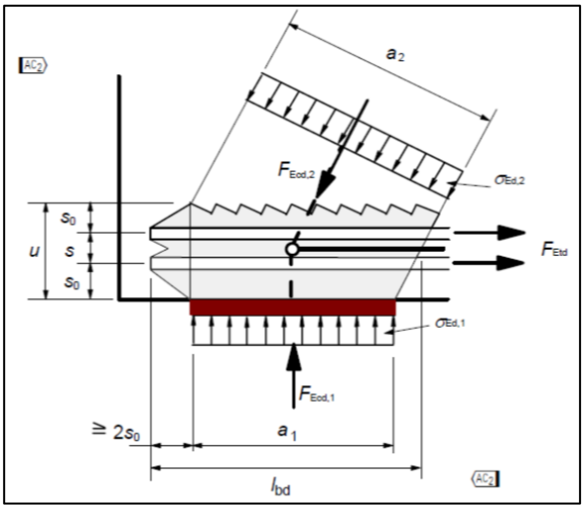
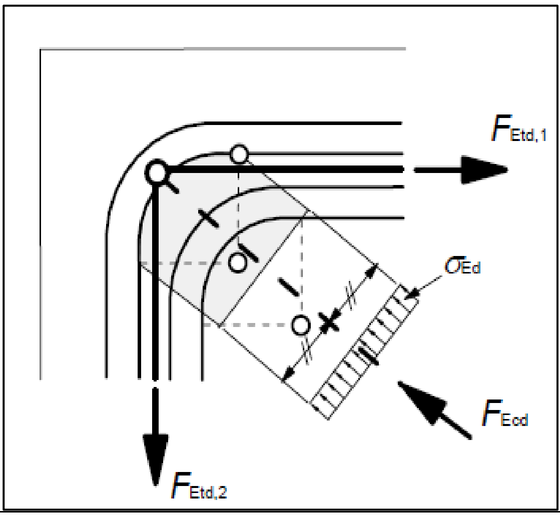
ii. Transverse Tension/Cracked Compression Zone:
$\sigma_{Rd,max} = 0.6\,v'\,f_{cd}$

where $v' = 1 - f_{ck}/250$
$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c$
$= 0.567$ (α_{cc}=1.0, γ_c=1.5)



C. Design of Nodal Zones

The ultimate compressive strength of a nodal zone can be taken as:

i) CCC node  	$\sigma_{Rd,max}$ $= k_1\, v'\, f_{cd}$ $= 0.567(1 – f_{ck}/250)\, f_{ck}$ where $k_1 = 1.0$, $v' = 1 – f_{ck}/250$ $f_{cd} = 0.567 f_{ck}$
ii) CCT node  	$\sigma_{Rd,max}$ $= k_2\, v'\, f_{cd}$ $= 0.482\,(1 – f_{ck}/250)\, f_{ck}$ where $k_2 = 0.85$, $v'=1– f_{ck}/250$, $f_{cd} = 0.567 f_{ck}$
iii) CTT node  	σ_{Rdmax} $= k_3\, v'\, f_{cd}$ $= 0.425\,(1 – f_{ck}/250)\, f_{ck}$ where $k_3 = 0.75$, $v' = 1 – f_{ck}/250$ $f_{cd} = 0.567 f_{ck}$ EC2 allows $\sigma_{Rd,max}$ to be increased by up to 10% for confined nodes or where all the angles between struts and ties are ≥ 55 degrees

2.3.4 Design Procedure

The primary designs will determine the following:

a) Boundary forces of the D region

b) Selection and maintaining continuity of the primary reinforcing steel from the B region into and anchored within the D region

Step 1 - Define the D region concrete dimensions and calculate the ultimate design boundary forces from imposed loads

Step 2 - Choose the basic internal supporting truss model and sketch to fit within the concrete dimension

The following guidelines may be adopted when choosing a suitable internal supporting truss model.

a) The supporting truss model should be determinated.
b) Boundary forces and section equivalent forces are applied at the nodes of the truss. The supporting truss must be in equilibrium with these forces as shown in Fig. 2.20.
c) Tension in concrete is neglected.
d) Strut and tie forces are uni-axial.
e) Struts are orientated parallel to the expected axis of cracking. They must not overlap other struts.
f) Ties may cross struts or other ties.

The angle between a strut and a tie that's joined at a node should not be less than 22 degrees (cot θ ≤ 2.5).

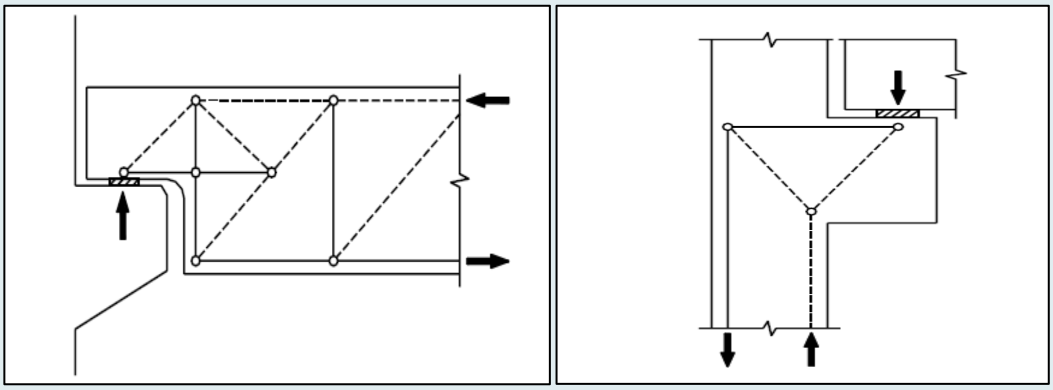


Figure 2.20: **Equilibrium in Boundary Forces and Section Equivalent Force in Internal Supporting Truss System**

Step 3 – Establish the truss geometry

The initial truss geometry may be set by geometric parameters such as bearing plate dimensions, centroids of struts and tie reinforcing steel layers that continue from the B into D regions.

It is for this reason that the analysis and design of the B region must precede the STM design of the D region as mentioned earlier.

Only after the initial truss system is established, the forces in the struts, ties and nodes can then be calculated.

Steps to be taken after calculating the strut and tie forces are as follow.

a) Determine the required area of tie reinforcing steel, select suitable bar sizes, bar spacing and layer of steel (in high steel content tie) in each of the ties. Establish the tie layout, dimensions and the resultant tie centroids.

b) Determine the geometry i.e. width and depth of the struts and nodes so that the strut forces can be converted into compressive stresses. This should be followed by checking against the limiting stresses imposed from the code.

   It should be noted that for CCT and CTT nodes, the length of the node needs to include the full development anchorage development length of the ties.

   In cases where it is not possible, mechanical anchorage as alternative to straight or hooked bar should always be considered.

c) Fit the ties, nodes and struts geometry into the concrete dimensions with the resultant strut and tie centroids that coincide with the initial assumed in the basic truss geometry.

If the ties nodes and struts cannot fit into the concrete member dimensions, the truss model must be redrawn to allow the ties, struts and nodes to fit and the forces in the model re-calculated.

Steps (a) to (c) above are repeated until all the truss components meet the allowable stress limits and fit within the concrete dimensions.

2.4 CONNECTION DESIGN EXAMPLES

The designs of structural connections for easy manufacturing and site assembly are illustrated in subsequent sections.

The jointing methods shown in the examples are selected from the usual practice in the industry. The main purpose is to demonstrate the principles and considerations that are involved in the design of a structural connection between various types of PC components.

Alternative designs by specialist with proprietary products are presented at the end of individual design examples. This is a comparison with the current industry practice. The specialist design alternatives are illustrated in a summary format, and the designer may approach the specialist for detail calculations, if necessary.

It should be noted that:

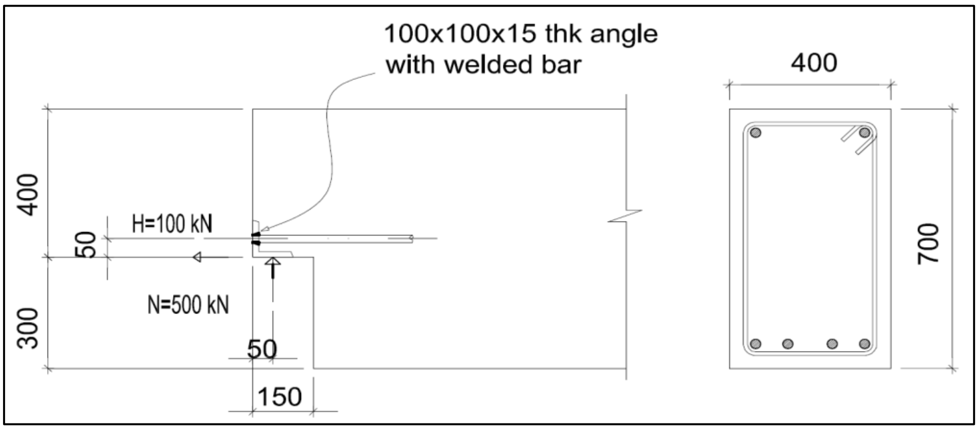
a) The use of links or looped reinforcement in vertical joints between PC walls is not considered friendly to the manufacturing and site assembly. But it is included as a comparison to the alternative recessed wire loops method.

b) There are equivalent connector products, available locally, to those shown in the specialist design alternatives.

The inclusion of a particular product as example does not imply any favours or the superiority of them, but purely comparative illustration

Design the beam half joint shown in the figure below for:

- Ultimate vertical reaction N = 500 kN
- Horizontal force of H = 100 kN
- Design concrete class C40/50
- f_{yk} = 500 N/mm^2 for reinforcement bars
- Concrete cover = 35mm to links



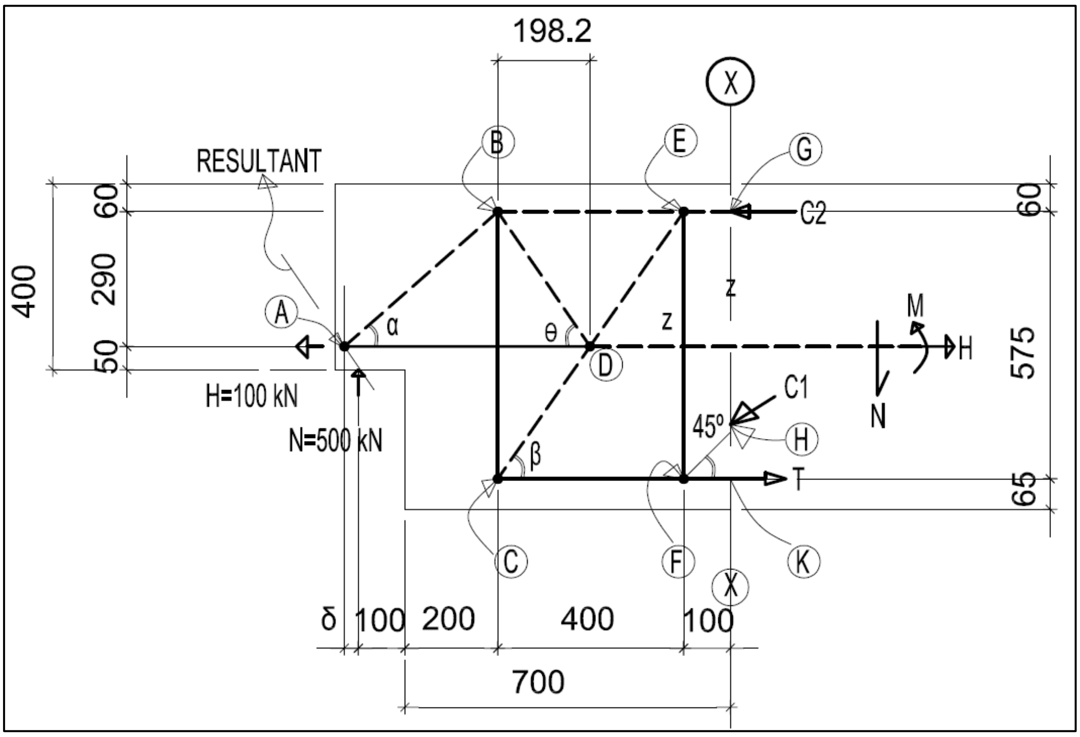
Check the Bearing Stress

Node type for the bearing is CCT. Maximum compressive bearing stress $\sigma_{Rd,max} = k_2\, v'\, f_{cd}$ where k_2=0.85, $v'=1-f_{ck}/250$, $f_{cd} = 0.567f_{ck}$

$\sigma_{Rd,max}$ = 0.85 x (1 – 40/250) x 0.567 x 40
= 16.19 N/mm^2

Design bearing stress = 500 x 10^3/(100 x 350)
= 14.28 N/mm^2 < 16.19 N/mm^2 **(OK)**

Strut and Tie Model



The tension ties, compressive struts and the nodes of an analogous strut and tie truss for the beam half joint are shown in the figure above.

The D-region is assumed to be one-member depth plus the overlapping D region of the half joint i.e. 150 + 700 = 850 mm as shown.

The offset from the resultant reaction at the level of the tie δ is:

$$\delta = 50 \times 100/500 = 10 \text{ mm}$$

The forces at the right end at section X are moment, axial tension force and shear force.

Shear force N = 500 kN
Moment M = 500 x 0.8 + 100 x 0.05 = 405 kNm
Axial tension H = 100 kN

The tension and compression forces in the truss at section X are:

	C_1	$= N/(\sin 45)$ $= 707.1$ kN
About point K	C_2	$= [405 - C_1 (\cos 45) \times 0.1 - 100 \times 0.575/2]/0.575$ $= 567.4$ kN
	T	$= C_1 (\cos 45) + C_2 + H$ $= 500 + 567.4 + 100$ $= 1167.4$ kN
Angles of inclination	α	$= \tan^{-1} [290/(\delta + 100 + 200)]$ $= 43.09^0$
	β	$= \tan^{-1} (575/400)$ $= 55.18^0$
	θ	$= \tan^{-1} (290/198.2)$ $= 55.64^0$

Strut Forces (+ tension, - compression)

Member	AB	BD	BE	DE	EG	CD	FH
Force(kN)	-731.9	-556.6	-220.0	-609.1	-567.4	-1168.8	-707.1

Tie Forces

Member	AD	BC	CF	EF	FK
Force(kN)	634.5	959.5	667.4	500.0	1167.4

Design of Ties

Required steel bar A_s = Tie force/(0.87 x f_{yk})

Member	AD	BC	CF	EF	FK
Force(kN)	634.5	959.5	667.4	500.0	1167.4
As (mm^2)	1459	2206	1534	1149	2684
Provide	3H25	6H16	4 H25	3H16	6H25
Remarks	Welded to angle	Close links @ 60 c/c	Welded to 130 x 25 thick plate	Close links @100 c/c	To be checked against mid span required steel area

Design of Struts

The maximum design stress in the struts without transverse tension is as follows:

$\sigma_{Rd,max}$ = f_{cd}
= $0.567f_{ck}$ (α_{cc} = 0.85, γ_c = 1.5)
= 22.68 N/mm^2

Upon inspection, nodes A, B, D and E are CCT and while nodes C and F are CTT. The maximum design stress at the nodal zone edges is as follows:

CCT (nodes A, B, D, E) $\sigma_{Rd,max}$ = $0.85 \times (1-f_{ck}/250) \times f_{cd}$
= 16.19 N/mm^2

CTT (nodes C, F) $\sigma_{Rd,max}$ = $0.75 \times (1-f_{ck}/250) \times f_{cd}$
= 14.29 N/mm^2

Let's assume conservatively that the maximum design stress in the struts is as follows:

Struts AB, BD, BE, DE, EG $\sigma_{Rd,max}$ = 16.19 N/mm^2

Struts CD, FH $\sigma_{Rd,max}$ = 14.29 N/mm^2

Strut Section

Strut width = 400 mm
Strut thickness (mm)

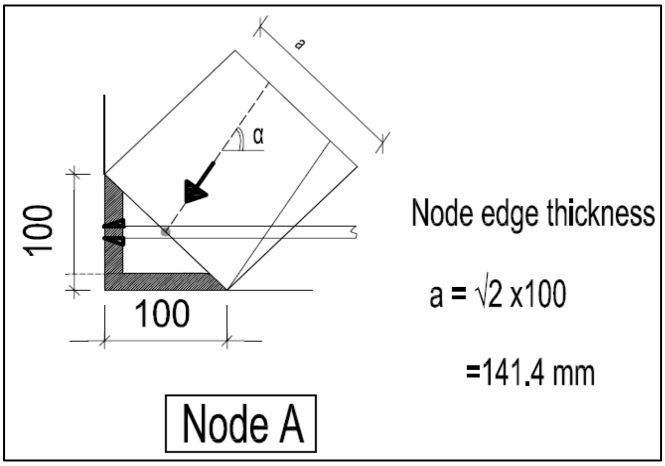
Member	AB	BD	BE	DE	EG	CD	FH
Force	-731.9	-556.6	-220.0	-609.1	-567.4	-1168.8	-707.1
$\sigma_{Rd,max}$	16.19					14.29	
Strut thickness	113.0	85.9	34.0	94.1	87.6	204.5	123.7

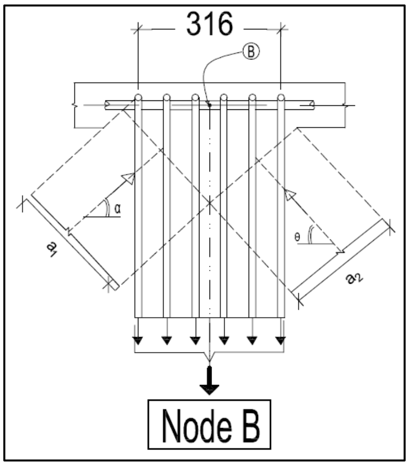
Check Nodal Zones

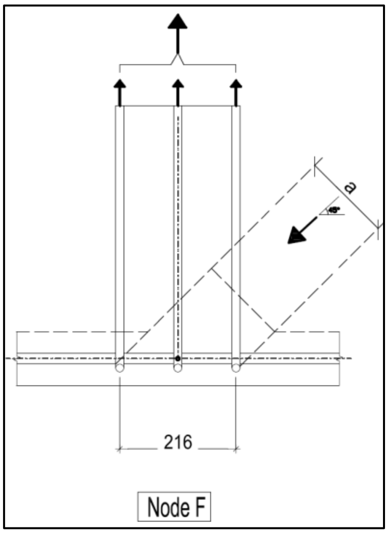
The required minimum thickness of the nodal zone edge for nodes A to F is similar to the respective strut thickness above.

Upon inspection, the critical nodal zones are nodes A, B and F.

Referring to the figures below, the available nodal zone edge thickness is:







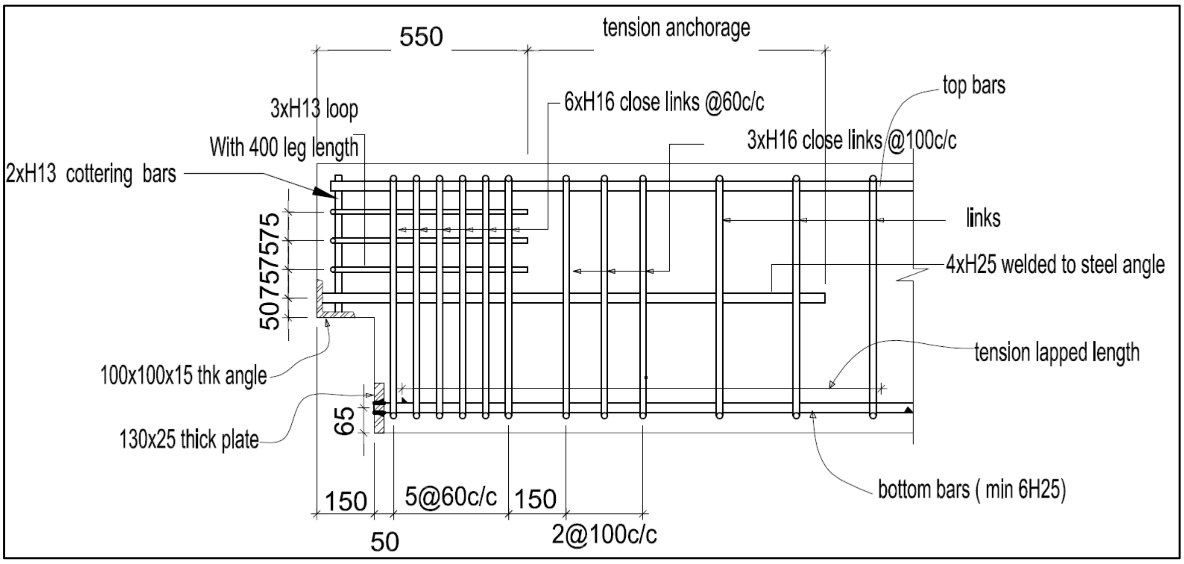
Strut AB	node A	a	$= \sqrt{2}$ x 100		
			= 141 mm	> 113.0 mm	(**OK)**
	node B	a_1	= 316 (sin43.09^0)		
			= 215.8 mm	> 113.0 mm	(**OK)**
Strut FH	node F	a	= 216 (sin45^0)		
			= 152.7 mm	> 123.7 mm	(**OK)**

Final Reinforcement Details

All reinforcement must be fully anchored beyond the nodes.

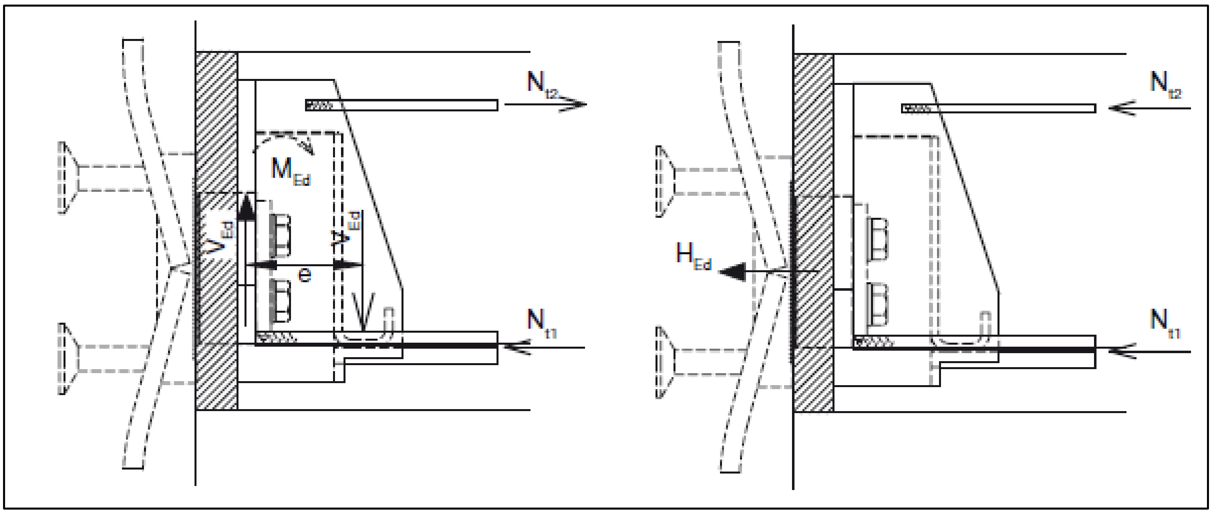
To improve crack control and ductility, you will need to provide a minimum reinforcement of $0.5A_s$, which is uniformly distributed at the half joint parallel to the bottom tie reinforcement.

Minimum A_s = 0.5 x 1459
= 730 mm^2 (Use 3 x H13 loops at 75 c/c)



Beam Half Joint → Selecting PC Beam Shoe (to be used with PCs Corbel)

The load transfer mechanism of PC Beam Shoe under vertical and horizontal loading is shown in the figure below.



PC Beam Shoes are pre-designed so that all components of the system have sufficient resistance against actions caused by external loads.

These design values of resistances are shown in the table below.

Design:
Vertical shear force in connection: $V_{Ed} = 500$ kN
Horizontal tensile force in connection: $H_{Ed} = 100$ kN

Select beam shoe connector:
Beam shoe **PC 5-H**
Shear resistance **$V_{Rd} = 520$ kN**
Tensile resistance **$H_{Rd} = 104$ kN**

$V_{Ed} < V_{Rd}$ → 500 kN < 520 kN
$H_{Ed} < H_{Rd}$ → 100 kN < 104 kN

Load	Notations	Units	PCs 2	PCs 3	PCs 5	PCs 7	PCs 10	PCs 15
Vertical load	V_{Rd}	kN	230	355	520	710	960	1500
Horizontal load	H_{Ed}	kN	46	71	104	142	192	300

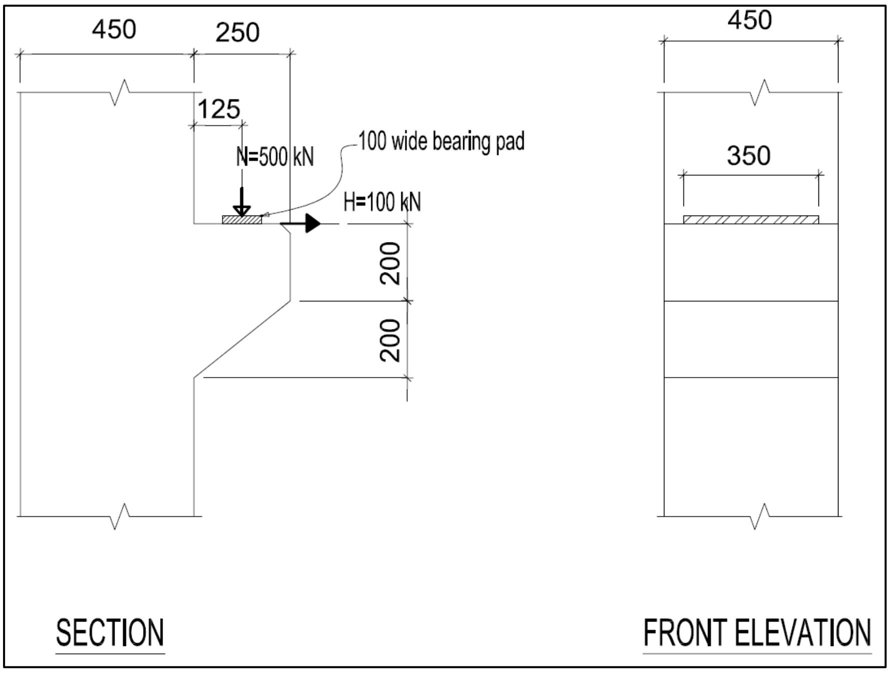
The following subjects must be verified from the technical manual of PC Beam Shoe:

a) Minimum required cross section of the beam
b) Positioning of the beam shoe in the beam
c) Required supplementary reinforcement
d) Other relevant instructions

DESIGN EXAMPLE 2
RC Corbel

Design a reinforced concrete corbel, as shown, to support:

- Vertical load N = 500 kN
- Ultimate horizontal force N = 100 kN
- The corbel is framed flushed with 450x450 RC column
- Design concrete class C40/50
- f_{yk} = 500 N/mm2 for reinforcement bars
- Concrete cover = 35mm to links



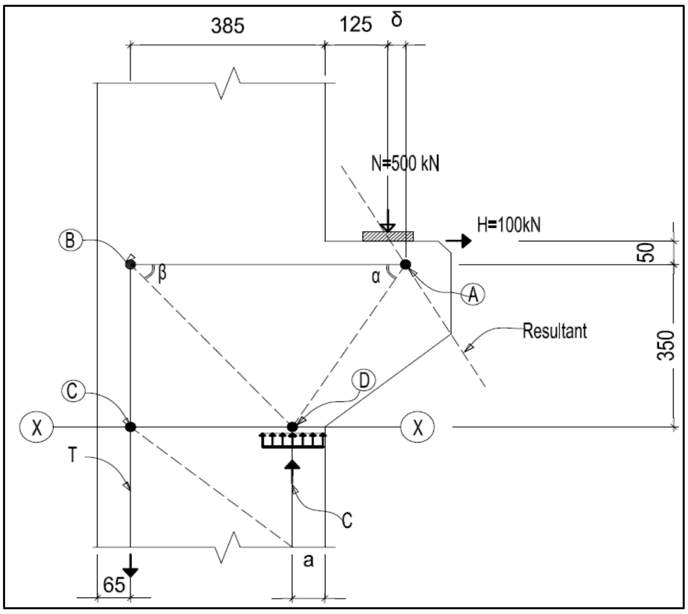
<u>Check Bearing Stress</u>

Node type for the bearing is CCT. The maximum compressive bearing stress $\sigma_{Rd,max} = k_2 v' f_{cd}$ where k_2=0.85, $v'=1-f_{ck}/250$, $f_{cd} = 0.567 f_{ck}$

$\sigma_{Rd,max}$ = 0.85 x (1 – 40/250) x 0.567 x 40
= 16.19 N/mm^2

Design bearing stress = 500 x 10^3/(100 x 350)
= 14.28 N/mm^2 < 16.19 N/mm^2 **(OK)**

Strut and Tie Model



The tension ties, compressive struts and the nodes of an analogous strut and tie truss for the corbel are shown in the figure above.

Tension ties are shown as solid lines and compression strut forces as dashed lines. The forces are assumed acting at the centre line of the ties and struts.

The offset from the resultant reaction at the level of the tie δ is as follows:

$$\delta = 50 \times 100/500$$
$$= 10 \text{ mm}$$

The external forces at section X are determined as follows:

Moment about node C

$C \times (385\text{-}a) = 500 \times (385 + 125 + 10) + 100 \times (400 - 50)$

$= 295 \times 10^3$

At node D, a CCT node type, the maximum compressive bearing stress $\sigma_{Rd,max}$ is as follows:

$\sigma_{Rd,max}$ = 16.19 N/mm^2

Compression force, C = $\sigma_{Rd,max}$ x 2a x 450

$\sigma_{Rd,max}$ x 2a x 450 x (385 -a) x 10^{-3} = 295 x 10^3

a x (385 -a) = (295 x 10^3) / (2 x 16.19 x 450 x 10^{-3})

a^2 - 385a + 20.245 x 10^3 = 0

Solving the equation, a = 62.8 mm

Angles of inclination, α = $\tan^{-1}$ (350/197.8)

= 60.53^0

β = $\tan^{-1}$ (350/322.2)

= 47.37^0

(Note: The angle of inclination for the strut in a corbel is limited to $1.0 \le \tan\theta \le 2.5$, i.e. $45^0 \le \theta \le 68^0$, EC2 Annex J.3)

Strut Forces (+ tension, - compression)

Member	AD	BD	Compression C
Force (kN)	-574.3	-564.8	-915.1

Design of Ties

Required steel bar A_s = Tension force/(0.87 x f_{yk})

Member	AB	BC	CD
Force (kN)	+382.5	+415.5	+100.0
As (mm^2)	879	955	230
Provide	5H16	5H16	2H10 (4 legs)
Remarks	-	Continued from AB ties	Bundled links

Design of Struts

The maximum design stress in the struts without transverse tension:

$\sigma_{Rd,max}$ $= f_{cd}$
$= 0.567 f_{ck}$ $(\alpha_{cc} = 0.85, \quad \gamma_c = 1.5)$
$= 22.68$ N/mm^2

Upon inspection, nodes A, D are CCT and nodes B is CTT.

The maximum design stress at the nodal zone edges:

CCT nodes A, D $\quad \sigma_{Rd,max} = 0.85 \times (1-f_{ck}/250) \times f_{cd}$
$= 16.19$ N/mm^2

CTT node B $\quad \sigma_{Rd,max} = 0.75 \times (1-f_{ck}/250) \times f_{cd}$
$= 14.29$ N/mm^2

Let's assume conservatively that the maximum design stress in the struts is as follows:

Struts AD $\quad \sigma_{Rd,max} = 16.19$ N/mm^2

Struts BD $\quad \sigma_{Rd,max} = 14.29$ N/mm^2

Strut Section

Strut width = 450mm
Strut thickness (mm)

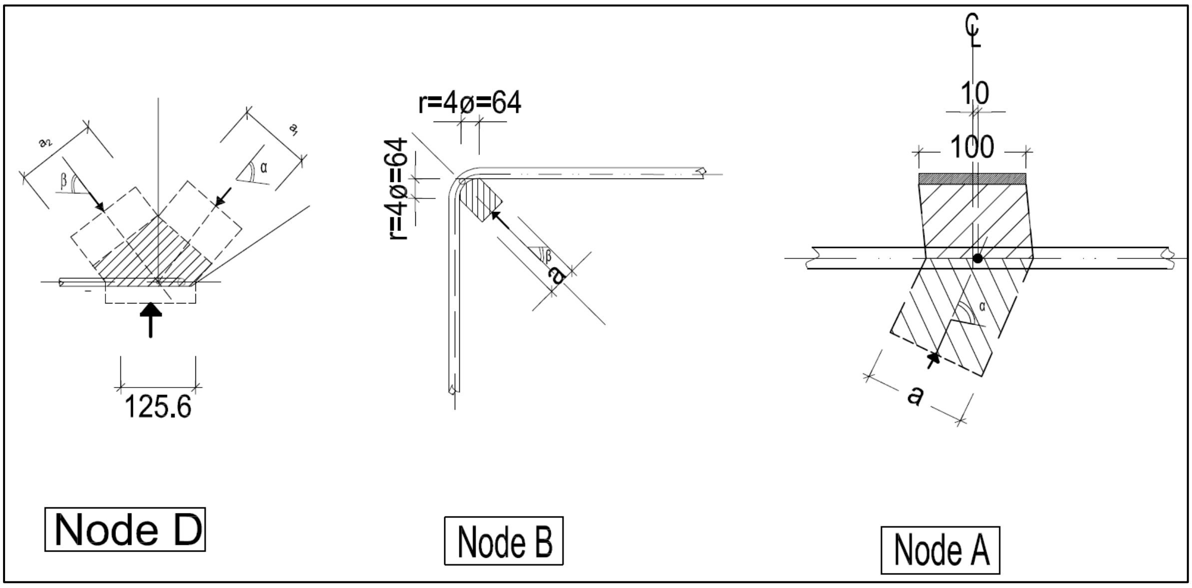
Member	**AD**	**BD**
Force (kN)	-574.3	-564.8
$\sigma_{Rd,max}$ (N/mm^2)	16.19	14.29
Strut thickness (mm)	78.8	87.8

Check Nodal Zones

The required minimum thickness of the nodal zone edge for nodes A to F is similar to the respective strut thickness above.

By inspection, the critical nodal zones are located in nodes A, B and F.

Referring to the figures below, the available nodal zone edge thickness is:



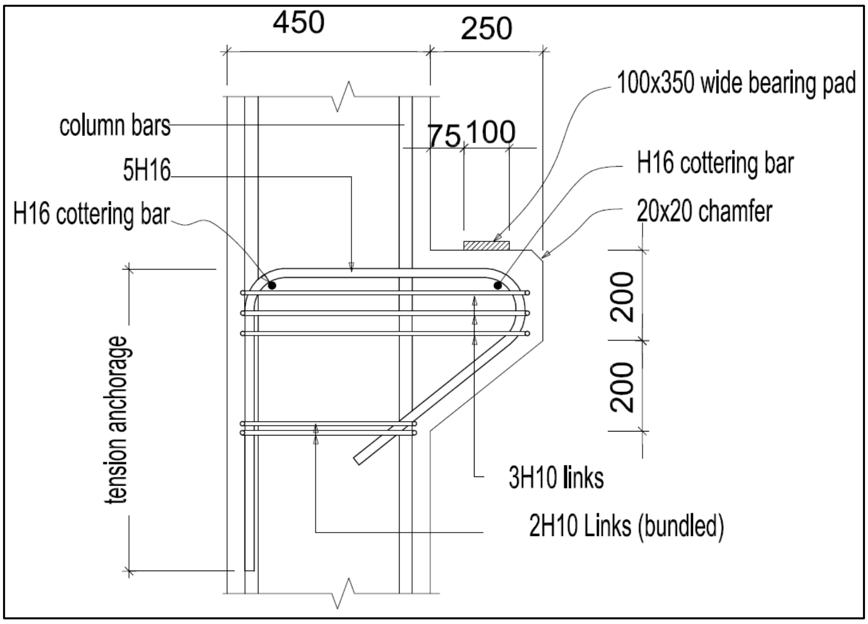
Strut AD,	node A	= 100 (sin 60.53^0)		
		= 87.1 mm	> 78.8 mm	**(OK)**
	node D	= 125.6 (sin60.53^0)		
		= 109.3 mm	> 78.8 mm	**(OK)**
Strut BD,	node B	= $\sqrt{2}$ x 64		
		= 90.5 mm	> 87.8 mm	**(OK)**
	node D	= 128.4 (sin47.37^0)		
		= 94.5 mm	> 87.8 mm	**(OK)**

Final Reinforcement Details

All reinforcement must be fully anchored beyond the nodes.

To improve crack control and ductility, you will need to provide a minimum reinforcement of $0.5A_s$, which is uniformly distributed, at the half joint parallel to the bottom tie reinforcement.

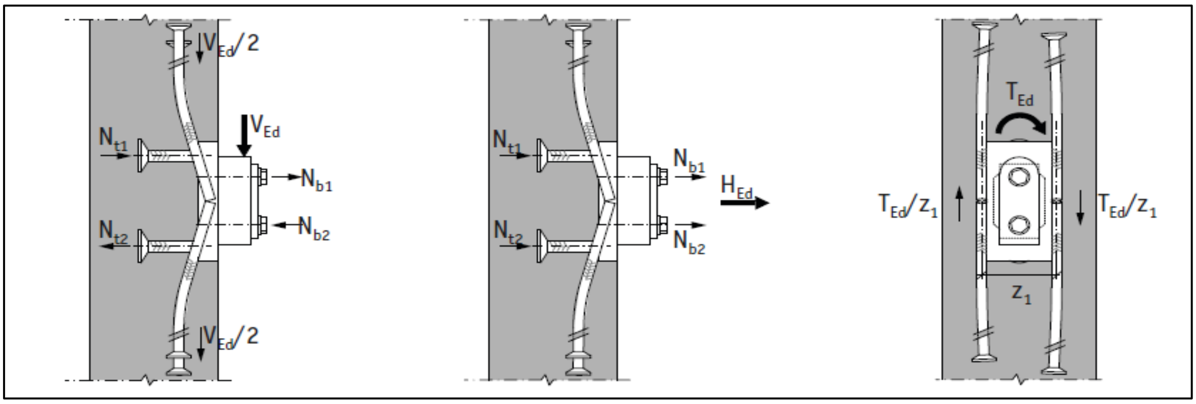
Minimum A_s	$= 0.5 \times 879$	
	$= 440\ mm^2$	Use 3H10 close links (6 legs)



DESIGN EXAMPLE 2
RC Corbel

Steel Corbel → Selecting Peikko® PCs Corbel (to be used with PC Beam Shoe)

The load transfer mechanism of PCs Corbel under vertical and horizontal loading is shown in the figure below.



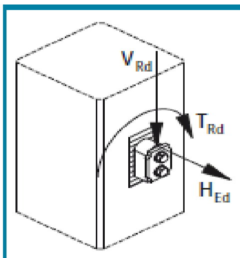
PCs Corbels are pre-designed so that all components of the system have sufficient resistance against actions caused by external loads.

The design values of resistances are shown in the table below.

<u>Design:</u>
Vertical shear force in connection: $V_{Ed} = 500$ kN
Horizontal tensile force in connection: $H_{Ed} = 100$ kN

<u>Select corbel:</u>
Corbel **PCs 5**
Shear resistance **$V_{Rd} = 520$ kN**
Tensile resistance **$H_{Rd} = 104$ kN**

$V_{Ed} < V_{Rd}$ → 500 kN < 520 kN
$H_{Ed} < H_{Rd}$ → 100 kN < 104 kN



Load		Units	PCs 2	PCs 3	PCs 5	PCs 7	PCs 10	PCs 15
Vertical load	V_{Rd}	[kN]	230	355	520	710	960	1500
Horizontal load	H_{Ed}	[kN]	46	71	104	142	192	300
Torsional moment	T_{Rd}	[kN/m]	7	15	25	50	75	170

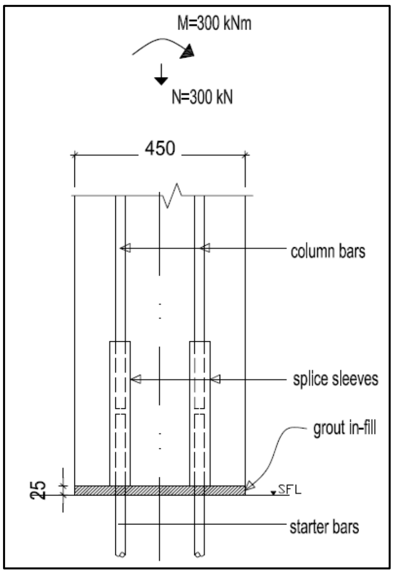
The following subjects must be verified from the technical manual of PCs Corbel:

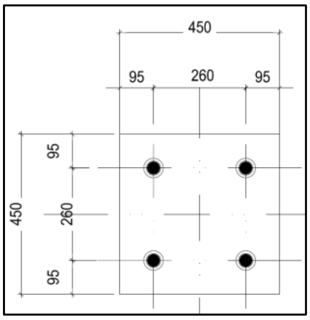
a) Minimum required cross section of the column
b) Positioning of the corbel in the column
c) Required supplementary reinforcement
d) Other relevant instructions

DESIGN EXAMPLE 3
Column Base

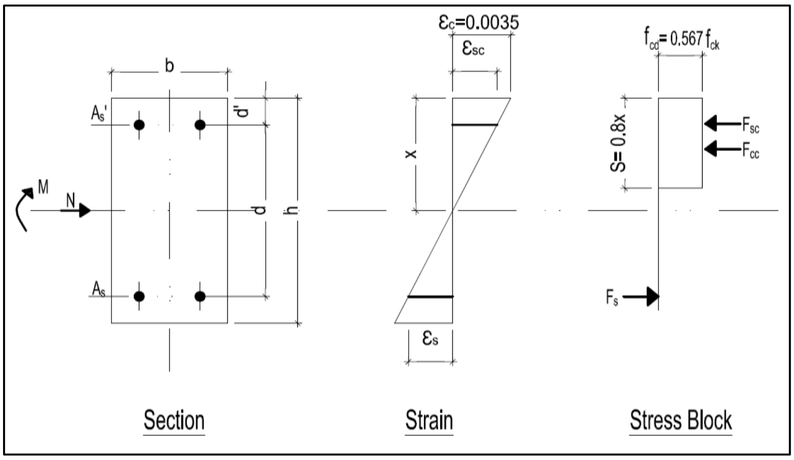
Design the column base connection with splice sleeves for a 450 x 450 column, to support:

- Vertical ultimate load of 3000 kN
- Bending moment of 300 kNm as shown in the figure
- Design concrete class C40/50
- f_{yk} = 500 N/mm^2 for reinforcement bars
- Concrete cover = 35mm to links
- Strength of grout is to be minimum C40/50





The figure below shows the cross section of the column at the joint subjected to bending and axial load.



Assuming symmetrical reinforcement arrangement, $A_s' = A_s$, the capacity of the joint connection is determined as follows:

(a) Force Equilibrium:

	N	$= F_{cc} + F_{sc} – F_s$	
where	N	$= 3000$ kN	
	F_{cc}	$= 0.567bf_{ck}s$	
		$= 0.567bf_{ck}(0.8x)$	
	F_{sc}	$= f_{sc}A_s$	
	F_s	$= f_sA_s$	
	fs	$= E_s\varepsilon_s$	
	ε_s	$= 0.0035\ (d\text{-}x)/x$	
	f_{sc}	$= E_s\varepsilon_{sc}$	
	ε_{sc}	$= 0.0035\ (x\text{-}d')/x$	
	d'	= 95 mm	
	d	= 450 - 95 = 355 mm	
Assume 4H32, total area		= 3217 mm2	
	A_s	= 3217/2	
		= 1608 mm^2	
Ultimate (elastic-plastic)	ε_{su}	$= 0.87f_{yk}/E_s$	
		$= 0.87 \times 500 / 200\times10^3$	
		$= 0.002175$	

By using the trial and error approach, the value of x results in close agreement in the equation is as follows:

N = Fcc + Fsc – Fs is

x	= 304.61 mm	
ε_s	= 0.0035 (355 - 304.61)/304.61	
	= 0.000579	$< \varepsilon_{su} = 0.002175$
f_s	$= 200\times10^3 \times 0.000579$	
	= 115.9 N/mm^2	
ε_{sc}	= 0.0035 (355 - 95)/304.61	
	= 0.00299	$> \varepsilon_{su} = 0.002175$
f_{sc}	$= 200\times10^3 \times 0.002175$	
	= 435 N/mm^2	

$F_{cc} + F_{sc} - F_s$ $= [\ 0.567 \times 450 \times 40 \times (0.8 \times 304.61) + 435 \times 1608 - 115.9 \times 1608\ ] \times 10^{-3}$

= 2487.0 + 699.5 - 186.4

= 3000.1 kN $\approx$ N = 3000 kN **(OK)**

(b) Moment Equilibrium

Taking moment about neutral axis x

M_{Rd} = - N(x - h/2) + Fcc (0.6x) + Fsc (x-d') + Fs (d-x)

= [-3000 x (304.61 - 450/2) + 2487.0 x 0.6 x 304.61 + 699.5 x (304.61- 95) + 186.4 x (355 - 304.61)] x 10-3

= - 238.8 + 454.5 + 146.6 + 9.4

= 371.7 kNm > M = 300 kNm **(OK)**

Column to column connection using 4H32 with splice sleeves is acceptable.

Theses splice sleeves must be threaded at one end with grout infill at the other end where starter bars are joined.

Note:

The steel bars provided for the column to column connection above will need to satisfy these following requirements:

a) Vertical wall ties through the connection which is given by $A_{s,tie} = N_{flr}/f_{yk}$, where N_{flr} is the ultimate floor load carried by the wall at the floor where the ties are determined

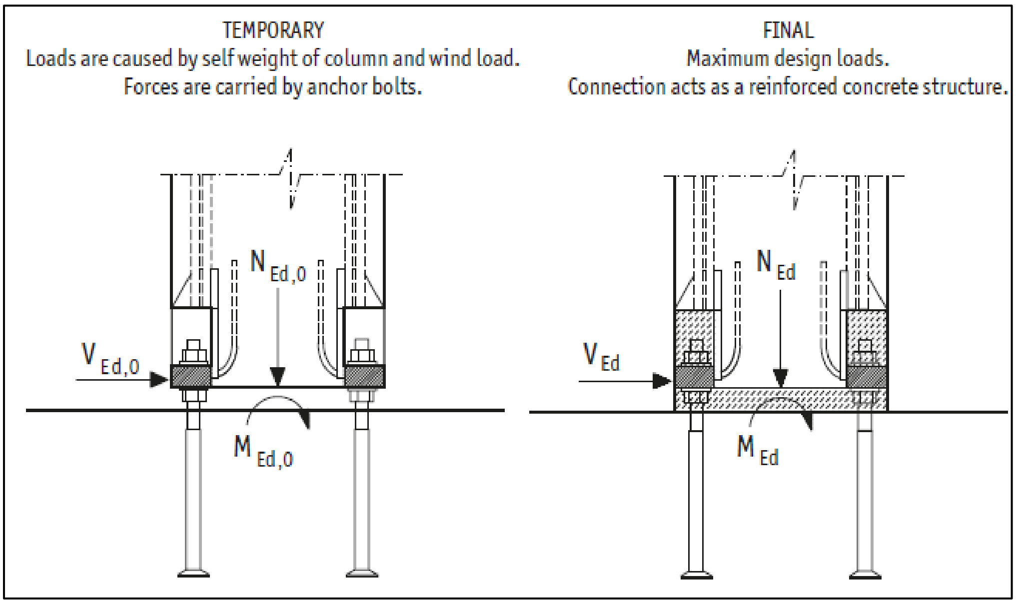
b) Minimum and maximum wall reinforcement of $0.002A_c \leqslant A_s \leqslant 0.04A_c$

DESIGN EXAMPLE 3
Column Base

Column Base → Selecting Peikko® HPKM and PEC Column Shoes (to be used with HPM and PPM Anchor Bolts)
Peikko® Column Shoes are designed to withstand tension and compression forces corresponding to the design values of resistances of Peikko® Anchor Bolts.

The maximum design values of resistances of individual Column Shoes are given in the table below.

Column Shoes and Anchor Bolts are pre-designed so that all components of the system have sufficient resistance against actions caused by external loads.

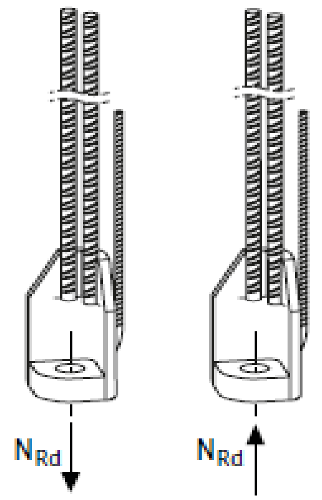


Design:
Tensile force in connection: N_{Ed} = 179 kN / 2 = 89 kN per column shoe

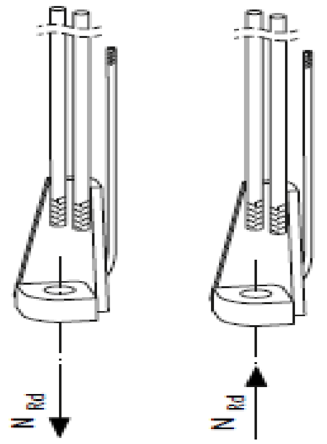
Select anchoring coupler and beam shoe:
Column shoe **HPKM 20, 4 nos**
Anchor bolt **HPM 20, 4 nos**
Resistance of the component **NRd = 96 kN**

$N_{Ed} < N_{Rd}$ → 89 kN < 96 kN

Column Shoe	Anchor Bolt	N_{Rd} [kN]
HPKM 16	HPM 16	62
HPKM 20	HPM 20	96
HPKM 24	HPM 24	139
HPKM 30	HPM 30	220
HPKM 39	HPM 39	383



Column Shoe	Anchor Bolt	N_{Rd} [kN]
PEC 30	PPM 30	299
PEC 36	PPM 36	436
PEC 39	PPM 39	520
PEC 45	PPM 45	697
PEC 52	PPM 52	938



It is recommended to calculate the resistances of the column connection by using Peikko Designer® software. It is possible to optimise connections and calculate resistances for temporary, permanent and fire conditions.

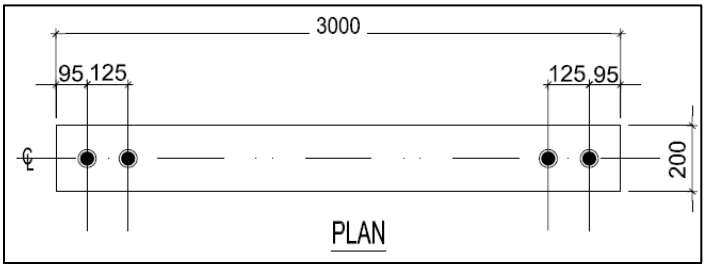
The following subjects must be verified from the technical manual of HPKM/PEC Column Shoe:

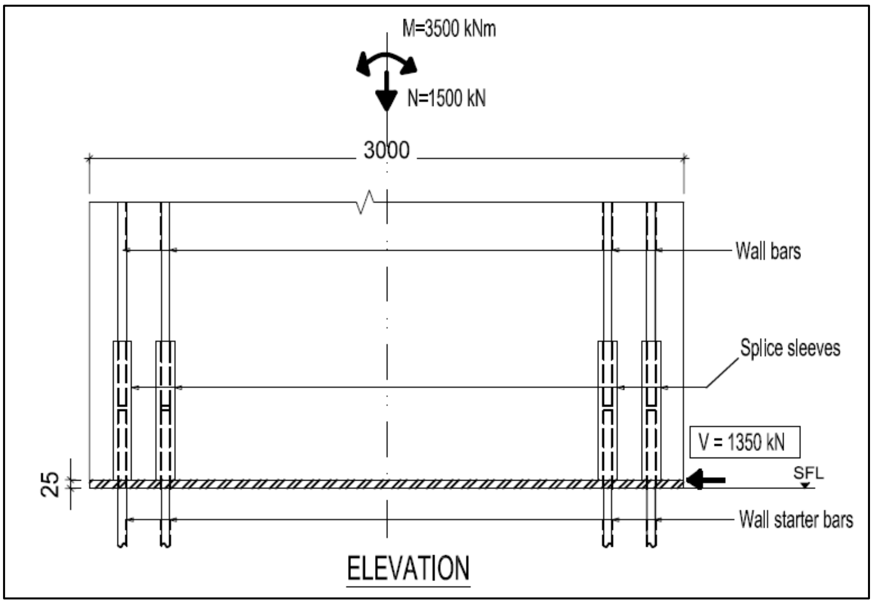
a) Minimum required cross section of the column
b) Positioning of the column shoe in the column
c) Required supplementary reinforcement
d) Other relevant instructions

Design the gable end shear wall base connection with splice sleeves, to support:

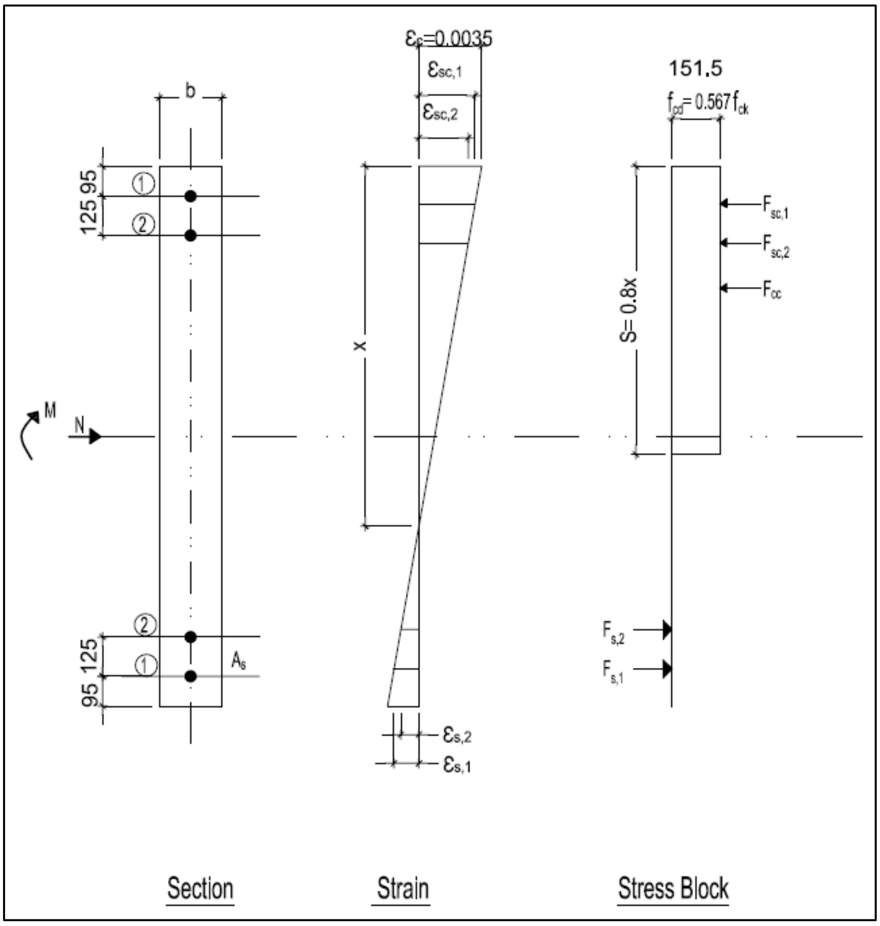
- Wall is 200mm thick and 3000 mm long
- Wall base is subjected to axial load N = 1500 kN, with the edge distance and spacing between sleeves as shown in the figures below.
- Design concrete class C40/50
- f_{yk} = 500 N/mm^2
- E_s = 200 kN/mm^2 for steel reinforcement bars
- Concrete cover = 35mm to links
- Strength of grout is to be minimum C40/50

The figure below shows the cross section of the wall base at the joint subjected to bending and axial load.





Assuming symmetrical reinforcement arrangement, $A_s' = A_s$, the capacity of the joint connection is determined as follows:



Referring to the stress strain figure for the wall section at the wall base joint above, the following design forces at the joint can be derived.

1. <u>Wall Connection Axial Load and Bending Capacity</u>

(a) <u>Force Equilibrium:</u>

$N = F_{cc} + F_{sc,1} + F_{sc,2} - F_{s,1} - F_{s,2}$

where $N = 1500$ kN

$F_{cc} = 0.567bf_{ck}s$
$= 0.567bf_{ck}(0.8x)$

$F_{sc,1} = f_{sc,1}A_{s,1}$

$F_{sc,2} = f_{sc,2}A_{s,2}$

$F_{s,1} = f_{s,1}A_{s,1}$

$F_{s,2} = f_{s,2}A_{s,2}$

$f_{sc,1} = E_s\varepsilon_{sc,1}$
$\varepsilon_{sc,1} = 0.0035\,(x-d')/x$ $\quad d' = 95$ mm

$f_{sc,2} = E_s\varepsilon_{sc,2}$
$\varepsilon_{sc,2} = 0.0035\,(x-d')/x$ $\quad d' = 95 + 125 = 220$ mm

$f_{s,1} = E_s\varepsilon_{s,1}$
$\varepsilon_{s,1} = 0.0035\,(d-x)/x$ $\quad d = 3000 - 95 = 2905$ mm

$f_{s,2} = E_s\varepsilon_{s,2}$
$\varepsilon_{s,2} = 0.0035\,(d-x)/x$ $\quad d = 3000 - 95 - 125 = 2780$ mm

Assume 4H32, total area = 3217 mm2 $\quad A_{s,1} = A_{s,2} = A_{s',1} = A_{s',2} = A_s$

$A_s = 3217/4$
$= 804\ mm^2$

Ultimate (elastic-plastic) $\varepsilon_{su} = 0.87f_{yk}/E_s$
$= 0.87 \times 500 / 200\times10^3$
$= 0.002175$

By using the trial and error approach, the value of x results in close agreement in the equation is as follows:

$N = F_{cc} + F_{sc,1} + F_{sc,2} - F_{s,1} - F_{s,2}$ is

$x = 433.5$ mm

$\varepsilon_{sc,1} = 0.0035\,(433.5 - 95)/433.5$
$= 0.002733 \qquad < \varepsilon_{su} = 0.002175$

$f_{sc,1} = 200\text{x}10^3 \text{ x } 0.002175$
$= 435 \text{ N/mm}^2$

$\varepsilon_{sc,2} = 0.0035\,(433.5 - 220)/433.5$
$= 0.001724$

$f_{sc,2} = 200\text{x}10^3 \text{ x } 0.001724$
$= 344.8 \text{ N/mm}^2$

$\varepsilon_{s,1} = 0.0035\,(2905 - 433.5)/433.5$
$= 0.01995 \qquad > \varepsilon_{su} = 0.002175$

$f_{s,1} = 200\text{x}10^3 \text{ x } 0.002175$
$= 435 \text{ N/mm}^2$

$\varepsilon_{s,2} = 0.0035\,(2780 - 433.5)/433.5$
$= 0.01894 \qquad > \varepsilon_{su} = 0.002175$

$f_{s,2} = 200\text{x}10^3 \text{ x } 0.002175$
$= 435 \text{ N/mm}^2$

$F_{cc} + F_{sc,1} + F_{sc,2} - F_{s,1} - F_{s,2} = [\,0.567 \text{ x } 200 \text{ x } 40 \text{ x } (0.8 \text{ x } 433.5) + 435 \text{ x } 804 + 344.8 \text{ x } 804 - 435 \text{ x } 804 - 435 \text{ x } 804\,] \text{ x } 10^{-3}$
$= 1573.1 + 349.7 + 277.2 - 349.7 - 349.7$
$= 1500.6 \text{ kN} \qquad \approx N = 1500 \text{ kN}$ **(OK)**

(b) <u>Moment Equilibrium</u>

Taking moment about neutral axis x

$M_{Rd} = N(h/2 - x) + F_{cc}\,(0.6x) + F_{sc,1}\,(x-95) + F_{sc,2}\,(x-220) + F_{s,1}\,(2905 - x) + F_{s,2}\,(2875 - x)$
$= [1500 \text{ x } (3000/2 - 433.5) + 1573.1 \text{ x } 0.6 \text{ x } 433.5 + 349.7 \text{ x } (433.5 - 95) + 277.2 \text{ x } (433.5 - 220) + 349.7 \text{ x } (2905 - 433.5) + 349.7 \text{ x } (2780 - 433.5)] \text{ x } 10^{-3}$
$= 1599.8 + 409.2 + 118.4 + 59.2 + 864.3 + 820.6$
$= 3871.5 \text{ kNm} \qquad > M = 3500 \text{ kNm}$ **(OK)**

Wall to wall connection using splice sleeves for 2xH32 at each end in single row arrangement along wall centre is acceptable.

The splice sleeves are to be threaded at one end with grout infill at the other end where starter bars are joined.

Note:

The steel bars provided for the wall to wall connection above will need to satisfy the following requirements:

a) Vertical wall ties through the connection which is given by $A_{s,tie} = N_{flr}/f_{yk}$, where N_{flr} is the ultimate floor load carried by the wall at the floor where the ties are determined

b) Minimum and maximum wall reinforcement of $0.002A_c \leq A_s \leq 0.04A_c$

2. Interface Shear at the Wall Base

Average interface shear at the wall base:

$$
\begin{aligned}
v_{Edi} &= V/(bh) \\
&= 1350 \times 10^3/(200 \times 3000) \\
&= 2.25 \quad N/mm^2
\end{aligned}
$$

The interface shear resistance at the base of the wall is determined by the expression 6.25 in EC 2 code:

$$v_{Rdi} = c\, f_{ctd} + \mu\sigma_n + \mu\rho f_{yd}$$

The surface of the wall base is conservatively considered as smooth surface and c = 0.2, μ = 0.6 (clause 6.2.5.(2))

$$
\begin{aligned}
cf_{ctd} &= 0.2 \times 0.14 f_{ck}^{2/3} \\
&= 0.2 \times 0.14 \times 40^{2/3} \\
&= 0.33 \quad N/mm^2 \\
\mu\sigma_n &= 0.6 \times N/(bh) \\
&= 0.6 \times 1500 \times 10^3/(200 \times 3000) \\
&= 1.50 \quad N/mm^2 \\
c\, f_{ctd} + \mu\sigma_n &= 1.83 \quad N/mm^2
\end{aligned}
$$

Required shear friction reinforcement is as follows:

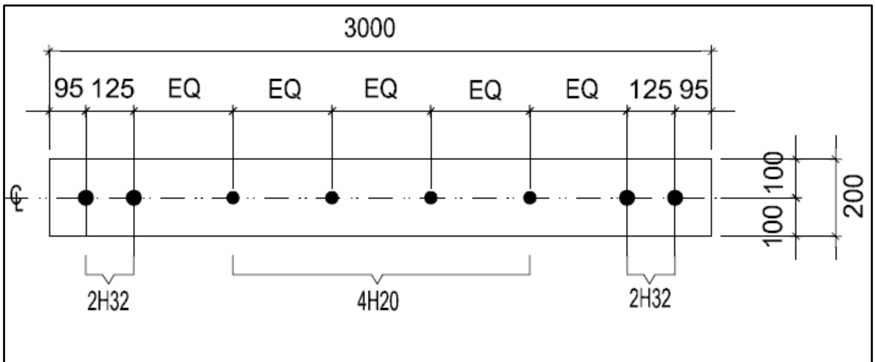
$$
\begin{aligned}
\rho &= (v_{Edi} - v_{Rdi})/(\mu f_{yd}) \\
&= (2.25 - 1.83)/(0.6 \times 0.87 \times 500) \\
&= 0.0016 \\
\text{Total } A_{sv} &= 0.0016 \times 200 \times 3000 \\
&= 966 \quad mm^2
\end{aligned}
$$

Use 4H20, single row arrangement between main connection bars at both ends

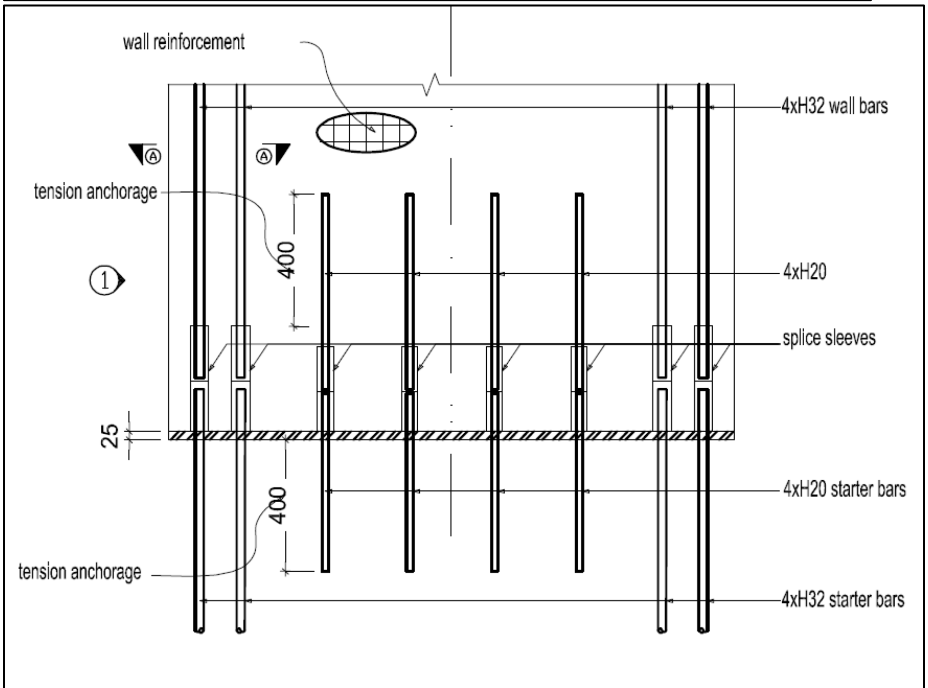
Note:

When determining the friction resistance, $\mu\sigma_n$, the axial compressive load N should be computed conservatively based on the more appropriate permanent self-weight of the structure excluding any variable superimposed dead and live loads.

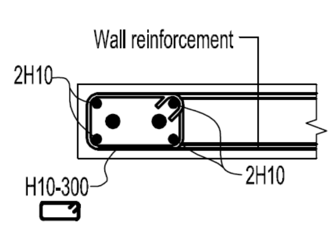
Final reinforcement at the wall to wall connection details are shown below.



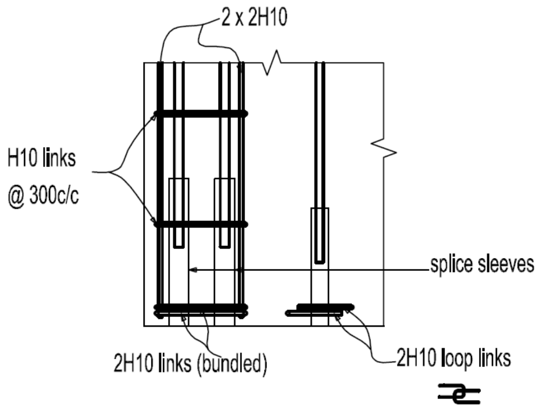
Wall Starter Bars



Wall Elevation



Seciton A



View 1

DESIGN EXAMPLE 4
Wall Base

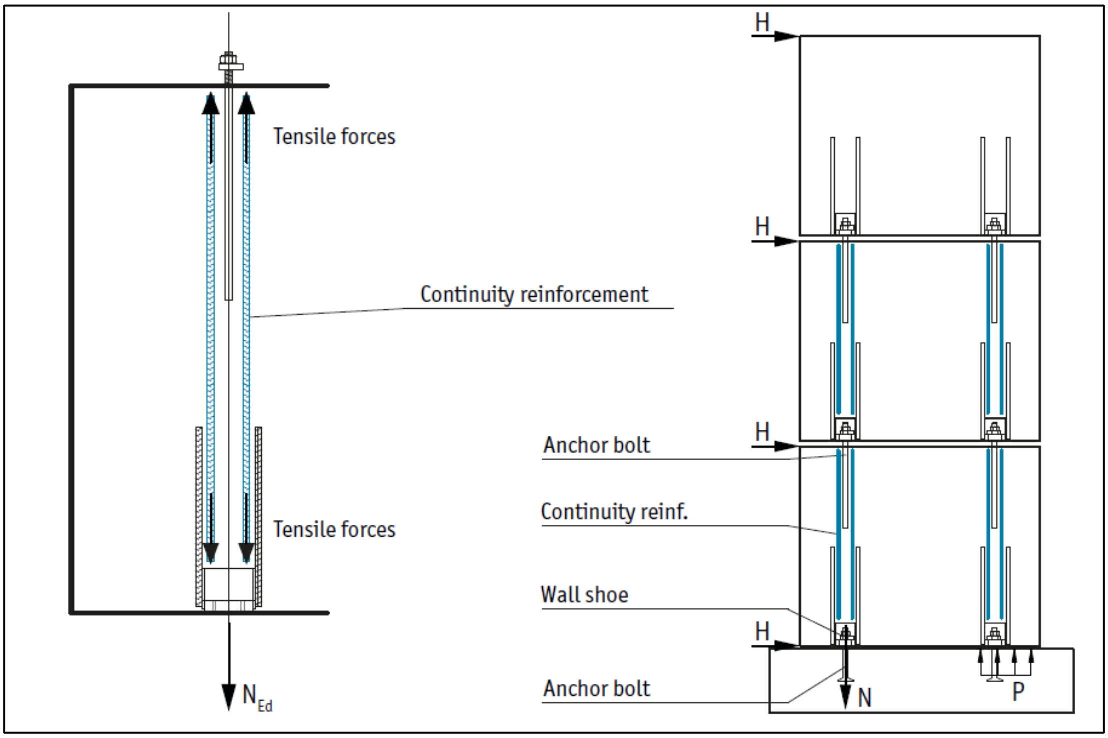
Specialist Design Alternative

Wall Base → Selecting Peikko® SUMO Wall Shoes (to be used with HPM and PPM Anchor Bolts)

Peikko® Wall Shoes are designed to withstand tension forces corresponding to the design values of resistances of Peikko® Anchor Bolts.

The maximum design values of resistances of individual Wall Shoes are given in the table below.

Wall Shoes and Anchor Bolts are pre-designed so that all components of the system have sufficient resistance against actions caused by external loads.



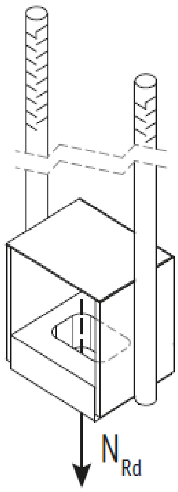
Design:
Tensile force in connection: N_{Ed} = 676 / 2 = 338 kN per wall shoe

Select anchoring coupler and wall shoe:

Column shoe	**SUMO 39H, 4 nos**
Anchor bolt	**HPM 39, 4 nos**
Resistance of the component	**N_{Rd} = 383 kN**

$N_{Ed} < N_{Rd}$ → 338 kN < 383 kN

Wall Shoe	Anchor Bolt	Washer	N_{Rd} [kN]
SUMO 16H	HPM 16	AL 16	62
SUMO 20H	HPM 20	AL 20	96
SUMO 24H	HPM 24	AL 24	139
SUMO 30H	HPM 30	AL 30	220
SUMO 39H	HPM 39	AL 39	383
SUMO 30P	PPM 30	AL 30	299
SUMO 36P	PPM 36	AL 36	436
SUMO 39P	PPM 39	AL 39	521
SUMO 45P	PPM 45	AL 45	697
SUMO 52P	PPM 52	AL 52	938



The following subjects must be verified from the technical manual of SUMO Wall Shoe:

a) Minimum required thickness of the wall

b) Positioning of the wall shoe in the wall

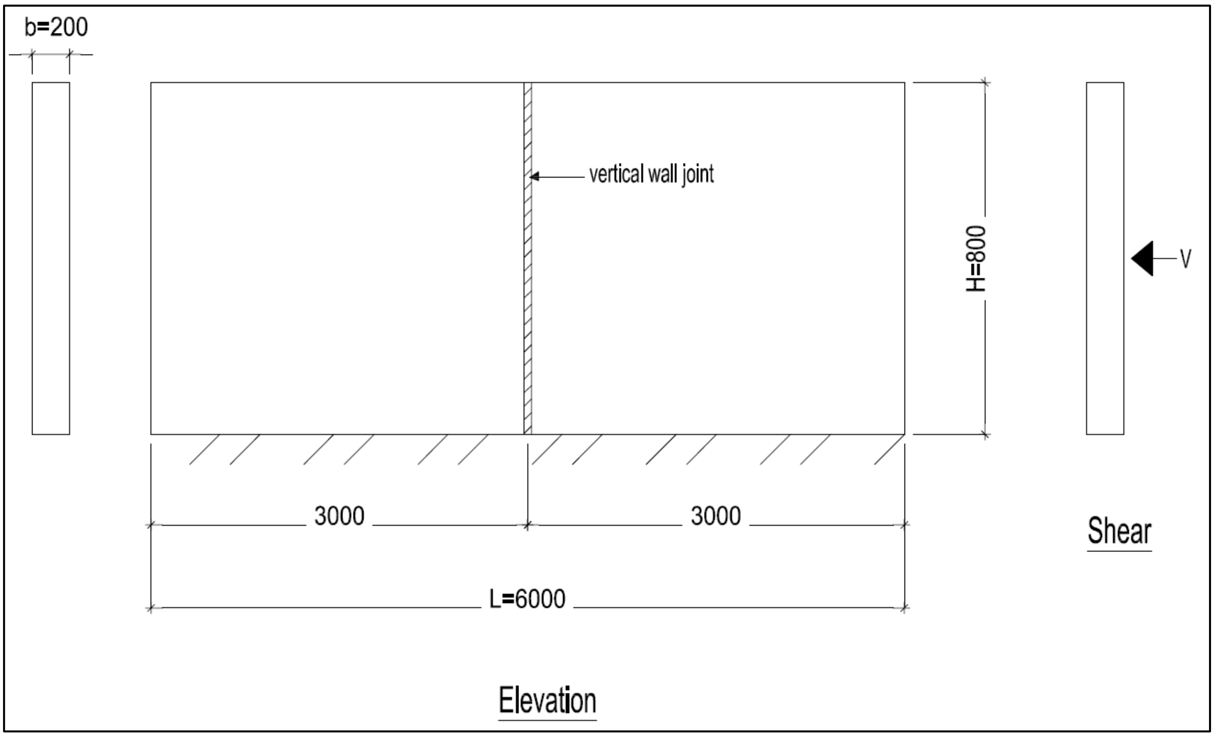
c) Required supplementary reinforcement

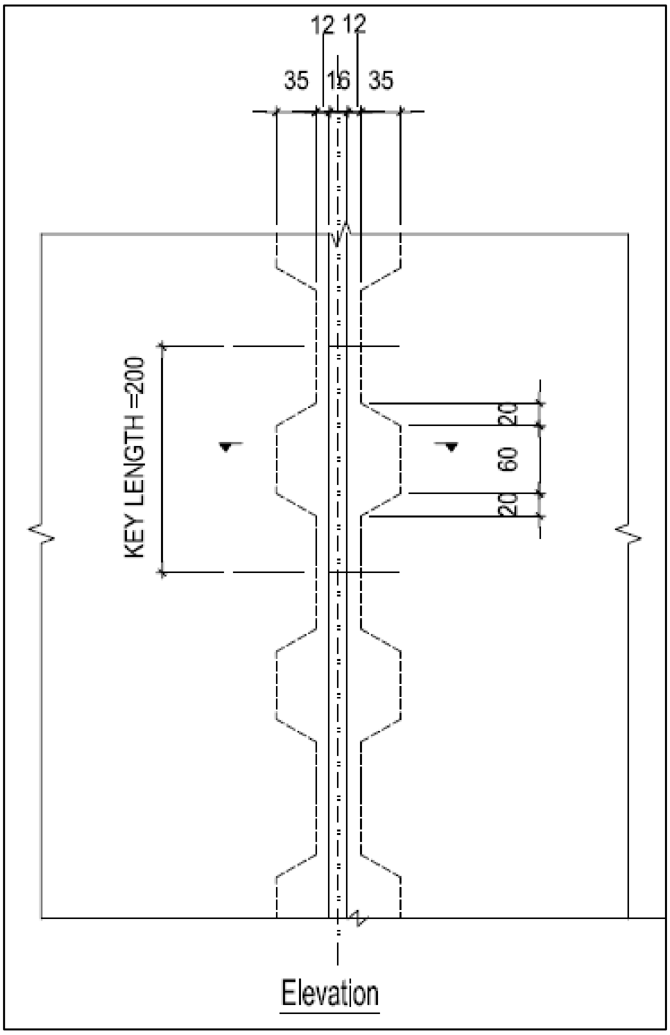
d) Other relevant instructions

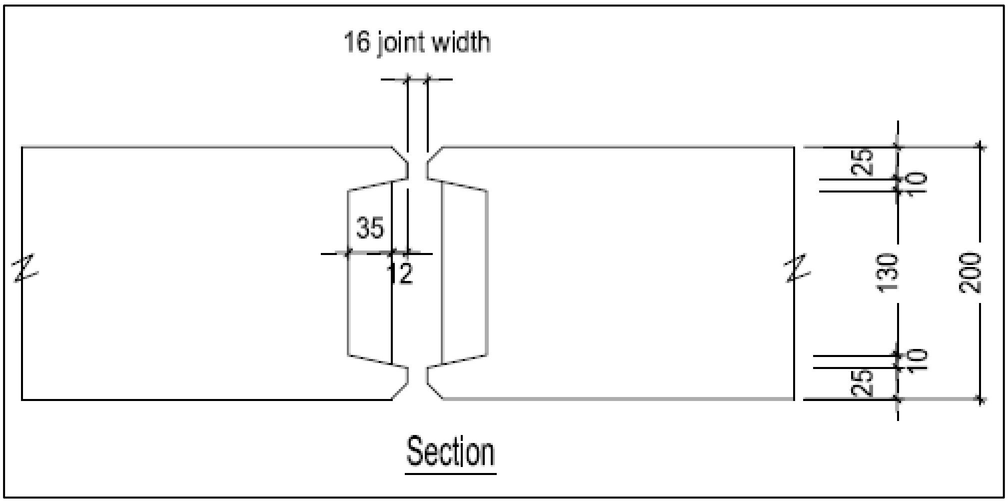
DESIGN EXAMPLE 5 **Vertical Wall to Wall Joint with Shear Keys**	**Design Principles**

A shear wall panel of height H = 2.8 m, length L = 6m and thickness b = 200 mm is split into two smaller wall segments of 3m length for ease of lifting.

- Subjected to an ultimate in plane shear force V = 1200 kN as shown in the figure
- Continuous shear keys are proposed for the vertical wall joint connection; the dimensions of a shear key unit are shown below
- Design concrete strength class C35/45 for use in wall panels
- Grout infill in the wall joint
- Steel reinforcement design yield strength is f_{yk} = 500 N/mm^2, E_s = 200 kN/mm^2
- Concrete cover = 30 mm

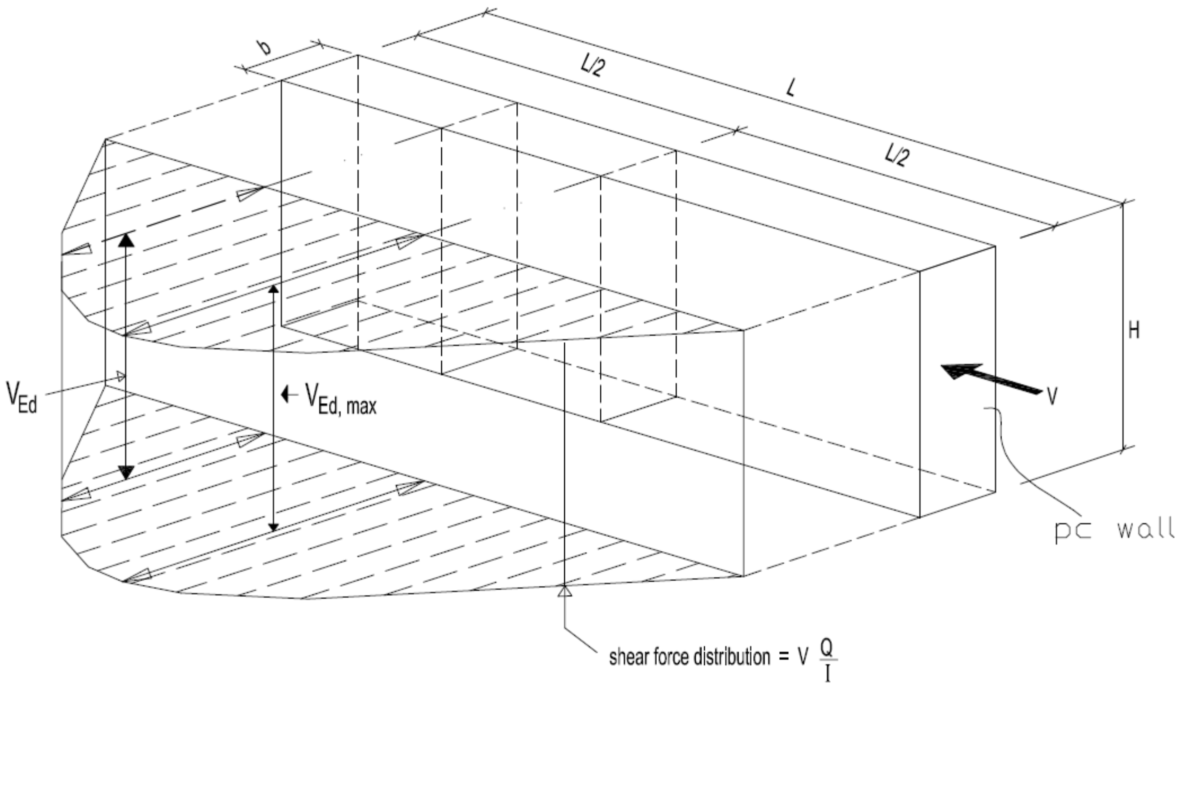






Design Shear Force in Vertical Wall Joint

Taking into reference the figure below, the design shear force at the wall joint may be calculated from the shear force distribution V_{Ed} = V Q/I



Design shear in vertical wall joint

V_{Ed} = V Q/I

where Q = first moment of area
I = second moment of area

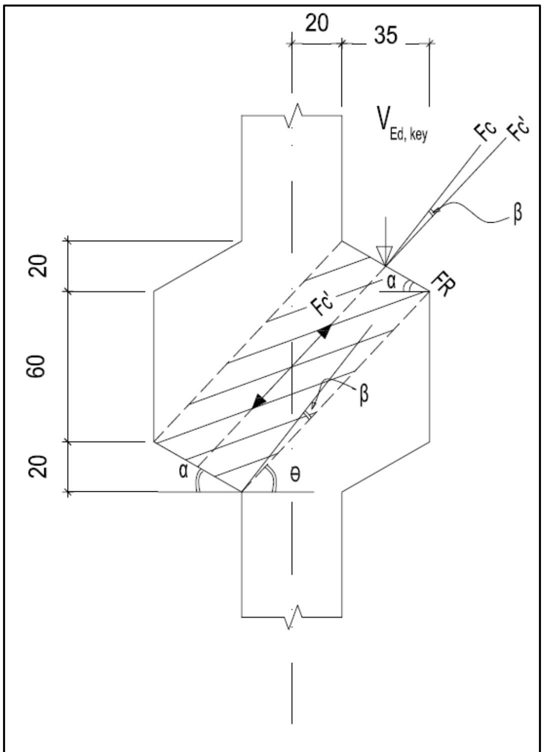
The distributed shear force V_{Ed} at the centre of a wall of b x L (width x length) is

V_{Ed} = V x (bL/2 x L/4)/(b x L^3/12)
= 1.5 V/L

By complementary shear, the interface shear force in the vertical wall joint is

V_{Ed} = 1.5 x 1200/6
= 300 kN/m

Checking the Shear Key



Forces in shear key

Check shear key length/depth ratio = 100/25
= 4 ≤ 6 **(OK)**

The shear key is designed as a 200 mm module and V_{Ed} in each shear key module is

$V_{Ed,key}$ = 300 x 0.2
= 60.0 kN/key

The forces acting in the shear key, as shown in the figure, can be obtained as below:

Angles α = $\tan^{-1}(20/35)$
= 29.74^0 ($\alpha \le 30^0$)

θ = $\tan^{-1}(80/75)$
= 46.85^0

β = $90^0 - (\alpha + \theta)$
= 13.40^0

Compression F_c normal to the slope face of the shear key

F_C = $V_{Ed,key} \cos \alpha$
= 60.0 x cos (29.74^0)
= 52.10 kN

Sliding force along the slope face of the shear key

$F_R = V_{Ed,key} \sin \alpha$
$= 60.0 \times \sin (29.74^0)$
$= 29.76$ kN

Friction factor (EC2, cl 6.2.5(2)) $\mu = 0.9$

Resistance against sliding $F_{R,Rd} = \mu F_C$
$= 0.9 \times 52.10$
$= 46.89$ kN $> F_R$

Checking the Compression Strut

Compression strut F_C' $F_C' = V (\cos \alpha / \cos \beta)$
$= 60 \times (\cos 29.74^0 / \cos 13.40^0)$
$= 53.55$ kN

Design strength of strut $\sigma_{Rd,max} = 0.6v'f_{cd}$
$= 0.6 (1 - f_{ck}/250) f_{cd}$
$= 0.6 \times (1-35/250) \times 0.567 \times 35$
$= 10.34$ N/mm^2

Required bearing area $A_{c,strut} = F_c'/\sigma_{Rd,max}$
$= 53.55 \times 10^3 / 10.34$
$= 5179$ mm^2

Available shear key area for bearing $= (130+135)/2 \times \sqrt{(35^2 + 20^2)}$
$= 5341$ mm^2 $>$ 5179 mm^2 **(OK)**

Tie Force

The horizontal push out force $F_H = F_C \cos\theta / \cos\beta$
$= 52.10 \times \cos 46.85^0 / \cos 13.4^0$
$= 36.63$ kN

Total push out horizontal force $= 36.63 \times H/0.2$
$= 36.63 \times 2.8/0.2$
$= 512.82$ kN

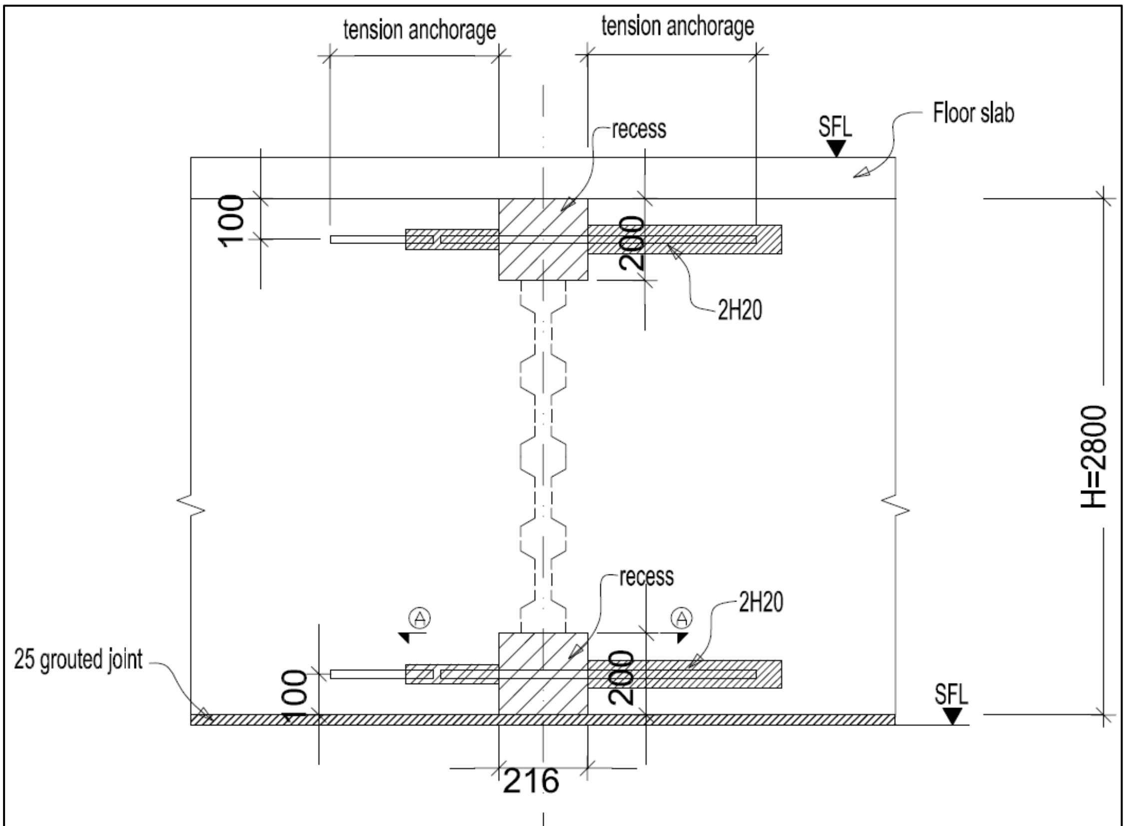
Total tie reinforcement required $A_s = 512.82 \times 10^3/(0.87 \times 500)$
$= 1179$ mm^2

Divide equally the ties in the topping concrete above the wall panel and at lower level above the floor.

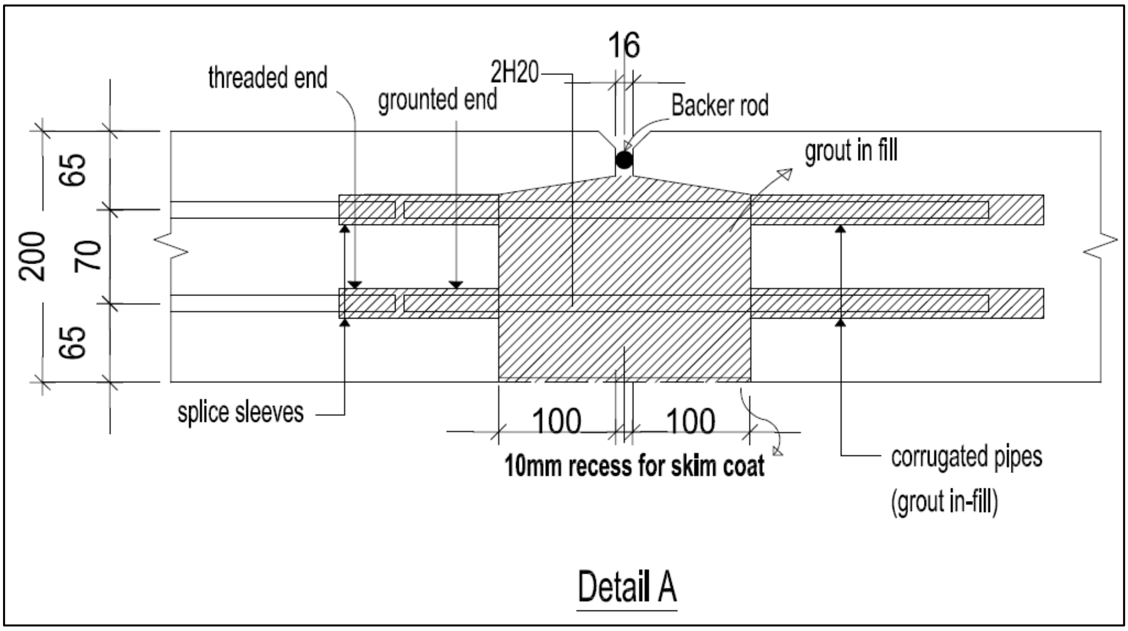
$A_s = 589$ mm^2 use 2H20 bars

Wall Details

The final wall details may be as shown in the figure. For the lower ties, other connection devices using column shoe bolting and welding may be adopted.



Typical tie bars



Note:

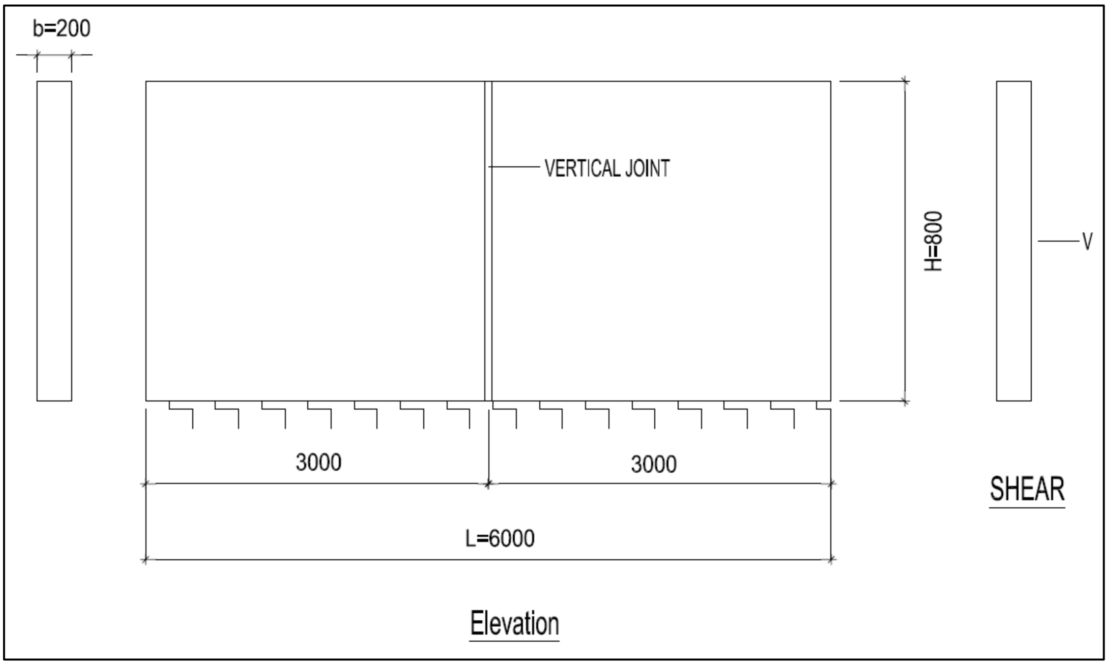
a) The H20 ties may use alternative proprietary products such as the column shoes with bolting. Other system may also be considered.

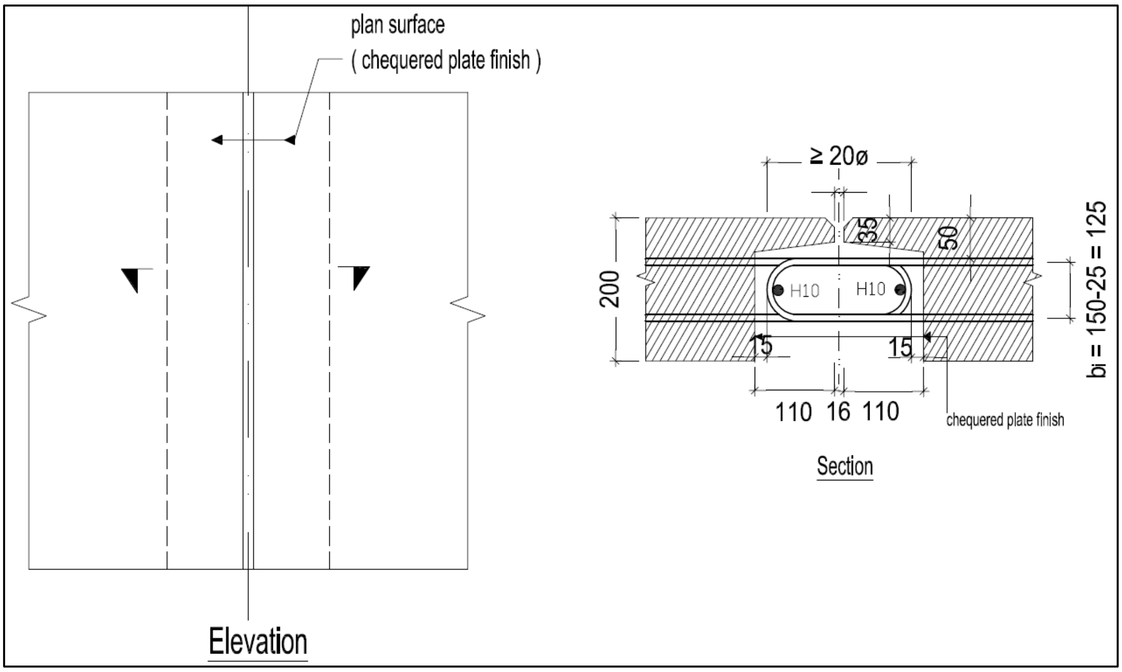
b) An alternative of the top H20 tie would be placing the 2H20 reinforcement bars directly across the wall vertical joint in the in-situ concrete topping.

DESIGN EXAMPLE 6 Vertical Wall to Wall Joint with Loop Reinforcement	Design Principles

A shear wall panel of height H = 2.8 m, length L = 6m and thickness b = 200 mm is split into two smaller wall segments of 3m length for ease of lifting.

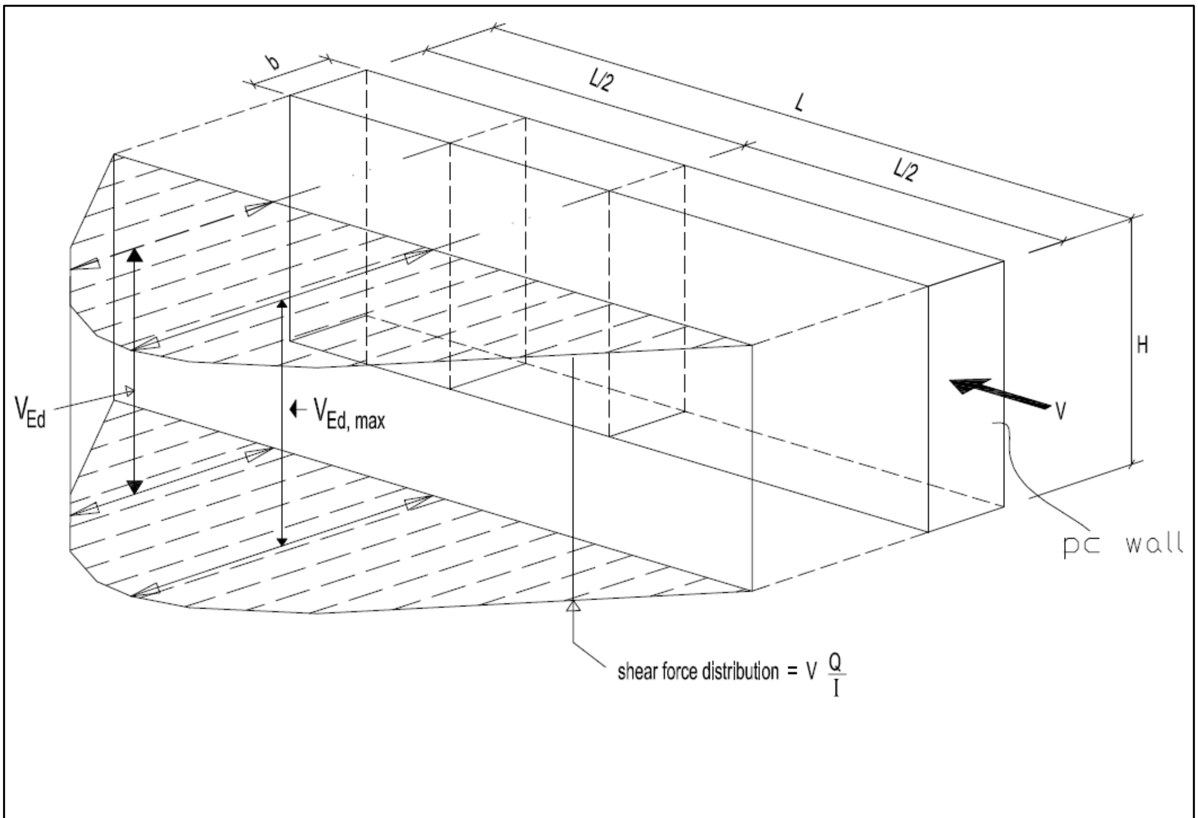
- Subjected to an ultimate in plane shear force V = 1200 kN as shown in the figure
- Interlacing loop reinforcement are proposed for the vertical wall joint connection as shown
- Design concrete strength class C35/45 for used in the wall panels
- Grout infill in the vertical wall joint
- Steel reinforcement design yield strength is f_{yk} = 500 N/mm^2
- Concrete cover = 30 mm





Design Shear Force in Vertical Wall Joint

Taking into reference to the figure below, the design shear force at the wall joint may be calculated from the shear force distribution $V_{Ed} = V\ Q/I$



Design shear in vertical wall joint

V_{Ed} = $V\ Q/I$
Q = first moment of area
I = second moment of area

The distributed shear force V_{Ed} at the centre of a wall of b x L (width x length) is

$$V_{Ed} = V \times (bL/2 \times L/4)/(b \times L^3/12) = 1.5\ V/L$$

By complementary shear, the interface shear force in the vertical wall joint is

$$V_{Ed} = 1.5 \times 1200/6 = 300\ \text{kN/m}$$

Design of Vertical Wall Joint

The capacity of the vertical wall joint is calculated as follows:

$$V_{Rd} \geq V_{Ed}$$

$$V_{Ed} / A_c = cf_{ctd} + \rho f_{yd}\mu$$

Wall joint surface is considered rough with c = 0.4, μ = 0.7 (clause 6.2.5(2))

$$f_{ctd} = 0.14\ f_{ck}^{2/3} = 0.14 \times 35^{2/3} = 1.50\ \text{N/mm}^2$$

Width of vertical wall joint is conservatively taken as b_i = 125 mm

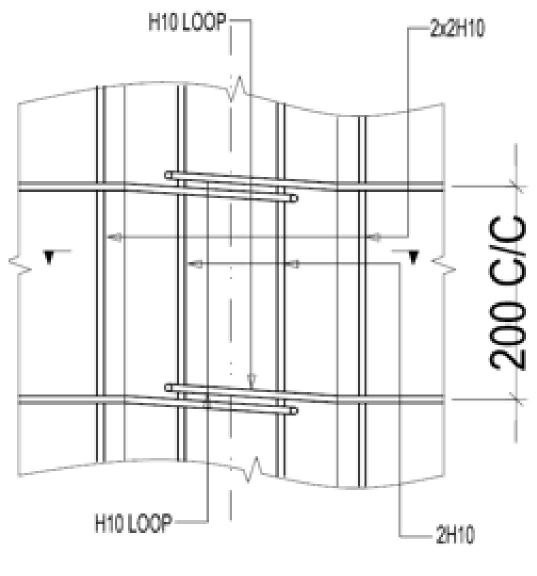
$$\rho = [\ 300 \times 10^3/(\ 125 \times 1000) - 0.4 \times 1.5)] / (0.7 \times 0.87 \times 500\ )$$

$$A_s = 0.00591 \times 125 \times 1000 = 739\ \text{mm}^2/\text{m}$$

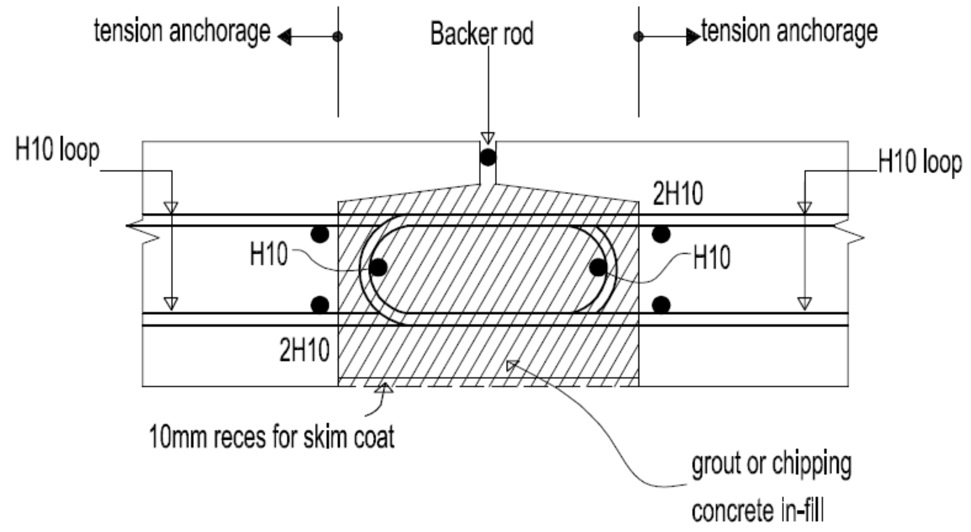
Use H10 loop reinforcement at 200 c/c (A_s = 78.5 x 2/0.200 = 785 mm_2/m > 739 mm_2/m)

Wall Details

The final wall details may be shown in the figure below. For the lower ties, other connection devices using column shoe bolting, welding and other devices capable to transmit direct tension forces may be adopted.



Elevation



Section

Note:

Alternative proprietary products such as the high strength wire loops may be used instead of the normal H10 loop reinforcement.

Designers may need to refer to specialist designs and consider suitability of the products for wall joints as the joints are susceptible to cracking.

DESIGN EXAMPLE 6 Vertical Wall to Wall Joint with Loop Reinforcement	Specialist Design Alternative

Vertical Wall Joint (Shear Keys) → Selecting Peikko® PVL and SOLO Connecting Loops

Connecting Loop resists shear forces with a “tension bar”, which consists of:

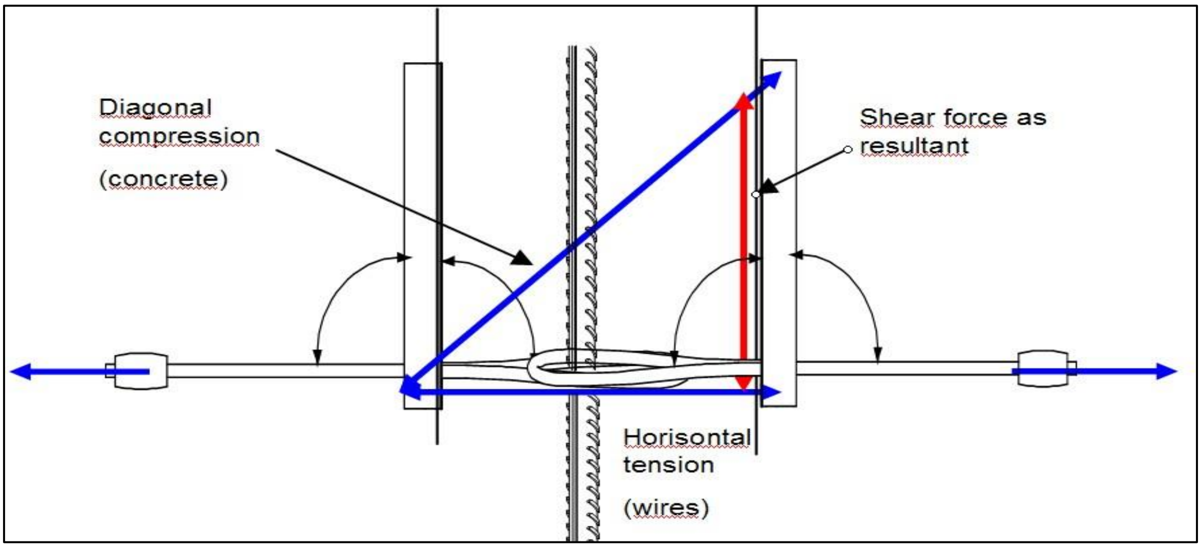
- Loops
- Vertical rebar in the joint
- ”A compression bar”

which is/are formed between the edges of the recess boxes from concrete.

Resistance of PVL Connecting Loop connections are defined according to loop spacing and compression strength of the concrete grout in the joint.

Resistance are calculated according to Eurocode 2 parts 1-1 and 1-2.

Connecting loop resists shear forces with a “tension bar”



Design:

- Shear force in connection: V_{Ed} = 300 kN/m

Select connecting loop:

- Connecting loop **SOLO 80, spacing 300mm**
- Resistance of the component **V_{Rd} = 311 kN**

$N_{Ed} < N_{Rd}$ → 300 kN < 311 kN

Resistances according to Eurocode 2

Concrete class (EC2)		C30/37 [kN]	C35/45 [kN]	C40/50 [kN]
Loop spacing [mm]	250	364	369	376
	300	311	316	323
	350	214	217	223
	400	191	195	200
	450	174	178	183
	500	160	164	169

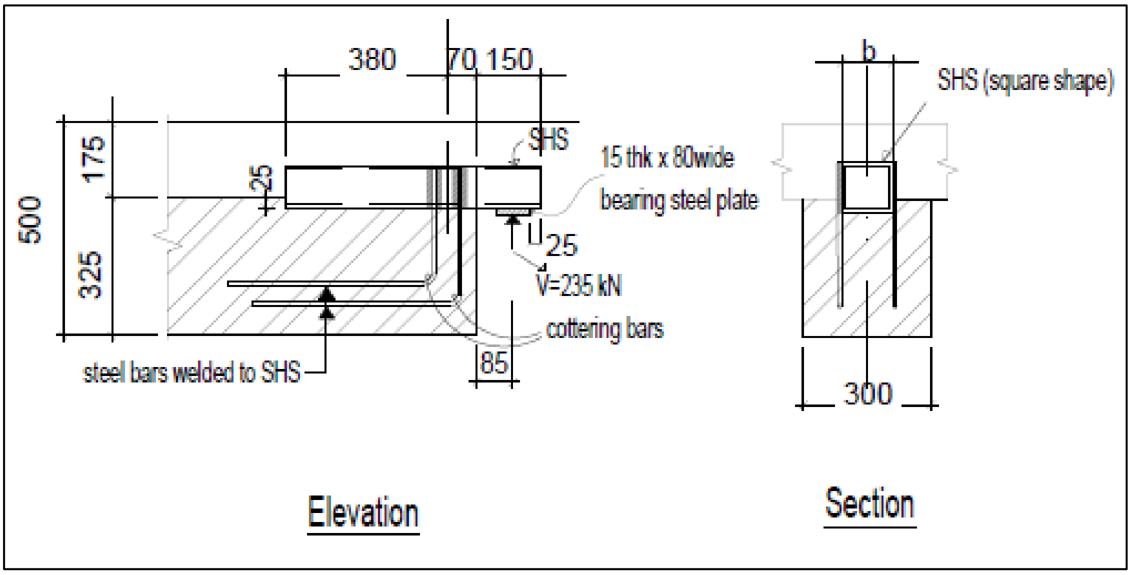
The following subjects must be verified from the technical manual of connecting loops:

a) Minimum required thickness of the wall
b) Positioning of the connecting loop in the wall
c) Required supplementary reinforcement
d) Other relevant instructions

DESIGN EXAMPLE 7 **Direct Steel Bearing Hanger**	**Design Principles**

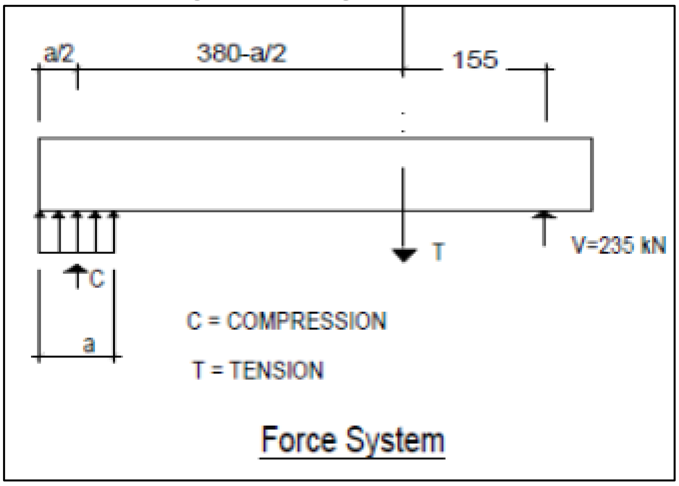
Square hollow section hangers are embedded at the top face at both ends of a semi precast beam. The hanger is subjected to:

- Ultimate reaction of 235 kN
- The semi precast beam is 300x325 (b x h)
- Form a composite beam of 300x500 deep with 175mm thick concrete topping
- Design concrete strength for both semi precast beam and concrete topping is C35/45
- Steel reinforcement design yield strength is f_{yk} = 500 N/mm^2
- Concrete cover = 35 mm to links
- All steel sections are to be graded as S355, f_y = 355 N/mm^2



Design of Steel Hollow Section

The forces acting on the hanger are idealised as shown in the figure below.



For the cantilever section, the bending moment at T

M_{Ed} $= 235 \times 0.155$
$= 45.0$ kNm

Plastic section required $S_{x,rqd}$ $= M_{Ed}/f_y$
$= 45 \times 10^3/355$
$= 127$ cm^3

$S_{x,prov}$ $= 146$ cm^3 > 127 cm^3 **(OK)**

Check shear resistance V_{Ed} $= 235$ kN

A_v $= Ah/(b+h)$ ($A = 35.2$ cm^2 ; b,h = 120 mm)
$= 35.2 \times 120/(120 + 120)$
$= 17.6$ cm^2

$V_{pl,Rd}$ $= A_v f_y / \sqrt{3}$
$= 17.6 \times 355 / \sqrt{3}$
$= 360.7$ kN > 235 kN **(OK)**

$V_{Ed} > 0.5\, V_{pl,Rd}$, apply reduction factor $\rho = (2V_{Ed}/V_{pl,Rd} - 1)^2$

ρ $= (2 \times 235/360.7 - 1)^2$
$= 0.092$

Reduced f_y $f_{y,\rho}$ $= (1-\rho)\, f_y$
$= (1-0.092) \times 355$
$= 322$ N/mm^2

$S_{x,rqd}$ $= 45 \times 10^3/322$
$= 140$ cm^3 $< S_{x,prov} = 146$ cm^3 SHS section **(OK)**

Design of Hanger Supporting System

Moment about T

$$C \times (380 - a/2) = V \times 155$$
$$C = a \times b \times f_{cd}$$
$$= a \times 120 \times 0.567 \times 35$$
$$= 2381.4\ a$$

$$2381.4 \times a \times (380 - a/2) = 235 \times 10^3 \times 155$$
$$a2 - 760a + 30591.2 = 0$$
$$a = 42.6\ mm$$
$$C = 0.567 \times 42.6 \times 120 \times 10^{-3}$$
$$= 101.4\ kN$$

$$T = V + C$$
$$= 336.4\ \ kN$$

Design of Ties

$$A_s = T/f_{yd}$$
$$= 336.4 \times 10^3/ (0.87 \times 500)$$
$$= 773\ \ mm^2$$

Use 4H16 welded to SHS

Checking the Welding

Weld strength of 6mm fillet weld $f_{vw,d} = f_u/(3^{0.5}\beta_w\gamma_{M2})$

$f_u = 355 \quad N/mm^2$

$\beta_w = 0.9$

$\gamma_{M2} = 1.25$

$f_{vw,d} = 182 \quad N/mm^2$

Weld thickness (round bar welded to plate)

$a_w = 0.2\phi$ ϕ = bar diameter

$= 0.2 \times 16$

$= 3.2$ mm

$F_{vw,d} = 2a_w \ f_{vw,d}$

$= 2 \times 3.2 \times 182 \times 10^{-3}$

$= 1.165 \quad kN/mm$

Length of weld per bar $l_w = F_s / F_{vw,d}$

$= 0.87 \times 500 \times 201 \times 10^{-3} / 1.165$

$= 75$ mm

Available weld length h =120 mm > 75 mm **(OK)**

Checking the Anchorage Length

Ultimate bond stress $f_{bd} = 2.25\eta_1\eta_2 f_{ctd}$ where η_1 , η_2 = 1.0

$f_{ctd} = 0.14 \ f_{ck}^{2/3}$

$= 0.14 \times 35^{2/3}$

$= 1.50 \quad N/mm^2$

$f_{bd} = 2.25 \times 1.0 \times 1.0 \times 1.50$

$= 3.37 \quad N/mm^2$

Basic anchorage length $l_{b,rqd} = \phi\sigma_{sd}/(4f_{bd})$

$= 16 \times 0.87 \times 500 / (4 \times 3.37)$

$= 516$ mm $\approx 32\phi$

Design anchorage length $l_b = \alpha_1\alpha_2\alpha_3\alpha_4\alpha_5 l_{b,rqd}A_{s,rqd}/A_{s,prov}$

$\alpha_1 = 1.0$

$\alpha_2 = 0.7$

α_3 , α_4 , α_5 = 1.0

$A_{s,rqd}/A_{s,prov} = 773/804$

$= 0.96$

$l_b = 1.0 \times 0.7 \times 1.0 \times 1.0 \times 1.0 \times 516 \times 0.96$

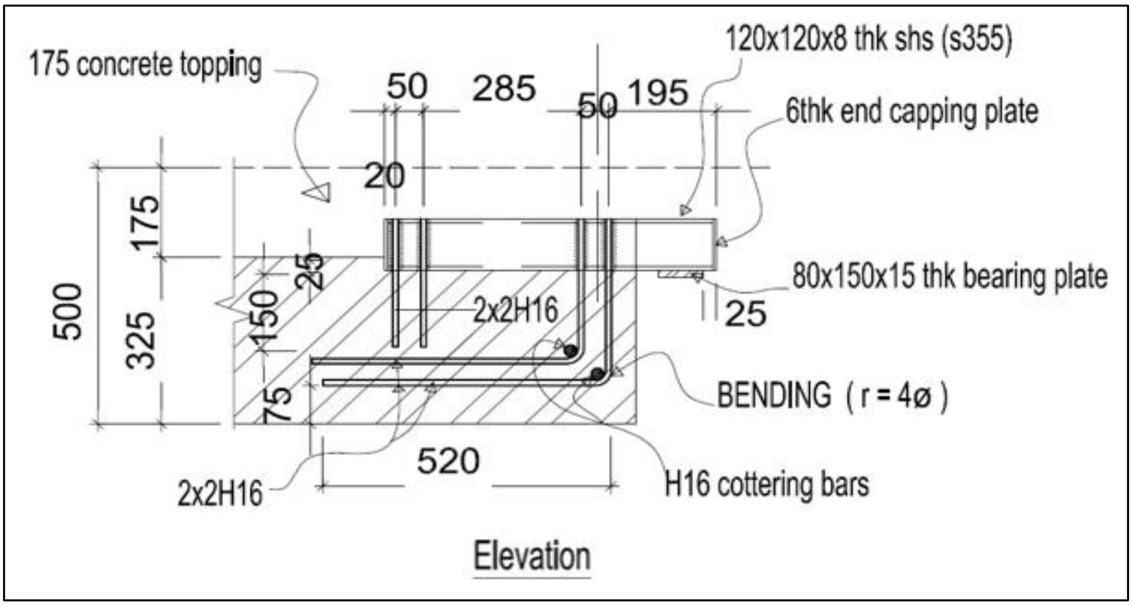
$= 347$ mm conservatively, adopt $l_{b,rqd}$

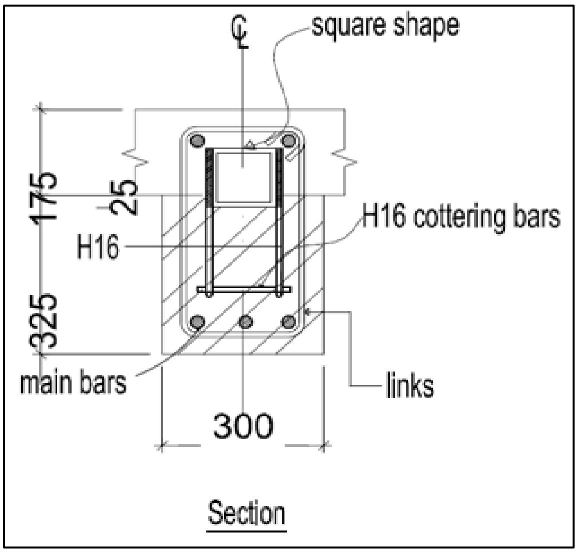
Design of Compression Bearing

The bearing provided by the concrete below the SHS may be weakened by having higher water cement ratio as water migrate upwards after concrete vibration. Conservatively, the compression bearing had been proposed to be replaced by welded reinforcement under the compression anchorage.

	A_s	$= C/f_{yd}$	
		$= 101.4 \times 10^3/(0.87 \times 500)$	
		$= 233$ mm^2	
		Use 4H16 welded to SHS	
Required	$l_{b,rqd}$	$= 32\phi \times A_{s,rqd}/A_{s,prov}$	
		$= 32 \times 16 \times 233/804$	
		$=148$ mm	Use 150 mm

Final Hanger Details

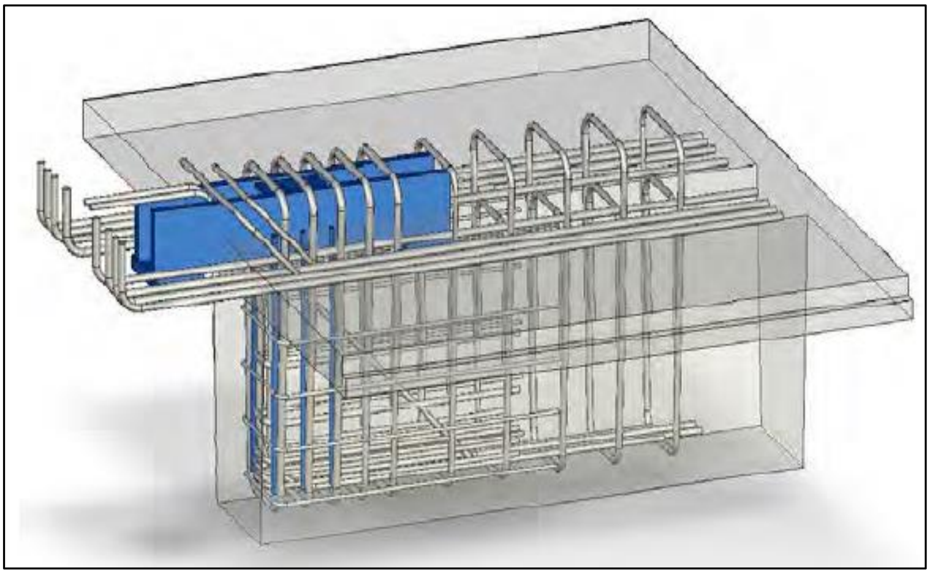




DESIGN EXAMPLE 7 **Direct Steel Bearing Hanger**	**Specialist Design Alternative**

Direct Steel Bearing Hanger → Selecting Peikko® PBH Corbel

Peikko® PBH Corbels are designed to withstand shear forces. Corbels are pre-designed so that all components of the system have sufficient resistance against actions caused by external loads.



These design values of resistances are shown in the table below:

Design:

Vertical force in connection: $V_{Ed,tot}$ = 235 kN (permanent load)

Select corbel:

- Corbel **PBH 1**
- Shear resistance **$V_{Rd,tot}$ = 235 kN**

$V_{Ed,tot} < V_{Rd,tot}$ → 235 kN < 235 kN

Vertical shear force in connection: $V_{Ed,erect}$ = X kN (Temporary erection shear forces, to be confirmed by the designer)

Select corbel:

- Corbel **PBH 1**
- Shear resistance **$V_{Rd,erect}$ = 80 kN**

$V_{Ed,erect} < V_{Rd,erect}$ → X kN (temporary erection shear forces, to be confirmed by the designer)

Based on the product catalogue, the selected connector’s shear capacity and web confinement reinforcement are as follows:

slab height h_{pl} [mm]	web height h_w [mm]	support reaction erecting state $V_{Rd,erect}$ [kN]	support reaction final state $V_{Rd,tot}$ [kN]	anchoring force Z_{Ed} [kN]	reinforcement Pos.1	Pos.1 l_b [m]	Pos.1 L_1 [m]	Pos.2	Pos.3	Pos.4	Pos.5	Pos.6	Pos.7	Pos.7a	Pos.8 [mm]	Pos.9	Pos.10
150	300	80	223	311	8Ø12	0,29	0,94	2Ø12	2Ø10	5Ø12	1Ø10	4Ø6	5Ø10	6Ø8	Ø10/210	1Ø6	-
	400	80	223	256	8Ø12	0,29	1,08	2Ø12	2Ø10	5Ø12	1Ø10	4Ø6	5Ø10	6Ø8	Ø10/210	1Ø6	-
	500	80	223	164	8Ø12	0,29	1,35	2Ø12	2Ø10	5Ø12	1Ø10	5Ø6	6Ø10	7Ø8	Ø10/210	1Ø6	2Ø10
	600	80	223	164	8Ø12	0,29	1,63	2Ø12	2Ø10	5Ø12	1Ø10	6Ø6	7Ø10	8Ø8	Ø10/210	1Ø6	2Ø10
	700	80	223	164	8Ø12	0,29	1,90	2Ø12	2Ø10	5Ø12	1Ø10	7Ø6	8Ø10	9Ø8	Ø10/210	1Ø6	3Ø10
	800	80	223	164	8Ø12	0,29	2,17	2Ø12	2Ø10	5Ø12	1Ø10	8Ø6	9Ø10	10Ø8	Ø10/210	1Ø6	3Ø10
160	300	80	235	302	8Ø12	0,28	0,88	2Ø12	2Ø10	5Ø12	1Ø10	4Ø6	5Ø10	6Ø8	Ø10/200	1Ø6	-
	400	80	235	246	8Ø12	0,28	1,01	2Ø12	2Ø10	5Ø12	1Ø10	4Ø6	5Ø10	6Ø8	Ø10/200	1Ø6	-
	500	80	238	157	8Ø12	0,29	1,26	2Ø12	2Ø10	5Ø12	1Ø10	5Ø6	6Ø10	7Ø8	Ø10/200	1Ø6	2Ø10
	600	80	238	157	8Ø12	0,29	1,50	2Ø12	2Ø10	5Ø12	1Ø10	6Ø6	7Ø10	8Ø8	Ø10/200	1Ø6	2Ø10

The following subjects must be verified from the technical manual of PBH Corbel:

a) Positioning of the corbel in the beam

b) Required supplementary reinforcement

c) Other relevant instructions

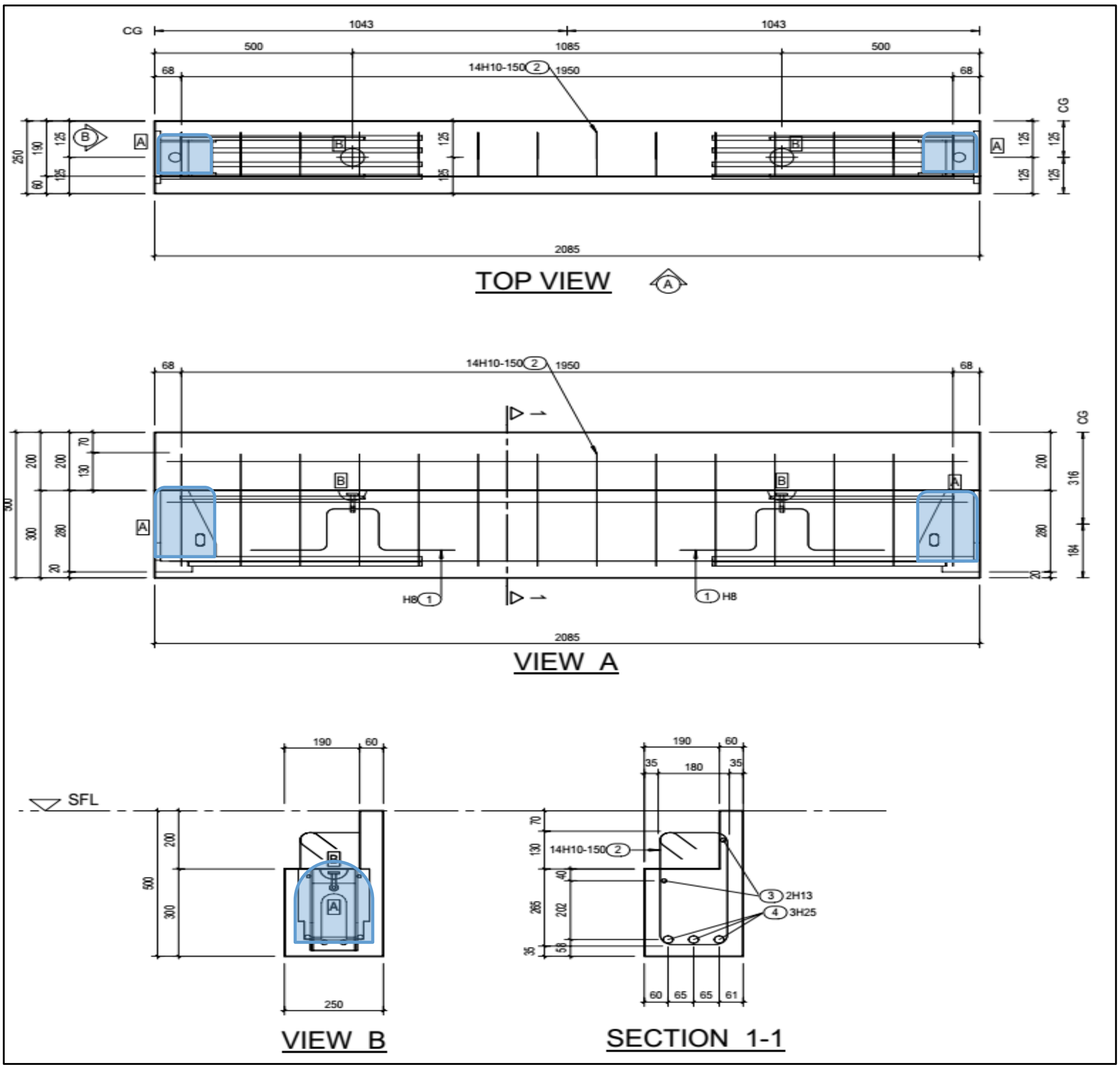
2.5 CONNECTION DETAILING EXAMPLES

When adopting proprietary mechanical connectors in PC design and manufacturing, the PC shop drawings should incorporate details as required by the specialists to ensure optimum performance of the connectors.

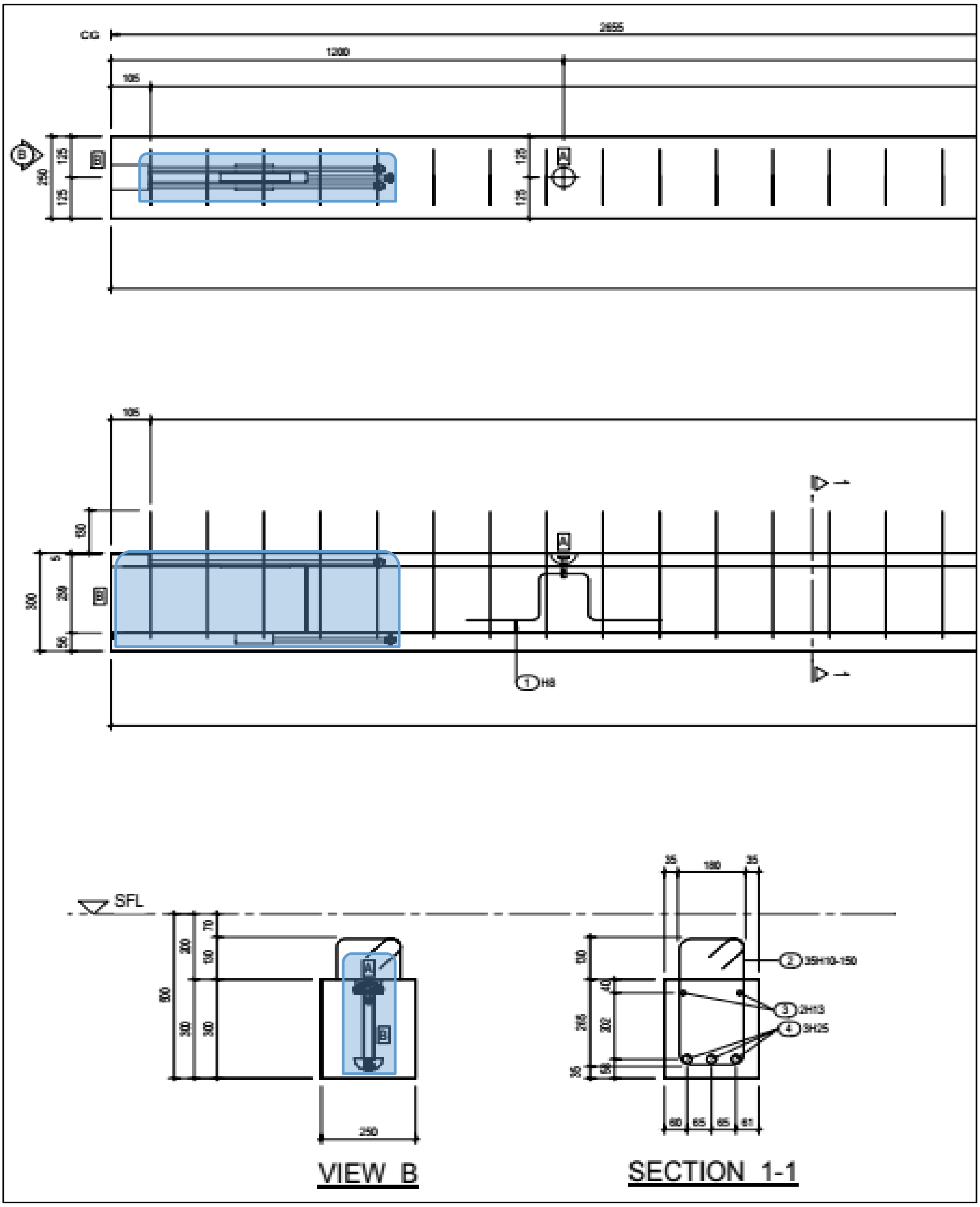
The detailing should also be verified by the connector specialists prior to manufacturing.

Example of a typical shop drawing which adopts mechanical connectors in PC is shown below.

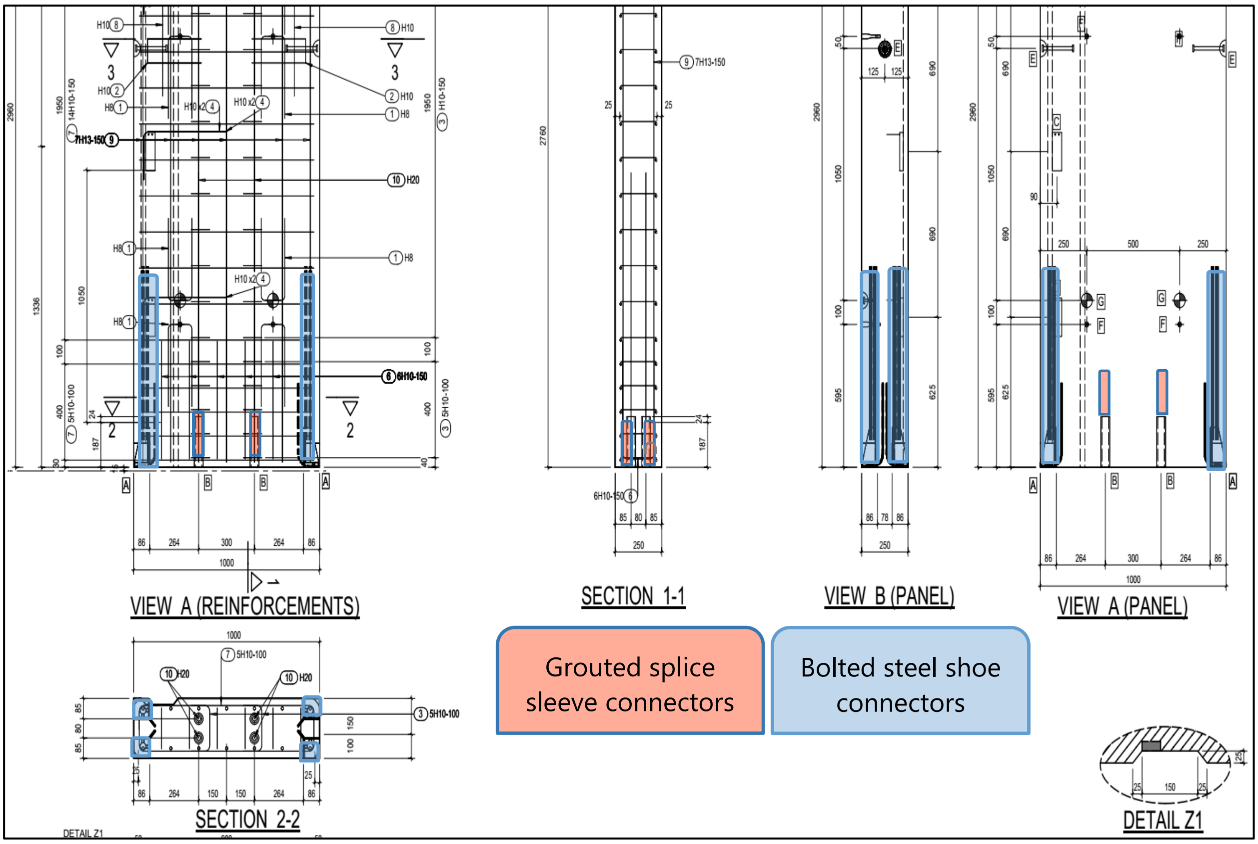
(a) Cast-in Steel Bracket Beam End Connector for Column with Bolted-on Corbel



(b) Cast-in Steel Telescopic Shear Plate Beam End Connector for Concealed Slot Support



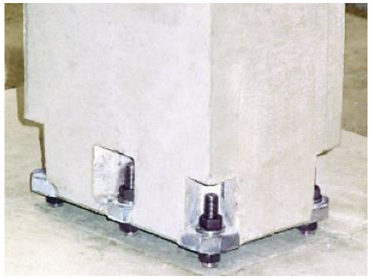
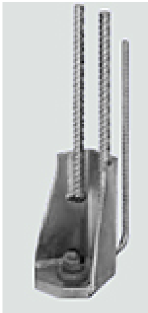
(c) Column Base with Cast-in Steel Shoe Bolted Connectors and Grouted Sleeves

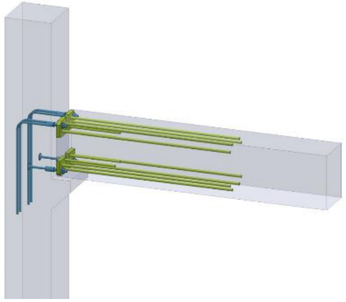


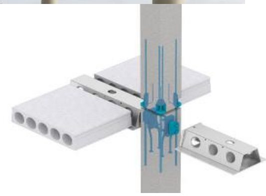
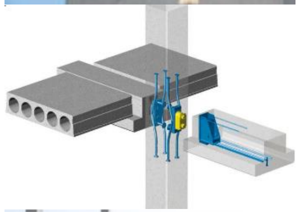
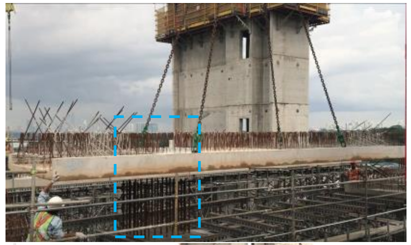
2.6 EXAMPLES OF PROPRIETARY MECHANICAL CONNECTION SYSTEMS

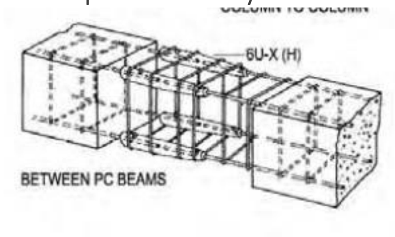
The following table provides examples of proprietary products and their applications, as provided by the various suppliers.

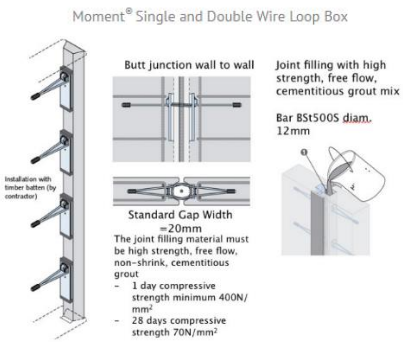
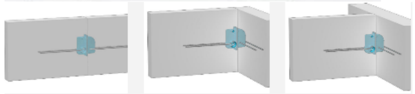
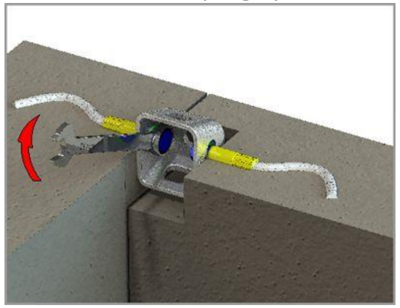
Note: Details provided are only strictly for illustration purpose without implicit product endorsement. The examples shown here are not exhaustive.

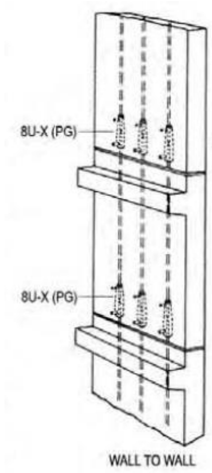
Connection System	Halfen-Moment	Jordahl & Pfeifer	Peikko	Others
1. Column to Base ***2. Column to Column***	Column Shoe  	Column Shoe 	Column Shoe 	NMB NMB Splice Sleeve System JM Grout sleeve (half grout and full grout) Spiral Connector

Connection System	Halfen-Moment	Jordahl & Pfeifer	Peikko	Others
3. Beam to Column – Moment Connection	Column Shoe	-	Beco Beam Shoes • Used in combination of MODIX® rebar coupler (for half-precast beam) and COPRA anchor bolts (for full precast beam). • The connections must be supported by corbel.  	NMB NMB Splice Sleeve System JM Grout sleeve (half grout and full grout)

Connection System	Halfen-Moment	Jordahl & Pfeifer	Peikko	Others
4. Beam to Column – Simply support only (i.e. no Moment Connection)	<u>Invisible Connection</u> • Using BSF telescopic beam connectors – shear "knife"   	-	Delta Beam   <u>PCs® Corbel</u> • Using 'bolted-on' corbel block • Pin connection   	<u>Hidden Connection System</u>    

Connection System	Halfen-Moment	Jordahl & Pfeifer	Peikko	Others
5. Beam to Beam	Rebar Coupler Moment® Coupler (Half & Full)  	Rebar Coupler PH Reinforcement Continuity System 	Rebar Coupler MODIX®  	NMB NMB Splice Sleeve System  ARJ Rebar coupler

Connection System	Halfen-Moment	Jordahl & Pfeifer	Peikko	Others
*6. **Façade Wall / Wall to Wall (Non-bearing) / Wall to Column [Vertical Connection]***	**Loop Box (with grouting)**   	**VS® Loop**  	**<u>PVL Connecting Loop</u>**  **<u>Tenloc Panel Connector</u>**   	<u>JM</u> Flexible loop  <u>BT-SPANNSCHLOSS</u> Turnbuckle clamping system 

Connection System	Halfen-Moment	Jordahl & Pfeifer	Peikko	Others
7. Wall to Wall [Horizontal connection]	-	Wall Shoe 	Wall Shoe 	<u>NMB</u> NMB Splice Sleeve System  <u>ASL</u> Wall shoe <u>Spiral Connector</u>
8. Precast Landing to Core Wall – Simply support only (i.e. no Movement Connection)	<u>Invisible Connection</u> Using TSS telescopic connectors for precast landing to core wall	-	-	-

CHAPTER 3

UNDERSTANDING DETAILING OF PRECAST CONCRETE COMPONENTS FOR AUTOMATED PRECAST PLANT

3.1 GENERAL

3.1.1 The manufacturing of PC components has evolved tremendously with the advent automation and robotic technology. Such highly mechanised and automated manufacturing process has started to be adopted locally in Integrated Construction Prefabrication Hubs (ICPHs) which aims to achieve higher productivity by reducing and replacing manual interventions with advanced technologies.

3.1.2 ICPHs are multi-storey advanced manufacturing facilities that produce prefabricated and PC components, with a high degree of mechanisation and/or automation. Besides circulating pallet production systems, ICPHs also include highly integrated manufacturing processes using serialised work stations that deploy various in-built robotic technologies. Such a concept is similar to an automated car assembly plant, where vehicle components are added in stages to the semi-finished product through a series of work stations along a moving production line.

3.1.3 The operational viability of ICPHs depend on design simplicity and standardisation, in both PC component profiles and edge connection details. Currently, manually interventions at some work stations are still required due to complex wet cast connection design which cannot be efficiently produced via the automated manufacturing process. The possible disruptions of these manual intervention works to the automated manufacturing process are illustrated in ***Figure 3.1.***

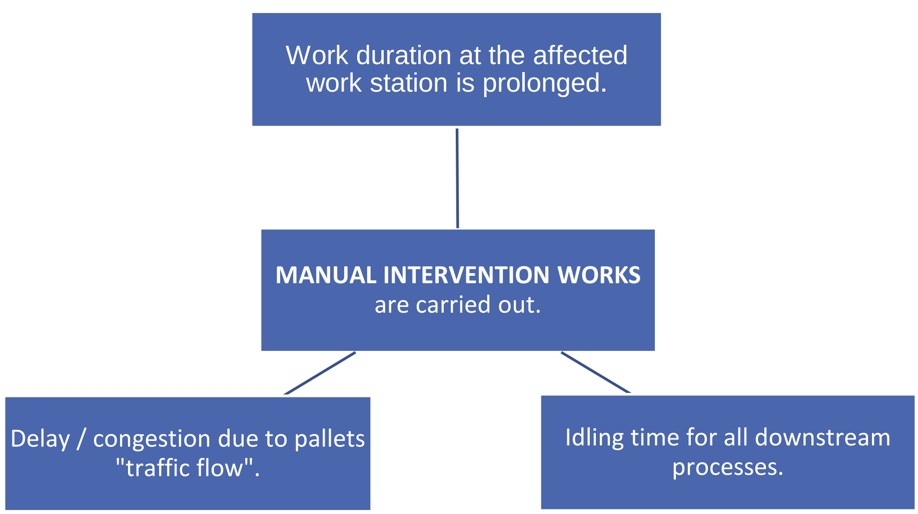
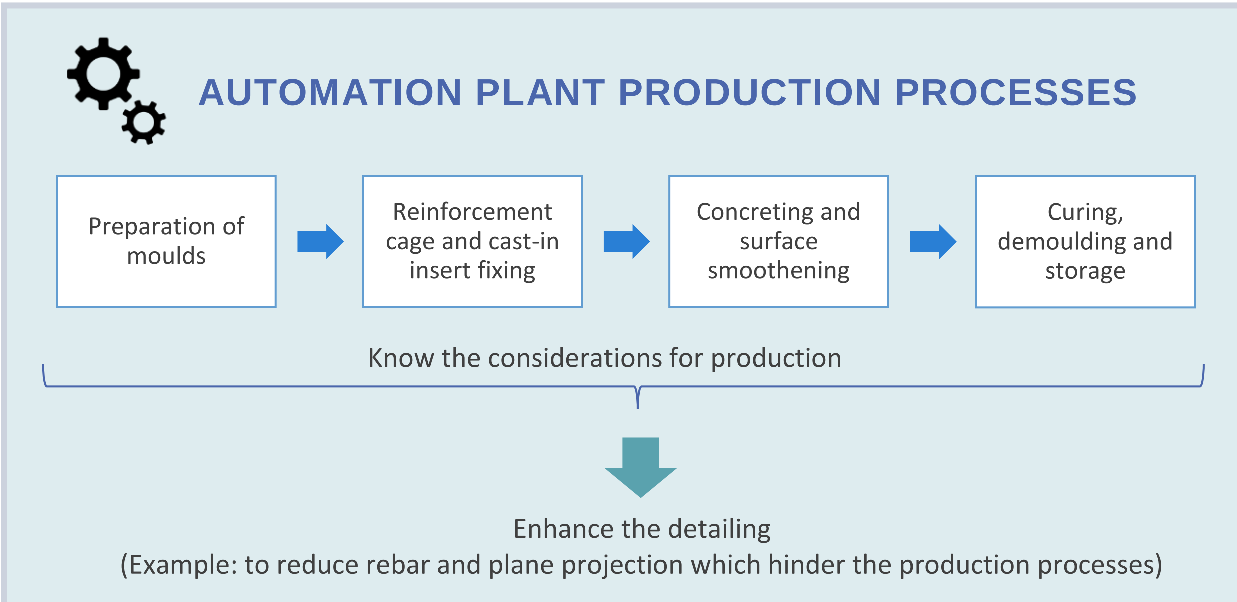


Figure 3.1: Possible disruption encountered in automated production

3.1.4 As such, the current PC connection designs need to be re-examined to enable optimal use of automation in the manufacturing process. Designers, who have sufficient understanding of the operating constraints in an automated manufacturing process, will be able to develop suitable PC connection design and details to reap maximum gains from the adoption of automation.



3.2 ASSEMBLY OF PRECAST MOULD

3.2.1 Advanced Work Method

Type of Moulds	
Steel Mould	Custom-made Mould
a) The most common mould type for mass-produced PC components b) Often the biggest cost component in precast manufacturing, especially when there is insufficient repetition in the design	a) Provides more flexibility to suit unconventional or irregular designs

3.2.1.1 Automated production adopts an assembly of standard mould modules like steel mould instead of custom-made mould. It has been proven to be the optimal choice in terms of cost-effectiveness and improve work efficiency.

Automated Pallet Circulation System for Mould

3.2.1.2 Standardised reusable mould modules and accessories are used to form the PC components' overall profile. They are handled and installed accurately by robots onto a flat table mould before being shifted to other work stations.

3.2.1.3 Unlike conventional manual mould assembly which is heavily reliant on manpower to fix and handle mould parts. For example, the correct dimension needs to first be adjusted before the complete setting of the mould. Although a much more complex design can be achieved via manual assembly, it is undesirable due to poor efficiency and lower productivity.

3.2.2 Understanding Advanced Work Method

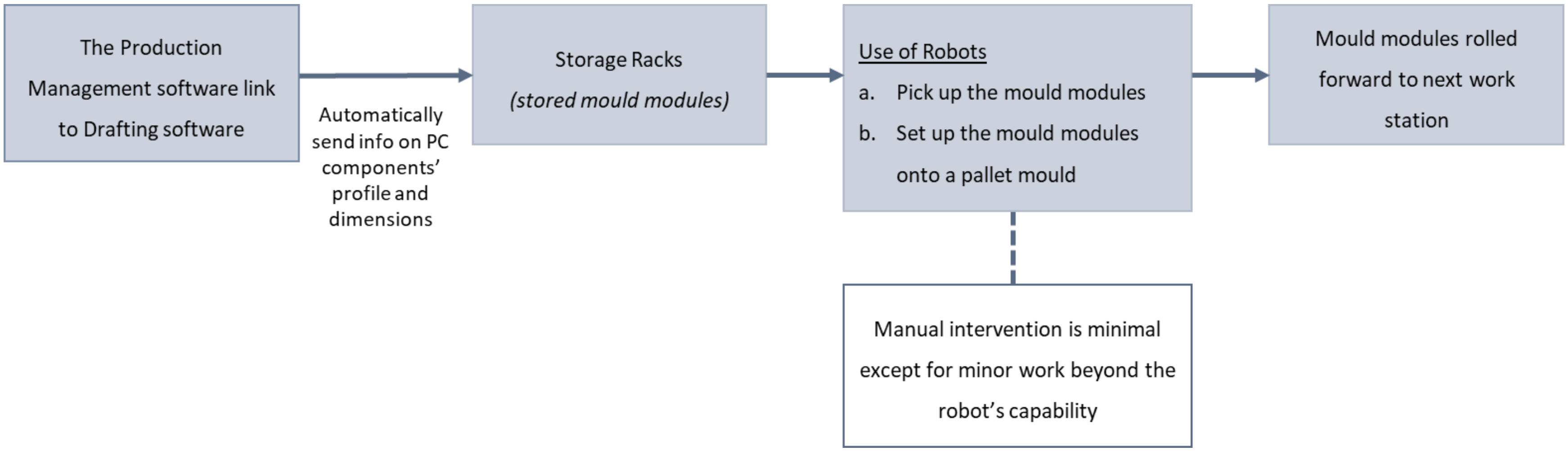


Figure 3.2: **The automated production process from the production management software to the mould setting-up.**

a) The production management software can be linked to the drafting software to read the PC components profile and dimensions automatically without manual input.

b) The correct modular moulds will then be selected from storage racks by using a robot to help set up the necessary components onto a pallet mould according to the production programme.

Manual intervention is kept minimal except for minor works which are beyond the robot's capability. The pallet, with completed mould settings on it, is then rolled forward to the next work station.

LIMITATIONS OF THE AUTOMATED PALLET CIRCULATION SYSTEM: MOULD

a) Unable to accommodate protruding starter rebars that extend through/beyond the edge moulds.

b) To avoid damaging the standard-edge moulds as a result of cutting holes to accommodate starter rebars, the use of threaded coupler inserts for adding starter rebars after casting is the only ideal solution. This will increase the manufacturing cost.

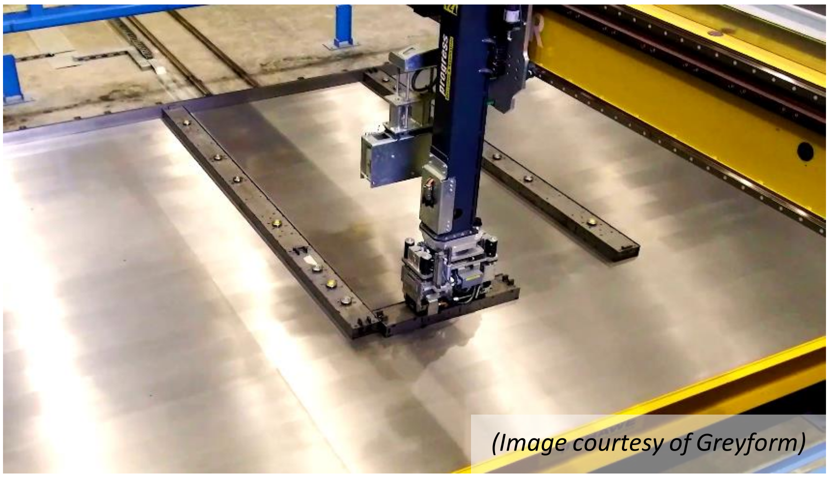
c) Current stirrup link details for lateral edge connections are not recommended as additional box-out moulds have to be fabricated and manually added.

3.2.3 Illustration of Advanced Mould Work

An advanced automated pallet table mould circulating system in an ICPH is shown in the photographs below.


(Image courtesy of Greyform)

Figure 3.3: **Magnetic side-forms are assembled onto the pallet table. The PC component profile information was extracted from shop drawings directly without the need for manual inputs.**



(Image courtesy of Greyform)

Figure 3.4 (left): **The computerised robot automatically selects and places standard side forms onto the pallet table, which are then secured with magnetic holders.**

Comparing to the advanced mould work, a conventional labour intensive set-up as in open precast yard shown in *Figure 3.5 (bottom left)* and *Figure 3.6 (bottom right).*


(Image courtesy of Advan-TIS)


(Image courtesy of Advan-TIS)

3.2.4 Other Mould Methods

3.2.4.1 Precasters should be consulted on the limitations and suitable out-of-plane projection dimensions of their automated manufacturing when casting 2-D PC components which have non-flat surfaces.

The use of rubber mould linings to create minor architectural protrusion features on façade wall surfaces remains possible without negative implications to the automated manufacturing operation.

For PC components with 3-D surface projection, such as fins and copings, a custom-made steel base mould used in conventional manual PC production will be added on top of the pallet mould. However, this method is significantly costlier and less productive as it restricts the degree of automation in the manufacturing process.

3.2.4.2 Designers should ensure that the majority of PC components are designed such that they can be produced via automated manufacturing processes and so as to avoid using conventional custom-made moulds. Nevertheless, PC components which require unique designs can still be accommodated in small proportions without increasing production the overall cost significantly.

3.3 REINFORCEMENT CAGE AND CAST-IN INSERT FIXING

3.3.1 Advanced Work Method

3.3.1.1 Reinforcement cages are fabricated by a separate automated weld mesh manufacturing facility adjacent to the main production areas.

3.3.1.2 The weld mesh can be fabricated just-in-time. Fixing of additional rebars may be required for much more complex design occasionally. In such cases, the automated weld mesh machine will pre-assembled in the cage modules in advanced. To reduce on-line production time, these weld meshes are pre-assembled into large sub-components off-line by the machine before being transferred and placed into mobile pallet table moulds.

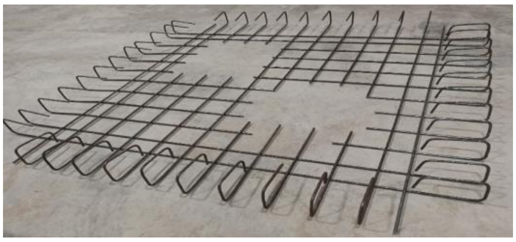
3.3.1.3 The completed reinforcement cages are then brought to the designated work stations to be installed into the moulds, or temporarily stored for later use.

3.3.1.4 Other small M&E cast-in items and lifting anchors etc. can also be pre-installed in the reinforcement cages before they are placed into the moulds.

Figure 3.7 (right): **Just-in-time pre-assembled reinforcement cages in temporary storage.**

Figure 3.8 (bottom): **Automated weld mesh fabrication facility with reinforcement details are extracted from shop drawing.**

(All images above courtesy of Greyform)



Rebar mesh *(Figure 3.9-above)* automatically placed onto the table mould *(Figure 3.10-bottom)* to produce PC panels with simple reinforcement details. High efficiency is achieved as a result.

Figure 3.11 (right): **Automated plant fabricating rebar mesh**

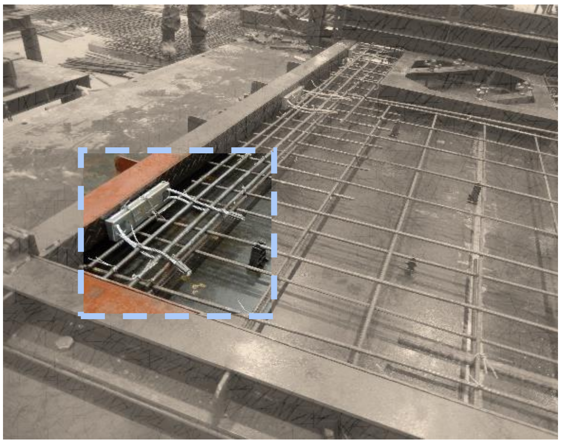
(All images above courtesy of Greyform)

3.3.2 Preparatory In-Process Work Details

3.3.2.1 Upon placing the reinforcement cage into the mould settings via an overhead crane, other cast-in accessories *(such as inserts, anchors, splice/steel connectors or electrical items which are pre-installed in the cage)*, will be secured into their final positions.

3.3.2.2 Upon completion, the pallet table is then rolled to the casting station for final quality check before concreting work commences.

3.3.2.3 This is unlike the conventional manufacturing which uses stationary moulds and workers for on-the- spot fixing works.



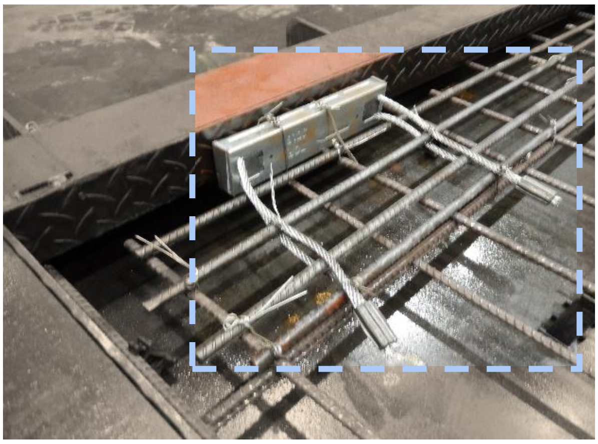
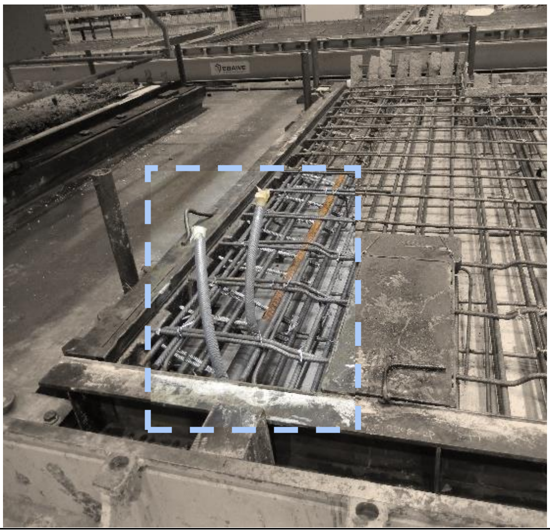


Figure 3.12 (left) and 3.13 (right): **Simple cast-in loop box for lateral edge connection as compared to the complex stirrup link details along panel edges. *(Images courtesy of Greyform)***



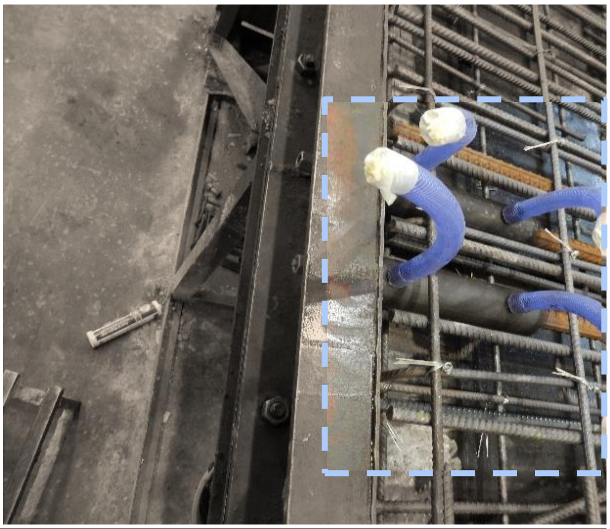


Figure 3.14 (left) and 3.15 (right): **Cast-in mechanical connector with half-grouted sleeves as compared to the long corrugated pipe sleeves. This can avoid rebar congestion at the PC connection area. *(Images courtesy of Greyform)***

(All images above courtesy of Greyform)

3.4 CONCRETING AND SURFACE SMOOTHENING

3.4.1 Automation in Work Process

3.4.1.1 The last two work stations in the automated pallet circulating system are focused on concreting and smoothening process. Both processes can be fully automated by using machinery with production software control.

3.4.1.2 Just-in-time batched concrete is delivered by auto-shuttle to the distribution hopper at the casting station with the right concrete quality and quantity. The concrete is then discharged and evenly distributed into individual moulds. (Refer to ***Figure 3.16***)

3.4.1.3 The entire pallet table is then mechanically vibrated for compaction purpose. Given the fully automated set-up, there is no need for workers to be present during this stage with the only exception during casting of out-of-plane upstand projections of PC components. (Refer to ***Figure 3.17***)

3.4.1.4 After initial concrete setting, the top concrete surface is troweled to a smooth finish by a suspended rotary machine or horizontal vibrating beam which can be operated automatically or manually.

3.4.1.5 Manual finishing is required for PC components with out-of-plane upstand projections or protruding rebars.

3.4.1.6 The finishing or troweling process can be omitted with the use of self-compacting and levelling concrete.

Figure 3.16 (left): **Automated concrete casting with the use of controlled discharge hopper to accurately and evenly distribute concrete into moulds on the pallet table.**

Figure 3.17 (bottom): **Vibrating table for concrete compaction**

(All images above courtesy of Greyform)

LIMITATIONS OF AUTOMATED PALLET CIRCULATION SYSTEM: CONCRETING AND SURFACE SMOOTHENING

Completely flat top surface without any upstand projection of profile or starter rebars.

Examples of PC components which do not facilitate the use of full automation in production are illustrated below.

Figure 3.18 (bottom) and Figure 3.19 (right): **Manual work is required for long or numerous upstand vertical starter rebars as it hinders the top surface finishing work.**

Figure 3.20 (left) and Figure 3.21 (bottom): **Non-flat and 3-D panels with/without starter rebars generally cannot be produced via the automated pallet production system and have to be manufactured manually.**

(All images above courtesy of Greyform)

3.5 CURING, DEMOULDING AND STORAGE

3.5.1 <u>Advanced Work Process</u>

3.5.1.1 Circulating pallet table production lines are equipped with curing chamber towers which provide an area to stack up and store pallets from the casting and smoothening stations.

3.5.1.2 Multi-level steel rack drawers of mostly pre-determined height (example about 600mm each) are used for the slotting in of the pallets for concrete curing at controlled temperatures and durations.

3.5.1.3 Stacking and withdrawal of pallets are mostly done with manual remote control.

3.5.1.4 Once the concrete has reached demoulding strength, the pallets will be transferred to the demoulding station.

3.5.1.5 The demoulding station is usually manned by workers for the manual removal of steel moulds and recycling of production accessories. In some factories, a tilting frame is used to upright wall panels together with the pallet for quick off-lifting and to avoid high concrete handling stresses.

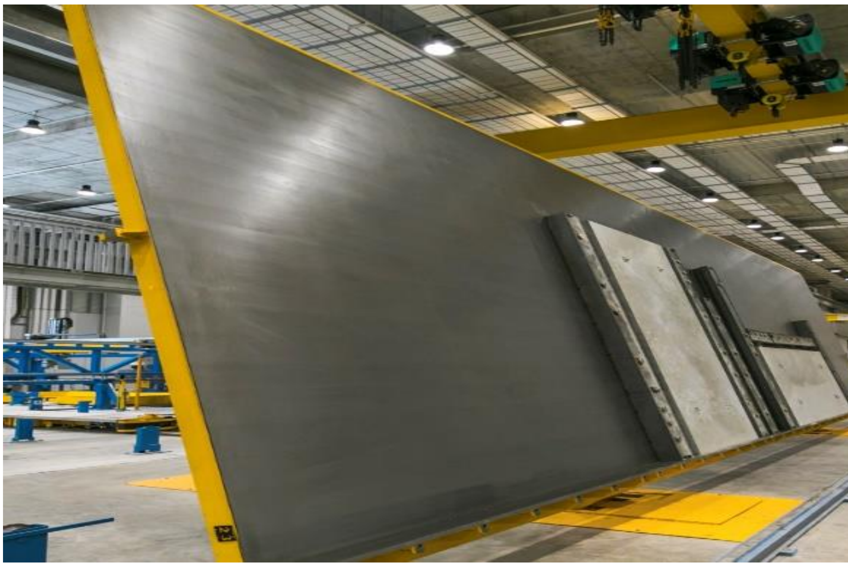
3.5.1.6 Other flat elements such as PC slabs which are to be stored horizontally do not require the tilting frame. A quality check is then performed after the PC components have been lifted off the pallet, before the finished components are transferred to the stock yard.

3.5.1.7 The cleared pallets are then channelled to the starting point of the production line for cleaning and oiling before the next production cycle commences.

***Figure 3.22 (left)*: Freshly cast panels being transported into the curing chamber**
***Figure 3.23 (right):* Enclosed multi-tier concrete curing chamber, with or without heating. It temporarily stores pallet tables with freshly cast panels for 6 to 12 hours curing.**

(All images above courtesy of Greyform)

Figure 3.24 (right): **After the concrete has achieved the desired demoulding strength, the pallet tables are extracted from the curing chamber and transferred to the tilting station to be demoulded.** ***(Image courtesy of Greyform)***



Figures 3.25 (bottom left) and 3.26 (bottom right): **The demoulded components are then lifted onto the temporary platform storage before being moved to the storage area.** ***(Images courtesy of Greyform)***

(All images above courtesy of Greyform)

LIMITATIONS OF AUTOMATED PALLET CIRCULATION SYSTEM: CURING CHAMBER

a) Drawer height restriction for PC components with upstand projection of profile or starter rebars.
b) Typically catered for flat PC components to maximise its capacity.
c) Components which cannot be fitted into the curing chamber will be sorted or re-directed to other curing areas (e.g. conventional open-yard air curing method).

3.5.2 Storage and Stock Management

3.5.2.1 Conventional open yard is equipped with overhead gantry cranes. Depending on the shapes and sizes, PC components are either stacked horizontally or vertically in supporting racks. Stock control and tracking using bar-code or RFI systems are widely practised.

3.5.2.2 Use of multi-level storage facilities is one solution to accommodate larger output capacity.

3.5.2.3 Automation in stock yard operations can be achieved with the use of information technology that focus on product tracking, quick and smooth management of products storage, as well as delivery with minimal manpower.

3.5.2.4 In a multi-tier storage system, finished components are required to be stored in mobile storage and delivery racks in a pre-sorted manner according to site requirements.

3.5.2.5 The loaded racks are then placed automatically into the designated multi-tier storage cells before delivery.

3.5.2.6 Pre-stacking and pre-sorting PC products into bundle storage is a productive feature which automates and streamlines the final delivery process.

3.5.2.7 The automated storage system enables fast storage and retrieval process. The waiting time of delivery trucks is significantly reduced with a bundled product loading process. This not only improves the round-trip delivery time, but also reduces the transportation resources required.

LIMITATIONS OF AUTOMATED PALLET CIRCULATION SYSTEM: STORAGE

a) Expensive storage method. This can be a costly and inefficient hardware investment if storage is not optimised due to ill-fitting PC components.
b) To consider the automated storage rack size limitation in PC components design.

3.5.3 Illustration of Automated Storage System

Figures 3.27 (left) and 3.28 (right): **Automated multi-storey storage cells for PC components.**

Figures 3.29 (left): **Standard metal rack for bundle handling and delivery;** ***Figure 3.30 (right):*** **Racks can be reused after delivery.**

Figures 3.31 (left) and 3.32 (right): **PC components are pre-sorted inside the factory after production and loaded into the modular standard storage or delivery rack**

(All images above courtesy of Greyform)

3.6 FULL PROCESS AUTOMATION

3.6.1 Integration of Production Process with Design and Other Management Software

3.6.1.1 Integrated design and production control software for automation in PC manufacturing has been available since the late 1990s. However, due to the extremely high upfront cost at that time, this method was not adopted in Singapore.

3.6.1.2 Advanced automated equipment with integrated hardware and software systems enables more efficient control during the manufacturing process with reduced labour input. Such equipment is widely adopted in new multi-storey PC manufacturing facilities. More of such new generation ICPHs with intensified land use will be developed in the coming years.

3.6.1.3 The total process of precast manufacturing business can be highly automated and fully integrated from design, shop drawings, to final production operation and product delivery to construction sites.

3.6.1.4 The investment costs of such plants have become relatively affordable with the advancement in computer hardware and information technology. Such transformation is also expected to help create new competitiveness and to increase the overall work productivity.

3.6.1.5 Designers is a critical human factor as they spearhead PC designs friendly to automated PC manufacturing facilities.

3.6.2 Capability of Integrated Design and Production Management Software

3.6.2.1 Proprietary software which are initially intended for producing shop drawings of PC components can now be linked with the automated PC production systems. These information can be extracted for production robots to execute their works. They can also be integrated with the management software for material planning and scheduling by the factory planning and purchasing departments.

3.6.2.2 Relevant design data which are fed into the production control system can be integrated with various work processes to accurately fabricate the PC components in stages with the aid of robots. Quality of final PC components are much more consistent due to lower risk of human error.

3.6.2.3 Greater standardisation and automation-friendly features in the design are the key success factors in such a system. This is done in order to optimise output efficiency with minimal manual interfacing activities.

3.6.2.4 Compilation of shop drawing design data allows the automation of production material planning, work scheduling and cost budgeting through suitable information technology.

The management software can also be used to optimise interfacing planning activities, produce production output records, and generate management reports on work progress, including production budget status.

3.6.2.5 The application of integrated information technology from design, production, management and accounting software can bring the following benefits:

a) Expedite generation of initial budgetary information
b) Automate operation management processes
c) Resolve work conflicts
d) Monitor real time work progress
e) Alert management staff for timely decision in project management

3.6.3 Integration with Full Construction Process Management Software

3.6.3.1 The use of Building Information Modeling (BIM) in the local construction industry to enable Integrated Digital Delivery (IDD) has fundamentally changed the way design, component manufacturing and construction management are carried out today.

3.6.3.2 Unsurprisingly, the information technology developed in the PC design and manufacturing has also matured significantly in recent years, and can be plugged into the relevant BIM software to make their application more versatile.

3.6.3.3 Design software for shop drawings of PC components have been developed by various software developers, such as Tekla Structures and Allplan. Besides enabling quick and accurate drafting in 3-D, these softwares also have the capability to integrate with other IT management software to achieve additional objectives such as automating quantity take-off and compiling material requirements.

3.6.3.4 Cross-discipline and function management activities are streamlined to offer substantial time and labour-saving in data processing. Updating of design information is also fast and accurate with data automation. This benefits all downstream management processes and operation activities with quick and reliable information access.

3.6.3.5 Modern business processes can now be created and controlled by design and manufacturing software integration, with reduced resource and manpower input.

3.6.3.6 As part of the production control software, delivery of finished products is automatically captured in the stock control software by barcode scanning or radio frequency identification technology (RFID). This helps to facilitate auto-generation of delivery orders and invoices, saving administrative manpower and costs.

3.6.3.7 Integration of design, production automation and accounting management software is possible to create improved collaboration between designers, manufacturers and builders facilitating productivity gains for the entire value chain.

3.6.3.8 In PC manufacturing, management and accounting software can also be linked to data outputs of individual project production status and its corresponding material input and consumption. This helps to obtain real time information for budget monitoring.

3.6.3.9 Management can now review and assess operational and business performances via accurate and up-to-date financial and management reports compiled from individual project data reports to facilitate timely decision.

AUTOMATED PRECAST PRODUCTION SYSTEM - LARGE PANEL SLAB SYSTEM (LPS)

The automated pallet circulation system or Automated Precast Production System (APPS) in ICPHs can help to enhance production productivity and output. However, such system has its limitations. For example, it is difficult to produce PC components with irregular profiles or components with long projected bars from the edges using this system.

Large Panel Slab (LPS) System used in Housing Development Board (HDB) Projects is a PC floor system that is customised to enable manufacturing using the advanced APPS which are commonly found in ICPHs. It can also be tailor-made to any sizes and shapes by leveraging on the Computer-aided Design and Computer-aided Manufacturing (CAD-CAM) capability of the APPS.

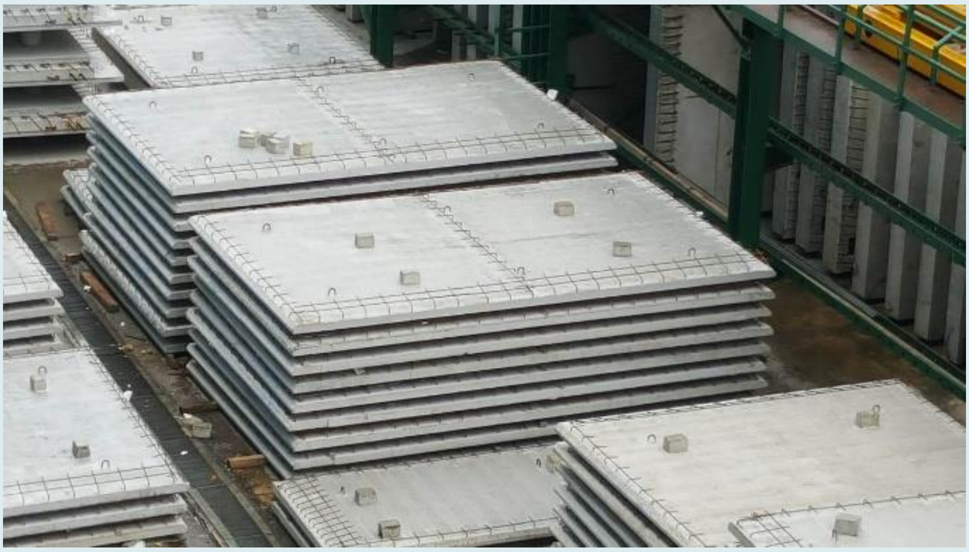


Figure 3.33: **LPS stacked up in a precast yard**

The slabs can be produced in panels that are as large as room-sized and still remain within the allowable handling and transportation dimensions. This essentially reduces the number of components to be produced and handled during the construction stage.

Such designs and detailing also help to facilitate the adoption of standard prefabricated reinforcement meshes.

Figure 3.34 (above) and 3.35 (left): Construction sites ultilising LPS system. LPS system can be leveraged on the APPS technology to achieve higher productivity. These components can be produced in tailor-made sizes.

(All information and images above courtesy of HDB)

CHAPTER 4
PRODUCTION PLANNING AND SITE MANAGEMENT

4.1 GENERAL

4.1.1 Logistics control is relevant for both input materials and finished products. As all manufacturing involves the processing of input materials into finished products, the success of manufacturing relies on:

a) Timely input of materials of the correct quality and quantity
b) Precise processing
c) Fast and accurate material tracking

4.1.2 In recent years, the integration of design and production software has introduced new possibilities and greater convenience in managing the planning and procuring of materials for PC production.

4.1.3 Scheduling can now be semi-automated with reduced manpower and monitoring by leveraging advanced information technology.

Besides automatically extracting relevant data from layout design and shop drawings, the management software built into the production control system can also compile this information to determine the required input materials and finished product quantity.

4.2 PLANNING FOR PRODUCTION MATERIALS

4.2.1 The use of production management software has automated manual work involved in information extraction and compilation for the manufacturing process.

In many construction projects, design changes are common and can create significant disruption to downstream activities.

4.2.2 As such, a fully integrated production and management software can ensure up-to-date design amendments, enabling smooth manufacturing processes.

The software can also disseminate updated information to the relevant suppliers, avoiding any costly abortive works.

4.2.3 Proprietary mechanical connections require lead time for delivery as certain sizes may not be readily available due to low demand.

Therefore, early decision and finalisation of connection design must be carried out to avoid delay in PC manufacturing.

4.2.4 Good planning and monitoring are needed for timely replenishment of stock. Batch ordering at regular intervals should be synchronized with the PC production progress.

Any lapses in replenishment of key cast-in items such as mechanical connectors, may result in disruption and delay site progress.

4.2.5 It is advisable to finalise the design by a certain date. This can be especially useful for projects with relatively short construction periods.

4.3 STOCK MANAGEMENT

4.3.1 Smooth automated PC manufacturing is important to achieve high production output. Any slowdown, interruption or stoppage during the manufacturing process may cause financial impact.

4.3.2 In poorly managed precast plants and construction sites, a lack of storage space due to poor planning or delays in site delivery can occur from time to time. This is undesirable for automated plants in terms of productivity, financial risks and investment cost.

4.3.3 Due to land scarcity, ICPHs in Singapore are mostly built on sites with limited land area, thus the need to intensify land use. As such, just-in-time production planning and stock management are critical to operate effectively on a small foot print.

While close monitoring using production management software can help to maintain an optimal stock, many external factors still exist to cause excessive stock build-up in the plant such as changes in erection planning on site.

4.3.4 Innovative storage method and stock management using automated high racking warehouse technology can be the solution for ICPHs to increase storage capacity and reduce risk of production choke-up.

Other contingency planning such as obtaining temporary storage area or external storage yards during peak production periods can be a short-term solution to overcome storage problem and reduce the risks of costly production disruption.

4.4 DELIVERY CONTROL AND TRANSPORTATION

4.4.1 In ICPHs, delivery control has to be well managed in view of limited land area. Apart from good stock management, speedy delivery preparation is highly desirable.

If PC components in the storage area are pre-sorted according to the erection sequence and bundled into the delivery storage racks, double handling on site can be minimised.

4.4.2 Currently, most plants practise advance notice of at least two days for site delivery, to sort out the finished PC, prior to loading them onto the trailers for delivery.

In contrast, the automated storage rack system helps to:

a) Shorten the delivery preparation process
b) Shorten the waiting time of trailers in the stock yard area
c) Optimise land usage
d) Maximise resource usage

4.4.3 In today's shared economy, it is beneficial to have some industrial standardisation in equipment such as storage racks, for easy and quick handling of finished products by a common pool of transporters. Once such practise is developed, all stakeholders in the PC manufacturing supply chain can benefit from the productivity gain and cost efficiency.

4.5 PLANNING AND ORDERING OF PROPRIETARY MECHANICAL CONNECTORS

4.5.1 Upon approval of the connection design, the various mechanical connectors to be adopted are preferably verified by the intended connector system suppliers (on connector design and application), and to check for availability.

Following which, the relevant suppliers should be consulted immediately on the delivery lead time. Prior to commencement of PC production, it is a good practise to order the required items in a pre-determined number of delivery batches at regular time intervals, to synchronize with production planning.

This should also apply to other accessories, tools and equipment that may be required or associated with PC components production, quality assessment and site erection works.

4.5.2 As with all proprietary products, some less popular types or models may not be available in bulk locally and require long lead time.

Therefore, it is good to exercise prudence by ordering such items well in advance with some contingency quantities.

4.6 TRAINING AND SITE EXECUTION OF CONNECTION WORKS

4.6.1 For construction sites using proprietary mechanical connection systems for the erection of PC components, the project management team should seek early consultation with the suppliers for guidance and training of erection crews prior to work execution.

This is especially important as different connection systems may require specific tools, work methods and installation sequence to join and erect PC components at the temporary stage.

4.6.2 The in-process quality check during site erection can also be unique, and may require specific test equipment and training for quality control supervisors.

Some of the suppliers can provide such certified training to facilitate proper work execution, troubleshooting and quality check on-site.

4.6.3 Suppliers of established brands will usually stipulate the recommended work methods to be used with their products. This covers the accessories needed for accurate positioning and securing of cast-in connectors during PC manufacturing, and also site alignment adjustment etc. during erection and permanent fixing of connectors.

4.6.4 These recommendations and instructions are to be studied and followed closely so as to maximise work quality and optimize product performance.

Deviations from the suppliers' recommendations may reduce work efficiency and performance reliability of the products, and invalidate the product warranty in extreme situation.

4.6.5 For the project Qualified Person (QP), a demonstration on erection work by the builder with the connector supplier in attendance, will offer good opportunity to review and improve quality procedures. Fine tuning of erection work method based on observations or comments from project personnel during trade demonstrations can help improve overall work efficiency and quality.

4.7 QUALITY CHECK AND TESTING

4.7.1 General

4.7.1.1 Work quality procedures of adopted connector products shall be incorporated into the project specifications for inclusion into the builder's quality plan, so that the basic requirement on work quality is clearly defined for the builder.

Sample size and frequency of independent tests, especially for associated in-situ grout strength to be conducted by external accredited laboratories, shall always be included in the work specifications as they are critical to ensure proper performance of grouted sleeve connectors.

4.7.1.2 For all site PC erection works carried out using mechanical connectors, both in-process and post-installation quality check and testing are necessary to assure the performance of the connector products.

4.7.1.3 All special tools/test equipment of connector products, that are required for carrying out in-process tests during site erection works, shall be provided by the builder and made readily available to quality check supervisors. These quality check supervisors should have attended relevant briefings or trainings by the connector supplier.

4.7.1.4 The site project personnel shall follow the suppliers' recommendations and instructions to establish the required quality procedures on critical in-process and final sampling strength tests. This could include the recommended strength and flowability of the grout used for sleeve connectors, tightening torque for threaded mechanical connectors, inspection of site laid connection cottering and confinement rebars in accordance with the recommended details etc.

Proper documentation of quality records should be kept for periodic review and assessment by QP, so as to ascertain work acceptability and avoid possible major lapses. These lapses can result in poor performance of connector products in service, which can be detrimental to structural safety.

4.7.2 Strength Test and Quality Check for Sleeve Connectors

4.7.2.1 Use of grouted sleeve connectors for rebar continuity has been implemented locally since the 1980s in public housing projects to connect structural walls/columns, as well as HDB's Main Upgrading Programme (MUP) projects.

MUP projects involved adding new stacks of three-dimensional (3D) volumetric PC components to existing residential blocks to enlarge the living space of dwelling flats. Subsequently, other grouted or bolted connectors such as corrugated sleeves and custom-made steel base brackets were also adopted in local school and industrial building in the early 1990s. Soon, it became a well-received practice in the base connection design of PC column and wall components.

Spiral connector is another type of grouted sleeve connector which was developed by HDB in early 2000s. Not only is it adopted in most public housing projects from 2004 onwards after extensive performance testing, it has also gone through continual product development to enhance efficiency for on-site installation.

There are three types of grouted sleeve connectors in general:

a) Long corrugated duct sleeve which relies on the lapping of long starter rebars with cast-in reinforcement inside the PC.
b) Short steel casing sleeve, e.g. sleeve connector which relies on casing strength as well as high strength non-shrink grout injected into the sleeve chamber which grips the short starter rebar in confinement high shear friction.
c) Spiral connector does not have a steel sleeve or rely on rebar lapping. Transfer of forces is achieved through concrete confinement by holding together the two shear cones developed at the ends of the spliced rebars inside the grouted spiral.

4.7.2.2 Due to the long starter rebars, the long corrugated duct sleeve is not only manufacturer-unfriendly but also inefficient.

The most efficient grout sleeve is the short steel casing type, which can be further divided into full or half grouted sleeve connectors. The half-grout type is a more recent product consisting of a threaded end embedded inside the PC and a grouted sleeve end.

Nevertheless, both types are highly dependant on the strength of the sleeve material, the bonding and confinement grout strength within the sleeve chamber, and also the mechanical threaded end of rebar as a complete system.

4.7.2.3 The passing criteria of connector strength test is usually taken as achieving breakage in the rebar during tensile strength test, or no slippage of confinement grout inside the sleeve chamber, without any sign of failure in the grout material or in the threaded end of the rebar.

Cyclical strength test may sometimes be conducted to verify the fatigue ductility performance of the sleeve connector system for certain design application, such as highway bridges or buildings with high seismic risks.

4.7.2.4 The tensile strength test for grouted sleeve and mechanical connectors can be easily and expeditiously carried out by local accredited laboratories or Singapore Accreditation Council (SAC) recognized foreign laboratories. For the test methods and requirement of international standards, please refer to ISO 15835 for the necessary compliance details.

4.7.2.5 Illustrations of tensile test samples are included below to illustrate the acceptable failure modes for the first two types of grouted sleeve connectors.

Tensile test for (A) Full-grout and (B) Half-grouted + threaded rebar coupler

(A)

Test set-up

(B)

Note: Bar break failure to show reliable connector system's structural strength.

Figure 4.1: **Tension or compression rebar connector pre-installation test**

4.7.3 <u>Grout Material of Sleeve Connector and In-process Tests</u>

4.7.3.1 The grouting of sleeve connectors is an operation separated from PC erection. This is carried out on-site upon erecting sufficient number of PC columns and/or walls which have met with the specified tolerance for setting out and verticality alignment.

4.7.3.2 Mass grouting operation is best executed by a small team of trained workers, who are skillful in flowable grout preparation and operating the pumping equipment.

Such work arrangements ensure higher work efficiency and reduced grout wastage.

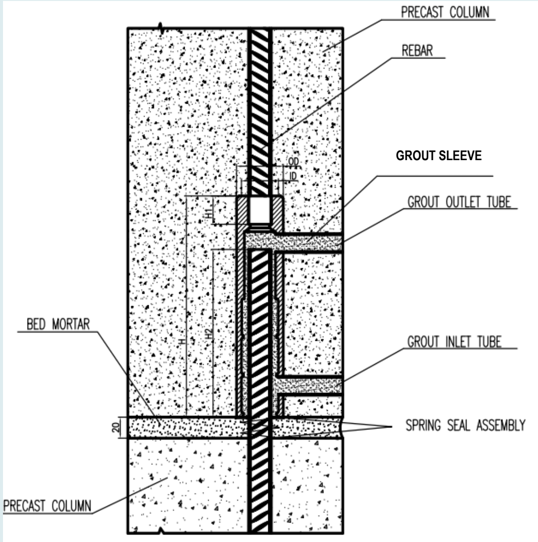
4.7.3.3 Work specifications for different grouted sleeve connectors can vary slightly, and the compressive strength of the grout material is generally in excess of 90N/mm^2.

The same connector manufacturer may also supply the grout as a full package. This option is preferred, as the guarantee covers the total performance of the complete system, and the reliability and performance consistency of the total connection system can be better assured. The use of alternative cheaper grout is not recommended even though some of them may exhibit similar strength to the specified one. This is particularly important since the performance characteristics may not be identical due to the difference in its constituents' composition and properties.

This is more so especially when no independent large-scale verification test has been conducted by the product principal supplier.

Examples of In-Process Test for Grout Material of Mechanical Sleeve Connector

- Grouting of rebar coupler, indicative specification of work



Rebar Size		Outer Diameter (OD)× Length(H)	Inner Diameter (ID)	Thread Depth (H1)	Rebar Embedment Depth(H2)	Grout Consumption (kg/pc)
With Thread Splice	With Grout Splice					
Φ13	Φ13,Φ12	Φ34×156	25	18	90~127	0.30
Φ16	Φ16,Φ13	Φ42×174	30	20.5	115~143	0.36
Φ20	Φ20,Φ16	Φ48×211	34	24	145~175	0.47
Φ25	Φ25,Φ20	Φ55×256	40	30	182~215	0.67
Φ28	Φ28,Φ25	Φ58×292	43.5	33	216~244	0.80
Φ32	Φ32,Φ28	Φ63×330	48	36	246~276	1.07
Φ40	Φ40,Φ32	Φ80×426	58	45	331~360	1.93

Examples of In-Process Test for Grout Material of Mechanical Sleeve Connector (Continued from previous page)

- Flowable grout mixing and injection into sleeve


- Test of flowable grout before injection and final strength

ITEM		UNIT	INDEX
Fluidity		mm	200～280
Bleeding Ratio		%	0
Compressive Strength	1d	MPa	≥30
	3d	MPa	≥60
	28d	MPa	≥100
Vertical Expansivity	3hrs	%	≥0.02
	between 3hrs & 24hrs	%	0.02～0.5

NOTE:
1.The fluidity indexes using cylindrical test module:Φ46×100 (mm);
2.The strength indexes using cube test module: 50×50×50 (mm).

Examples of In-Process Test for Grout Material of Mechanical Sleeve Connector (Continued from previous page)

- Accessories for sleeve connector grouting work

4.7.3.4 For spiral connector, HDB shall be consulted on the design technical details, material and installation work specifications to ensure proper application.

Some of its' key features and installation work process are illustrated in Section 4.8.1.

4.8 ILLUSTRATION OF SITE USE OF VARIOUS TYPES OF CONNECTORS

There are five main types of mechanical connectors in PC construction. Each has its own structural function. They are:

Tension or Compression Rebar Continuity Individual Connector

Bearing or Corbel Connector
(primarily designed as seating support but may also combine with tensile or compressive capacity)

Shear or Hang-up Connector
(of mostly simply supported design or corbel bracket)

Moment Connector
(which can consist of an assembly of individual connectors especially designed for a given PC connection)

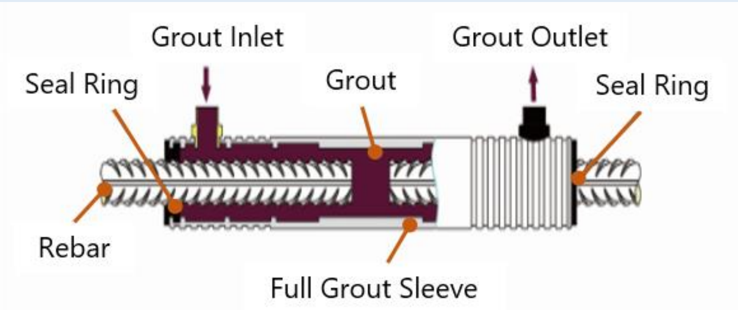
Composite Interface Shear Enhancement Connector

Apart from being used as structural connectors for permanent loads, some of these connectors may also act as temporary support for PC erection.

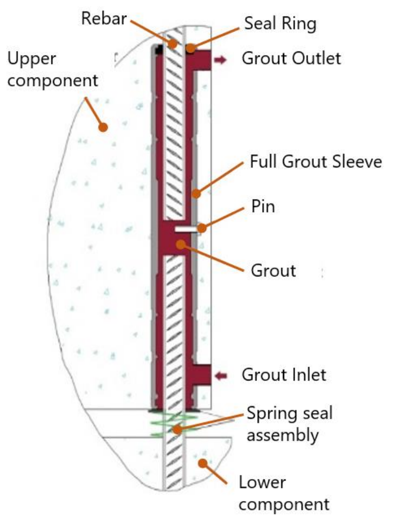
This enables prop-free construction of PC structures and high efficiency in manpower and construction resources.

4.8.1 Tension or Compression Rebar Continuity Individual Connector

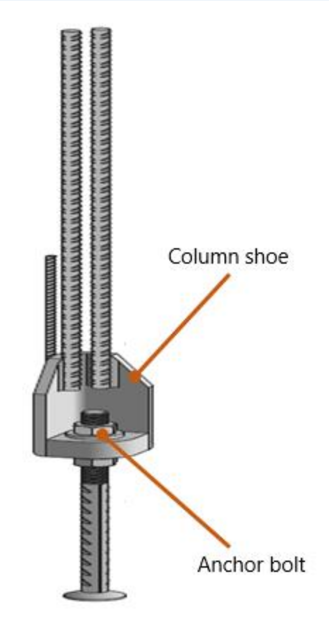
- Full grout sleeve (horizontal – beam)



- Full Grout Sleeve (Vertical – column)



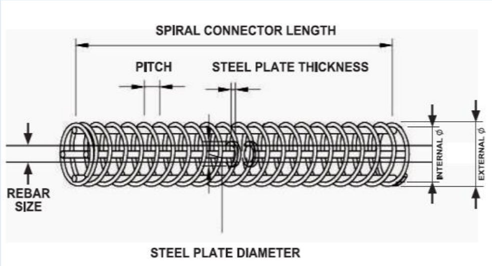
- Steel Column Shoe



Note: Demonstration of possible application without implicit endorsement of any product

- Spiral Connector

Specification



Type	Rebar Diameter (mm)	Spiral Connector Length (mm)	Pitch (mm)	Internal Diameter (mm)	Thickness of Enlarged Ends (mm)	Diameter of Enlarged Ends (mm)
SC4	16, 20	500	25	65	5	30
SC7	25	500	25	75	6	40
SC8	28	500	25	75	6	40

Grout

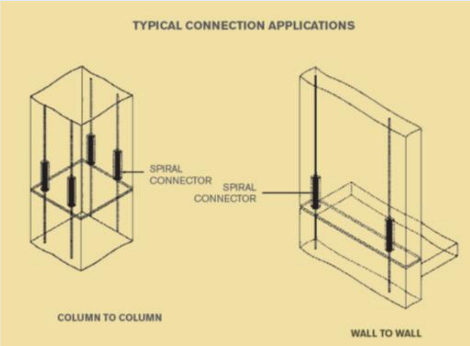
The spiral connector requires flowable high-strength non-shrink grout (Grade 70) for pressure grouting the space between the spiral and the bars.

A preformed chamber in the precast element is required to receive the spiral connectors. Using the same grout material, the base can be grouted in a single operation.

Only grout material that are tested by HDB and meeting the specifications (Please refer to www.hdb.gov.sg for details) can be used.

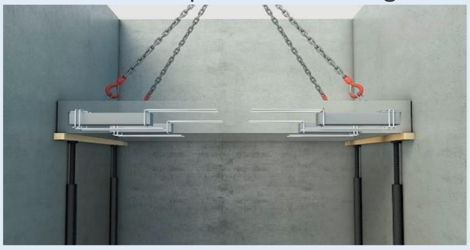
The grouting process should be conducted using a mortar pump through the lower inlet.

Application



4.8.2 Bearing or Corbel Connector
(which is primarily designed as seating support but may also have tensile or compressive capacity)

- Telescopic corbel seating – horizontal (slab, staircase landing, etc.)

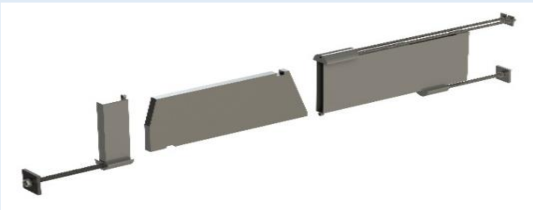
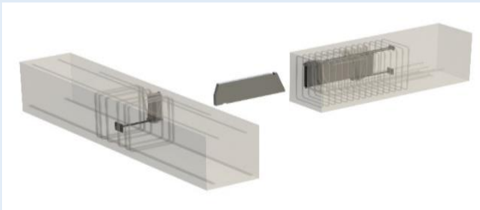



- Steel bolted on corbel – horizontal (beam)

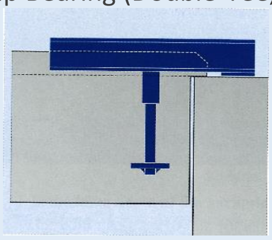
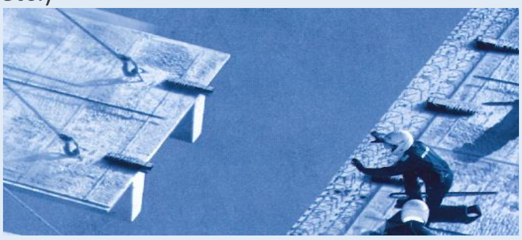

Note: For demonstration of possible application only. The above is not an implicit endorsement of any product.

4.8.3 Shear or Hang-up Connector (of mostly simply supported design)

- Telescopic Steel Shear Plate – Horizontal (Beam)

- Steel Hang-up Bearing (Double-Tee, etc.)

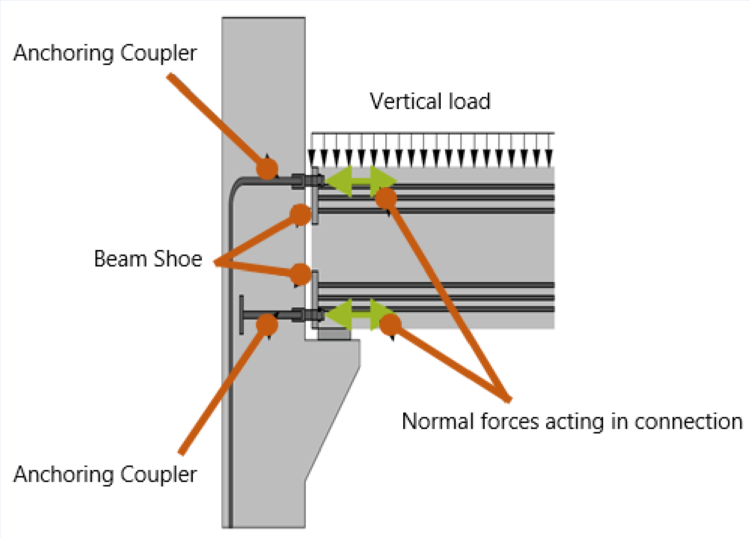



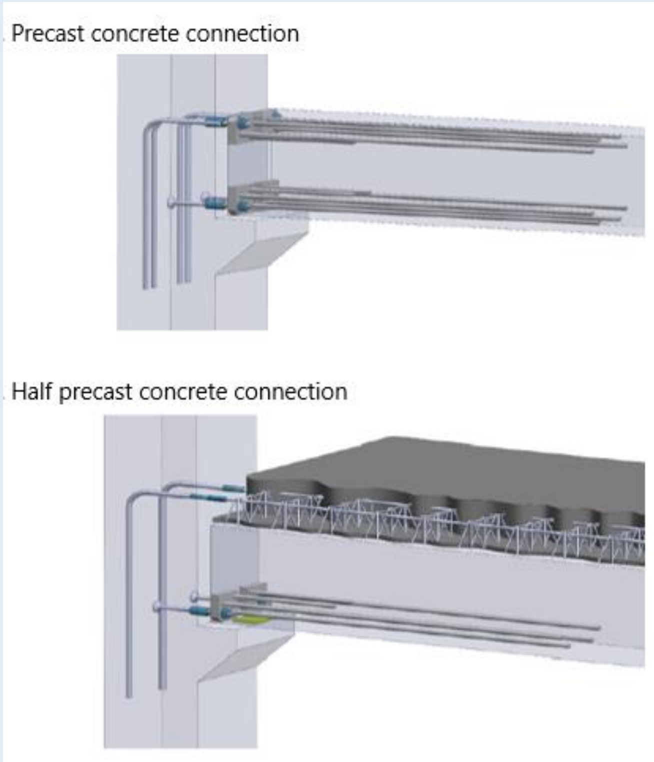
Note: For demonstration of possible application only. The above is not an implicit endorsement of any product.

4.8.4 Moment Connector

(which can consist of an assembly of individual connectors specifically designed for a given PC connection)

- Moment Connector – Horizontal (Beam)

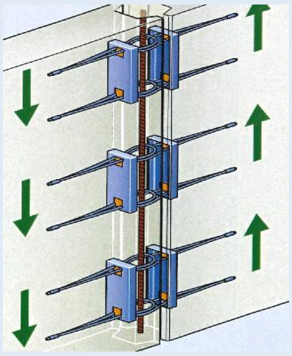


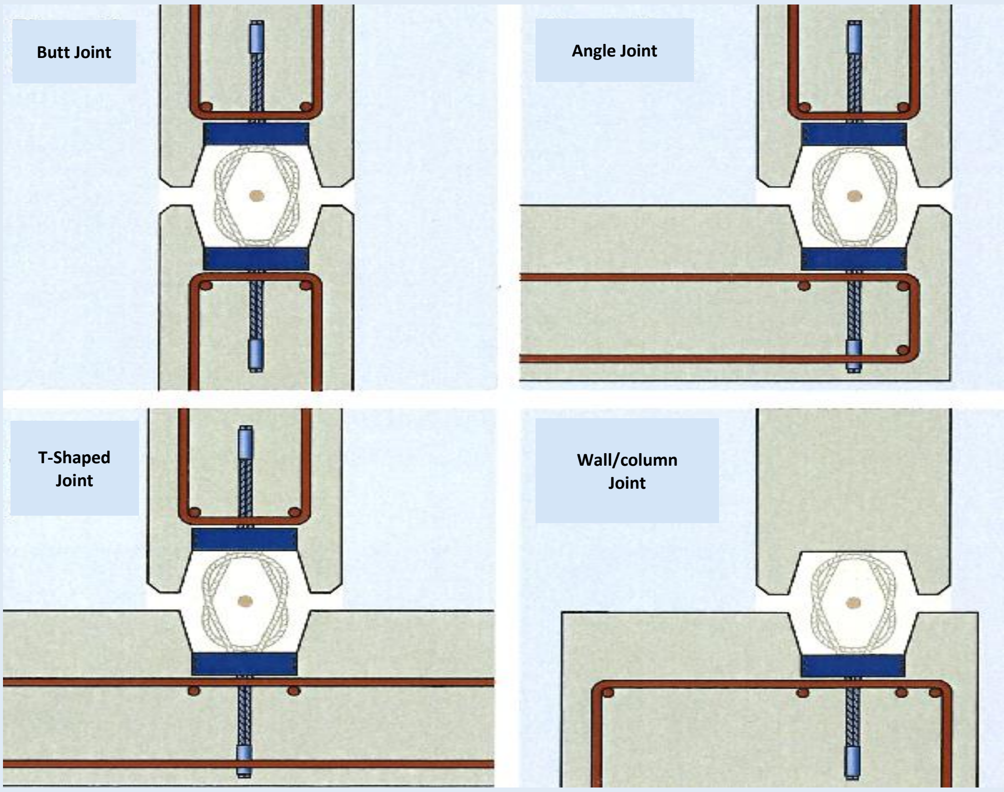


Note: For demonstration of possible application only. The above is not an implicit endorsement of any product.

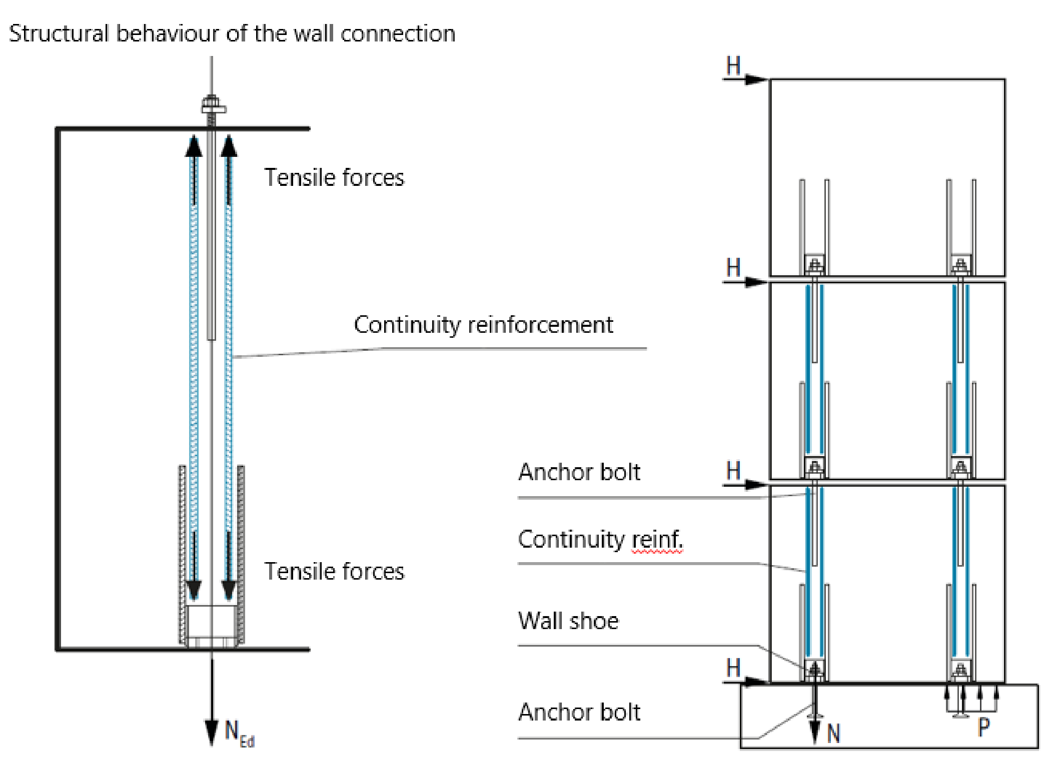
4.8.5 Composite Interface Shear Enhancement Connector

- Flexible cable loop box





- Steel wall shoe



Note: For demonstration of possible application only. The above is not an implicit endorsement of any product.

CHAPTER 5
COMPLIANCE TO REGULATIONS

5.1 GENERAL

5.1.1 Majority of the proprietary mechanical connectors are developed to provide temporary stability during erection of PC components and as permanent structural connection.

5.1.2 Some of these products have been tested and used in the developed countries in the past two decades. Products with test certificates issued by international testing agencies or have obtained the ETA (European Technical Approval) and CE-marking, may be considered as highly reliable.

5.1.3 Warranties and insurances that guarantee performance and product quality are often offered by suppliers or principals of established products.

Test certificates issued by foreign laboratories are generally acceptable if they bear recognition from the Singapore Accreditation Council (SAC) for the required test methods under the international mutual recognition arrangement.

Nevertheless, due diligence checks should be carried out by qualified persons to ensure the products satisfy the intended design requirements.

5.1.4 As similar connector products from lesser known sources are also available, it is important to verify products that claim to possess very similar design and performance before adopting them into any design works.

These products tend to have relatively short product development and application history. Before acceptance and adopting them in the design, the QP should assess the products and review results of verification tests to check on possible risks in product quality, reliability and performance consistency.

5.1.5 Product quality assurance for ferro-mechanical connectors shall cover both the material specifications and the manufacturing process for product quality consistency. Most of these mechanical connectors are fabricated using high strength steel materials, involves processes such as machining, welding, threading and forging.

Hence, overall process work quality is important to ensure performance reliability of the total product system in service condition. In view of these processes, the manufacturing facilities should preferably possess the relevant process certification such as ISO/IEC17065 or its equivalent to ensure compliance with relevant product quality standards. Otherwise, more local tests may be necessary to verify the product quality.

5.1.6 Similar to steel rebar supply, the source of product origin may give indication of the likely manufacturing quality. Product performance verification test must be conducted by an independent laboratory to serve as initial assessment prior to selection.

5.1.7 After adopting the product into the design, the builder is responsible for the workmanship during on-site installation.

Further quality tests should be conducted in accordance to requirements specified by the supplier or stipulated by the designer to ensure work acceptability.

5.1.8 For full-grout or half-grout sleeves connectors which uses different suppliers for connector and grout, it may be difficult to identify the cause of failure in the event of product failure.

Work responsibilities in such connection systems can be complex. To ensure total system quality, it is vital to establish and implement a comprehensive project quality plan for testing of connector and in-process work.

5.2 DESIGN RESPONSIBILITY ON USE OF PROPRIETARY MECHANICAL CONNECTOR

5.2.1 The designer must have basic understanding of the product design principle.

Notwithstanding, the designer must be familiar with the transmission and distribution of internal forces through the connector into the reinforced concrete section.

5.2.2 Some established proprietary connector manufacturers provide free in-house design software for designers to check on the design capacity of their connections.

Suppliers who can offer detailed technical and design advice on the use of their proprietary products are often the preferred sources for quality reliability.

5.2.3 The primary design responsibility in adopting any proprietary connectors lies with the QP.

He/she who makes the decision to accept the connector products as part of the overall structural system; whether for use in in-situ or PC construction.

Designers' basic technical competency in design and due diligence in supervising the works must always be emphasised in the proper selection and application of connector products.

5.2.4 The QP is responsible for assessing connector product quality and reliable performance. This can be done through examination of technical specifications, test certificates from accredited laboratories and/or conducting appropriate local verification tests.

It is essential for the QP to be involved in these due diligence checks before accepting the connector system as appropriate for a particular structural application.

Reference may be made to ISO15835 for details on materials and test requirements for compliance to various international standards.

For grouted connector application, the complete system of connector sleeve and compatible grout should be considered together instead of individually to ensure proper performance of the connector system.

5.2.5 The half-grouted sleeve has a threaded end. As such works are often often carried out by local workshops of varying competency, the designers must pay strict attention to the threading quality.

Threading work must adhere to the sleeve manufacturers' requirements on diameter and pitch, so that they fit precisely to the half-grouted sleeve connector.

5.2.6 Proprietary connectors generally act as alternatives to lapping of site laid rebars, for continuity of reinforcement in connections of reinforced concrete structures.

The use of connectors technology can enhance the structural performance of building structure, such as for use in seismic design when lapping of rebars at critical sections are not allowed.

Meanwhile, the adoption of connection system with superior performance can also complement the building construction technology, with advantages in enabling more productive construction execution.

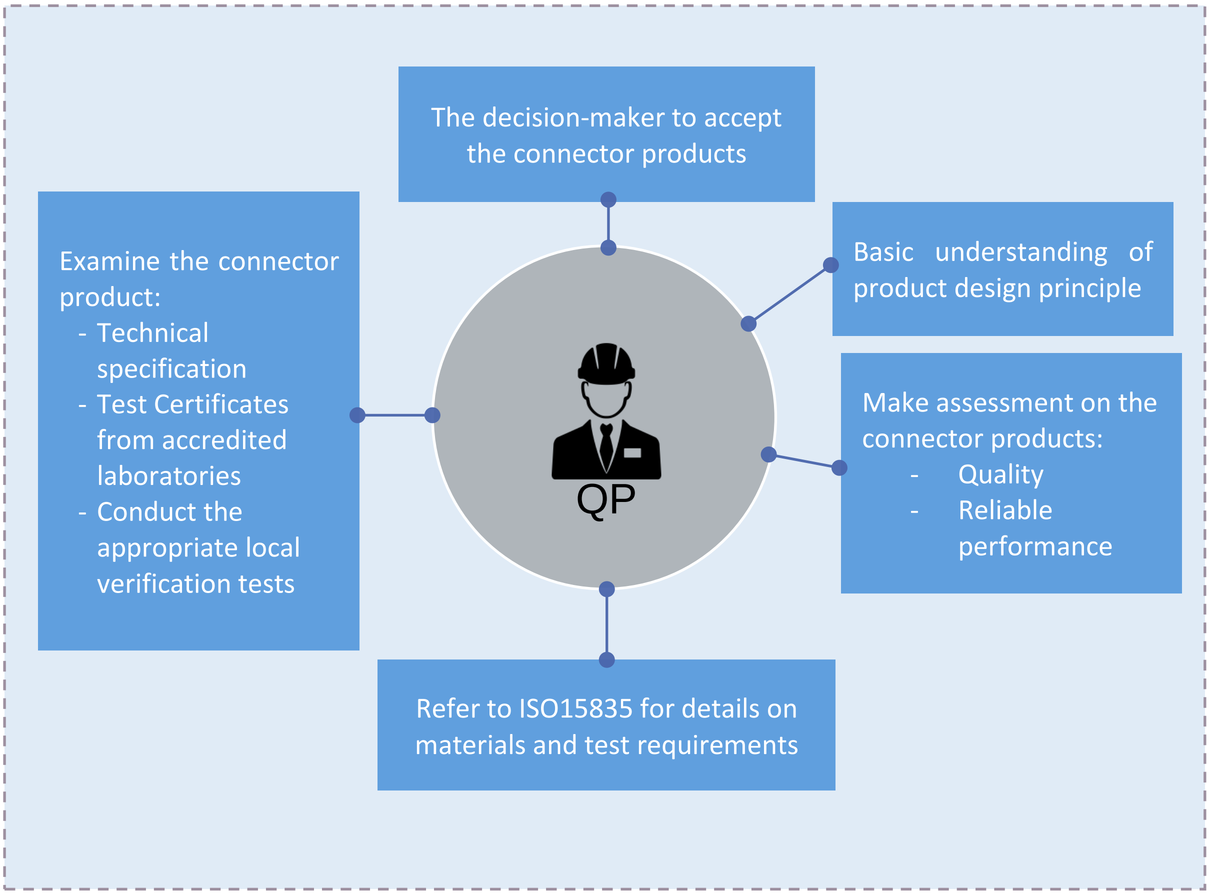


Figure 5.1: **Responsibilities of a QP**

5.3 WORKS INSPECTION AND TESTING

5.3.1 As required in the Building Control Act, all structural works are to be supervised by the QP to ensure that the final executed work achieves quality in compliance with the design intent and work specifications.

5.3.2 The QP shall therefore dictate and implement quality check procedures when adopting proprietary connection systems. This can be done with reference to manufacturer's recommendations of the selected product and together with any other requirements deemed necessary.

Proper quality assessment and regular audit system on work execution are means to facilitate effective supervision. This helps to ensure that the builder's work are adequately verified by in-process tests to monitor work quality.

5.3.3 Prior to mass work execution, it is recommended to carry out trade demonstration organized by the builder and connector product suppliers, so that the QP can assess the actual work process. Adjustment to the in-process work quality check or tests and documentation may sometimes be necessary to cover work areas that might not have been sufficiently covered in the product manufacturer's recommendations.

5.3.4 The basic types of inspection and testing requirements for any selected connector products are:

Pre-installation	Product material checks Sample performance verification tests
Post-installation	In-process work quality verification tests

The QP must pay particular attention in these two aspects to ensure satisfactory in-service performance of connection system products in the final structure.

5.4 SERVICE MAINTENANCE AND PERIODIC INSPECTION

5.4.1 Most proprietary mechanical connector products are designed to be maintenance-free.

They are generally protected with galvanizing, cementitious grouting or embedded with in-situ concrete after construction, for fire resistance and durability against exposure to the environment.

5.4.2 When the building is completed and in operation, the performance of connectors is comparable to reinforcing steel rebars as they are also part of the RC structure.

Any distress signs in the concrete structure such as cracking at connection of structural components, may indicate inadequate capacity or deterioration of the connector product.

It is therefore useful to highlight the types of connector products in PC components used in the completed building in the building maintenance manual. This is to ensure that the renovation designers and contractors are aware of their use, when doing structural alteration or partial demolition during major renovations or upgrading works.

5.4.3 Where significant connection cracking in the PC structure is found during periodic inspection, the connector products may need to be exposed by localized controlled concrete hacking for detailed investigation.

A visual inspection and assessment on the condition of exposed connectors will then be conducted to decide on the appropriate rectification or any necessary strengthening needed.

5.4.4 As for all building structural defects investigation, a professional engineer must be engaged to ensure proper investigation. All assessment reports must be submitted to the building authority.

After which, the rectification work proposal can be prepared for approval and to make good the building defects.

5.5 QUALITY ASSESSMENT PLAN ON PROPRIETARY CONNECTION SYSTEMS

The QP shall adopt the quality assessment plan prior to selection and acceptance of a particular proprietary connection system for use in their projects. The following suggestions may be considered for inclusion in the project quality assessment plan at the user's discretion.

GOOD PRACTICE FOR QUALITY ASSESSMENT OF PROPRIETARY CONNECTION SYSTEMS

Preliminary Assessment	a) Are the connector products from reliable supply sources? b) Are these connector products suitable for use in the intended connection design and able to achieve the required work quality? c) Is there any independent certification on the product from a reputable testing institution?
Product Pre-installation Quality	a) Are there sufficient technical information on product application performance provided by the supplier? b) Are recent verification products tests and certifications by accredited laboratories available? c) Are the material specification and product characteristics in compliance with relevant deign code-requirement? d) Are there any applications in past and current projects as reference for possible third-party user experience feedback?
Local Test Verification	a) Do the products have sufficient local test verifications from recent projects? b) Upon selection of connection system, are local assessment testing necessary to check on product performance?
Product Post-installation Quality	a) What are the proper connector installation methods recommended by the suppliers? b) What are the in-process tests recommended by the suppliers? c) After reviewing the 'trade demonstration' conducted for on-site installation, are the above recommendations sufficient to ensure total work quality?

Project Inspection and Test Plan	a) Are the works for connector products included in the quality plan? For example, does it state the specified tests, frequency and sample size required for quality verifications? b) What are the quality documents and records needed in order to monitor and verify the consistency of work quality? c) What are the precautions needed to ensure correct usage of the connector products?