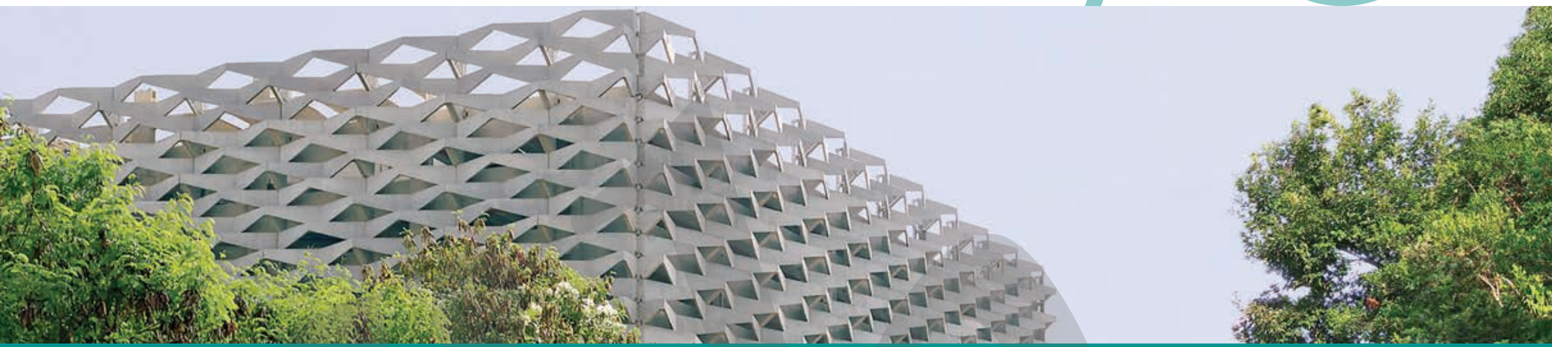
Bulletin 78



Precast-concrete buildings in seismic areas

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State-of-the-art report

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State-of-the-art report

Task Group 6.10 and PCI

March 2016

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Category:	**Draft to be approved by:**
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Manual / Guide to good practice / Recommendation	Technical council
Model code	General assembly

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

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Foreword

Since the mid-twentieth century the industrial prefabrication of concrete structures has acquired an increasing share of the construction market worldwide thanks to its well-known advantages. These advantages include the best use of materials, structural efficacy, flexibility in use, speed of construction, quality consciousness, durability, eco-friendliness and sustainability.

Over this same period, *fib* Commission 6: *Prefabrication* has continued the work of the FIP on issues directly related to precast concrete, such as structural elements, detailing, connections, systems, production, handling, assembling, demounting, as well as items that are indirectly related, like materials technology, structural analysis, building physics, equipment and sustainable development.

Construction in seismic areas is being afforded increasing attention due to the progress of both seismology and seismic engineering. The latter covers the identification of areas considered to be at risk and a deeper understanding of structural response. All types of construction, including those using structural concrete in general and precast concrete in particular, are affected by this development.

The theme was approached in *fib* Bulletin 27: *Seismic design of precast concrete building structures* (2003) by *fib* Task Group 7.3 of *fib* Commission 7: *Seismic Design* (as it was in the old structure), with contributions by this commission. It offered a comprehensive overview of the theoretical bases, design approaches and practices in most advanced countries at that time.

Further experience with precast concrete structures enduring earthquakes of various intensities has come to the fore since then. Most structures have performed well, whereas some, particularly the older ones, showed shortcomings that elicited additional inquiry into their behaviour to learn how to enhance the quality of future designs. Comprehensive research projects run internationally yielded important results on the capacity of precast structures to withstand seismic events. They were particularly useful for improving the local detailing, as well as for evaluating the overall ductility, which proved to be quite comparable with that of cast-in-place structures, thereby helping to define appropriate behaviour factors.

Today we can say that much has been clarified and that the recent experience can be transferred to practice so as to create new buildings that exhibit the performance required. This report aims to update designers on developments and help them in their daily practice. Thanks are due to all members of the task group, particularly to its convener

Spyros Tsoukantas, not only for carrying out such useful and ambitious work and for imparting it through this valued report, but also because they have committed themselves to a further study of how to upgrade existing structures, including structures possibly damaged by earthquakes, that were built when practice was less qualified.

U.S. design and construction practice for precast concrete structures was used as basis throughout the preparation of this report. Nothing in here is contrary to U.S. practice. This report has been approved by the Technical Activities Council of the Precast/Prestressed Concrete Institute (PCI) as a joint *fib*-PCI publication.

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Contents

1 Introduction

1.1 General

The technology of prefabrication in buildings has been in use for more than half a century and is spreading considerably and successfully every day throughout the world.

Experience in earthquakes has shown generally good behaviour, even in regions of high seismicity, in precast structures if they are properly designed and built. In particular, precast frame systems, which are very commonly used for one-story industrial buildings, combine their capacity for high resistance with a strong attenuation of the seismic action because of a long period of natural vibration and a large capacity for ductile deformation. Most of the precast buildings that suffered excessive damage with partial or total collapse were built using ill-conceived conceptual designs, inadequate structural schemes and load paths and/or the poor detailing of connections between structural elements, as well as between structural and 'non-structural' elements, such as claddings and heavy infills.

In general, for all types of structures and structural systems, good behaviour under seismic situations depends on the following fundamental elements:

- overall design
- construction details of the structure

Good seismic design depends on the experience and skills of the designer and the state of knowledge at local and international levels in terms of theory, analytical/numerical and experimental observations, and design guidelines and codes at the time of design. In general terms, knowledge of current 'best practice' is required. This often goes beyond the prescriptions contained in the latest codes. Design codes, standards or by-laws, in fact and by definition, set 'minimum' requirements. It is up to the owner to discuss requirements with the engineer and the other parties involved so as to set the desired design targets, which may be higher than the code requires, and to investigate options and solutions to achieve them in a cost-effective manner. Ultimately, it is best to be mindful of Professor Tom Paulay's dictum: Earthquakes do not read the code. Earthquakes seem to be particularly effective at showing up the hidden structural weakness of a system - the so-called weakest link in the chain.

Good construction depends on the experience, skills, available equipment and so on of the construction company as well as adequate supervision and inspection to ensure that all design details are properly implemented.

In the particular case of precast construction (when compared to monolithic construction) additional design rules, concepts, code provisions and building procedures and details are required to provide a level of safety under seismic actions that is equal to or better than those for cast-in-situ construction. Furthermore, given the peculiarities and recognized advantages of precast construction, such as high quality control, speed of erection, modular construction, it is essential to have an integrated and collaborative approach from the owner and the design and construction teams. A precast structure is, in general, less forgiving than its cast-in-situ counterpart in terms of tolerances and post-design on-site modifications of any kind. Hence, the entire building system has to be looked at holistically rather than at a component level. At the same time, specific attention needs to be paid to the final detailing of each connection between elements in order to guarantee the expected performance, while maintaining ease of constructibility.

1.2 Scope

This bulletin is mainly addressed to engineers who are involved in the design, construction and/or production of precast buildings. It is also intended for civil engineers and architects who want to become acquainted with the main principles of design, detailing and construction, and in general, the behaviour of different precast structural systems under seismic actions.

The scope is broad and does not focus on design issues. Precast construction under seismic conditions is treated as a whole. The main principles of the seismic design of different structural systems, their behaviour and their construction techniques are presented through rules, construction stages and sequences, procedures and details. The purpose is to allow precast structures to be built in seismic areas that comply with the fundamental performance requirements of collapse prevention and life safety in major earthquakes and limited damage in more frequent earthquakes.

The content of this bulletin is largely limited to conventional precast construction and although some information is provided on the well-known technology of PRESSS (jointed-ductile dry connections), this latter solution is not covered in detail. A systematic description of PRESSS technology may be found in the PRESSS Design Handbook (NZCS, 2010) and other publications.

Detailed information about the seismic design of precast-concrete building structures may be found in *fib* Bulletin 27 (2003), while the design and behaviour of structural connections for precast-concrete buildings is thoroughly dealt with in *fib* Bulletin 43 (2008).

The general overview in this bulletin of alternative structural systems and connection solutions that are available for achieving desired performance levels intends to provide engineers, architects, clients and end-users in general with a better appreciation of the wide range of applications that modern precast-concrete technology can have in various types of construction, from industrial to commercial as well as residential. Lastly, the emphasis on practical aspects, from conceptual design to connection detailing, aims to help engineers rid themselves of the habit of blindly following prescriptive design codes and instead to return to basic principles in order to achieve a more robust understanding and thus greater control of the seismic behaviour of the structural system as a whole, as well as of its components and individual connections.

1.3 Key features

The key features outlined in this bulletin include:

- Basic principles of seismic design with particular reference to precast structures
- Role of connections and their relationship to selected behaviour factors q (as used in the Eurocode or, as per other codes, ductility factors μ or reduction factors R)
- Basic principles of conceptual design
- Description of hinged and moment-resisting frame systems, including jointed systems and construction details of beam-to-column connections
- General information about and details of column-to-foundation, beam-to-beam and column-to-column connections
- General information about precast large-panel-wall systems and construction details
- General information about wall-frame systems (dual systems) and construction details
- General information about the seismic behaviour of floor systems (diaphragm action)
- General information about and construction details of precast double wall systems
- General information about and construction details of precast cell systems

Design examples are also given using both the more traditional 'force-based design' (FBD) approach and the emerging 'displacement-based design' (DBD) approach, for comparison.

2 Terms and definitions

Precast structures consist of structural elements manufactured elsewhere than in their final position within the structure. In the structure, the elements are interconnected to ensure the required structural integrity.

Wall: a structural element having an elongated cross section with a length to thickness ratio *lw* / *bw* greater than four (according to Eurocode).

Frame system: a structural system in which both the vertical and the lateral loads are resisted by space frames. Frame systems may have hinged or moment-resisting connections between beams and columns. Frames with hinged beam-column connections are herein designated as **HCF** (hinged-connection frames). Frames with moment-resisting beam-column connections are herein designated as **FCF** (fixed-connection frames).

Wall system: a structural system in which both vertical and lateral loads are mainly resisted by structural walls.

Large wall-panel construction: a structural system made up of large precast wall panels placed in two orthogonal directions and suitably connected to form a resisting system.

Shear wall-frame systems (dual systems): structural systems in which lateral forces are shared by shear walls and frames.

Cell structures: three-dimensional precast-cell systems that emulate monolithic construction.

Diaphragms: horizontal structural members subject to in-plane forces transferring those forces to the vertical elements of the lateral-force-resisting system.

Ductility: the ability of a structure to sustain its load-carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake.

Ductile frame: a structural frame specially detailed to provide ductility.

Behaviour factor *q* or reduction factor *R*: the factor used to reduce the design forces below those needed for the elastic response to the design earthquake and that accounts for the nonlinear response of a structure.

The **capacity design method** ensures that undesirable brittle mechanisms within an element or a connection between elements are protected by developing an appropriate hierarchy of strength so that the weakest link of the chain will behave in a ductile manner under severe deformations while the other structural elements are provided with sufficient relative overstrength for the chosen means of the ductile mechanism to be maintained.

Connectors are connecting elements embedded in concrete to transfer forces between precast members or between precast and cast-in-situ members.

3 Basic principles of earthquake-resistant design

3.1 General

3.1.1 Performance-based design philosophy

In response to the recognized critical need to design, build and maintain facilities guaranteeing better damage control, a major effort has been made in the past decade to prepare ad-hoc, performance-based design guidelines involving the whole building process, from conceptual design to construction aspects.

The Structural Engineers Association of California (SEAOC-1995) Vision 2000 document defined performance-based seismic engineering (PBSE) as 'consisting of a set of engineering procedures for design and construction of structures to achieve predictable levels of performance in response to specified levels of earthquake, within definable levels of reliability'. Interim recommendations were developed to implement the approach.

Within such a framework, expected or desired performance levels are coupled with levels of seismic hazard by performance design objectives as illustrated by the performance design objective matrix shown in Figure 3-1.

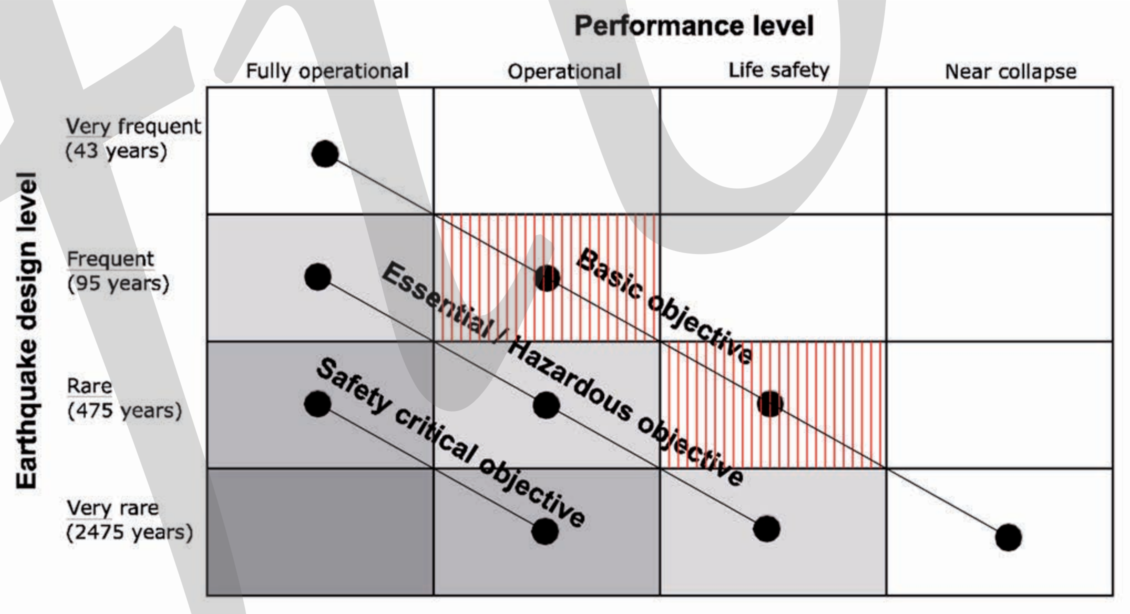


Fig. 3-1 Seismic Performance Design Objective Matrix (based on SEAOC, 1995)

Performance levels are thus expressions of the maximum acceptable extent of damage under a given level of seismic intensity, including losses and repair costs due to damage to both structural and nonstructural elements.

Four performance levels are typically recognized, namely **fully operational**, **operational**, **life safety** and **near collapse**. Depending on the structural importance level (from minor low-hazard to critical facilities), the four performance levels are correlated to four levels of seismic intensity expressed in terms of the return period of the earthquake (approximately 50, 100, 500, and 2,500 years). These return periods can also be expressed in terms of the probability (approximately 100%, 50%, 10% or 2% in 50 years) of an earthquake exceeding this set level during the lifetime of a structure.

The main challenge and still controversial part of a reliable performance-based design approach is the difficulty in correlating the level of damage, qualitatively described in general terms, to engineering demand parameters (for example, deformations, drifts and accelerations) for the individual structural and nonstructural elements as well as for the whole 'system', including the soil-foundation system.

Similarly, for precast structures, the selection of appropriate engineering parameters in order to guarantee the achievement of targeted performance objectives should be recognized as a critical and fundamental step during the design phase, depending on the expected use of the structure (residential, commercial or industrial facilities) and targeted post-earthquake business interruption.

With reference to Figure 3-1, and considering an ordinary structure (residential or commercial building), operational and life safety are the two performance levels typically adopted in design code provisions and verified, respectively, for the frequent and the rare earthquake design levels. For the fully operational performance level, the maximum level of interstory drift should be limited to protect nonstructural elements and services, while the maximum level of floor acceleration should be limited to protect instrumentation, machines, and other acceleration-sensitive devices. These requirements may be assumed for particular building designations (such as hospitals) and verified for the very frequent earthquake design level. The near collapse performance level typically refers to the need to check that under a maximum considered earthquake event, associated with a 2,500-year return period, the structure may experience significant damage but still retain structural integrity.

Different importance levels, e.g., from ordinary to essential and critical facilities, will dictate the adoption of higher levels of performance, thus a lower level of damage for the same level of expected intensity or return period. For example, critical post-earthquake facilities such as hospitals or police stations would require a higher level of protection and immediate functionality after a severe earthquake.

3.1.2 Performance requirements

For the sake of simplicity, the philosophy adopted in most seismic codes for the design of structures against seismic actions is based on only two performance levels (out of the general four previously mentioned and outlined in the performance matrix of Fig. 3-1).

3.1.2.1 Damage-limitation limit state

The structure is expected to experience limited structural and nonstructural damage during frequent earthquakes. At this limit state, the structural members should retain their strength and stiffness, negligible permanent (residual) deformations/drifts are expected to occur and limited repair is needed. The corresponding seismic action is usually termed as the serviceability earthquake. The definition of this earthquake level varies in different codes but a reasonable probability of exceedance is, for ordinary structures, 10% in 10 years (for a mean return period of 95 years). According to the performance matrix of Figure 3-1, this level would thus correspond to the operational level. Compliance criteria for the damage limitation limit state are usually expressed in terms of deformation limits.

3.1.2.2 Collapse-prevention limit state

For a rare earthquake with a relatively smaller possibility of occurrence during the life of the structure, the prevention of collapse and the retention of the structural integrity and sufficient residual structural capacity should be ensured. Although significant damage might occur, the structure should be able to maintain the capacity to carry vertical loads and retain sufficient lateral strength and stiffness to protect life during aftershocks. The corresponding seismic action is referred to as the design-level earthquake. For structures of ordinary importance, this earthquake level would be considered to have a 10% probability of exceedance in 50 years (for a mean return period of 475 years). For structures of higher importance, a longer return period is assigned.

According to the performance matrix of Figure 3-1, this level would thus correspond to the life safety level.

It is worth noting that in modern seismic codes, including Eurocode 8 (2004), compliance criteria for the no-collapse limit state have typically been expressed only in terms of forces (force-based seismic design). More recently, in the New Zealand Design Standard (NZS1170.5, 2004), a displacement-focused approach has been introduced and a maximum drift level of 2 to 2.5% is suggested for ultimate limit state design. A more explicit and direct displacement-based design approach has also been introduced in the (normative) Appendix B of the Concrete Standard (NZS3101, 2006) as a fully code-compliant design approach (thus not only an alternative method) specifically for precast-concrete jointed ductile connections, also referred to as PRESSS technology. Similar displacement-based design concepts were also introduced in the (informative) appendix section of Eurocode 8 (2004). In fact, the extent of the damage of structural members is more correctly related to their deformation (ultimately, to the level of strain in the materials) rather than to the forces induced to them during the seismic action. For this reason, and for the time being, according to most codes, a proper check of the actual displacement demand versus capacity should at least be carried out, even when following a force-based design approach, before finalizing the design of the structure. Furthermore, more recent trends in earthquake engineering are oriented towards a displacement-based design approach whereby an acceptable level of displacement and deformation demand, associated with a specific level of performance and damage, is assumed and thus controlled as target design values at the beginning of the design process, and not at the end.

3.1.3 Seismic actions

3.1.3.1 Acceleration spectrum

Following a traditional force-based design approach (FBD), the seismic action is usually represented by an elastic acceleration response spectrum of the kind shown in Figure **3-2**. This spectrum gives the spectral acceleration S_a of an elastic single-degree-of-freedom (SDOF) structural system with 5% damping, subjected to a base motion with peak acceleration S_{ag} as a function of its natural period T. In this figure, $a_g = \gamma_I \, a_{gR}$ where γ_I is the importance coefficient and a_{gR} is the ground acceleration for stiff soil that corresponds to the reference return period. S is the soil factor, T_B is the lower limit of the period of the constant spectral acceleration branch, T_C is the upper limit of the period of the constant spectral acceleration branch and T_D is the value defining the beginning of the constant displacement response range of spectrum. T_B and T_C depend on the soil properties and T_D is usually set by the code/standard being considered.

In an alternative representation of the seismic action, a nonlinear time-history analysis using natural earthquake records or artificial ground motions can also be performed. Specific criteria are defined in the codes for the selection and the scaling of such records to be 'compatible' with the pertinent design spectrum.

In general, a minimum of three records with response spectra is required that 'match' (within certain limits) the code spectrum that corresponds to the design-level earthquake. Maximum values of the engineering demand parameters (such as internal forces and drift level) obtained in the nonlinear time-history analyses will, in this case, be used for the design. Alternatively, some codes (EC8-1, 2004; NZS1170.5, 2004) propose using seven records and consider for design the average values of the same selected engineering demand parameters. However, it should be highlighted that ground motions with apparently similar response-spectrum characteristics may yield significant variations of the inelastic response, mainly due to period shifts. Various studies have shown that in order to achieve a reliable, stable probabilistic evaluation of the seismic random event, the time-history analysis should be repeated with hundreds of records. For this reason, many codes or design guidelines allow or propose the use of a nonlinear time-history analysis, preferably as a verification method, and not as a direct design method, of the results of other simpler techniques such as equivalent static analysis, dynamic modal analysis or nonlinear static (pushover) analysis.

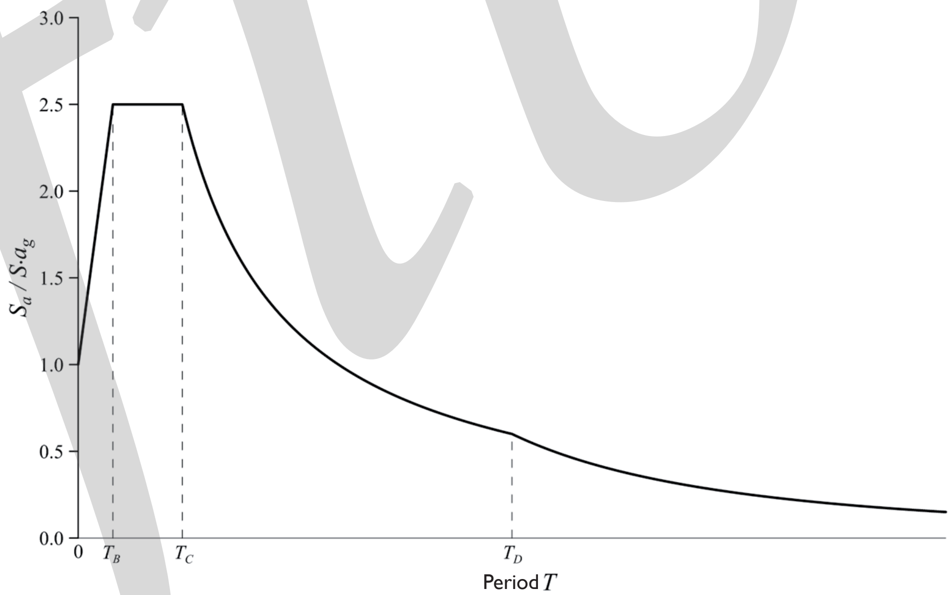


Fig. 3-2 Typical elastic-acceleration-design spectrum (5% damping)

3.1.3.2 Elastic-displacement spectrum

A design displacement spectrum can be directly used when either:

- using a displacement-based design approach (see Appendix C for design examples); or
- aiming at directly checking the displacement demand of the SDOF equivalent designed based on the previously mentioned force-based design.

The elastic acceleration design spectrum provided by any code can be converted to a design (pseudo-)displacement spectrum using the relation in Equation 3-1, which is valid for zero damping and approximately valid for the small values of damping commonly used in engineering structures (up to 20%):

$$S_d = \frac{S_a(T)}{\omega^2} \tag{3-1}$$

where
S_d is the spectral displacement
S_a is the spectral acceleration
ω is the natural frequency given by $\omega = 2\pi / T$
T is the structural period

Therefore, the spectral displacement can be defined by: $S_d = \dfrac{S_a(T) \times T^2}{4\pi^2}$

It is worth noting that the symbol S_a and S_d are also referred to as S_{ae} and S_{de} to clarify that they represent the spectral ordinate corresponding to the elastic spectra.

Note that recent studies on displacement spectra for long periods (Faccioli et al., 2004) suggest that the displacement demand be capped at a certain level (plateau), starting at the corner period T_D (which itself should be a function of the magnitude and distance of the earthquake event) and ending at the period T_E, a function of the soil conditions, and then reduced to the level of the peak ground displacement d_g for very long periods T_F (see Fig. 3-3). Formulas for the peak ground displacement d_g are usually provided in the codes. In Eurocode 8 (2004), the ground displacement is defined as:

$$d_g = 0.025 \times S \times a_g \times T_C \times T_D$$

Suggested values for the characteristic periods T_E and T_F are also provided in Appendix A of Eurocode 8 (2004), depending on the ground category. The suggested values for T_E range from 4.5 seconds (for stiff soils, such as rocky or rock-like geological formations with at most 5 metres [16.4 ft] of weaker material at the surface) to 6.0 seconds (for soft soils, such as deep deposits of dense or medium dense sand, gravel or stiff clay with a thickness of several tens to many hundreds of metres and very bad soils, such as deposits of liquefiable soils or sensitive clays). The suggested values for T_F is 10 seconds for all types of soils. However, it is important to note that more detailed information on the local seismicity might propose the adoption of different values for the corner period T_D.

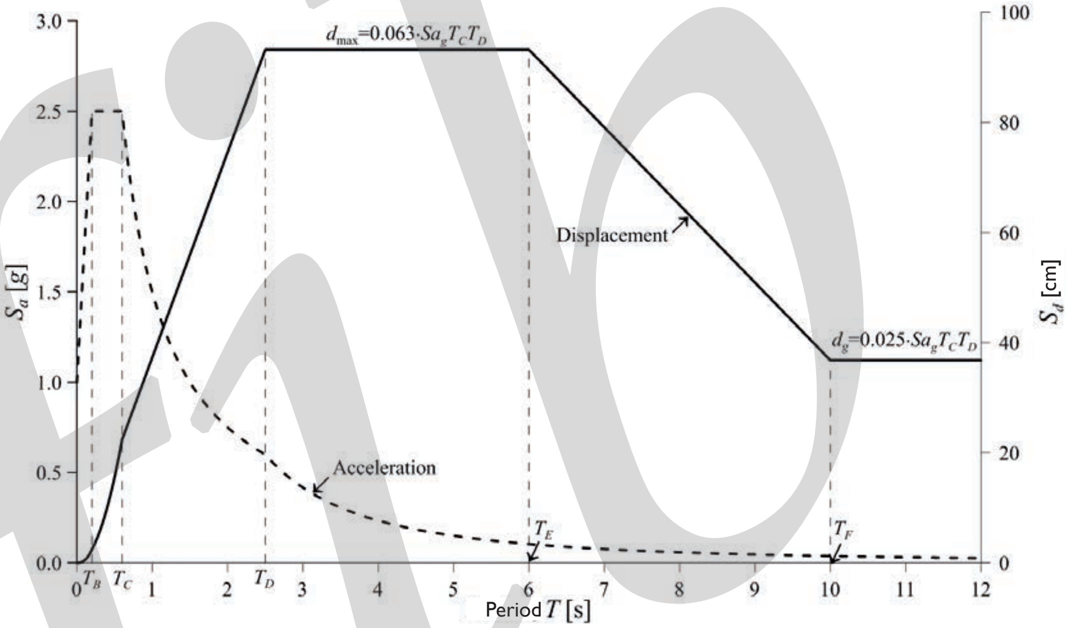


Fig. 3-3 Displacement spectrum from acceleration spectrum for S × a_g = 1 g, T_B = 0.20 s, T_C = 0.60 s, T_D = 2.50 s, T_E = 6.0 s and T_F = 10.0 s (5% damping)

3.1.4 Equivalent damping for hysteretic response

The dissipation of energy due to inelastic deformation (hysteretic response) can be accounted for by reducing the elastic design spectrum in accordance with the equivalent viscous damping coefficient $\xi_{eq} = \xi_{el} + \xi_{hyst}$ where ξ_{hyst} is the additional damping (beyond the elastic damping ξ_{el}) associated with the total energy dissipation due to the yielding of the system. This in turn is related to the ductility demand of the system.

To calculate the spectral values for the equivalent damping from the elastic spectrum for 5% damping, reduction coefficients are typically used. The European seismic code Eurocode 8 (2004) allows the elastic design acceleration and displacement spectra to be reduced by the so-called damping correction factor η, which is defined as follows (with ξ_{eq} in percentage):

$$\frac{S_a(T, \xi_{eq})}{S_a(T, \xi_{eq} = 5\%)} = \frac{S_d(T, \xi_{eq})}{S_d(T, \xi_{eq} = 5\%)} = \eta = \sqrt{\frac{10}{5 + \xi_{eq}}} \tag{3-2}$$

An alternative reduction factor presented in Eurocode 8 (1998) and supported by Priestley et al. (2007) for its better compatibility with real earthquake acceleration records is given in Equation 3-3 below:

$$\eta = \left(\frac{7}{2 + \xi_{eq}}\right)^{\alpha_{SF}} \tag{3-3}$$

where $\alpha_{SF} = 0.25$ for sites located close to a major fault with ground motions affected by near-fault directivity phenomena and $\alpha_{SF} = 0.50$ for sites located elsewhere.

3.1.5 Design concept

3.1.5.1 Force-based design

According to the force-based seismic design approach (FBD), structures can be designed to behave inelastically with a strength capacity (design seismic forces) lower than that required for a fully elastic response to the design earthquake. The life-safety performance level is achieved from the ductility and overall deformation capacity of the structure, which in turn can be achieved with proper detailing.

In this sense, the forces corresponding to the design seismic action are derived by dividing the elastic ones by a reduction factor denoted by *R* (reduction factor) or *q* (behaviour factor used in Eurocode 8, 2004) or directly by a ductility factor μ, as shown in Figure 3-4 for *q* (or *R*) = 4.0 (solid line). As the structure is expected to experience plastic deformation during the design earthquake, an overall ductile behaviour type should be ensured in order to comply with the life-safety (in other codes, ultimate-limit-state) criterion. This is achieved through the proper dimensioning and detailing of the structural elements and connections that should provide the structure with an adequate capacity to deform beyond its elastic limit without a substantial reduction of the overall resistance against horizontal loads and the capacity to carry vertical forces. In addition, capacity design concepts are applied to ensure that the ductile modes of failure at both global and local levels (for instance, beam-sway and flexural hinging) precede the brittle modes of failure (for instance, column-sway and shear failure) with sufficient reliability.

The appropriate value of the behaviour factor q (or Reduction factor R) that should be used in the analysis depends on the material and the structural system and technology. In some codes, alternative maximum values for the behaviour factor are allowed, thus permitting ductility and dissipation capacity to be traded for strength. For example, in Eurocode 8 (2004) and for buildings in areas with significant seismicity, one can design a structure either for a ductility class medium (DCM) or a ductility class high (DCH). In the former case, a smaller value of the behaviour factor q is used than in the latter. In a design for DCM, certain detailing rules are relaxed, making the design of slightly easier-to-build structures possible, which structures, however, provide good performance during moderate earthquakes. DCH requires stricter detailing criteria that ensure large plastic deformation capacities related to the high values of the reduction factor and, thus, provides higher safety margins against local or global collapse.

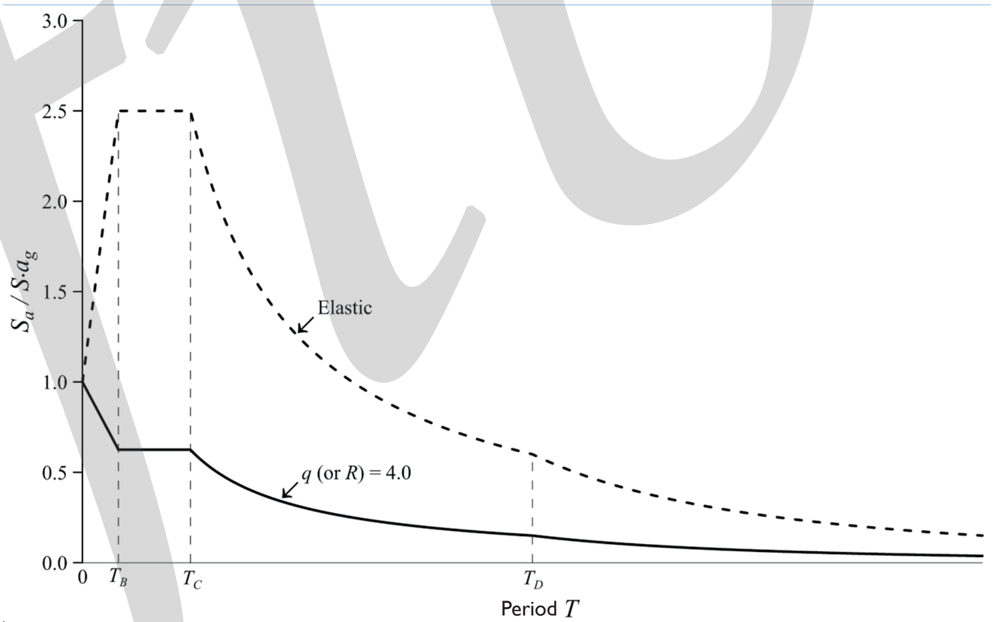


Fig. 3-4 Typical pseudo-inelastic acceleration design spectrum, derived from the elastic spectrum reduced by the q or R factor (5% damping)

A force-based design procedure follows these steps (see Fig. 3-5):

- evaluating or estimating the 'initial' or 'fundamental' period of the structures (or multi-periods if a modal analysis is followed)
- entering the acceleration spectrum from the X-axis (periods)
- selecting the corresponding damped or inelastic acceleration spectrum curve
- deriving the corresponding spectral acceleration $S_a(T)$
- calculating the design base shear, overturning moment and internal design forces

It is worth noting that too often the force-based design procedure is concluded after the base shear based on the first estimation of the period has been evaluated. However, as a result of an incorrect evaluation of the initial period, a force-based design procedure without an appropriate iteration on the initial period and to the reduction factor (as per a common-practice FBD approach) can lead to variable results, leading to an underestimation:

- of the actual displacement demand, which could lead to exceeding strain limits in members, being exceeded, pounding against adjacent buildings or not meeting code drift requirements, *or*
- of the base shear requirements, with possible unexpected failure mechanisms in the superstructure or foundation level.

In fact, the 'actual' stiffness (secant to the yielding of the equivalent SDOF elastoplastic system) of the structure, thus the 'actual' period to be used when entering the acceleration spectrum, varies with the base shear (which is not known at the beginning of the design process), while the yielding displacement remains approximately constant for given geometrical properties. The initially assumed value of the period, and resulting base shear, should thus be adjusted and iterated on until convergence is reached. An example of such an iterative process is shown in more detail in Appendix C.

3.1.5.2 Displacement-based design

In a displacement-based design approach (DBD) (see Fig. 3-6) the maximum accepted displacement/drift is the main objective of the design and, thus, the starting point of the procedure, rather than a secondary parameter to be checked post-design, as it happens in a force-based design approach. Rather than an acceleration spectrum, an elastic (pseudo-) displacement spectrum is used and derived following Equations 3-1 and C-5 (in Appendix C).

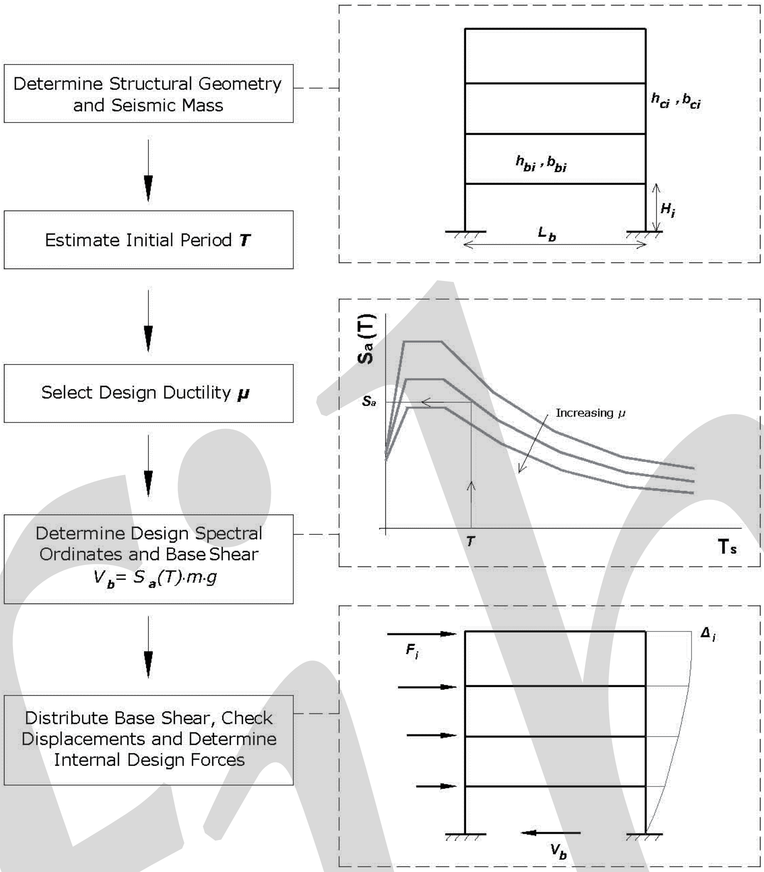


Fig. 3-5
Force-based design procedure (based on Sporn and Pampanin, 2013)

Entering the spectrum with the design target displacement Δ_d (see Fig. 3-6d) and the assumed level of hysteretic damping (see Fig. 3-6c), the effective period T_{eff} and thus the corresponding design effective stiffness K_{eff} are evaluated. This stiffness represents (as indicated in Fig. 3-6b as K_e) the secant stiffness to the target displacement of a single-degree-of-freedom (SDOF) system equivalent to the original multi-degree-of-freedom system (see Fig. 3-6a), for instance, a multistory frame or wall. The design-base shear for the whole structure is then obtained by multiplying the secant stiffness and the target displacement: $V_b = K_{eff} \Delta_d$. The yield displacement Δ_y is, with good approximation, constant regardless of the required base shear and is a function only of the geometrical characteristics of the structural system.

Following this approach, a more direct control of the structural response can be guaranteed and different performance levels (such as limit states) can be achieved under varying

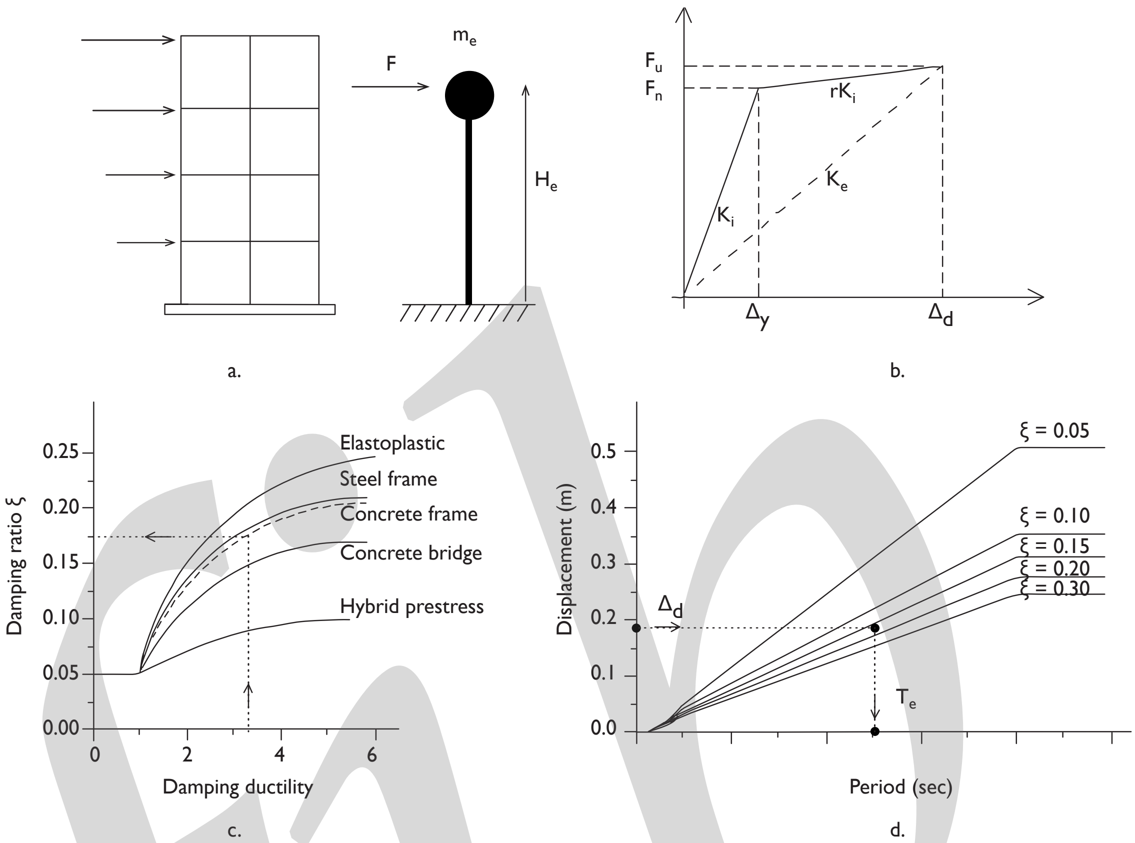


Fig. 3-6 Displacement-based design (DBD) (based on Priestley at al., 2007)
a. SDOF simulation
b. Effective stiffness Ke
c. Equivalent damping vs ductility
c. Design displacement spectra

intensities of the seismic input motion. Figure 3-7 illustrates the general flow chart of a displacement-based design procedure.

It is worth noting that both alternative design approaches (force-based or displacement-based) aim at evaluating, through different hypotheses and procedures, the design base shear to be applied to the structure before proportioning and detailing the structural elements. Once the base shear is evaluated, the design internal forces are obtained by distributing them over the height of the building (for instance, according to the first mode shape, as typically suggested by the design codes or, more appropriately, using an equilibrium approach, as proposed by Priestley et al. (2007). When proportioning the structural members, capacity design (hierarchy of strength) principles should be followed, so that the occurrence of inadequate global collapse mechanisms (for instance, weak-column/strong-beam systems with a tendency to a soft-story mechanism) or brittle local failures (such as shear failure) can be avoided.

In Appendix C a full design example of an SDOF system - a one-story industrial building with hinged connections - is presented with a 'traditional' force-based design (FBD), a displacement-based design (DBD) (based on a formulation by Priestley et al., 2007) and a closed-form solution for an iterative FBD (based on a recent proposal by Sporn and Pampanin, 2013).

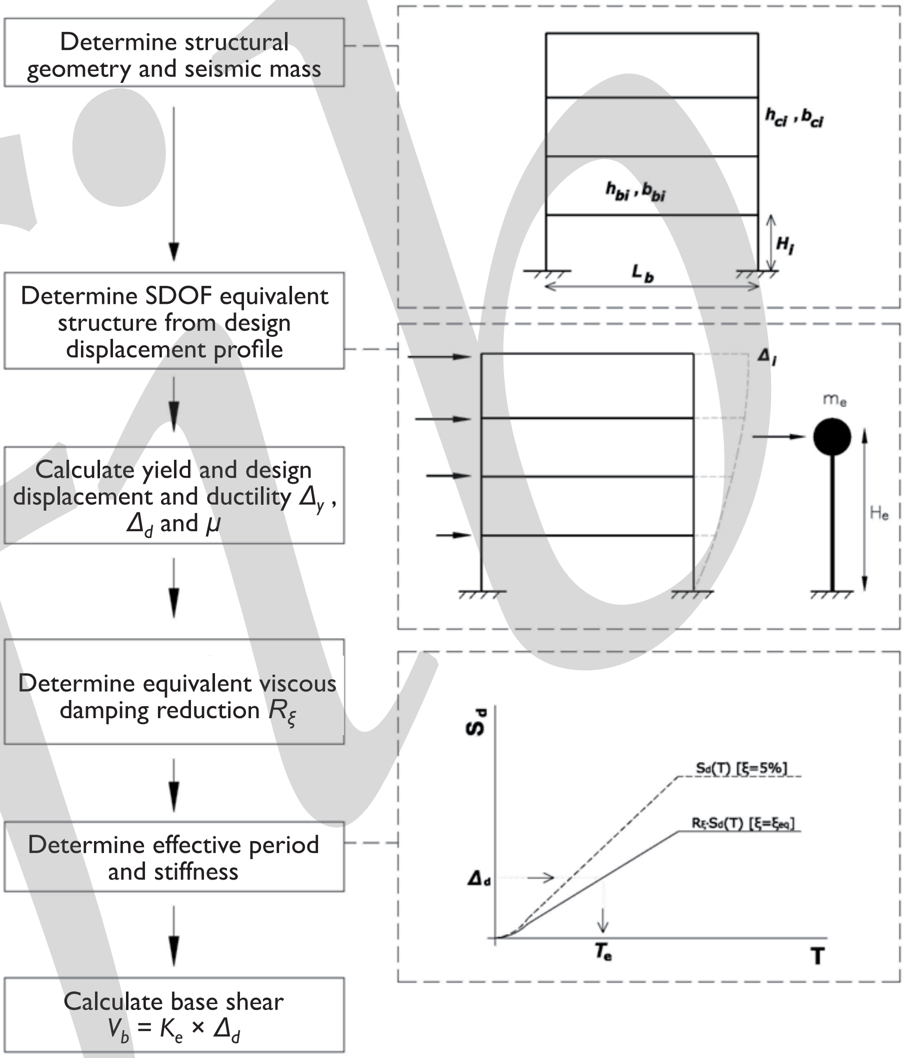


Fig. 3-7 Displacement-based design procedure (based on Sporn and Pampanin, 2013)

3.2 The case of precasting

The major difference between traditional cast-in-situ reinforced-concrete structures and precast structures is that the latter are composed of various members cast in a different place from their final position in the structure and, therefore, the structure consists of a set of 'elements' and 'connections'. In precast concrete construction, the response of the total structure under seismic action is largely controlled by the behaviour of the connections. Special attention has to be paid to developing adequate structural detailing and connection solutions in the primary seismic resisting systems (including connections with the foundations), between floor/diaphragm/roof and seismic resisting systems, between structural and nonstructural elements, as well as in the secondary (gravity-only or gravity-dominated) structural systems, with respect to the deformation and displacement compatibility of the overall structure.

However, precast structures have passed the test of moderate and strong earthquakes and have been subjected to extensive experimental and analytical/numerical investigations performed at prominent international research centres. From these experiences, some key conceptual design aspects have been identified as critical to the good seismic performance of precast structures. Some of these key aspects are listed below. They do not cover all the design aspects at this stage but refer to the main points in the seismic behaviour of precast structures that should be carefully considered in the design phase.

3.2.1 Performance-based design philosophy

The behaviour of precast structures in earthquakes is governed by the types of connections used, which affect their strength, stiffness, deformability and ductility.

A proper design could, in principle and depending on a specific configuration, achieve levels of horizontal displacement, stiffness and seismic response similar to monolithic systems at serviceability and ultimate-limit states.

In the case of structural systems with hinged beam-to-column connections (see Section 5.2) and fixed (moment-resisting) column-to-foundation connections, both typically adopted in the Mediterranean countries and elsewhere (in Chile, for example), the column cross sections and possibly the total reinforcement content are larger than those required for equivalent monolithic systems for the provision of sufficient lateral-load resistance.

In the United States emulative design is commonly followed and connections have to be evaluated based on code (see Section 3.2.3e).

In all cases, the deformations should be evaluated via elastic analysis to verify that the requirements for the damage-limitation limit state are met. To this end, the interstory drift should be evaluated as the difference of the lateral displacements at the top and bottom of the storey under consideration and compared to the limiting value. Usually, the interstory-drift limits range from 0.005h (or 0.5% drift) to 0.01h (or 1% drift), with *h* equal to the storey height, depending on the characteristics of the nonstructural elements and their connection to the primary structure.

Lower values may apply to the brittle elements. Upper values may apply to elements that are connected to the structure in such a way as not to interfere, up to certain level of relative displacement, with the deformations of the main structure. Intermediate values may be used for ductile elements.

When one is dealing with inherently more flexible hinged connections systems, it is important to note that the fulfilment of the requirements of damage limitation may lead to structural member sizes that are larger than those required to prevent collapse. In other words, serviceability-limit-state (SLS) design considerations are likely to control the design of precast industrial buildings versus ultimate-limit-state (ULS) design requirements.

3.2.2 Connections

Connection details that rely solely on the friction caused by gravity loads must not be used. This is often the case in earlier construction practice based on hinged-connections with beams simply supported on columns with no types of horizontal connections and no means of shear transfer other than through friction. Under earthquake shaking, the supported element can jolt out of the bearing and become unseated. This can affect supports both between beams and columns and between floor elements and beams, leading to a partial or total collapse of the structure. Dry bearings with interposed rubber pads and with no other connector can provide a support between beams and columns sufficient for static actions (such as gravity and wind) but they must be excluded in seismic applications, where other forms of mechanical connections are required, namely those that can transmit horizontal actions without relying on the aforementioned shear friction mechanisms.

3.2.3 Ductility properties of the structure

a. Every type of structure built in a seismic area should possess adequate ductility or, in general terms, a deformation capacity consistent with the material and the structural system chosen, the seismicity of the area in which building will take place and the reduction factor *R* (or behaviour factor *q*) used in the analysis and the corresponding detailing.

b. In the case of precast concrete structures, the overall mechanism and behaviour of the structure, including its energy dissipation capacity, is much affected by the type and position of the connections in the structural system.

c. Therefore, in the modelling and detailing of precast structures, the identification of the effect of the connections on the overall behaviour of the structure should also be made, while all the general requirements for moment-resisting frames built with cast-in-situ concrete will be followed according to code specifications unless more specific guidelines are provided, as in the case of jointed ductile connections in the New Zealand Concrete Standard (NZS3101:2006) (see Appendix B).

d. When using precast frame systems, the following distinctions should also be made between the connections, as shown schematically in Figure 3-8, based on EC 8 (2004):

- connections located well outside the critical regions of the structural members, not affecting the energy dissipation capacity and the inelastic response of the structural system (see Fig. 3-8a)
- connections located within critical regions but adequately overdesigned with respect to the rest of the structure, so that under seismic loading they remain elastic while the inelastic response occurs in critical regions adjacent to the connections (see Fig. 3-8b)
- connections located within critical regions, provided with substantial ductility capacity such that the inelastic response can be concentrated in the connection itself (see Fig. 3-8c)
- hinged beam-to-column connections (see Fig. 3-8d) in frames with fixed (moment-resisting) column-to-foundation connections

If this approach is followed, a more direct control of the structural response can be ensured and different performance levels can be achieved under varying intensities of

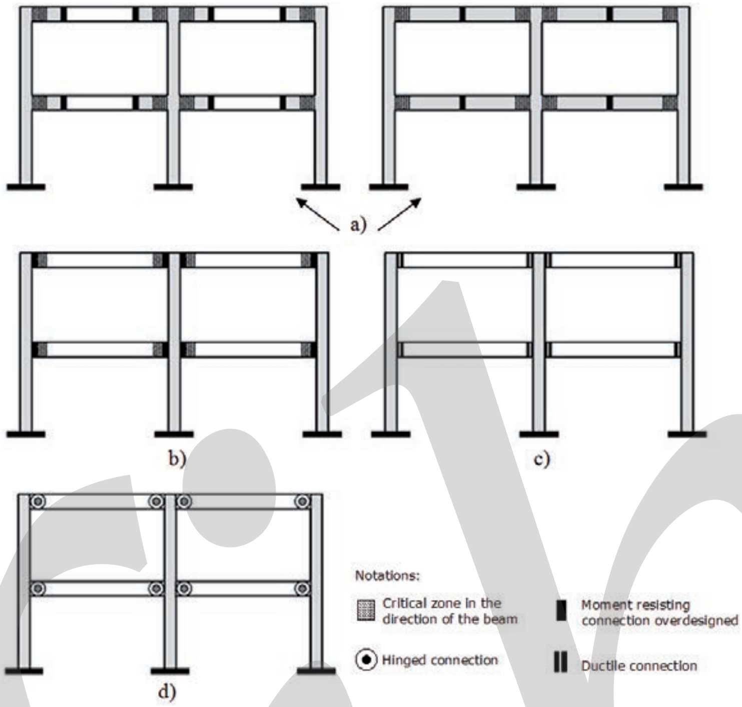


Fig. 3-8

Distinctions between connections depending on their location in the structural system
a) Beam-column connections well outside the critical regions
b) Beam-column connections inside the critical regions but overdesigned
c) Beam-column connections inside the critical regions and ductile
d) Hinged beam-column connections and moment-resisting connections at the base columns

the seismic input motion. Figure 3-7 illustrates the general flow chart of a displacement-based design procedure.

It is worth noting that both alternative design approaches (force-based or displacement-based) aim at evaluating, through different hypotheses and procedures, the design base shear to be applied to the structure before proportioning and detailing the structural elements. Once the base shear is evaluated, the design internal forces are obtained by distributing them over the height of the building (based, for example, on the first mode shape, as is typically proposed in design codes or, more appropriately, an equilibrium approach, as proposed by Priestley et al. [2007]). When proportioning the structural members, capacity-design (hierarchy-of-strength) principles should be followed, so that the occurrence of inadequate global collapse mechanisms (such as weak-column/strong-beam systems prone to a soft-story mechanism) or brittle local failures (such as shear failure) can be avoided.

In Appendix C a full design example of an SDOF system - a one-story industrial building with hinged connections - is presented with a 'traditional' force-based design (FBD), a displacement-based design (DBD) (based on a formulation by Priestley et al., 2007) and a closed-form solution for an iterative FBD (based on a recent proposal by Sporn and Pampanin, 2013).

In Eurocode 8 (2004) the behaviour factors of precast structures q_p are roughly expressed in the following equation:

$$q_p = k_p \times q \geq 1.5 \tag{3-4}$$

where
q_p is the value of the (maximum acceptable) behaviour factor to be applied to the considered precast structure;
q is the (maximum acceptable) value of the behaviour factor of an equivalent monolithic structure, depending on the type of structural system and on its regularity in elevation. For buildings that are not regular in elevation, the value q should be reduced by 20%;
k_p is the reduction factor related to the connection properties (for example, k_p = 1.0 for connections complying with the rule in point b. above; for other experimental verified connections k_p = 0.5 to 1.0).

Figure 3-9 is a flowchart depicting the selection of the k_p factor.

More detailed information about the maximum values of the adoptable and the actual behaviour factors resulting from the design (compatible with the strength-stiffness proportionality of a concrete section and an overall structural system) can be found in Appendixes A and B.

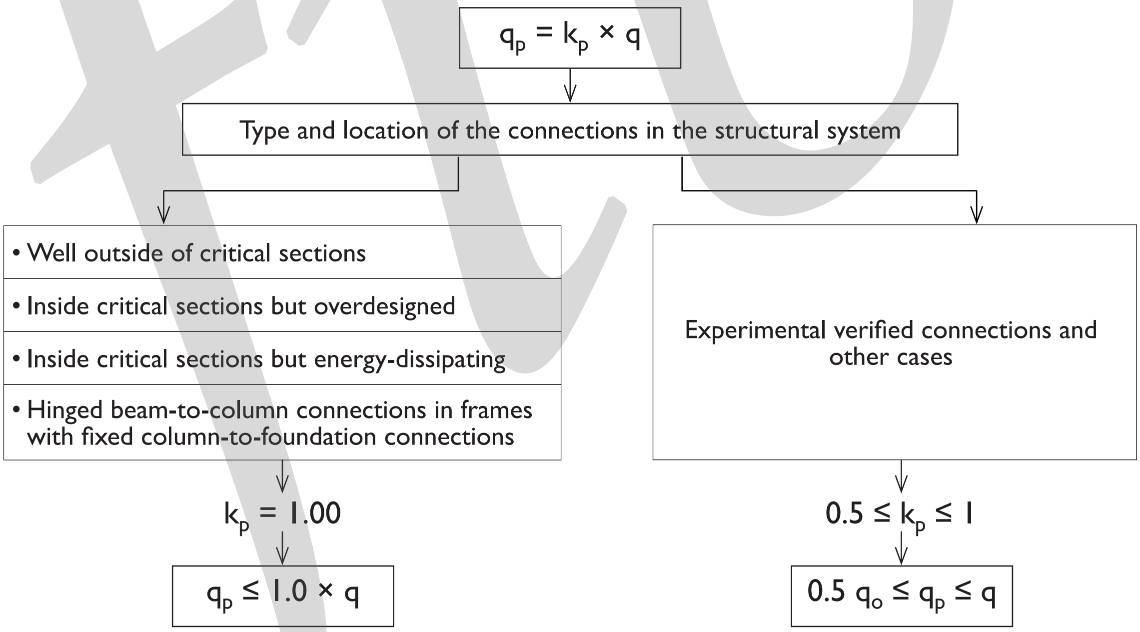


Fig. 3-9 Selection of the k_p factor

e. Chapter 21.8 of ACI 318 (2011) refers to 'special moment frames constructed using precast concrete.' Clause 21.8.2 and 21.8.3 give detailing provisions are given for frames that respond to design displacements in essentially the same way as monolithic special frames.

- Clause 21.8.2 refers to special moment frames with ductile connections. In this case:

  - all the requirements for special moment frames built with cast-in-situ concrete should be kept, and
  - the nominal shear strength V_n should not be less than $2V_e$ where:

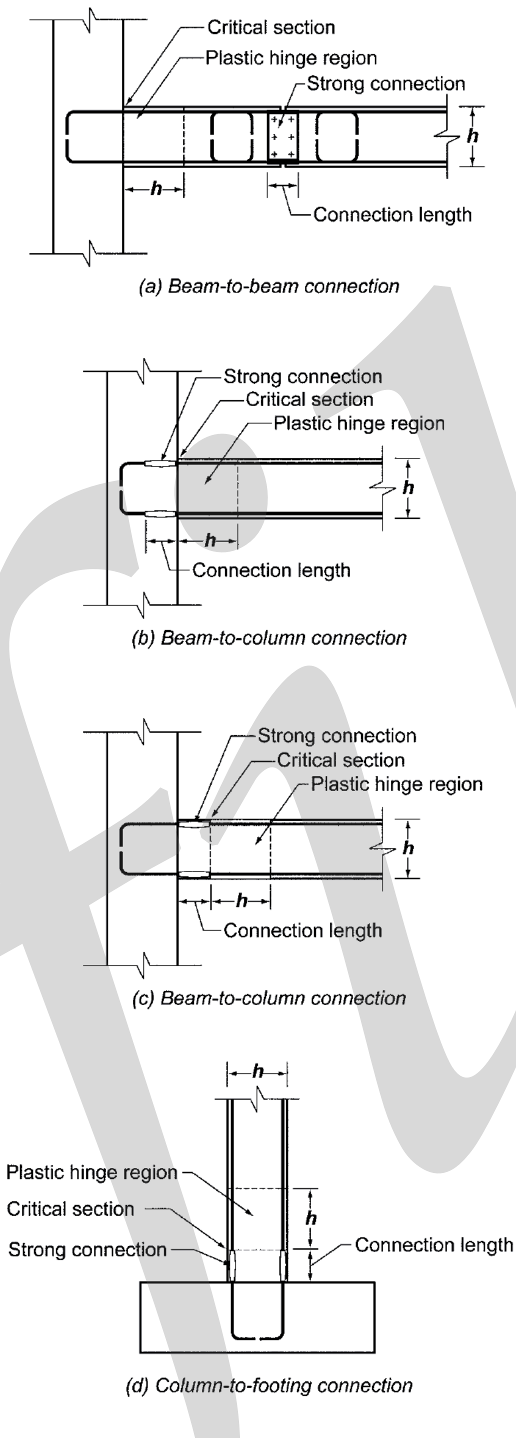
$V_n = A_{vf} \times f_y \times \mu$, with A_{vf} = area of shear friction reinforcement, mm² (in²)
f_y = special yield strength of reinforcement, MPa (psi)
μ = coefficient of friction in accordance with clause 11.6.4.3
V_e = the design shear force, which should be determined by taking into consideration the effect of the statical forces on the portion of the member situated between the faces of the joints. It should be assumed that moments of opposite sign corresponding to the probable flexural moment strength M_{pr} act at the joint and that the member is loaded with the factored tributary gravity load along its span.

Clause 21.8.3 refers to moment frames with strong connections.

In this case all the requirements for special-moment frames built with cast-in-situ concrete should be kept, as well as the additional requirements a, b, c, and d, represented schematically in Figure 3-10.

3.2.4 Supports

The correct positioning and proportioning of the supports in a precast structure represent a fundamental requirement in both seismic and nonseismic applications. The stability of elements depends on this support arrangement. Considering that the connections of precast structures can significantly affect the behavior of the overall structural system, proper criteria must be applied for their dimensioning in seismic applications.



'21.5.1.2 — Clear span for member [...] shall not be less than four times its effective depth.' (ACI 318 [2011], p. 340)

'21.8.3 [...] (b) Design strength of the strong connection, ϕS_n, shall be not less than S_e' (ACI 318 [2011], p. 354)
where:
'ϕ - strength reduction factor' (ACI 318 [2011], p. 27)
'S_n - nominal flexure, shear, or axial strength of connection' (ACI 318 [2011], p. 25)
'S_e - moment, shear, or axial force at connection corresponding to development of probable strength at intended yield locations, based on the governing mechanism of inelastic lateral deformation, considering both gravity and earthquake effects' (ACI 318 [2011], p. 25)

'21.8.3 [...] (c) Primary longitudinal reinforcement shall be made continuous across connections and shall be developed outside both the strong connection and the plastic hinge region' (ACI 318 [2011], p. 354)

'21.8.3 [...] (d) [...] ϕS_n shall not be less than $1.4S_e$. [...] ϕM_n shall be not less than $0.4M_{pr}$ for the column within the storey height, and ϕV_n of the connection shall be not less than V_e determined by 21.6.5.1.' (ACI 318 [2011], p. 354)
'M_n = nominal flexural strength at section, N × mm' (ACI 318 [2011], p. 23)
'M_{pr} = probable flexural strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile stress in the longitudinal bars of at least $1.25f_y$ and a strength reduction factor, ϕ, of 1.0, N × mm (in × lb)' (ACI 318 [2011], p. 23)

Fig. 3-10 Strong connections (illustration on left reproduced form ACI 318, 2011, with permission of ACI)

3.2.5 Second-order effects

Given the inherent flexibility of frame systems used for industrial buildings, an important design aspect is second-order or *P-Δ* effects, which may lead to extensive structural collapse, as occurred in reinforced-concrete (RC) structures of all types, precast or not during the Izmit earthquake of 1999 in Turkey (Wallace, 1999). The design of excessively slender columns without explicit deformation limits and without due consideration to second-order effects caused by the large displacements expected under seismic action can lead to catastrophic collapse. When designed with the maximum allowed force-reduction factor *R* (or behaviour factor *q*) related to a specific structural system, an early collapse before the attainment of ultimate strengths of the critical sections can occur prior to the mobilization of the overall ductility capacity and, thus, the related force-reducing capacity of the systems.

Special care is needed to evaluate and control *P-Δ* effects on the critical base-column moment-resisting connections, particularly when adopting hinged beam-column connection solutions. This would require a correct evaluation of the expected displacement demand *Δ* of the primary lateral load-resisting system. Underestimation of such a displacement demand due to the incorrect evaluation of the initial (secant to yielding) stiffness and the yielding point of the structure could thus have very dramatic consequences. More information on this critical step can be found in Appendix C.

3.2.6 Cladding-panel connections

In the United States such exterior cladding walls are typically employed as shear walls, which resist lateral forces. However, in Europe, for structural analysis, it is common practice to assume a seismic-resisting system with a bare frame (structural skeleton) consisting of columns, beams, floors and a roof, in which the peripheral cladding panels (assumed as nonstructural elements) are used as masses without any rigidity. Following such an analysis, the frame elements (structural components) are designed in terms of strength, ductility and deformations, while the cladding panels (non-structural components) are connected to the structure with fixed fastenings dimensioned only for local actions. Experiences in recent earthquakes have shown that this practice needs to be improved. In fact, while the seismic behaviour in general of the structures themselves was good, the failure of connections, which resulted in the collapse of heavy cladding panels, created severe, life-threatening situations in a number of cases. If the structural analysis is based on a model consisting of a bare frame only, the connections of the cladding system must be designed to effectively allow the large relative displacements between the primary structural system and the non-structural components calculated from the analysis. On the other hand, if the cladding panels are fixed to the structure, they must be included with their stiffness as an integral part of the structural system in the analysis; the overall structural design including the design of these connections must then be based on the results of such an analysis.

3.2.7 Shear failure

Another significant concern is the shear failure of short (or captive) columns, which has been fairly common in all types of RC systems, precast or cast-in-situ, subjected to earthquakes. In some cases, the seismic analysis of precast buildings may consider the bare structure only, neglecting the stiffening contribution of the cladding walls and, in general terms, the interaction between the structural systems and the 'nonstructural' elements such as internal partitions or external claddings. This contribution is actually relevant and often leads to higher resistance. However, there are situations in which the presence of cladding walls could lead to a different structural response, possibly characterized by much lower ductility and fewer local brittle mechanisms. This is the case when an interior heavy partition/infill wall or cladding wall is rigidly fixed to the columns, extending beyond the spandrels. The effective length of the column between the wall panels becomes shorter with a higher stiffness, which draws higher seismic forces, and has brittle shear behaviour. This phenomenon is also referred to as a short or captive column mechanism. However, if isolated from the cladding walls with a proper allowance for a relative displacement between the two elements, the columns are likely to have adequate flexural ductile behaviour.

3.2.8 Design of diaphragms

Chapter 8 has further information about this topic.

In precast structures, the precast-floor system consists of many distinct gravity-load components. These components are often designed as simply supported, single-span elements and are characterized by rather large spans and the use of prestress.

Floor and roof systems play an important role in the lateral response of the structure, providing diaphragm action, which is needed to:

- ensure the uniform response of the structure without in-plane distortions; and
- transfer the lateral forces to the lateral force-resisting elements.

Precast-floor and roof diaphragm systems are as varied as the components that form the horizontal framing and may be designed to develop their diaphragm action by relying either on a topped or untopped (precast-floor) solution.

Special care should be taken in cases where:

- large openings are located in the vicinity of the vertical structural elements that may affect the effectiveness of the diaphragm to transfer the horizontal forces to these vertical elements;
- considerable changes (greater than 50%) in diaphragm stiffness occur from one storey to the next, which may lead to undesirable seismic response; and
- diaphragm aspect ratios are large (≥ 4.0).

Rigid diaphragm behaviour may be desirable at the intermediate floors of a multi-story building so as to preserve the integrity of the nonstructural elements, such as partitions supported by the above-mentioned floors.

Special care should be taken to ensure proper connections between diaphragm elements and between the diaphragms themselves and the lateral-resisting system.

When diaphragms are not rigid, the in-plane stiffness of the diaphragm and its dynamic interaction with the lateral resisting system should be considered in the structural analysis. For the connections, capacity design rules should be applied.

3.2.9 Stability of beams supported on columns

An important case of structural deficiency in hinged precast frames is related to the potential for the unseating of the beams from their supports over the columns. These supports must prevent the possible lateral overturning of the beams that can be induced by seismic actions. Strong transverse horizontal forces can act together with vertical shaking, which decreases the stabilizing effects of gravity loads. Furthermore, nonsynchronous motion between the columns can lead to an excessive displacement demand and the unseating of beams from their support. Therefore, adequate lateral supports that are able to stabilize the beams even in the absence of gravity forces must be provided, which will prevent unseating.

3.2.10 Structural integrity of precast structures - Ties

In all precast structures, a suitable interaction should be established between the bearing elements of the system (beams-columns-walls) and the floor system to ensure the structural integrity of the whole edifice. In order to achieve the latter, connections leading to continuity-of-force transfer (robust-load path) should be provided between all the structural parts of the systems. This is usually obtained through a three-dimensional grid of ties (see Figure 3-11, top). Other typical solutions are bolting or grouting dowels in sleeves in the beams.

Ties are generally continuous tensile elements consisting of reinforcing bars (of proper length and size) placed in longitudinal, transverse and vertical directions in the cast-in-situ topping, infill strips, sleeves or joints between precast elements. Their role is not only to transfer forces (between units) originating from earthquakes but also to provide a level of redundancy in the structural system against the unexpected failure of single connections and elements under higher-than-expected seismic actions, as well as additional safety to the structure to withstand to a certain extent actions due to accidental loading (such as gas explosions and collisions). The function of ties can also be provided by the reinforcement in the elements when due continuity is ensured through their connections.

- In Figure 3-11 a general arrangement of ties and some details of ties in a frame structure are shown for a floor system consisting of prestressed hollow-core units with topping:
  - Floor elements are tied together (1).
  - Load paths for internal diaphragm action are allowed to possibly tie floor units together across beams (2).
  - Ties are perpendicular to beams (also potentially acting as starter bars or shear connectors between floor and lateral resisting systems) (3).
  - Ties are situated along the beam between slabs (4).
  - Ties are situated in the direction of the precast units (in their support) and perpendicular to the beams (5) or along the beams on which precast units sit (6)
  - Ties are perpendicular to the direction of the precast units, which run along a peripheral beam. In frames with moment-resisting beam-to-column connections, the development of plastic hinges in the beams can lead to beam elongation effects. In this situation, the use of detailing as indicated in 7a is to be limited to low-seismic regions and low-ductility demands at the beam-to-column connections. Detailing as indicated in 7b would be a preferred option instead for moderate-high seismic regions and/or high ductility demands at the beam-column connections.
  - Ties are situated along the peripheral beam (8).

- Additional ties are situated in the corner of a building with the floor and beams around the internal part of the column (9).
- Additional ties are situated between a column and the floor when there are no transverse (perpendicular to a one-way frame) beam frames in the column (10).

Ties 4, 6 and 8 should be continuous around parts of the whole floor.

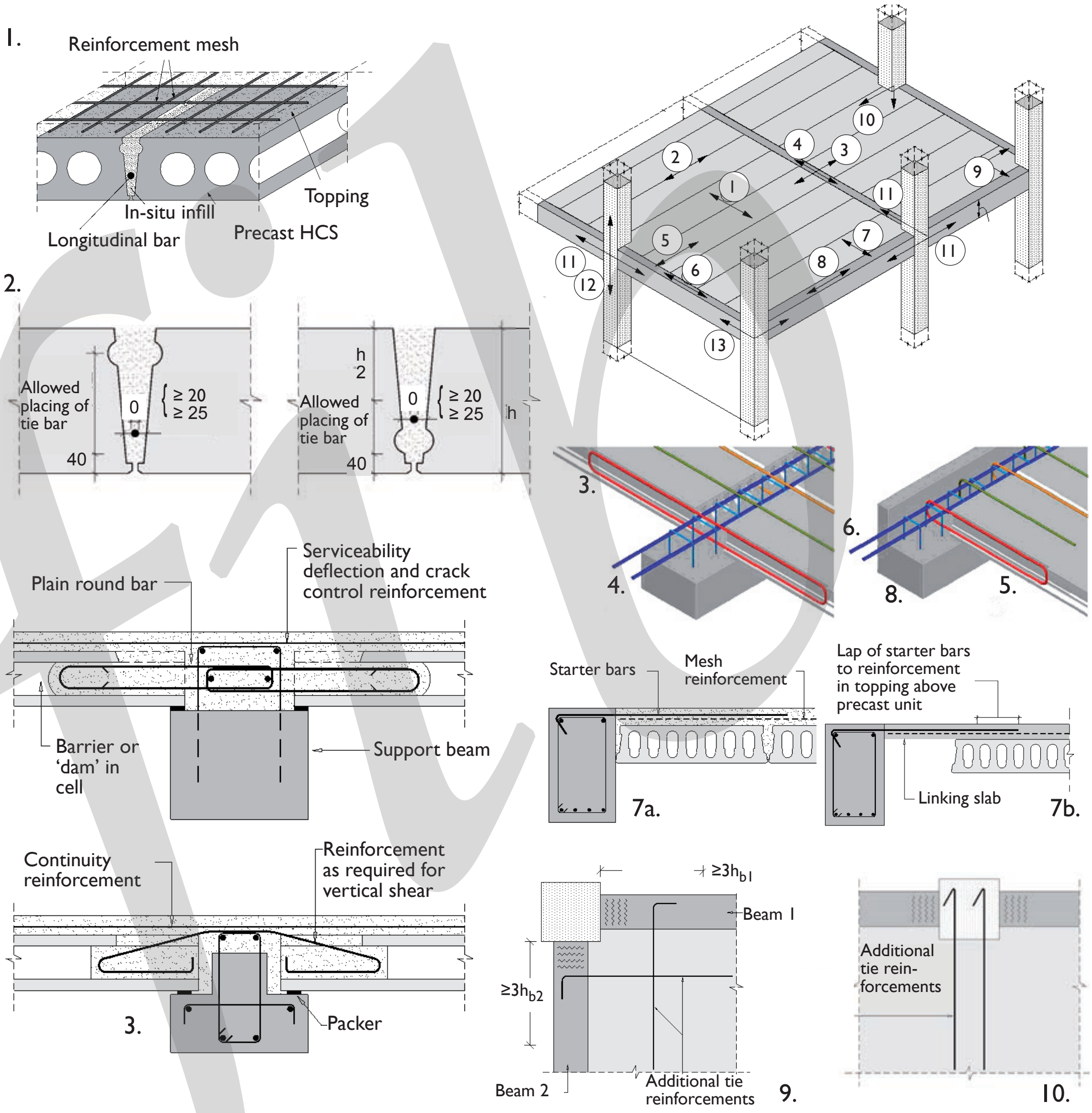


Fig. 3-11 Location and types of ties in precast skeletal structures in which the floor contains prestressed hollow-core slabs

Generally in precast frame systems under seismic actions, ties (11), (12), and (13), are in general provided directly by the beam-to-column and column-to-column connections.

In the case of lateral resisting systems consisting of shear walls, the floor should also be properly connected/tied with the walls in such a manner to transfer the lateral forces while allowing for the (vertical) displacement incompatibility between floor and wall (due to the uplifting of the wall when forming a plastic hinge at the base).

- In ACI 318 (2011) there are specific code provisions for structural integrity in precast concrete construction that are based directly on research conducted by the Portland Cement Association (PCA) and that take into consideration the notional removal of walls in large panel (load-bearing) wall structures. The general configuration of the conventional assembly of large panel structures was evaluated to establish alternate load path mechanisms for this class of building. These alternate mechanisms were subjected to design and experimental verification. From this research, it was possible to develop a minimum detailing practice for vertical, horizontal, and transverse ties to establish a suitable degree of continuity and ductility, ensuring integrity through indirect design. This approach was broadened to all forms of jointed precast construction by requirements in ACI 318 (2011) with general provisions for integrity ties (see Section 6.8.3).

3.3 Basic principles of conceptual design (to satisfy the fundamental requirements of collapse avoidance and damage limitation)

3.3.1 General

A satisfactory seismic performance of any structural system can be achieved by adopting proper earthquake-resistant design principles.

The design process generally includes three distinct phases:

- Conceptual design and framing of the structure (including preliminary dimensioning)
- Analysis (calculation of external load effects including effects of seismic loads)
- Resistance and deformation verifications

As far as the analysis phase is concerned, it is well recognized that the actual values of internal actions and deformations/displacements may vary widely due to disproportionately large uncertainties related to seismic input data combined with changes in the stiffness of the system during strong earthquakes.

For reliable dimensioning and verification, the fundamental importance of the proper modelling of the global and local structural mechanisms, with specific attention to the connections, needs to be recognized. The significance of these models appears more crucial in the case of seismic actions under which a rather large degradation of material properties (depending on the level of the seismic action) is expected, leading to modifications both in stiffness and in-force-response characteristics (nonlinear cyclic behaviour), which can be substantial. Furthermore, control of the inelastic mechanism developed in a structural system responding to earthquake actions should be visualized. That is, appropriate care should be taken to enhance the basic principles of the capacity-design philosophy. It needs to be pointed out that the 'constructibility' and 'maintenance' of a structure are not ensured by means of relevant specific calculations and/or code provisions. This aspect should also be given due importance. Experience based on the history of seismic engineering and the behaviour of reinforced concrete structures in recent earthquakes showed that, despite the enormous evolution of computing facilities, the satisfaction of the fundamental performance objectives cannot be directly achieved only by means of apparently sophisticated calculations. Several basic design concepts proved to be more important.

This section will emphasize the major importance of the first step of the seismic design process, the so-called conceptual design. Every analysis has to be carried out on a preconceived structural scheme; several decisions have to be made prior to any analysis being carried out so as to minimize uncertainties related to the seismic response of the structure. Local and global mechanisms have to be predicted and controlled well in advance by the engineer in order to target the reliable seismic response of the overall systems. Following the conceptual design phase, the development and implementation of adequate structural detailing are of crucial importance to achieve and maintain the desired mechanisms.

3.3.2 Basic principles of conceptual design

Experience from past earthquakes has repeatedly shown that the behaviour of precast structures (as well as other structures) designed using certain basic principles of conceptual design (as listed below) satisfied the fundamental requirements of collapse prevention and damage limitation.

3.3.2.1 Structural simplicity

It is important from the early stages of the design of a precast building to develop a configuration that is simple and clear. Short and direct (and/or alternative) paths for the transmission of the seismic forces (load path) should be provided. This approach minimizes the uncertainties related to the modelling, analysis, dimensioning, detailing and prediction of the seismic response of the structure. Structural simplicity is characterized by the uniformity and regularity in configuration of the structural system in plan and/or elevation (see Fig. 3-12).

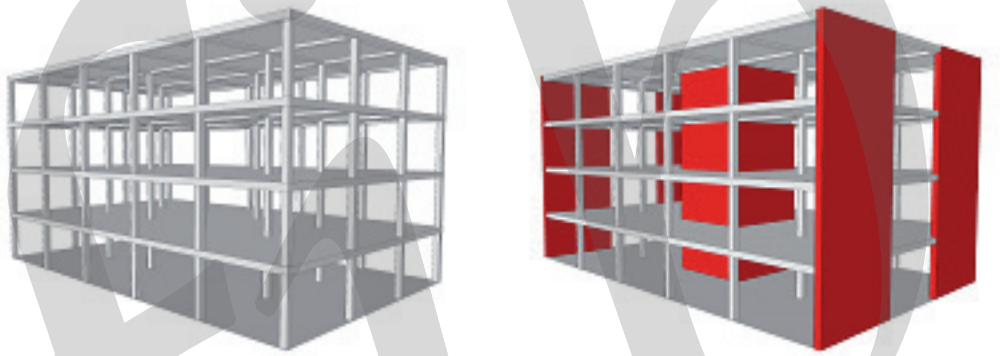


Fig. 3-12 Schematic examples of structural simplicity (Chastre, Lúcio et al., 2012)

3.3.2.2 In-plan uniformity - regularity

- Regularity in plan is much affected by the geometrical configuration of the building. Therefore:
  - The configuration should be compact and clear. In-plan set-backs (re-entrant corners or edge recesses) or *L, T, Π, E* and ⏋⎿ shapes, among others, should be avoided (see Fig. 3-13), or otherwise limited and given special treatment (see Fig. 3-14). For buildings that are rectangular in plan, the plan aspect ratio should not be greater than four.
  - The distribution of the lateral stiffness and mass should be closely symmetrical

in plan with respect to both orthogonal axes (see Fig. 3-15). Based on the size and shape of the entire structure in plan and elevation (imposed by architectural considerations), it may be necessary to divide the entire structure into dynamically independent units by means of seismic joints (see Fig. 3-14a). Special attention should be given to the width of the seismic joints in order to avoid the hammering of the individual units (see Fig. 3-16), particularly when the individual units are of different height. Figure 3-17 depicts a case of the improper arrangement of walls, which is to be avoided. Figures 3-18 and 3-19 show damage to some buildings after strong earthquakes caused by the disregard of the rules of in-plan uniformity-regularity.

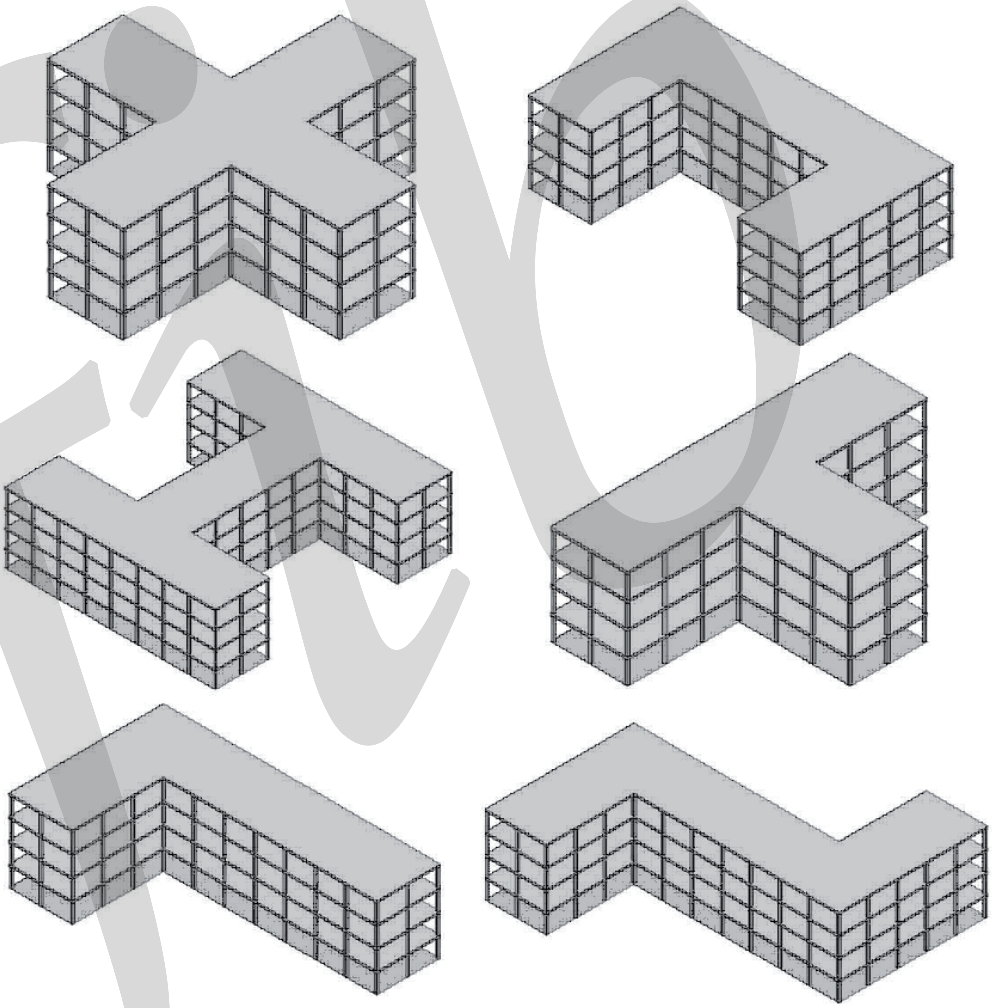


Fig. 3-13 Special attention should be given to these more complex building plan configurations

- In-plan uniformity - regularity:

  - leads to a desirable, even distribution of the structural elements in plan, which contributes to better mass and stiffness distribution throughout the system;
  - increases redundancy and contributes to well-distributed energy dissipation across the entire structure;
  - reduces possible torsional effects.

  An attempt should be made while devising the structural system to reduce the distance between the centre of mass and the centre of rigidity so as to minimize torsional effects as far as practicable. It should be noted that in multi-storey buildings subjected to seismic excitations, the centres of stiffness at the various floor levels can be located only approximately. Attention should also be given to the inelastic nature and characteristics of the torsional mechanisms of ductile systems (Paulay, 2000).

  The load standard in the United States, ASCE 7-10, directly considers the uncertainty of the locations of centre of mass and centre of stiffness by requiring that an allowance for an additional offset of 5% of the length of the building in each direction be included in the distribution of lateral forces to the vertical system at each level.

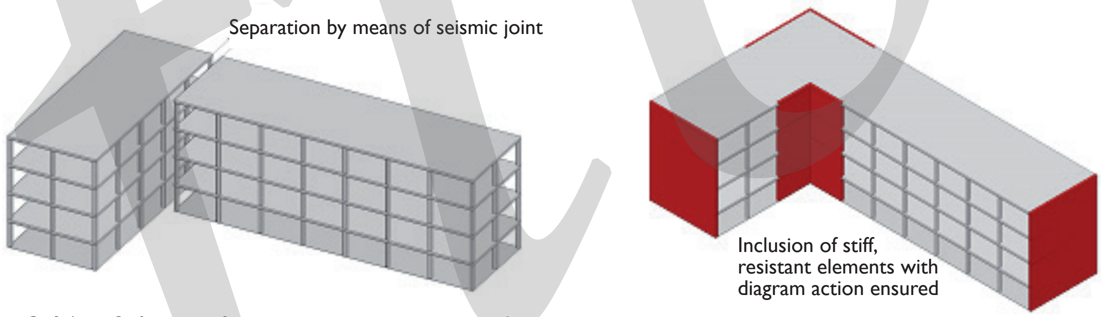


Fig. 3-14 Solutions for re-entrant corner condition

3.3.2.3 Vertical uniformity - regularity

For a building to be considered as regular in elevation:

- almost all lateral force-resisting elements such as cores, structural walls and columns in frame systems should preferably be continuous from their foundations to the top of the building (see Fig. 3-15). However, in some buildings, depending on the usage, the columns supporting the floors above may be allowed not to extend to the roof of the building (see Fig. 3-20);

- both the lateral stiffness and the mass of the individual stories should remain constant or reduce gradually;
- an uninterrupted flow of forces should be ensured by avoiding beams with vertical offsets or, even worse, discontinuous columns.

Generally, a uniform vertical distribution of 'safety margins' (V_R-V_S and M_R-M_S) is desirable (where *R* denotes resistance or capacity and *S* action or demand), so that any unpredictable increase of seismic action does not produce local failures. Figure 3-21 schematically shows cases to be avoided where the vertical discontinuities of structural members may lead to soft stories on the ground floor (see Fig. 3-21a) or upper floors (see Fig. 3-21b). Figure 3-22 schematically shows cases to be avoided where vertical discontinuities in stiffness and strength disrupt the natural flow of forces (load path) and may produce an abrupt concentration of stresses leading to local as well as global failures (see Fig. 3-23 and 3-24).

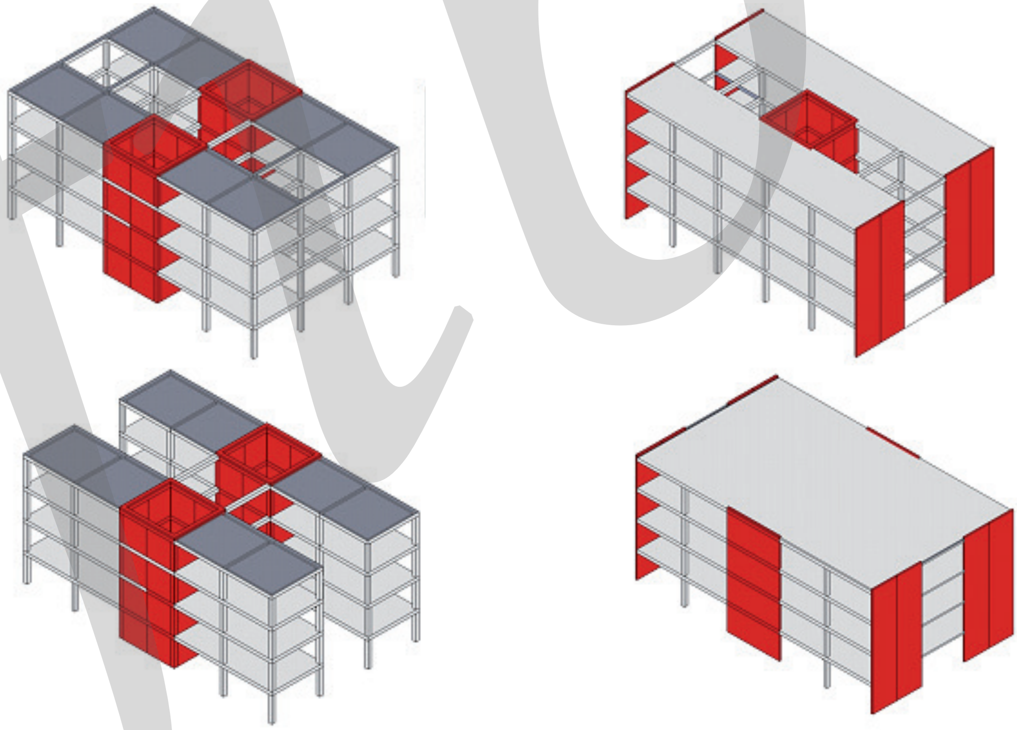


Fig. 3-15 Examples of symmetrical in-plan distribution of lateral stiffness and mass (based on Chastre, Lúcio et al., 2012)

Fig. 3-16
Detail of damage to Hotel de Carlo, showing intermediate floor failure

(Photo courtesy of NISEE-PEER, University of California, Berkeley)

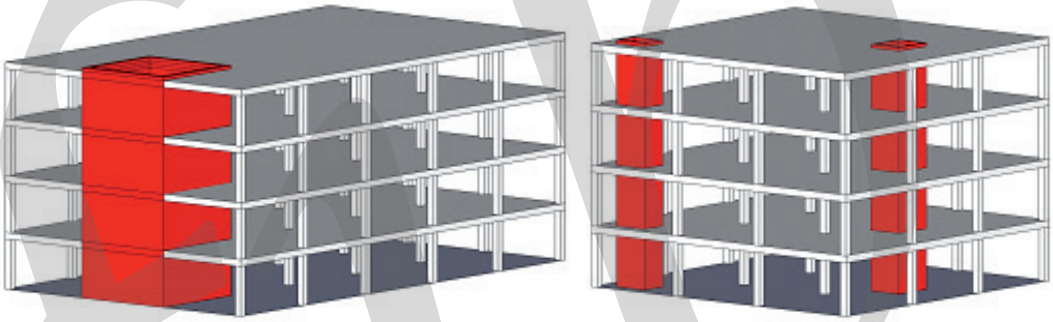


Fig. 3-17 Shear wall configurations to be avoided: walls are not symmetrical in plan with respect to both orthogonal axes and severe torsional mechanism can develop (based on Chastre, Lúcio et al., 2012)

Fig. 3-18
Damage due to stress concentration at notch of shallow L shaped building (West Anchorage High School) after 1964 Alaska earthquake

(Photo courtesy of NISEE-PEER, University of California, Berkeley)

Fig. 3-19 Damage to hospital building in Sepulveda (USA) after 1994 earthquake. Seismic joints separated and blocks pounded together (Photo courtesy of NOAA/NGDC, Mehmet Celebi, US Geological Survey)

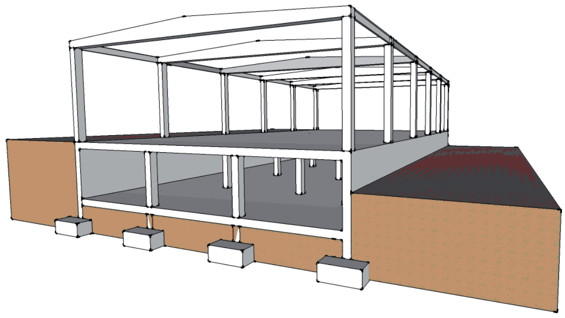
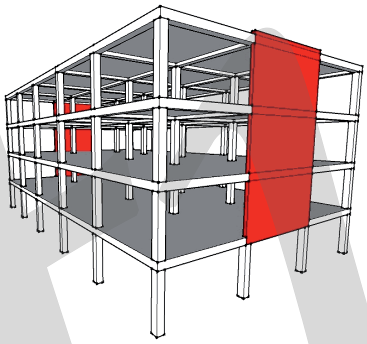


Fig. 3-20 Schematic presentation of building with basement

a.



b.

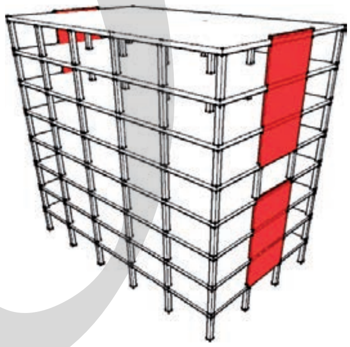


Fig. 3-21 Vertical discontinuities of structural members potentially leading to soft storey on ground floor (a) or upper floors (b)

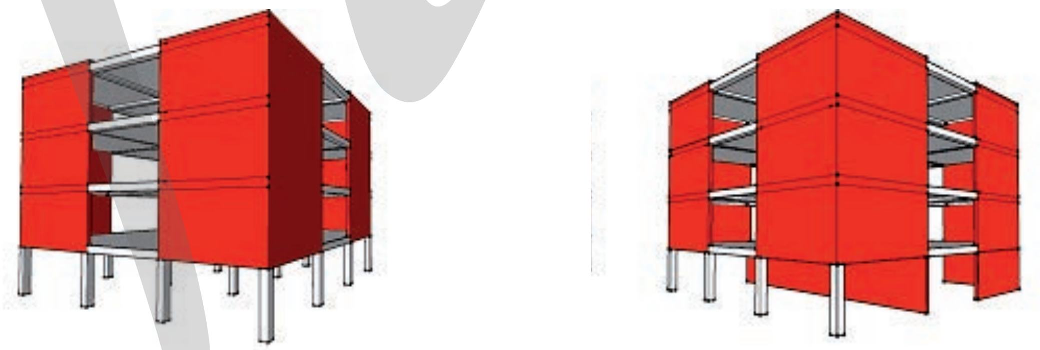


Fig. 3-22 Discontinuities of structural members leading to vertical discontinuities in stiffness and strengths (based on Chastre, Lúcio et al., 2012)

Fig. 3-23 Olive View Hospital, San Fernando earthquake, 1971, showing extrame deformation of columns above plaza level (Photos courtesy of NISEE-PEER, University of California, Berkeley)

Fig. 3-24 Soft-storey or 'pancake' collapse of the third floor of a building during Kobe earthquake, Japan, 1995 (Photo courtesy of NISEE-PEER, University of California, Berkeley)

3.3.2.4 Bidirectional resistance, torsional resistance and stiffness

Each structural system should be able to resist seismic forces in any direction. This can be achieved through the proper arrangement of the structural elements (columns and/or walls) in an orthogonal grid, which provides comparable strength and stiffness in orthogonal directions. Torsional irregularities should be avoided as much as possible.

In all cases, special attention should be given to the position of the elevators and staircases within the structure. These parts of the building, which are usually enclosed by structural walls (in order to prevent their collapse and ensure rapid evacuation after a seismic event), significantly affect the torsional behaviour of the overall structural system, depending upon their location and the arrangement of the other vertical structural elements. Figure 3-25 shows an example of the effects of torsion on a building subjected to an earthquake.

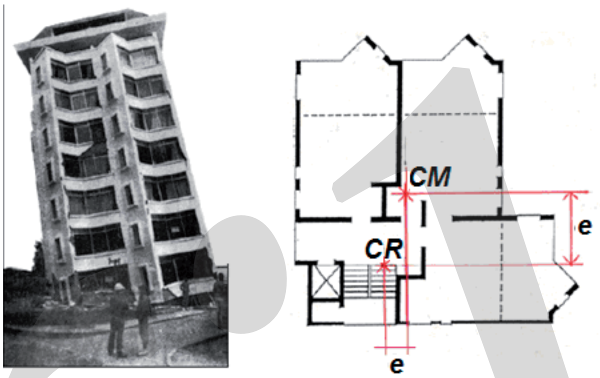


Fig. 3-25
Left: Building damaged in an earthquake. Right: Floor plan showing the centre of mass (CM), centre of rigidity (CR) and eccentricity along two axes (courtesy of Earthquake Engineering Research Institute (EERI)

3.3.2.5 Effects of the contribution of infills, partitions and cladding (strong beam - weak column)

In the design of precast structures and, in particular, frame systems, close attention should be paid to the interaction between secondary (often referred to as 'nonstructural') elements such as infills, partitions and cladding and appropriate measures should be taken regarding them (see Section 3.2.6):

- The contribution of the infills to the lateral resistance of the structural system is generally neglected in the analysis. However, attention should be paid to the mechanical characteristics of the infills and their interaction with the structural members. Infills, depending on their nature and connections, may significantly alter the behaviour of the structural system when compared to what is expected from an analysis neglecting their contribution (such as modelling the bare structural system).

- In the particular case of masonry infill walls within frame systems, where infills are typically in contact with the columns, the possible shear failure of these columns under shear forces induced by the diagonal strut action of the masonry infills should be considered. Short column effects, which may develop in the columns of the structural system in cases where the height of the infills is smaller than the clear height of the adjacent columns, lead to the undesirable behaviour of the columns (see

Fig. 3-26), if, as in most cases, they are not suitably reinforced and confined to develop this mechanism.

- Generally, the possible consequences of irregularity in plan and/or elevation that may be caused by infills, partitions or cladding should be taken into account.

Appropriate measures should be also taken to avoid the brittle failure of the secondary elements; these panels should be connected so as to accommodate the movement of the supporting structure. The connections also need to avoid unintended out-of-plane movement that may lead to the collapse of the cladding panels. Recent developments in low-damage solutions for drywall partitions, clay infills as well as precast heavy façades can be found in Tasligedik et al. (2013) and Baird et al. (2013).

Experience from past earthquakes has shown that inadequate connection details between (mostly rigid) cladding panels and structural elements led to the out-of-plane collapse of these cladding panels during seismic excitation (see Fig. 3-27, 3-28, 3-29, 3-30 and 3-31).

Fig. 3-26 Shear failure of columns under shear forces induced by inadequate arrangement of infills

(Photo courtesy of NISEE-PEER, University of California, Berkeley)

Fig. 3-27 Out-of-plane collapse of cladding panels during L'Aquila earthquake, 2009 (Toniolo and Colombo, 2012)

Fig. 3-28

Collapse of panels during Chile earthquake, 2010

(Photo courtesy of Mark Pierepiekarz, MRP Engineering LLC)

Fig. 3-29

Collapsed panel due to connection failure at roof level, Chile, 2010

(Ghosh and Cleland, 2012)LLC)

Fig. 3-30 Precast-concrete cladding collapsed as supporting structure underwent local failure, Chile 2010 (Ghosh and Cleland, 2012)

Fig. 3-31 Damaged precast-concrete cladding, Chile 2010 (Ghosh and Cleland, 2012)

3.3.2.6 Adequacy of foundation

In seismic situations, the interaction of the soil-foundation system with the structure (often referred to as soil-structure interactions [SSI] or soil-foundation-structure interactions [SFSI]) should be carefully studied and the structural system, the foundation system and the type of soil taken into account.

The design of the foundation elements depends on the way the superstructure is designed:

- Where capacity design that accounts for the development of possible over-strength is carried out, no inelastic mechanisms are expected and thus no ductility and energy dissipation is required in the foundation elements.

- In addition, for the design of the foundation elements, the corresponding design rules for elements of the superstructure should be followed (such as ductile behaviour, the protection of brittle mechanisms and displacement compatibility).

- In the latter case, for the tie beams and foundation beams, the design shear forces have to be derived on the basis of capacity design concepts; further considerations of repairability and the associated costs of the foundation system need to be accounted for.

Generally, the design and construction of the foundations should ensure that the whole building is subjected to uniform seismic excitation. In this respect the stiffness of the foundations should be adequate. It is better to use only one foundation type for the same structure.

Unless a structure is founded on competent, rocky soil (namely, type A) the following rules are considered fundamental for a good structural behaviour:

- Tie-beams should be provided between individual foundation elements in both orthogonal directions.

- Foundations should not be placed at different levels. When this is not possible, the equal horizontal movement of the different foundation levels needs to be ensured.

4 Precast-building systems

4.1 General

A precast concrete structure is an assembly of individual components with proper connections.

It is of primary importance that the design loads (both gravity and lateral, including seismic forces) be transferred from their point of origin to the foundation without the brittle failure of the structural members and their connections. The ductile behaviour of the entire structural system should be guaranteed.

The choice of the lateral-force-resisting system depends on the seismicity of the area, the soil conditions, the number of storeys, the function of the building and also economic considerations.

The following structural systems are usually used (and are discussed here after):

- Frame systems (Chapter 5)
- Wall systems - large panel systems (Chapter 6)
- Wall-frame systems (dual systems) (Chapter 7)
- Floor-framing systems (Chapter 8)
- Double-wall systems (Chapter 9)
- Precast-cell systems (Chapter 10)

The above-mentioned structural systems are used along with staircases and façades in a complete structure. Staircases and façades are not studied in this document. They were examined by Task Group 6.12 in *fib* Bulletin 74: *Planning and Design Handbook on Precast Building Structures* (2014).

5 Frame systems

5.1 General

Frame systems refer to assemblies of precast, linear, vertical elements (columns) and horizontal elements (beams).

Frame systems are suitable for buildings where a high degree of flexibility of use and open-space features are needed. This is mostly the case for industrial buildings, shopping malls and office buildings, among others. However, in residential buildings as well the frame-system concept is useful since it allows freedom in planning and opens up possibilities for changing the use by rearranging the partition walls, which are normally independent of the structural system. A high degree of freedom is also left to the architect in the choice of the façade.

Reinforced concrete precast frame systems may be distinguished as follows:

- Systems that are characterized by hinged beam-to-column connections, while the columns (usually manufactured in one piece) are fixed (moment-resisting) at the base-to-foundation connection.

  In the United States, these systems are designated as 'cantilevered column' systems in the table of permissible seismic systems (12.2-1 in ASCE 7-10, 2010).

  These types of precast frames are henceforth referred to as HCF (hinged-connection frames).

- Systems in which precast members are connected together and to the foundations through moment-resisting connections.

  Such frame systems are usually designed to meet the requirements of equivalent monolithic ones, and are henceforth referred to as FCF (fixed-connection frames).

5.2 Frames with hinged beam-to-column connections and HCF (cantilevered-column system in ASCE 7-10)

It is generally desirable in precast structures (usually comprised of prestressed horizontal members) to design beam-to-column connections that allow rotation for several reasons:

- Pretensioned members perform satisfactorily under unrestrained conditions
- Beam-to-column connections that provide moment continuity at the supports can be complex and costly
- Restraining volume changes (caused by such issues as concrete creep, shrinkage and temperature) occurring in rigid connections may cause unsatisfactory performance

In frames with hinged-beam-to-connection connections the beams are simply supported either directly on columns (namely, they are on top of them) or via a corbel, depending on the storey level.

The lateral forces are resisted by the flexural mechanism of the columns. In this case the columns are continuous (one piece or lapped) for the full height of the structure and are fixed to the foundations by moment-resisting connections so that they act as a cantilever in both directions of the building when subject to horizontal earthquake loadings (see Fig. 5-1). The column-to-foundation connections should be designed accordingly (see Section 5.3.4). Columns should also be designed and detailed to ensure strength and ductility.

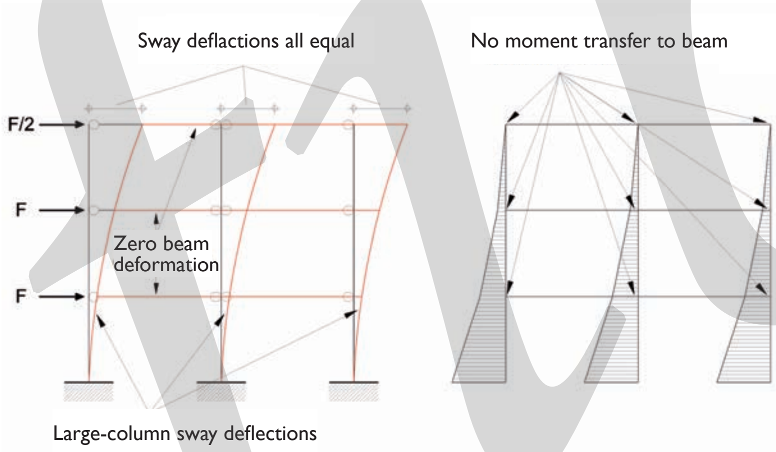


Fig. 5-1
Deformations and bending moment distributions, caused by lateral loads, in unbraced, three-storey hinged-connection frame (HCF)

Where a uniform sway deflection between columns is required and is assumed in the design, the floor and roof system should be such as to develop proper diaphragm action (see Section 8.2).

Generally, such HCF systems are used in low-rise buildings and in areas of low or moderate seismicity. However they may also be integrated as gravity-dominated systems in buildings where the lateral loads are principally carried by fixed-connection frames

(FCFs) or by lateral resisting systems of other types, for instance, shear walls or dual systems (combining walls and frames).

In Figure 5-2 a typical precast frame system with an HCF is depicted schematically on the left, with different precast floor elements. Figure 5-2b shows a typical real case.

In the United States, these systems are called 'moment-resisting frames'. Table 12.2-1 in ASCE 7-10 includes three classes of reinforced concrete moment-resisting frames: ordinary, intermediate and special. The assignment to these classes depends on the amount of ductility that is assured by detailing requirements. For precast concrete construction, only ordinary and special moment frames are specifically recognized.

Fig. 5-2 Left: Typical HCF structural system (hinged beam-to-column connections and fixed column-to-foundation connections)
Right: Case under construction in Calicut, Kerala, India
(Photo credit: A Dienst; photo provided courtesy of KEF Infrastructure India Pvt. Ltd)

5.2.1 Hinged beam-to-column connections for HCF

The role of the beam-to-column connections in hinged frames and with fixed connections at the foundation (thus, when a different lateral force-resisting system is not being used) is to ensure the transmission of the lateral forces from the roof (or intermediate floor) to the columns.

Varieties of hinged/pinned connections exist worldwide.

Those mostly used are pinned connections with one or two parallel vertical steel dowels arranged in the direction normal to the plane of the frame.

The two principal directions of the frame should be considered based on stability:

- one in the plane of the frame (in-plane direction)
- one perpendicular to the plane of the frame (out-of-plane direction)

In Europe the hinged connection in the plane of the frame is usually achieved by using two parallel dowels (see Fig. 5-3); the connection is adequately anchored in the body of the column (or in a corbel of the column) and protrudes into grouted ducts in the beam. These dowels provide resistance against horizontal relative displacement and actions between the beam and column (see Fig. 5-4a) as well as some moment resistance against the out-of-plane overturning of the beams (see Fig. 5-4b).

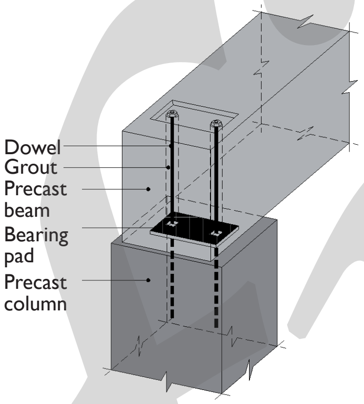


Fig. 5-3 Typical hinged beam-to-column connection with parrallel dowels (other reinforcements not shown for clarity)

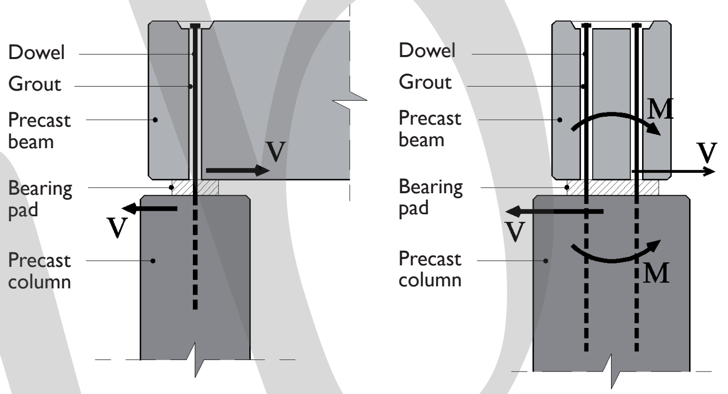


Fig. 5-4 Hinged beam-to-column connection with parallel dowels
a. Actions in plane of frame
b. Actions perpendicular to plane of frame

In the United States sand or vermiculite is placed in the lower 10.16 cm (4 in.) of the sleeve to permit some volume change strain.

Bolted caps on top of the dowels (see Fig. 5-5) are desirable for the following reasons:

- To secure the connection before its completion in the unlikely event of a seismic excitation during erection

- To eliminate the failure of the connection under large shear sliding between the beam and column caused by an unexpectedly strong seismic event or, in general terms, nonsynchronous motions between columns

The essential components of the connection are:

- the bearing (contact surface) between the top of column/corbel and the underside of the beam;
- one or more dowels protruding from the column/corbel and passing through vertical ribbed metal ducts at the beam ends, which are filled in-situ by nonshrink grout, to ensure transmission of lateral forces (shear); and
- the nonshrinkable grout.

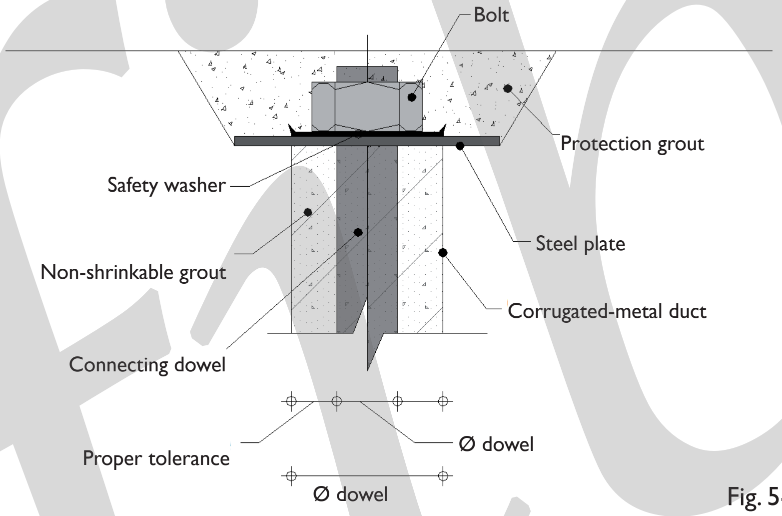


Fig. 5-5 Detail of dowel bolting

Bearing pads are recommended for the protection of the concrete edges of the column and the beam against spalling, caused by the concentration of stresses owing to the flexural deformation (relative rotation) between the beam and the column (especially in the case of rather slender columns).

Flexible or rigid pads may be used:

- Flexible pads should be designed to allow rotation in the top of the column. A typical solution consists of a neoprene pad with appropriate thickness (plain or reinforced by internal steel layers) in order to prevent concrete-to-concrete contact under the design lateral displacement (see Fig. 5-6).

In the case of flexible pads, care should be taken with the time required for pouring the grout into the dowel ducts. This is because after the hardening of the grout any additional load induces compression in the dowels owing to the bond that develops between the dowels and the grout. The bond stresses are activated by the vertical flexibility of the bearing pads. Even if no loss of bond occurs with any increase in additional loads, the compression forces that are induced in the steel dowels generally reduce the shear resistance of the connection under induced displacements.

In the United States, when vertical rod connections are used, it is a common practice to add a loose compressible fill into the sleeve to about half its height before the grout is placed so that volume change restraint is reduced. This modification reduces the direct shear demand on the rods since they act primarily in tension after small local bending causes yielding. The resistance in shear increases significantly when deformation causes the rods to come into contact with the surrounding sleeve. The nonlinear response history analysis of a system using this modification shows that a beneficial secondary effect occurs by reducing the ground motion transferred into the floor framing.

- An alternative solution is the use of rigid steel pads, which do not allow vertical deformations at the joint and thus eliminate probable additional loading applied to the main beam after the hardening of the grout does not cause compressive stresses in the dowels and does not affect the strength of the connection.

When steel pads are used, beam rotation can cause beam or column spalling.

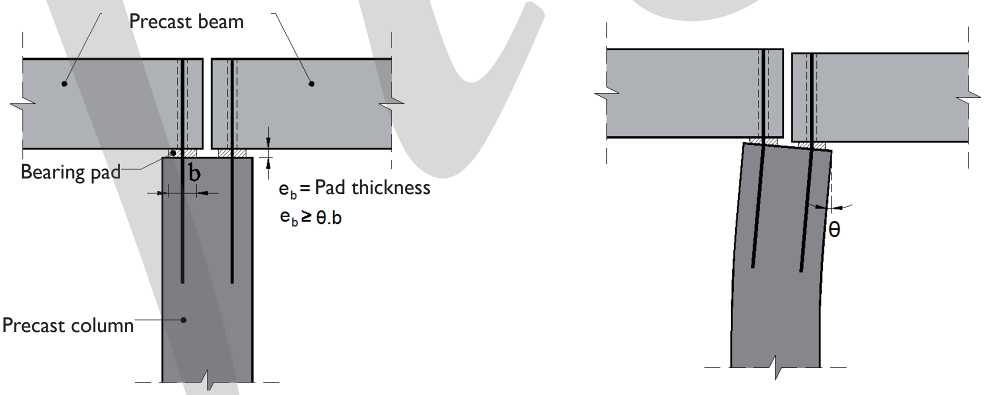


Fig. 5-6 Schematic representation for the estimation of the thickness of the pad as a function of the relative rotation between beams and column

Failure modes can be avoided by designing hinged connections in the following way:

- In the plane of the frame (see Fig. 5-4a)
  The shear yielding of the connection (under combined shear, axial and bending actions in the steel dowels and/or shear sliding under horizontal actions) due to the inadequate:

  - cross section and steel grade of the dowels;
  - concrete strength (of the column and/or beam ends);
  - strength of the poured-in-situ grout;
  - concrete cover of the dowels (in the direction of the horizontal shear force and in the orthogonal direction);
  - confinement reinforcement (such as horizontal stirrups) around the dowels.

  Spalling of the concrete edge of the beam and/or column due to the dowel action and/or the inadequate bearings thickness and/or local detailing.

- Perpendicular to the plane of the frame (see Fig. 5-4b)
  - Flexural failure around the joint due to the out-of-plane moment *M*
  - Pullout of the dowel due to tension caused by the out-of-plane moment *M*

There are **factors** that affect the **shear resistance** of a hinged beam-to-column connection with grouted dowels. The information that follows summarizes findings in international theoretical and experimental research (SAFECAST [2012a & b], Utescher and Herrman [1983], Vintzeleou and Tassios [1987], *fib* Bulletin 43 [2008], among others).

- The concrete grade and steel grade of the dowels
  Tests over decades have shown that the concrete and steel grades contribute to shear resistance by the square root of the product $f_{cc} \times f_{sy}$, in other words:

  $$\sqrt{f_{cc} \times f_{sy}}$$

  where f_{cc} is the concrete confined compressive strength and the grout used in the connection, in conformance with *fib* Bulletin 43 (2008), and f_{sy} is the yield strength of the dowels.

- The area A_d of the dowels
  The shear resistance of the connection increases almost proportionally to the total area A_d of the dowels, as shown in Figure 5-7.

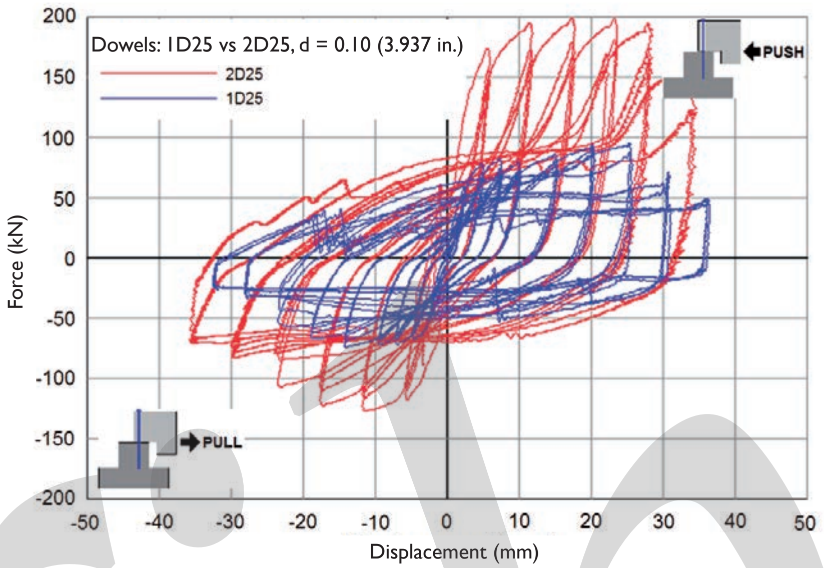


Fig. 5-7 Shear strength vs. shear sliding of hinged beam-to-column connection (under cyclic loading) using one or two dowels of 25 mm (0.984 in.) in diameter. *D* is diameter of dowels in millimetres and *d* is concrete cover of dowels in metres. (Test results from European research programme SAFECAST, 2012a.)

- The grade of the grout that surrounds the dowels in the beam ends (see Fig. 5-8). A higher grade of grout leads to higher shear resistance.

- Confining reinforcement perpendicular to the dowels
  When the confining reinforcement is properly arranged along the length of the dowels and concentrated close to the shear interface of the column and the beam ends, the shear resistance of the connections is improved (see Fig. 5-9).

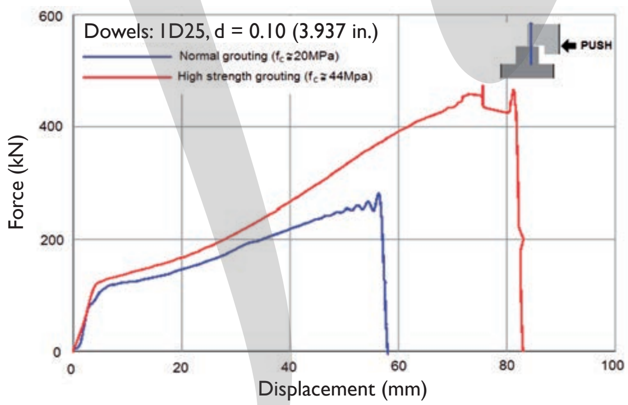


Fig. 5-8

Shear strength vs. shear displacements (sliding) of hinged beam-column connection using one D25 dowel with cover *d* =100mm (3.937 in.) and different grades of grouting material (based on SAFECAST, 2012a)

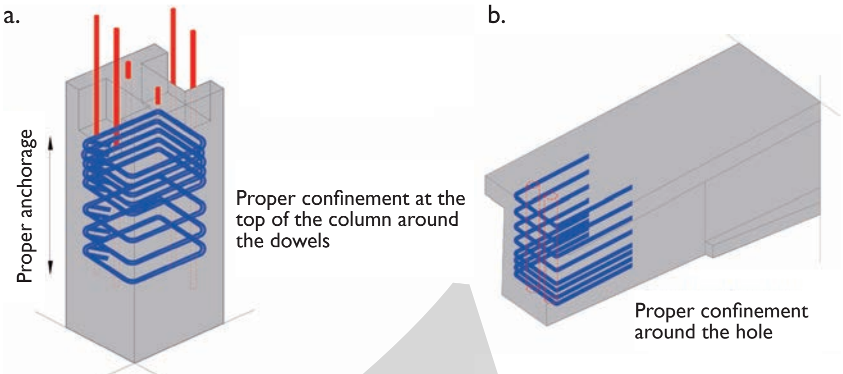


Fig. 5-9
Detailing of confining reinforcement around dowels

- The eccentricity of the shear force with respect to the interface of the connected members
  According to Utescher and Herrman (1983), the eccentricity e of the shear force to be transferred from the beam to the column, represented in Figure 5-6 by the effective thickness e_b of the bearing pad, has negative effects on the shear resistance of the connection; in other terms, an increase of e_b would result in a reduction of the shear strength (albeit not significantly).

- The direction of loading (push vs. pull)
  As observed in several experimental tests (see Fig. 5-7 and 5-10), the beam-column-connection shear-capacity and force-displacement (slip) behaviour is not symmetric but weaker in the pull direction and stronger in the push direction (due to the different concrete cover usually found around the dowels in the pull and push direction). This phenomenon, which is more pronounced under monotonic loading than cyclic loading, should be carefully accounted for when considering the global design and response of the building since excessive slippage in one direction might lead to premature damage (if not unseating and failure) at one end of the beam-end connection, with the redistribution and the concentration of seismic actions on fewer elements.

- Monotonic or cyclic loading types
  In addition to the direction of loading, the type of loading drastically affects the shear resistance of the connection.
  Under cyclic loading, the shear resistance of a connection can significantly decrease in comparison with its shear resistance under monotonic loading. This effect should be accounted for in the design and analyses. Figures 5-10 and 5-11 show a 50% decrease under the specific conditions of the test referenced in Figure 5-10.

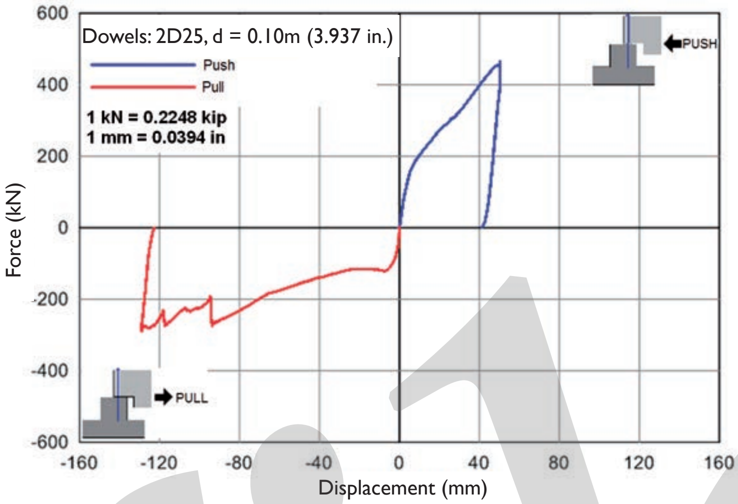


Fig. 5-10
Shear strength vs. shear displacement of beam-to-column connection under monotonic loading using two D25 dowels and cover *d* =0.10 m (3.937 in.) (based on SAFECAST, 2012a)

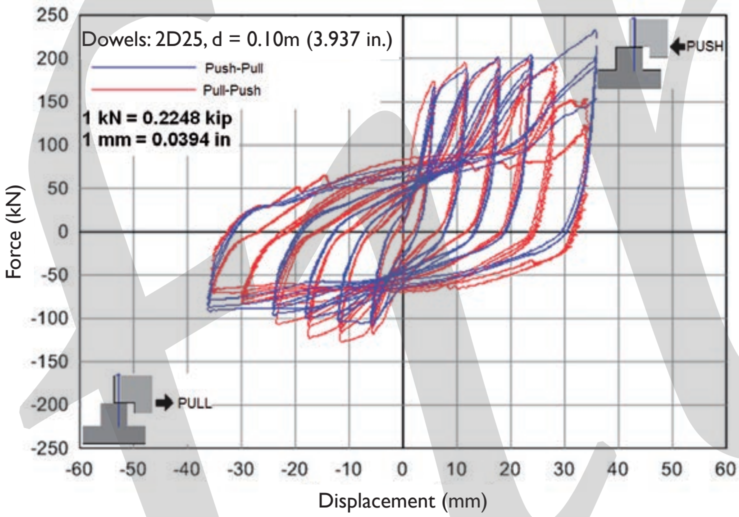


Fig. 5-11
Shear strength vs. shear displacement of beam-to-column connection under cyclic loading using two D25 dowels and cover *d* =0.10 m (3.937 in.) (based on SAFECAST, 2012a)

- The concrete cover of the dowels
  The concrete cover of the dowels is an important factor affecting the shear resistance of hinged connections.

Note: In what follows, the term 'concrete cover of the dowels' refers to the distance between the centreline of the dowels and the edges of the concrete elements.

Figure 5-12 shows the minimum practical distance of a dowel to the concrete edges (in both a beam and column section) with reference to the usual arrangement of reinforcement and code-compliant concrete covers. Evidently, this minimum practical distance may be assumed at approximately 4×*D*, where *D* is the diameter of the dowel.

In the framework of the European research program SAFECAST (2012a & b) shear tests were performed on specimens of the concrete covers of the dowels in the direction of the horizontal shear force of 4×*D*, 6×*D* and 8×*D*, while in the transverse direction the concrete cover was either 4×*D* or 8×*D*. In each case, the shear resistance of the connection was measured experimentally. A summary of the result is shown in Figure 5-13.

Figure 5-13 shows the variation of the shear resistance F_R (= a × F_R) with different edge-to-dowel distances. It can be concluded that the edge-to-dowel distance in the direction of the force governs the shear capacity of the overall connection when the concrete cover of the dowel perpendicular to the applied force is equal to or greater than 4×*D*. Thus, for practical applications, it is recommended that the concrete cover is taken as a minimum of 4×*D* in all cases and a minimum of 6×*D* in the direction of the applied shear.

If dowels are placed with a cover that varies (due to construction restraints) between 4×*D* and 6×*D* in the direction of shear, a reduction factor linearly varying from 0.60 for 4×*D* to 1.00 for 6×*D* needs to be applied to the shear resistance (see Fig. 5-13a and c).

Figures 5-14 and 5-15 show a typical beam-to-column connection at the top of a column.

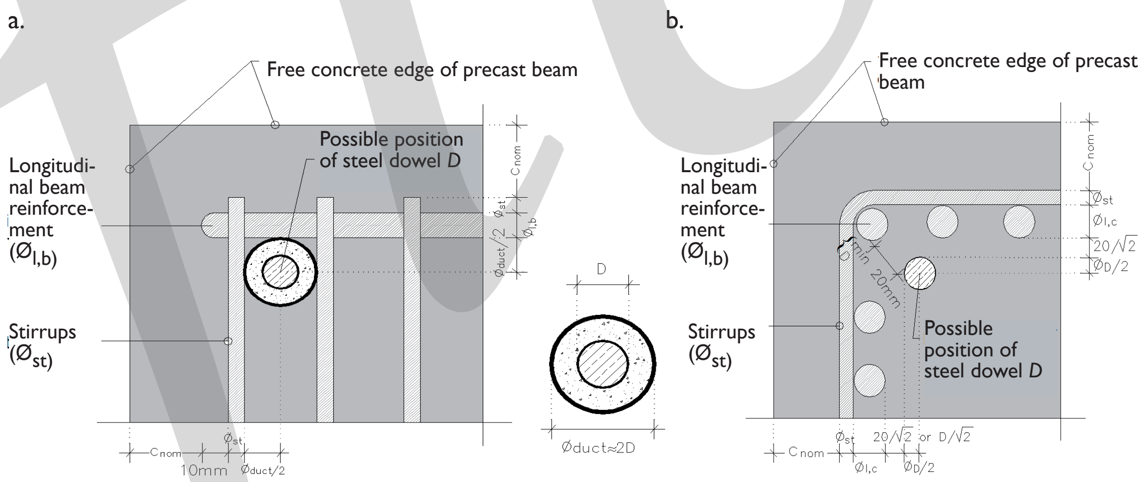


Fig. 5-12 Minimum practical distance of dowel to free edges of:
a. concrete beam
b. concrete column (reinforced in conformance with codes)

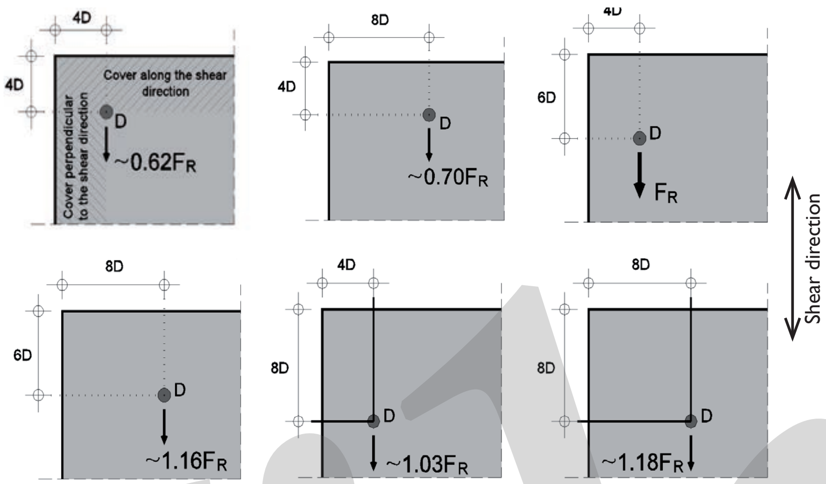


Fig. 5-13
Shear resistance of dowels as function of concrete cover in direction of shear and of distance in perpendicular direction (based on SAFECAST, 2012a)

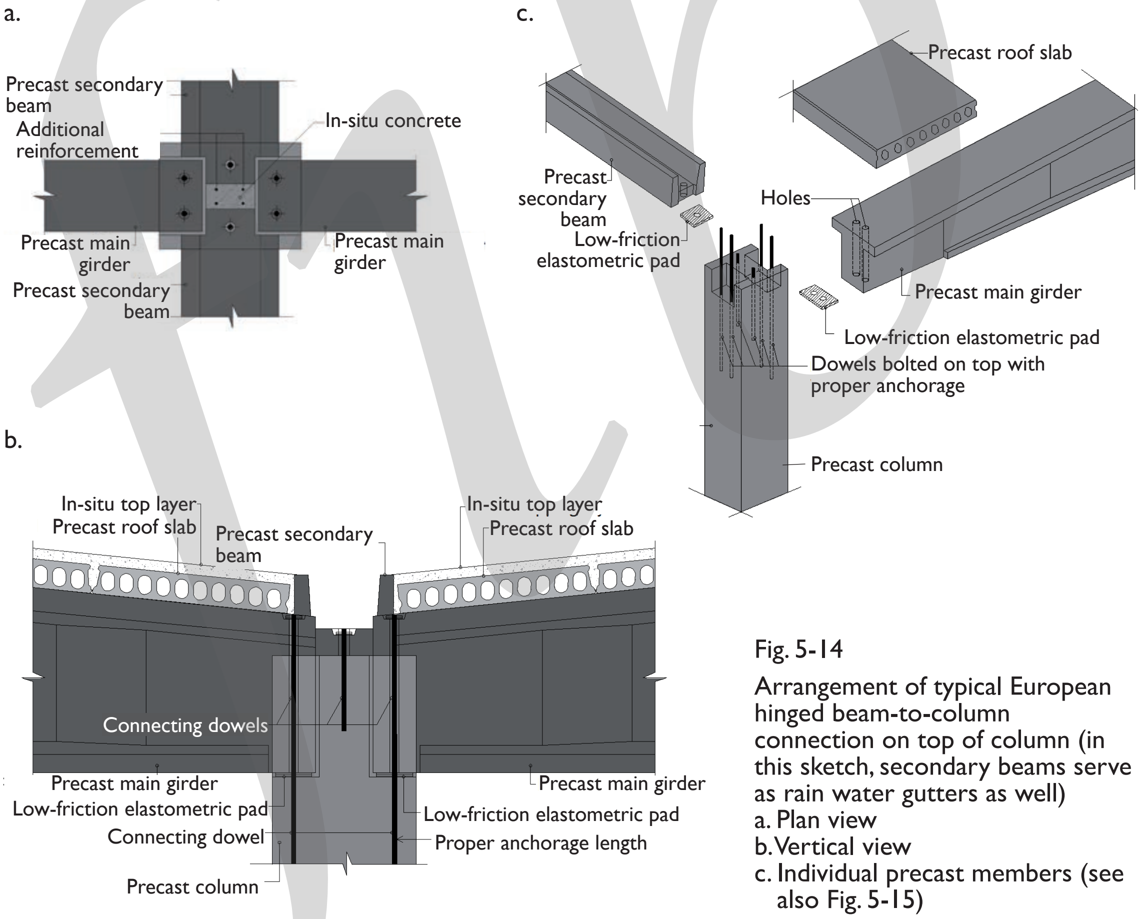


Fig. 5-14
Arrangement of typical European hinged beam-to-column connection on top of column (in this sketch, secondary beams serve as rain water gutters as well)
a. Plan view
b. Vertical view
c. Individual precast members (see also Fig. 5-15)

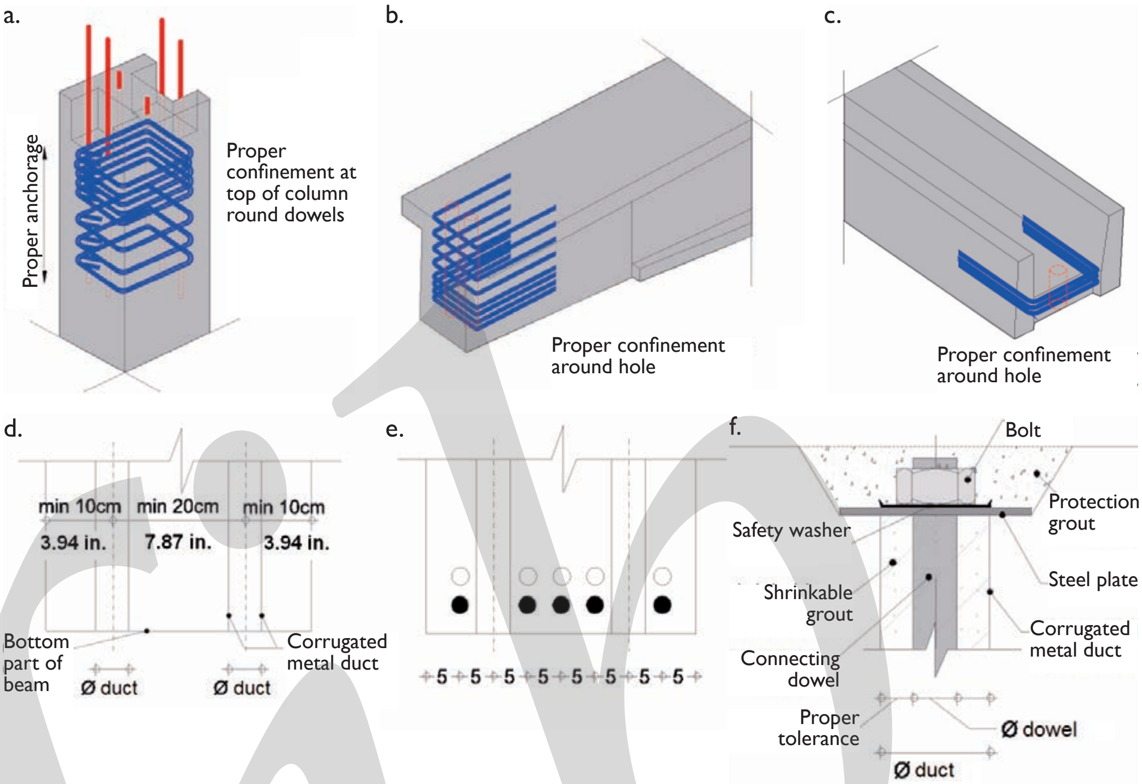


Fig. 5-15 Details of Fig. 5-14. (Other reinforcements of precast members are not shown for clarity.)
Confinement around connecting dowels: a. in columns; b. in main girder; c. in secondary beams
Proper arrangement of ducts for two dowels: d. geometry; e. proper arrangement between pretensioned bars, bolting; f. recess, grouting & tolerances

5.3 Frames with moment-resisting connections

5.3.1 General

Fixed-connection frames (FCFs), or frames with moment-resisting beam-column connections, are built with a set of properly connected, independent, precast or semi-precast linear elements (columns and beams) or a set of precast units (such as beam-column cruciform units) to form a stable frame system with moment-resisting connections. Lateral forces are resisted primarily through the flexural action of the frame members. The challenge in incorporating precast concrete elements in moment-resisting frames is to find economical and practical means of connecting the precast elements together to ensure adequate stiffness, strength, ductility and stability.

Beam elements are usually connected using cast-in-situ concrete, either in the beam-to-column joint regions, also referred to as 'wet connections', or along their length, also referred to as 'strong connections', indicating that the connection itself between units occurs far away from the critical section where the inelastic mechanism is expected.

In the United States such connections are also accomplished with post-tensioning or proprietary splice sleeves.

Precast concrete columns are typically spliced, either at the base connection with the foundation and above the beams/floors or at mid-height between floors. They are connected either through proprietary grouted steel sleeves (couplers) or by connecting the column longitudinal bars through corrugated metal ducts and grout (such as drossbachs). Figure 5-16 shows the elastic deformations and bending-moment distribution in a FCF under horizontal static loads.

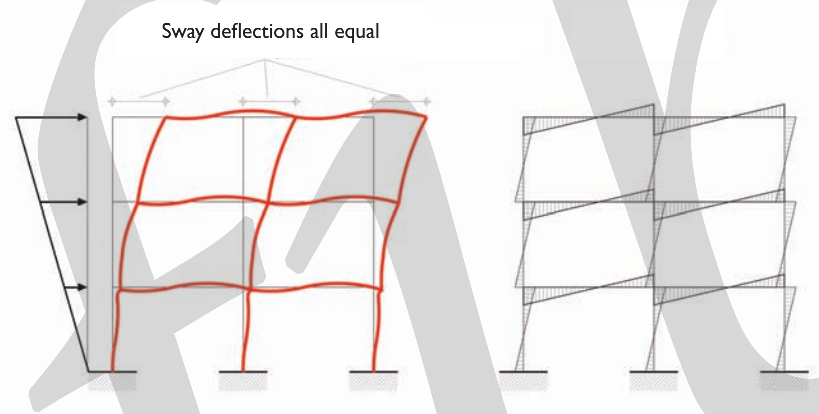


Fig. 5-16
Elastic deformations and moment distributions in frame with moment-resisting connections

FCFs are designed to provide lateral-force resistance in one or two directions (see Fig. 5-17).

One-way frames are built without (see Fig. 5-17a) or with (see Fig. 5-17b) transverse pin-ended beams, whose main purpose is to carry gravity loads.

The lateral resistance of the overall system in both orthogonal directions can be achieved by:

- relying upon independent one-way FCFs in each direction; or

- combining a one-way FCF in one direction with shear walls, dual systems or other lateral-load-resisting systems (such as braces) in the orthogonal directions; or
- adopting a two-way FCF.

In some practical cases and in regions of low seismicity, one-way moment-resisting frames in one direction are combined with hinged connection frames (HCFs) in the orthogonal directions. This solution should be carefully considered and analysed, and where possible, avoided in regions of moderate-to-high seismicity, in consideration of the possibility of significantly different behaviour of the structure in two directions, in particular in terms of flexibility, which could lead to an undesirable torsional response.

An advantage of two-way FCFs (see Fig. 5-17c) is that inertia forces originating in diaphragms would generally require a rather short load path before they are collected and transferred into the columns, thus providing, overall, an appreciable level of redundancy in the seismic-resisting system.

In seismic applications FCFs are, as per any other seismic resisting system, designed to inelastically resist lateral forces that are much smaller than those required to resist the design earthquake elastically. Nonlinear deformations in the plastic hinge regions at the base column and at the beam-column connections under the design earthquake are therefore expected. Thus, such frames need to be designed and detailed to possess large nonlinear deformation capacity.

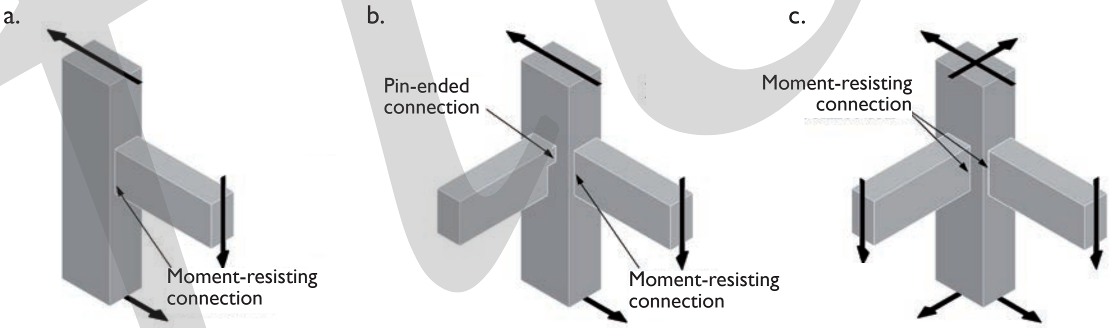


Fig. 5-17 Classification of FCFs by considering in-plane lateral-force resistance
a. One-way frame
b. One-way frame with pin-ended transverse beam
c. Two-way frame

FCFs, which are designed and detailed to respond inelastically under lateral forces, may be grouped into two broad categories:

- Equivalent monolithic systems
- Jointed ductile systems

Equivalent monolithic systems aim to emulate cast-in-situ frames (reinforced or prestressed) in terms of stiffness, strength, ductility and energy-dissipation characteristics. The best means of achieving ductile post-elastic deformations in moment-resisting frames is by flexural yielding at selected plastic hinge locations, properly designed and detailed to develop the required ductility capacity. Post-elastic deformations due to flexural yielding result in stable load-displacement hysteretic loops without significant degradation of strength, stiffness, and energy dissipation.

Significant post-elastic deformations due to shear or bond mechanisms are to be avoided.

The preferred overall mechanism for precast-concrete-equivalent monolithic moment-resisting frames is a beam-sideway mechanism (see Fig. 5-18) with the formation of plastic hinges in the beams at the beam-column interfaces. A beam-sideway mechanism occurs as a result of strong-column-weak-beam design. In accordance with the basic principles of capacity design and hierarchy of strength, while plastic hinge regions are detailed for ductility, other regions and elements of the structural system are overdesigned to ensure that they will remain in their elastic range. A column-sideway mechanism should generally be avoided.

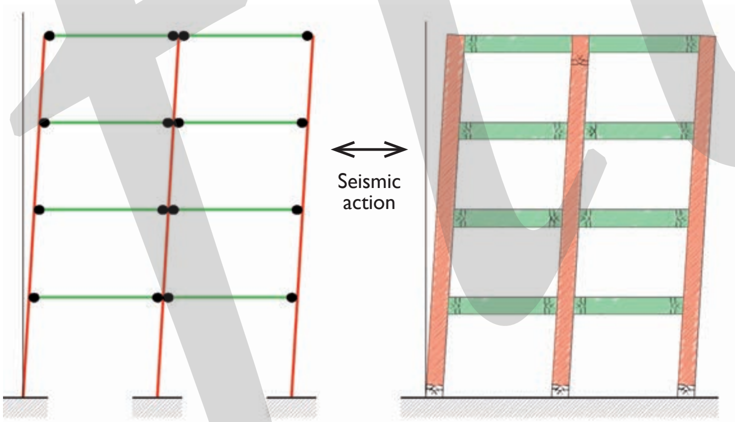


Fig. 5-18
Development of plastic hinges at beam ends and bases of columns (beam-sideway mechanism)

ACI 318 (2011) deals with emulative design. Sections 21.8.2 and 21.8.3 are intended to produce frames that respond to design displacements (namely, displacements that are expected when the structure is subjected to the design level earthquake) essentially in the same way as monolithic special-moment frames.

Precast-frame systems composed of concrete elements with ductile connections (see Section 21.8.2 in ACI 318 [2011]) are expected to experience flexural yielding in the connection regions. Precast-concrete frame systems composed of elements joined using strong connections (see Section 21.8.3 in ACI 318 [2011]) are intended to experience flexural yielding outside the connections. Strong connections include the length of the coupler hardware. Capacity design principles are used to ensure that the strong connection remains elastic following the formation of plastic hinges in the connected members. Additional requirements are provided to avoid the hinging and strength deterioration of the column-to-column connections.

Examples of the brittle fracture of reinforcing bars at the faces of mechanical splices caused by strain concentrations have been observed in laboratory tests of precast beam-to-column connections. Therefore, designers are cautioned to select the locations of strong connections carefully or to take other measures, such as the debonding of reinforcing bars in highly stressed regions, so as to eliminate strain concentrations that have the potential to cause the premature fracture of the reinforcement.

ACI 318 (2011) also recognizes the vulnerability of mechanical splices in ductile connections, prohibiting their use closer than $h/2$ from the face of the joint between the beam and column.

Jointed ductile systems are permitted by ACI 318 (2011) in special moment frames built using precast concrete provided that they satisfy the requirements of ACI 374.1-05 (2005), which provides acceptance criteria for moment frames based on the structural testing of jointed ductile systems, even if they do not satisfy the requirements of monolithic emulation.

The New Zealand Concrete Standard NZS3101:2006 provides a special 'Normative Appendix B' for the design of jointed ductile connections.

One non-emulative frame connection and system that has been successfully used in the United States and elsewhere is the hybrid solution described below (Priestley et al., 1999; *fib* Bulletin 27, 2003; Pampanin 2005, 2012; NZCS, 2010). The hybrid beam-to-column connection (see Fig. 5-19), part of the wider family of jointed ductile systems, uses a system of post-tensioning strands that run through a duct in the centre of the beam and through the column. Mild steel reinforcement is placed in (typically, metallic corrugated) ducts at the top and the bottom of the beam and through the column and is then grouted. A key feature of the hybrid-frame connection is that the grouted mild reinforcing bars must be deliberately debonded for short distances in the beams adjacent to the beam-to-column interfaces in order to reduce and control the high cyclic strains that would otherwise occur at these locations. The amount of mild steel reinforcement and post-tensioning steel is proportioned so that the frame recentres itself after a major seismic event. The inelastic demand is concentrated at the beam-column interface, where an opening and closing of a gap, also referred to as the controlled rocking mechanism, occur during the seismic sway. The post-tensioning can also provide high shear resistance for the beam at the interface with the column, eliminating, in principle, the need for corbels, although in the New Zealand and European code-design approach and construction practice, a shear key/corbel to carry out the gravity load is required and adopted. Mild steel (or other types of external and replaceable dissipaters) provides ductility and energy dissipation in the connection region by yielding. The post-tensioning strands (tendons) or threaded bars remain elastic throughout the seismic event while the mild steel is yielding. The post-tensioning helps the structure return to its initial position (re-centring) with negligible residual displacements.

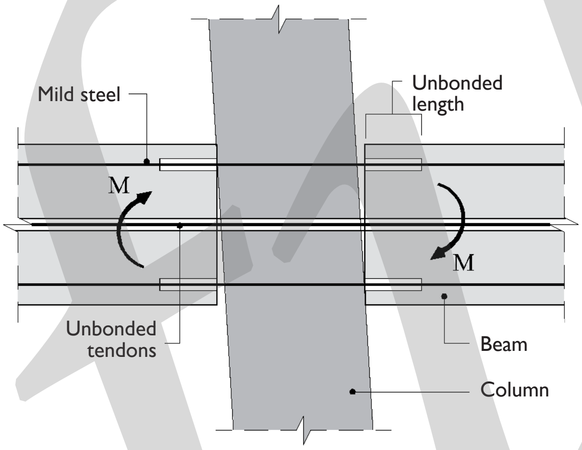


Fig. 5-19
Hybrid beam-to-column connection

Precast units of different suitable shapes are sometimes used to form one-way or two-way frames of the FCF type. The shape of these units depends on the specific features of the building and its serviceability needs (such as high-rise, low-rise and one-storey buildings) and the spans between columns in both directions. Economical aspects need also be considered. The idea is to produce precast vertical monolithic units of different shapes covering 1, 2 or 3 storeys (of the building) in one piece, including horizontal cantilever arms (parts of beam) at each level. The length of these horizontal arms provided in one or two directions always exceeds the corresponding critical plastic hinge region along the span of the beam. The length of the critical plastic hinge region of beams, in normal conditions, may be taken as 1.0 hb (according to EC8, 2004), where hb is the depth of the beam.

In Figure 5-20 a one-way FCF, using two different types of precast units interconnected by beams or directly to one another, is schematically presented. Care should be taken to ensure the compatibility of deformations between the floor/slab-system and its supports on the beams. In the United States such frames are typically one storey tall and have an 'H' or an inverted 'U' or 'M' shape, with connections occurring at column mid-height and beam midspan, as seen in the middle of Figure 5-22. If connections along the height of the precast units are needed, they should preferably be placed at mid-height of the storeys, where a point of contraflexure under lateral loading is expected (see Fig. 5-21) and should be moment-resisting connections to be able to transfer all the internal actions with continuity. In Figure 5-22 various configurations of precast frame elements are shown.

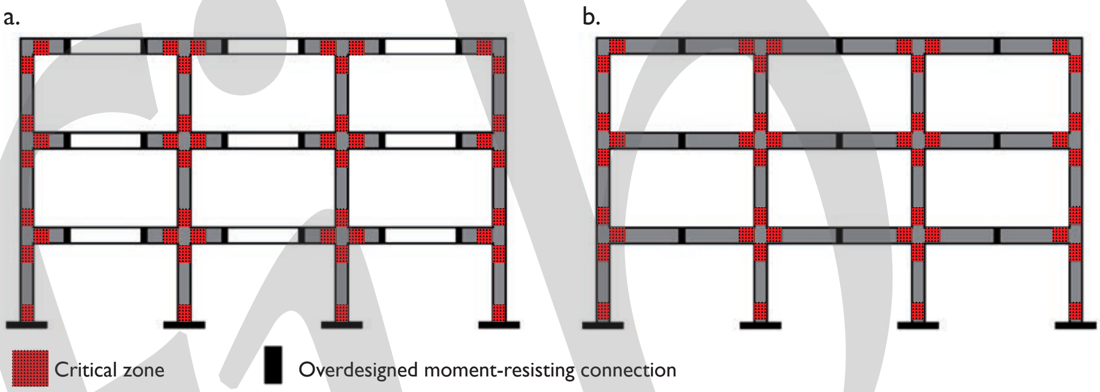


Fig. 5-20 Plain FCF with vertical cruciform units:
a. interconnected by beams
b. interconnected directly to one another

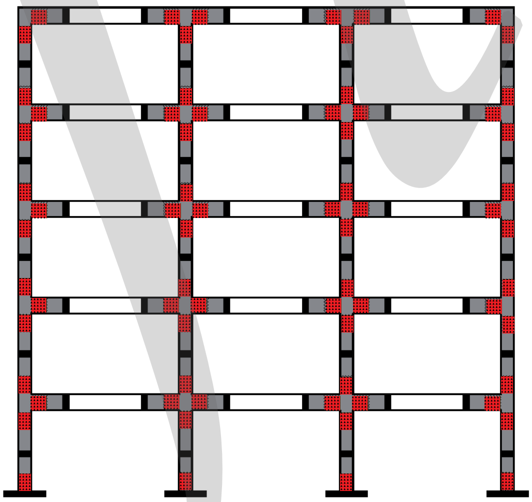


Fig. 5-21
Plane FCF with T-shaped (⊣)and cruciform-shaped (+) units interconnected directly or by beams

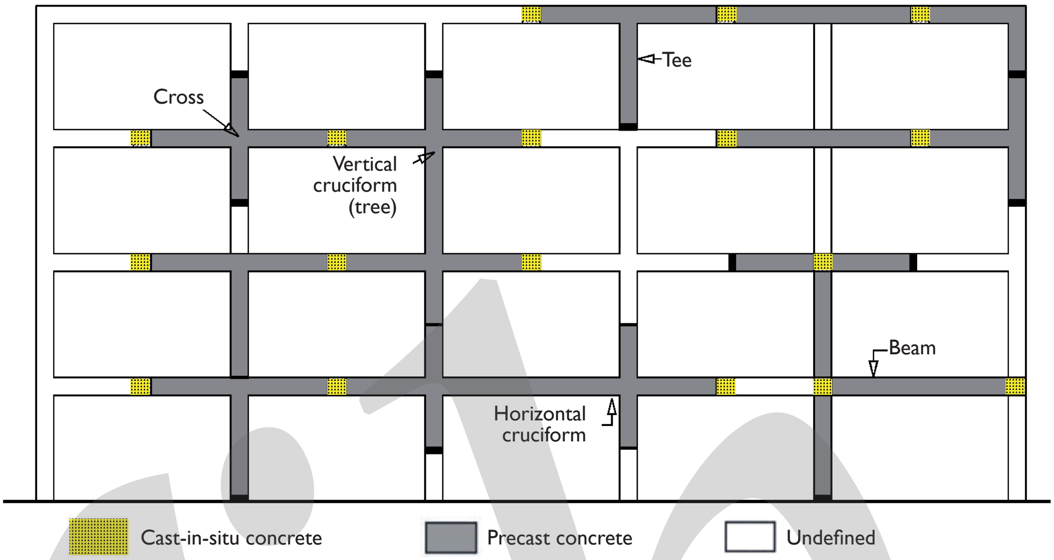


Fig. 5-22 Various configurations of precast-frame elements (based on ACI 550.1R-09, 2009)

5.3.2 Equivalent monolithic moment-resisting beam-to-column connection systems

Note: The following systems are to be considered as possible solutions, offering the practitioners a variety of options for frame system construction. However, the optimum/proper solution to be applied in practice may differ and should be carefully selected and implemented on a case-by-case basis.

5.3.2.1 System S1

In Figure 5-23 a moment-resisting beam-to-column connection is shown (plan view) that consists of:

- a precast or cast-in-situ rectangular/square column;
- two inverted T-shaped precast beams;
- hollow-core slabs; and
- cast-in-situ concrete to complete the beam-column connection and floor topping.

This is a typical beam-to-column connection system extensively used in, among other countries, New Zealand (mostly for perimeter frames) and Japan, where the precast beam elements span from column to column and sit on a short ledge of the column (see Fig. 5-23).

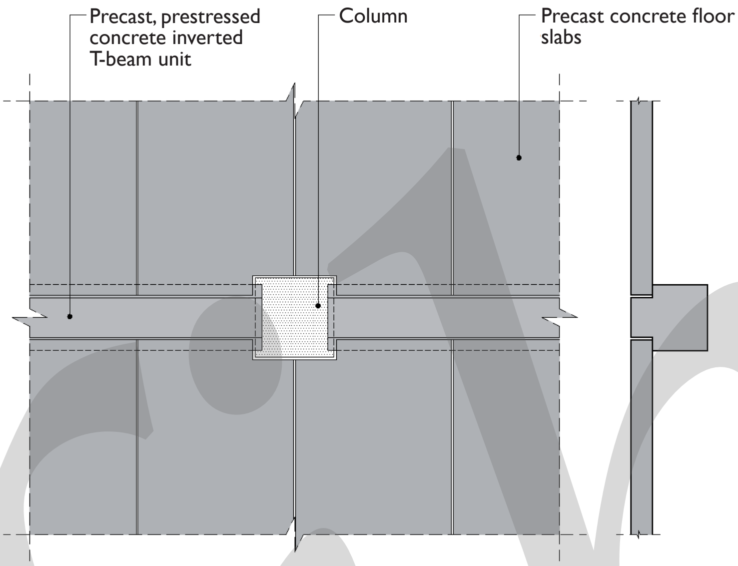


Fig. 5-23
System S1 plan view of a moment-resisting connection with inverted T-shaped beams (other details not shown for clarity)

To cover the positive moment developed at the connection, the protruding longitudinal bottom bars of the precast beams are bent upwards and anchored in the cast-in-situ concrete of the joint core (see steps 2, 3 and 4 in Fig. 5-24). Alternatively, as shown in the next system, S2, U-shaped shell beams can be adopted to allow for both positive and negative reinforcement to be inserted prior to the casting of the connection.

For the negative moments, suitable reinforcement will be placed in the area of the cast-in-situ concrete during the construction stages (see steps 2, 3 and 4 in Fig. 5-24).

The **construction steps** (1 to 7) are shown in Figures 5-24, 5-25 and 5-26.

- The columns are erected in their final position within the allowable tolerance limits. The columns may also be cast-in-place with in-situ concrete.
- The precast beams, made with reinforced or prestressed concrete and provided with protruding bottom bars, are placed between columns seated either on the cover of the columns or suitably propped adjacent to the column.
- Precast slabs, which form the floor system, are placed on the ledge of the beams.

As can be seen in Figure 5-23, the hollow-core slab units (or alternative flooring systems) sit directly on the inverted T-beams, except at the location of the columns where a gap exists between the bottom of the precast slabs and the top of the column. At the location of the columns where no beams exist, the slab units sit partly on the beam and partly have no support (see step 5a in Fig. 5-25). Alternative solutions in practice adopt a cast-in-situ floor strip at the location of the column. This is also done to better accommodate the displacement incompatibility between the floor system and the lateral-resisting system, including beam elongation effects (see Section 8 for more information). For the unsupported length of the hollow-core units along the two sides of the column, temporary local propping (or permanent support) is better used before the relevant hollow-core units are placed, depending on the extent of the unsupported length of the unit (see step 5a in Fig. 5-25).

- Stirrups will be properly arranged on the column part of the junction, while the whole area of the floor (above beams and slab units) will be suitably reinforced (see step 6a in Fig. 5-25 and steps 6b and 7 in Fig. 5-26). Proper shuttering is placed on the sides of the column (see step 5b in Fig. 5-25) to ensure proper in-situ concreting.

- Cast-in-situ concrete is poured over the floor system (topping) and the joints between precast beams and precast slab units, as well as the beam-to-column joint cores (see step 7 in Fig. 5-26). The concrete topping is reinforced with steel mesh properly designed to provide the required diaphragm action. Ductile mesh reinforcement should be used, at least in regions of moderate-to-high seismicity, to accommodate displacement incompatibility effects between floor and lateral resisting systems.

- Columns of the next storey are then built using normal RC detailing if the columns are cast-in-situ, or by other means (such as grouted steel sleeves to connect vertical bars) if the columns are precast.

The next paragraphs provide **system requirements** and **additional information**.

Experimental tests (Restrepo et al., 1995) on such systems have shown a performance similar to that achieved with cast-in-situ concrete construction. In this respect:

- If the beams are seated on the cover concrete of the column then the bearing width s_d should satisfy the followings (Restrepo et al., 1995):

$$s_d \geq \frac{V^o}{0.85\, f'_c \times b_w}$$

$$s_d \geq 0.03 h_b$$
$$s_d \geq 30\text{mm (1.18 in.)}$$

where
b_w is the width of the beam; h_b is the beam depth; V^o is the shear force at the column face corresponding to the development of the flexural overstrength. The flexural overstrength is generally assumed to be 1.25 times the nominal flexural strength calculated with the longitudinal beam reinforcement as detailed.

The capacity of the cover area and concrete strength of the cover on which the beams rest should also be able to withstand all relevant loadings.

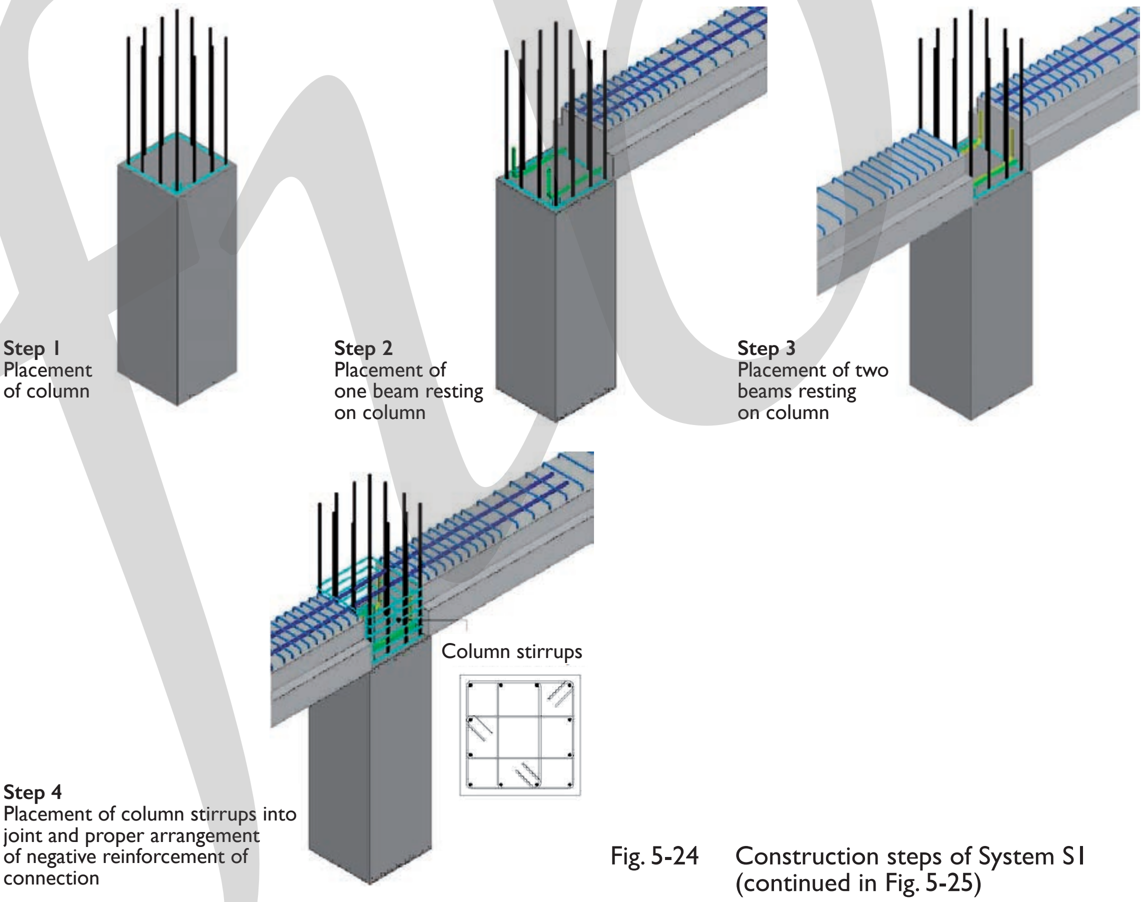
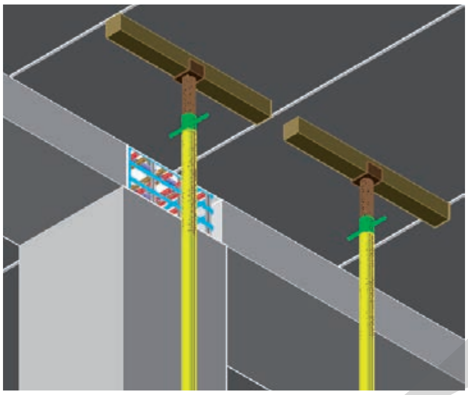
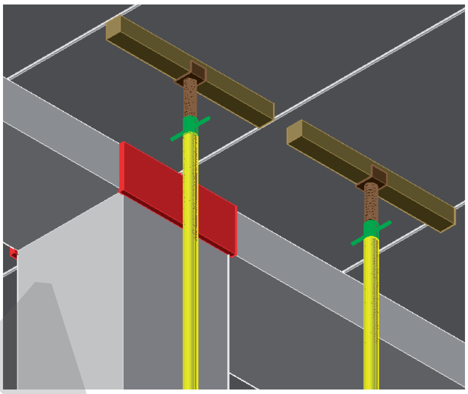


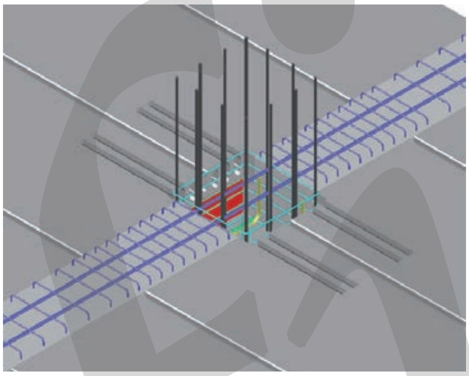
Fig. 5-24 Construction steps of System S1 (continued in Fig. 5-25)



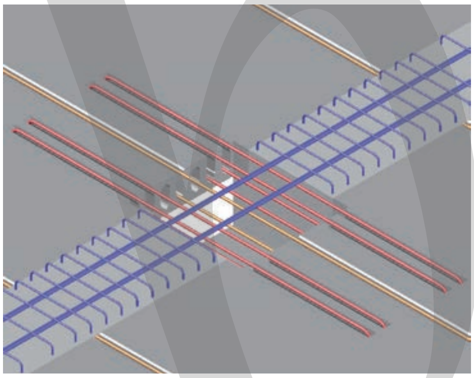
Step 5a
Propping for hollow-core units resting partially on beams at column beam junction



Step 5b
Closure of gap connection with cast-in-situ concrete (under hollow-core units)

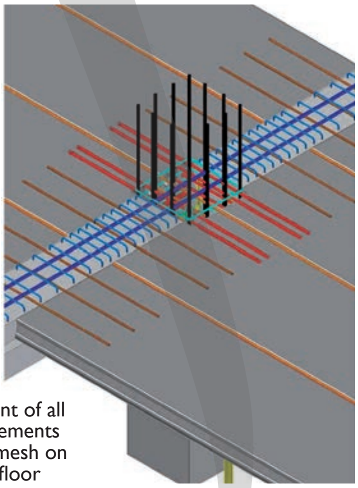


Step 5c
Placement of hollow-core units

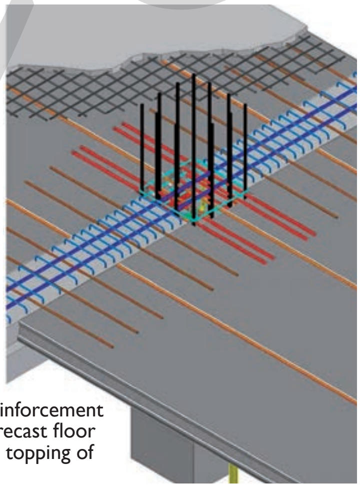


Step 6a
Placement of reinforcements on open cells of hollow-core units and between slab units. (Other reinforcements in core not shown for clarity)

Fig. 5-25
Construction steps of System S1 (continued in Fig. 5-26)



Step 6b
Placement of all reinforcements except mesh on precast floor



Step 7
Placement of reinforcement mesh all over precast floor and of concrete topping of whole slab

Fig. 5-26
Construction steps of System S1

- The bent-up bottom bars inside the joint core:
  - need to protrude (or extend) from the ends of both beams in such a manner as to permit the easy placement of the beams during construction and the correct positioning of the bars in the joint core (see steps 2 and 3 in Fig. 5-24);
  - need to be anchored at the far side of the joint core. The overlapping length between these bars should not be less than:

    $l_o = l_{b,eq} + 8d_b - a$

    where
    $l_{b,eq}$ is the development length of the hooked bar;
    d_b is the diameter of the hooked bar;
    a is the clear cover to the extension of the hook.

- The surface of the joints should be clean, free of laitance and wet before the pouring of the topping.

- The interface between in-situ concrete and beams in the beam-to-column joint cores should be made intentionally rough enough to accommodate shear. It is recommended that mechanical shear keys are created at the vertical ends of the precast beams during the casting of the precast beams (see Fig. 5-27).

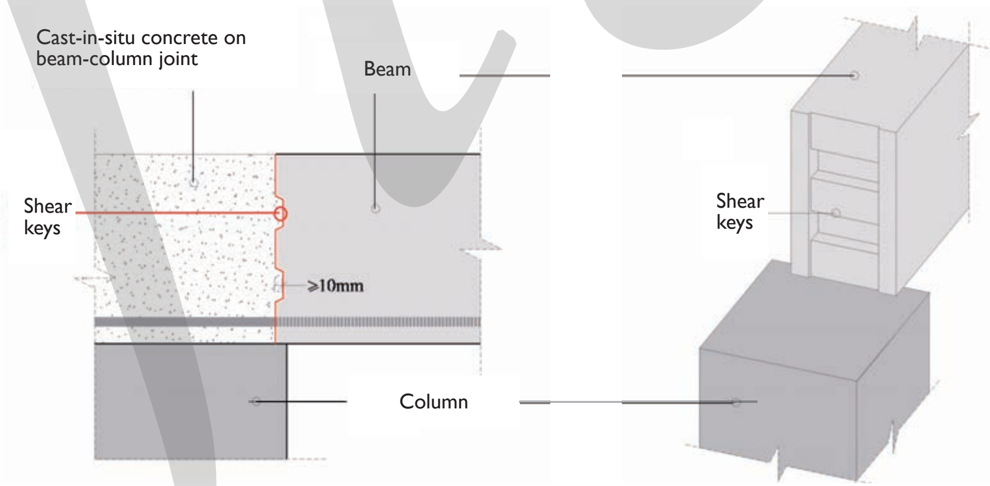


Fig. 5-27 Castellated beam ends

- Smooth vertical joints between the in-situ concrete of the cores and cross sections of the beam ends may also be used but for rather low shear stress at the interfaces between the ends of the beam and the cast-in-situ concrete of the core.

- If beams are seated on the concrete cover of the columns prior to connection via concreting of the core, particular attention should be given to the production of the members and particularly to erection tolerances, which should be very tight. Otherwise, the beams should be suitably propped adjacent to the columns.

- Corbels on column may also be used for better and easier erection methods (see Fig. 5-28).

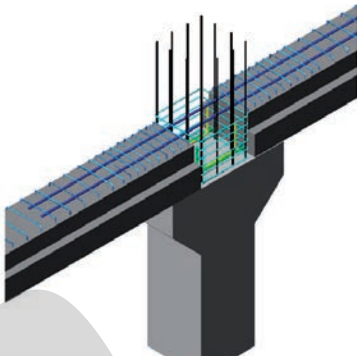


Fig. 5-28 Columns with corbels

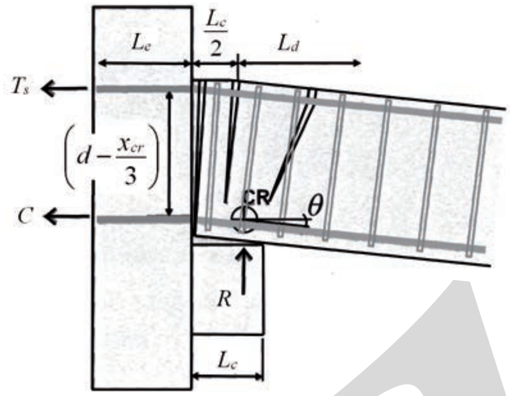
The **use of column-cover concrete** to seat the precast beams generally creates significant constraints and difficulties, such as:

- Erection with very tight tolerances
- The need to ascertain that the available cover concrete of the column has the required strength
- Suitable propping of the beams adjacent to the columns, if propping is used

All the above constraints can be avoided if corbels (or similar alternative solutions to providing a reliable shear key as slotted metallic insert) are used.

When **corbels** are used the following should also be considered:

- The centre of rotation (due to the reaction of the beam at the corbel) is expected to relocate at the compression zone of the horizontal joint between the end of the beam and the corbel (see Fig. 5-29) when compared to the counterpart solutions without corbels (see Fig. 5-30). If allowances for such mechanisms are made by using rigid body considerations around the corbel edge as a pivot point and including proper reinforcement for the corbel and limited length of the corbel itself, this relocation might not significantly affect the ultimate behaviour of the connection, but it might be useful within the structural analysis for the evaluation of displacements at the serviceability limit state (SLS).



Elongation of steel bars

$$\Delta_s = \varepsilon_s \cdot \left(L_e + L_d + \frac{L_c}{2} \right)$$

Beam-end rotation

$$\theta = \frac{\Delta_s}{d - \frac{x_{cr}}{3}}$$

Fig. 5-29
Idealization of deformed configuration of beam-to-column connection with concrete corbel (based on Ferreira et al., 2010)

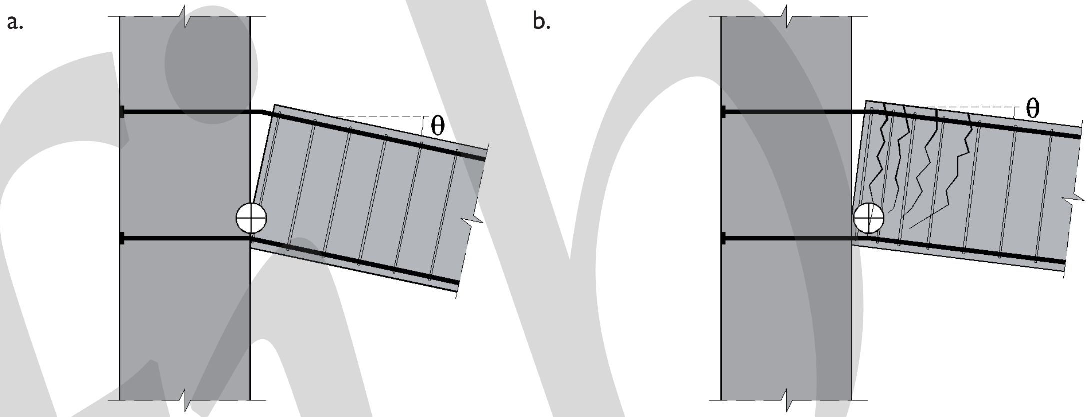


Fig. 5-30 Deformation and crack configurations in beam-to-column connection without corbel
a. Crack opening and center of rotation at beam-to-column interface
b. Crack propagation at end of beam (based on Ferreira et al., 2010)

In general, care should be taken in checking the displacement incompatibility effect between the rotation of the beam and the presence of the corbel. High-strength pads under the beam at the corbel support (or similar solutions) should be used to allow the beam to develop its natural flexural mechanism at the column interface without compromising the structural integrity of the element and thus the capacity of the connection.

- The initial loadings are:
  - the self-weight of the beam;
  - the self-weight of the precast floor units (seating on the beams); and
  - the self-weight of the fresh concrete of the topping (which corresponds to the beam).

These are supported by the flexural capacity of the simply supported beam (see Fig. 5-29) and are transferred directly to the column (through the corbels). Consequently, negligible moment demand is created in the beam-to-column connection under the above loadings, since the wet concrete in the connection's core does not cause any restraint to the beam-to-column connection (that is, the deformations of beams and column are developed before the hardening of the topping concrete takes place).

The above is also true in the case of beams sitting on the cover concrete of the columns if the cover has appropriate capacity but it is not true if beams are propped between columns. In the latter case, the self-weight of the beam and the floor slabs and the topping create negative moments once the props have been removed after the hardening of the concrete in the topping and the core of the connection.

- Under the above circumstances, when corbels are used or when beams are seated on the column-cover concrete, the negative moments acting in the beam-to-column connection are only those developed due to:

  - the dead loads of finishes (superimposed dead load);
  - live loads;
  - the possible effects of the upper levels (in regard to each connection); and
  - the seismic excitation of the total building.

  This leads to a lower level of total negative reinforcement at the beam-to-column connections.

  For these reasons, the constructibility, construction time, economy and final appearance of the finished structure can affect the choice of the construction method for the construction of the beam-to-column connections. The construction method chosen, in a similar way, can also influence the design.

5.3.2.2 System S2

In Figures 5-31 and 5-32 a moment-resisting beam-to-column connection is shown (in plan view) that consists of:

- a precast or cast-in-situ rectangular/square column;
- two U-shaped precast prestressed beams (shell beams);
- hollow-core slabs; and
- cast-in-situ concrete to complete the beam-column connection and floor topping.

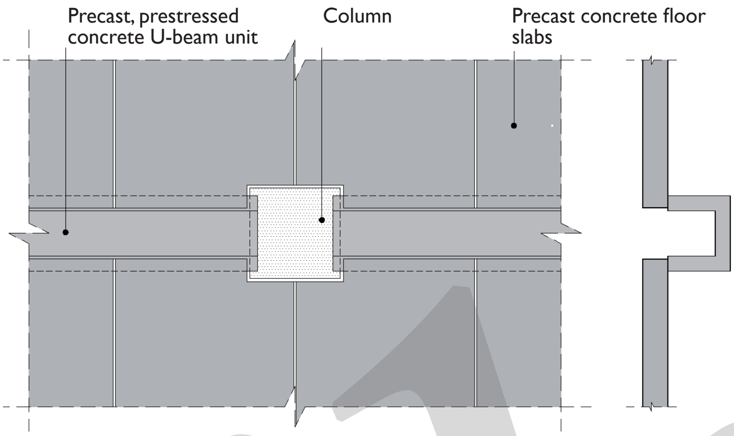


Fig. 5-31

System S2 plan view of a moment-resisting beam-to-column connection using U-shaped precast beams (other details not shown for clarity)

In System S2 the precast beams, also referred to as shell beams, are U-shaped, pretensioned and left permanently in position after the pouring of the cast-in-situ reinforced concrete. As per the previous system - S1 - the U-shaped beams may be supported either on the concrete cover of the columns or be suitably propped adjacent to them, or on column corbels.

During construction the beams are designed to carry their self-weight, the weight of the precast slab-units and the weight of the wet cast-in-situ concrete completing the beams, as well as the topping. If the beams are not propped and rest directly on the cover of the columns, the level of prestress necessary to accommodate the above loadings at midspan should be carefully considered, as well as the shear resistance at their ends, since the full prestress will be developed rather beyond the critical shear section of the beams.

After the cast-in-situ concrete has hardened, the U-shaped beams act in a composite manner (precast cell and cast-in-situ concrete core) for any other additional loading (floor finishing and live loads).

The **construction steps** are fundamentally similar to those of the previous system.

- After the erection of the columns, the beams are placed and supported either on the concrete cover of the columns (see Fig. 5-31 and 5-32) or suitably propped adjacent to the columns, or on column corbels.

- The slabs are placed on the flanges of the U-shaped precast beams.

- The unsupported part (if any) of some hollow-core units (or similar floor systems) at the column locations on opposite sides of the columns should be propped and shuttering carried out for the gap in the column below the hollow-core units.

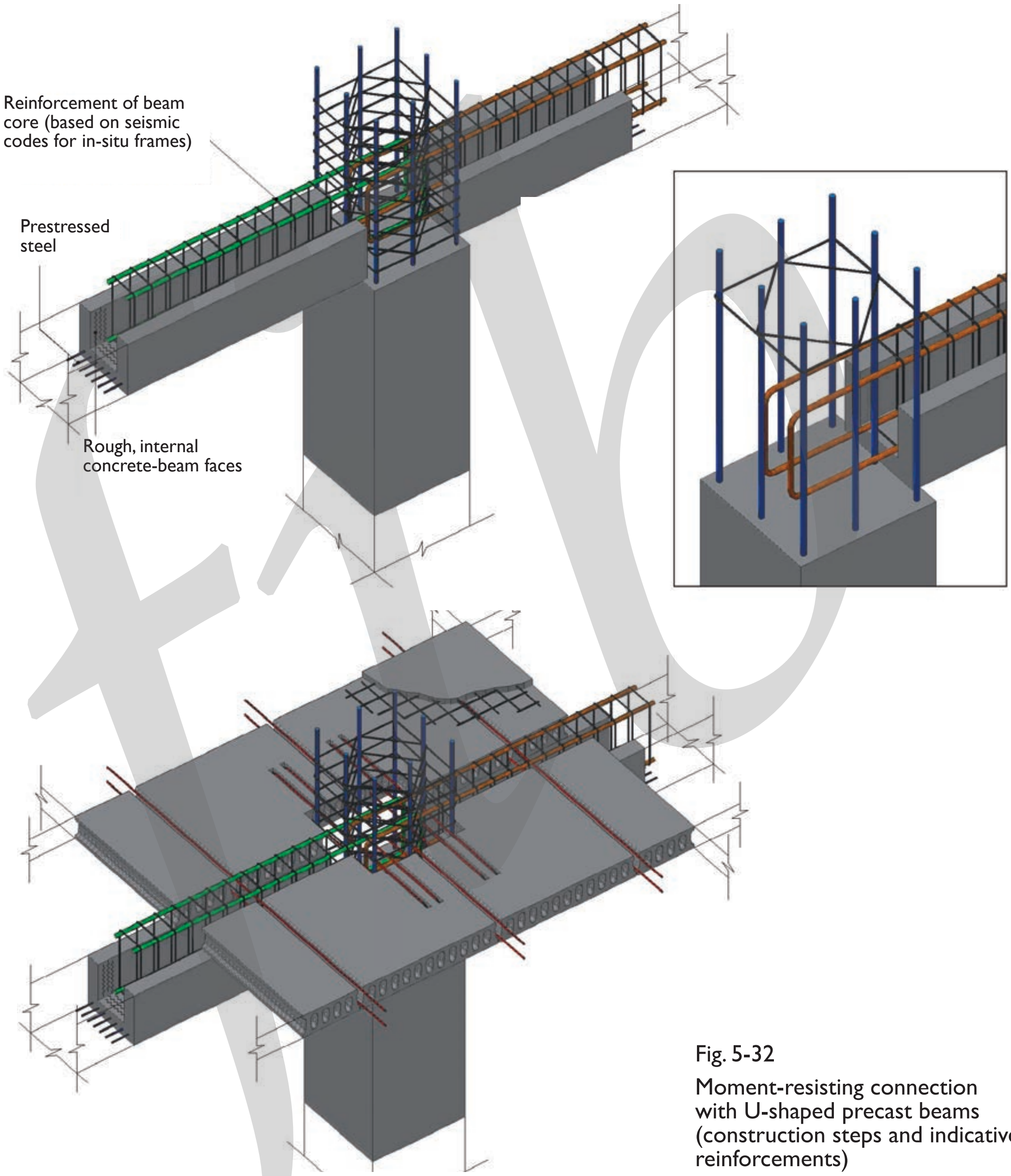


Fig. 5-32

Moment-resisting connection with U-shaped precast beams (construction steps and indicative reinforcements)

- Continuity reinforcing is placed in the beam that passes through the column; in the beam column-joint core; in the infill cells of some of the hollow-core units that may not sit on beams (or in permanent supports) at the two sides of the column; between adjacent hollow-core units; and above the precast slabs. Another similar method is to position the 'U' cross section only at the ends of a beam of sufficient length to develop the connection bars. This creates a stronger precast beam and eliminates the need for shoring.

- Cast-in-situ concrete is used for the whole slab (topping and cores).

- The columns of the next storey are then built using normal RC detailing if the columns are cast-in-situ or, if the columns are precast, by other means, such as grouted steel sleeves to connect vertical bars.

The following points cover **system requirements** and **additional information**.

- Tests by Bull et al. (1986) and Park (1995) have shown that such systems are suitable for use in ductile moment-resisting frames (referred to as FCFs in this document).

- Particular attention should be paid to the interfaces of the precast beams and cast-in-place concrete core of the beam, which should develop a proper bond (under additional loading after the hardening of the in-situ concrete) in order to achieve composite action and develop a proper plastic-hinge length. Normally, composite action may be assumed when the inner surface of the U-beams is intentionally roughened, for example, with a depth of 3 to 5 millimetres, or 0.12 to 0.2 inches (see Fig. 5-32). Stirrups protruding from the U-beams may be used to improve the interface shear strength. In all cases, it is important during construction to ensure that the inside surfaces of the U-shaped beams are very clean when the cast-in-situ concrete is placed. Otherwise, sufficient bond in the interfaces or precast and cast-in-situ concrete cannot develop.

- The imposed horizontal shear stress at the interface of the contact surfaces between the U-beam and cast-in-situ concrete core arise, during positive bending moment, from the transfer of the prestressing steel tension force from the U-beam to the core, and, during negative bending moment, from the transfer of the reinforcing steel force from the core to the U-beam flange.

  The horizontal interface shear stress τ_{hor} can be estimated using the following simplified expression proposed by Bull et al. (1986):

$$\tau_{hor} = V_u / b_{int} \times h_{eff}$$

where
V_u is the vertical shear force at ultimate stage;
b_{int} is the total width of interface (two sides and bottom surface);
h_{eff} is the effective depth of the composite section.

However, the transfer of vertical shear stresses across the interface will be more critical at the ultimate load if the end support of the U-beam in the column cover is lost during seismic loading. In this case, the vertical shear stress, which should be checked, will arise from all (factored) dead and live loads acting on the beam.

In Figure 5-33, a schematic section of a composite beam is presented.

The critical sections for flexure in the beams under gravity and seismic loading occur, in general:

- for positive moments, away from the columns;
- for negative moments always at the beam ends.

The positive moment flexural strength at the beam ends in the beam-to-column connection will be provided only by the longitudinal reinforcement of the cast-in-situ concrete of the core of the beam and the concrete of the joint core and topping, since the prestressing strands at the bottom of the precast beams terminates at the beam ends. Away from the beam ends and after the full development of the prestress of the precast beam, the positive moment flexural strength is characterized by the full composite action of the section (see Fig. 5-34).

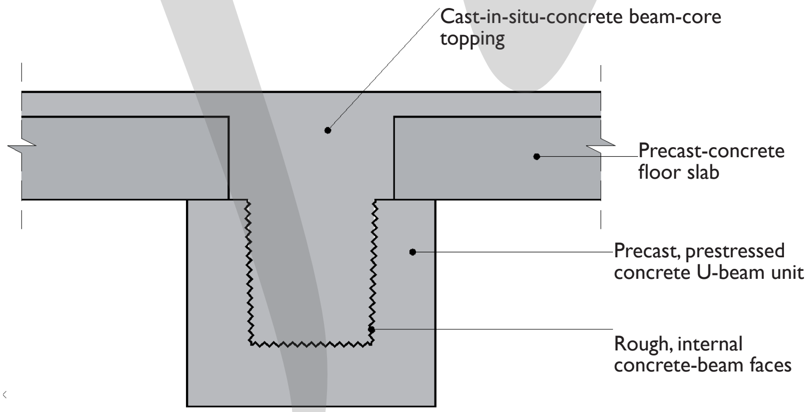


Fig. 5-33
Section of composite beam (other details not shown for clarity)

The lower limit of negative moment flexural strength at the beam ends is provided by the cast-in-place reinforced concrete core alone. This refers to the case when the precast beams seats on the column concrete cover and either this or the interface bond between the joint core concrete and the precast beam breaks down during seismic loading. The upper limit of the negative moment flexural strength away from the ends will be due to the composite section of the beam.

Figure 5-34 shows an application in Italy of precast shell beams supported on small corbels.

Fig. 5-34 Application of System S2 in Italy
Left: Shell beams are supported in small column corbels
Right: Slabs seated in beam ledges
(C.A.S.E. project - L' Aquila earthquake - April 6, 2009; photos courtesy of Stefano Pampanin)

5.3.2.3 System S3

In Figure 5-35 a moment-resisting beam-to-column connection is shown consisting of:

- a precast or cast-in-place column;
- one precast beam passing through the column top;
- hollow-core slabs; and
- cast-in-situ topping concrete.

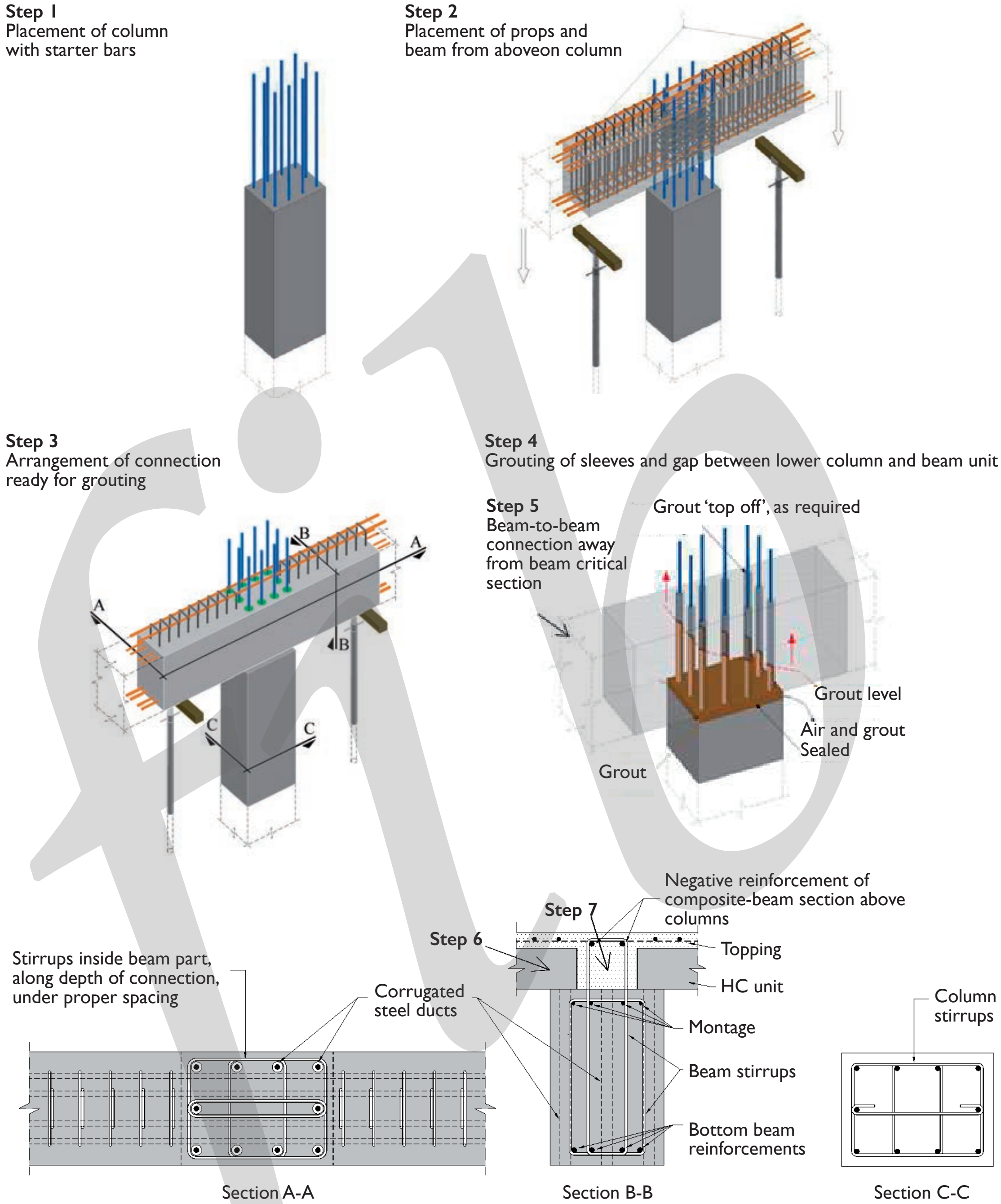


Fig. 5-35 Beam-to-column connection in which precast beam units pass through column

System S3 is characterized by continuous beams-on-single-storey column elements. The precast beams thus incorporate the beam-to-column joint region (see Sections A-A and B-B of Fig. 5-35), thereby avoiding the need to pour cast-in-situ concrete in the congested beam-to-column joint core region, as per the previous two systems (S1 and S2). This type of connection belongs to the family of strong connections, where the precast elements are connected away from the location where the inelastic mechanism is expected to occur. The system is designed to develop a weak-beam and strong-column mechanism, following capacity design principles. Ductile plastic hinges are thus expected to occur at the beam-to-column interface, as per any traditional, monolithic solution. The length of the precast beam is such that it extends away from both sides of the columns for a distance of at least 2.5 × h_b, where h_b is the depth of the beam (that is, intended to be far away from the possible plastic-hinge length of the beams) or extends from midspan to midspan, depending on the spans of the beam and other construction restraints. The precast beam includes, in the region over the columns, the proper arrangement of the joint-core reinforcement. The precast beam is placed on the concrete column with suitable material at the connection interface to allow proper grouting (see details for column-to-column connections in the relevant section of this document) and is propped beyond the column for construction stability (see steps 2 and 3 in Fig. 5-35). The lower-column longitudinal starter bars pass through the corrugated-steel ducts (see Section A-A in Fig. 5-35) of the proper diameter provided in the beams and extend above the top surface of the finished floor. The precast beam and the column are then connected by grouting the corrugated ducts and the gap between the lower column and the beam unit (see step 4 in Fig. 5-35).

The **construction steps** are the following (see Fig. 5-35):

Step 1: Columns are placed.
Step 2: Propping is arranged to support the portions of the precast beam beyond the columns and the beam is placed in such a way that a 20-millimetre (0.79-inch) construction joint is allowed in the horizontal connection interface between the column and beam for grouting.
Step 3: The connection is arranged in this way before grouting.
Step 4: The horizontal gap between the column below and the precast beam unit is sealed around the perimeter and the grouting operation is performed.
Step 5: Beam elements are connected to each other either at their midspan or elsewhere, but away from their critical section (which corresponds to a strong connection solution), by, for example, the lapping or welding of the bars in a cast-in-situ joint.
Step 6: The precast-floor system is placed on top of the precast beams and spans across them (see Section B-B).

Step 7: After placing the reinforcement at the top of the precast beams and slabs, the topping concrete is poured.

The columns of the next storey are then built using typical reinforced-concrete detailing, if the columns are cast-in-situ or, if the columns are precast, by other means, such as grouted steel sleeves to connect vertical bars.

The following points cover **system requirements** and **additional information**.

- Experimental tests (Restrepo et al., 1995) have shown the excellent performance of System S3, which is used extensively in New Zealand, among other countries.

- Attention should be paid to the production and erection tolerances, which are very tight.

- Corrugated ducts similar to those used in grouted post-tensioned construction should be used. Plastic or smooth metallic ducts are unacceptable as they cannot develop adequate bond conditions.

- The diameter of the duct should accommodate tolerances, plus a recommended additional 10 ÷ 12 mm (0.39 ÷ 0.47 in.) clearance between the duct surface and bar surface to allow grout to flow between the duct and bar. Generally, the duct diameter ranges from two to three times the nominal diameter of the bar.

- It is highly recommended that the compressive strength of the grout is about 10 MPa (1450 psi) greater than the compressive strength of the concrete of the precast beam.

- Grouting should be carried out very carefully.

  Regarding the grouting of this type of beam-to-column joint, the following procedure, with two methods of grouting, should be used, as reported in Park (1995):

  Method 1: The horizontal joint at the beam-to-column interface is first sealed around the outside and then grout (typical nonshrink, cement-based) is pumped in at an inlet port (or tube) at one corner of the horizontal joint to displace air progressively across the interface. If the grout has a high viscosity, it may start to flow up the open ducts, starting at the duct closest to the grout inlet. It is recommended that outlet ports are provided at the other three corners of the interface. These are progressively plugged once grout without air bubbles *flows out. When all the outlets are plugged, further pumping of grout will result in the ducts being filled upwards from the bottom. The duct nearest the inlet should fill first while the one furthest away (opposite corner) may require topping up by use of a tremie tube, or by pouring-in from a dispenser in such a way that grout runs down in contact with the reinforcing bar to avoid air-locks. The inlet tube or port is plugged once injection is completed.*

Method 2: The horizontal joint at the beam-to-column interface is first sealed around the outside and then grout is poured in from a dispenser down one corner duct. Progressively, the grout will flow across the interface and up the remaining ducts. It is recommended that, as with Method 1, outlet ports be used to confirm the progress of the grouting of the interface. Topping off of ducts remote from the filling position may be necessary. Again, care must be exercised so that no air is trapped in the ducts.

The grouting operation must be undertaken using proper quality-assurance procedures to ensure that all voids are properly and solidly grouted.

- A possible advantage of this system is that, under proper detailing, the potential plastic-hinge regions in the beams occur within the precast elements, away from the beam-to-column jointing faces.

Figure 5-36 shows an example in New Zealand of the application of an emulative approach using System S3.

Fig. 5-36
Application of emulative approach using System S3 in IRD Building, Christchurch, New Zealand
(Photo courtesy of Stefano Pampanin)

5.3.2.4 System S4

Figures 5-37 and 5-38 show a wet/welded, moment-resisting beam-to-column connection consisting of:

- a precast, rectangular column with corbels (multi-storey column elements or lapped one-storey column elements);
- two inverted T-shaped precast beams;
- hollow-core or double-tee slabs; and
- cast-in-situ concrete topping.

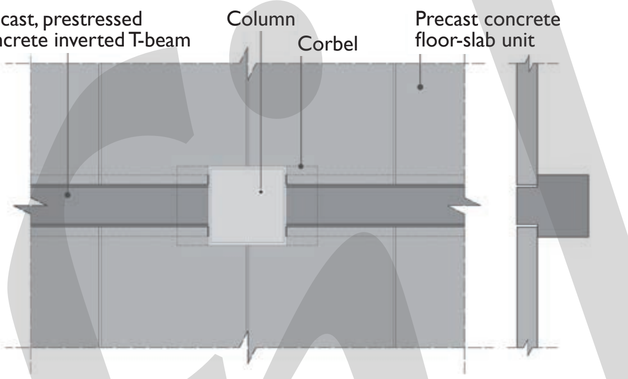


Fig. 5-37
System S4 plan view and cross section of a moment-resisting wet/welded connection using inverted T-shaped beams (other details not shown for clarity)

The connection may well be used for heavy service loads due to its inherently high gravity load transfer capacity. Gravity loads are transferred directly through corbels, instead of by shear mechanism between the connecting members.

The precast-concrete columns with corbels are either cast per storey, allowing the negative moment reinforcement to be continuous over the column, or cast as multi-storey elements, with openings at storey levels allowing for the beam top reinforcement to pass through for negative moment continuity. The openings in the columns, mostly in the form of non-concreted segments of 10 to15 centimetres (3.9 to 5.9 inches) in height, are grouted during the pouring of the floor concrete topping. On the other hand, the continuity of the beam positive moment is achieved through the welding of two steel plates, one located on the beam end (bottom face) and the other on the corbel (top face) (see Fig. 5-38).

In the United States, where pretensioned beams are typically employed, there is concern about volume-change strains and the ends of the beams are typically not welded. The connection of the beam to the column is achieved with dowels from the column corbel passing through sleeves in the ends of the beams, which are then grouted. Bearing pads are provided at the beam to the corbel interface.

Special reinforcement detailing is used to connect the steel plates to both the corbels and the beams (see Fig. 5-39). The connection between the reinforcement and the steel plate is obtained through welding. The steel plate on the beam end is mainly welded to the beam bottom reinforcement in order to transfer the steel tensile stresses, hence the beam positive moment. Along with the beam bottom reinforcement, special detailing rebars are also welded to prevent any local diagonal failures caused by the concentrated pull force on the plate.

Beam shear reinforcements at the beam ends, over the steel plate, are welded to the steel plate itself in order to provide concrete confinement.

Experimental tests under quasi-static cyclic loading on the wet/welded moment-resisting connections (Karadogan et al., 2012), showed satisfactory behaviour in terms of the suitability of the connection in seismic regions.

The cyclic behaviour of the connection was characterized by ductile and energy dissipating loops, with the ultimate and yield capacities being easily predicted by section analysis (Karadogan et al., 2012).

Since the beams are seated on corbels, no special shoring is necessary until the concrete topping is poured. Shoring only needs to be carried out for the flooring members (hollow-core or double-tee) in the regions adjacent to the columns, as discussed in System S1. The welding of the steel plate is performed before the concrete topping is poured and the system starts behaving as a monolithic connection only after the topping is cured.

In some regions the reinforcing bars are produced with high (equivalent) carbon content in order to comply with strength requirements. In such cases, quality assurance should be obtained.

The **construction steps** are the following:

- The columns with corbels (steel plates located on the top surface of the corbel) are placed and the beams with steel plates (at each beam end, lower surface) are seated on these corbels. No special shoring is needed. The beams are simply supported at this stage of construction and bearing pads may be provided.

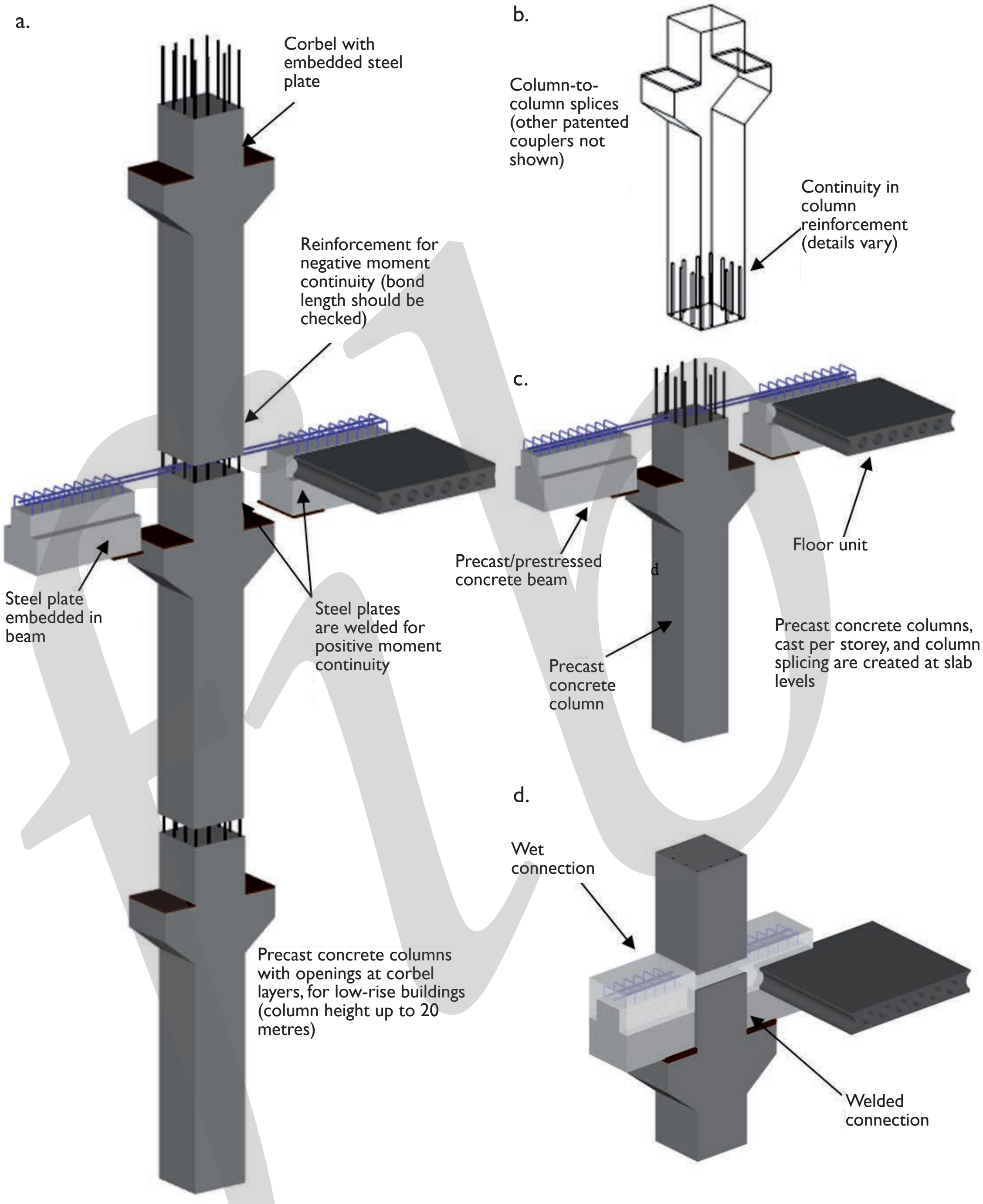


Fig. 5-38 Moment-resisting wet/welded connection with inverted T-shaped beams (indicative reinforcements)
(Other details not shown for clarity)

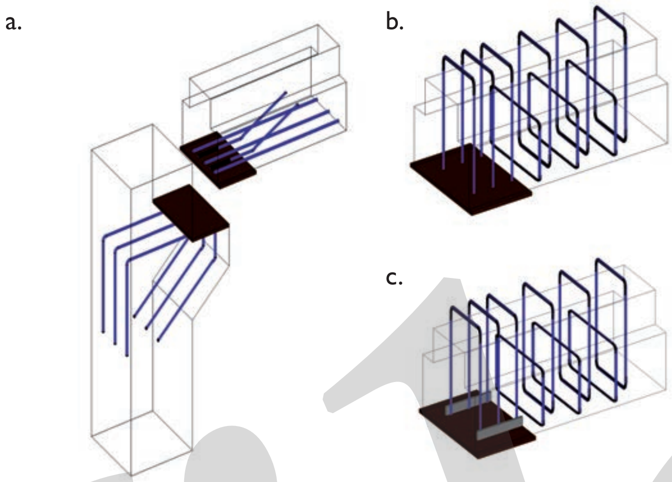


Fig. 5-39
Shear-reinforcement detailing at beam end and positive reinforcement welded to steel plates

- The floor units, either hollow-core or double-tee, are seated on the inverted T-beams. The edge beams should be checked carefully for torsional stability.
- Props or column supports are used for the unsupported floor units or parts, particularly in the region adjacent to the columns; shutters are to be placed on the column sides before pouring the concrete.
- The beam negative-moment-continuity reinforcement, the column core confinement reinforcement, the floor-unit tie reinforcement and the diaphragm topping reinforcement are placed in the form of a mesh.
- Welding of the steel plates (corbel to beam) is carried out
- Quality checks are performed on the welding. The reinforcement and the steel plates must be weldable.
- Cast-in-situ concrete is poured for the whole slab (topping and cores). It should be noted that the system becomes statically indeterminate only after the curing of the concrete.

The following points cover **system requirements** and **additional information**.

- Special attention should be paid to the selection of weldable beam flexural reinforcements and weldable steel plates. Since the equivalent carbon content (carbon equivalent) is an important indicator of both hardness and weldability, the quality control of the carbon equivalent of reinforcing bars has primary importance. A high amount of carbon and other elements such as manganese, chromium, molybdenum, vanadium and nickel tend to increase steel hardness while decreasing the weldability, leading to weld cracking.

- The shear/confinement reinforcement at the beam ends, where the positive reinforcement is welded to the steel plates, should have a special configuration, as shown in Figure 5-39c.

- The anchorage length of the negative reinforcement should be increased by 40% to account for the adverse effects of the thin layer of the concrete topping. For the edge and corner columns, where the column dimension is less than the calculated anchorage length, either rebars with hooked ends or U-shaped rebars should be used.

- The thickness of the steel plates, which assures the moment continuity through welding, should not be less than 15 millimetres (0.59 inches) in both the corbel and the beam itself. The actual steel-plate thickness should be selected such that the in-plane stresses, principal stresses and shear stresses are well below the steel yield limit.

- The weld itself, either plate-to-plate or reinforcing bar-to-plate, should carry all the forces induced by the moment capacity of the connecting beam. The force to be transferred through tension (from the beam positive reinforcement-to-steel plate, from the steel plate-to-steel plate, from the steel plate-to-corbel) is to be calculated with the ultimate strength of the reinforcing bars taking the strain hardening stage into consideration. The weld capacity should be at least 50% higher than the ultimate tension force to be transferred.

- The moment capacity of the connection is calculated through simple section analysis, provided that the welds and anchorages do not prematurely fail.

5.3.2.5 System S5

Figures 5-40 and 5-41 show a wet-in-beam moment-resisting beam-to-column connection consisting of:

- a precast, rectangular column with protruding U-shaped beam reinforcement;
- two inverted T-shaped precast beams; ends with protruding positive reinforcement;
- hollow-core or double-tee slabs; and
- cast-in-situ concrete topping.

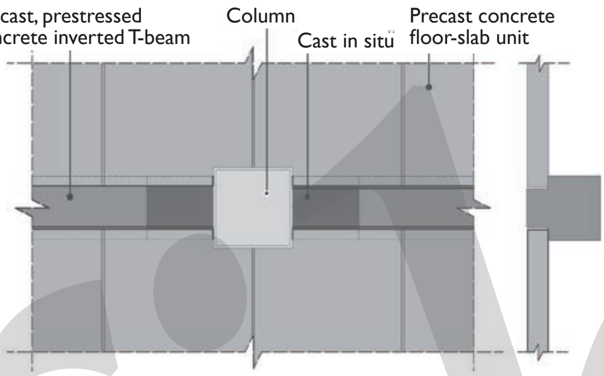


Fig. 5-40
System S5 plan view and cross section of a moment-resisting wet-in-beam connection using inverted T-shaped beams (other details not shown for clarity)

The wet-in-beam moment-resisting connection is a sort of emulation of the cast-in-situ connection, where the columns are either spliced at storey mid-heights or continuous in the case of mid-rise buildings. The column surface and the beam section in the connection region are roughened in order to transfer the gravity loads through shear. The wet-connection, which lies over a length approximately equal to the beam depth, is typically filled with fibre-reinforced concrete to improve the rebar bond properties and the connection shear strength.

The shoring in wet-in-beam connections are more robust compared to other types of precast connections since the member loads, the construction loads and the load of the topping concrete have to be carried solely by the shoring until after the concrete topping is cured.

The columns in this connection type are spliced mainly at storey mid-height. Such a splice location reduces the strength demand on the splice, resulting in a more robust seismic response of the overall structure. On the other hand, in such connections, the beam reinforcement protruding from the face of the column is usually greater than that of the beam flexural reinforcement, resulting in the plastic hinge being relocated away from the column interface. It should be noted that such a relocation of the hinge point leads to a higher rotational ductility demand in the beam x-section for the same drift ratio, when compared to the case of plastic hinges forming right at the column interface.

Generally, the connection capacity can be calculated easily through section analysis, as per traditional monolithic connections.

The **construction steps** are the following:

- The columns with horizontal reinforcement protruding from faces (for positive and negative moment continuity) are placed with relatively higher tolerances compared to other precast connections.

- The required amount of shear reinforcement in the lap-spliced length is hung on the reinforcement protruding from the column.

- The beams with protruding positive flexural reinforcement are seated on heavy shoring; the formwork, either from timber or steel, is prepared for the pouring of the concrete into the wet-in-beam connection. The beam flexural reinforcements protruding either from the column or the beam elements should be spliced thoroughly (see Fig. 5-41).

- The negative moment reinforcement is tied under the protruding shear reinforcement of the beam. Adequate bond length should be supplied.

- The shear reinforcement must be equally spaced, and tied to the longitudinal reinforcement.

- Floor units are seated on the beams. The floor units adjacent to the columns and the splice region in the beam should be seated on shores/propping.

- Formwork is applied to the connection region, and the concrete for the connection and for the diaphragm action is poured after the mesh reinforcement in the topping and the continuity reinforcement for the floor units have been placed.

The following points cover **system requirements** and **additional information**.

- The connection inherently requires heavy shoring on site. Therefore, structural analysis for the shoring should be part of the overall design process. Furthermore, the dead load and the construction loads of the upper storey should be taken into consideration in the design of the storey beneath.

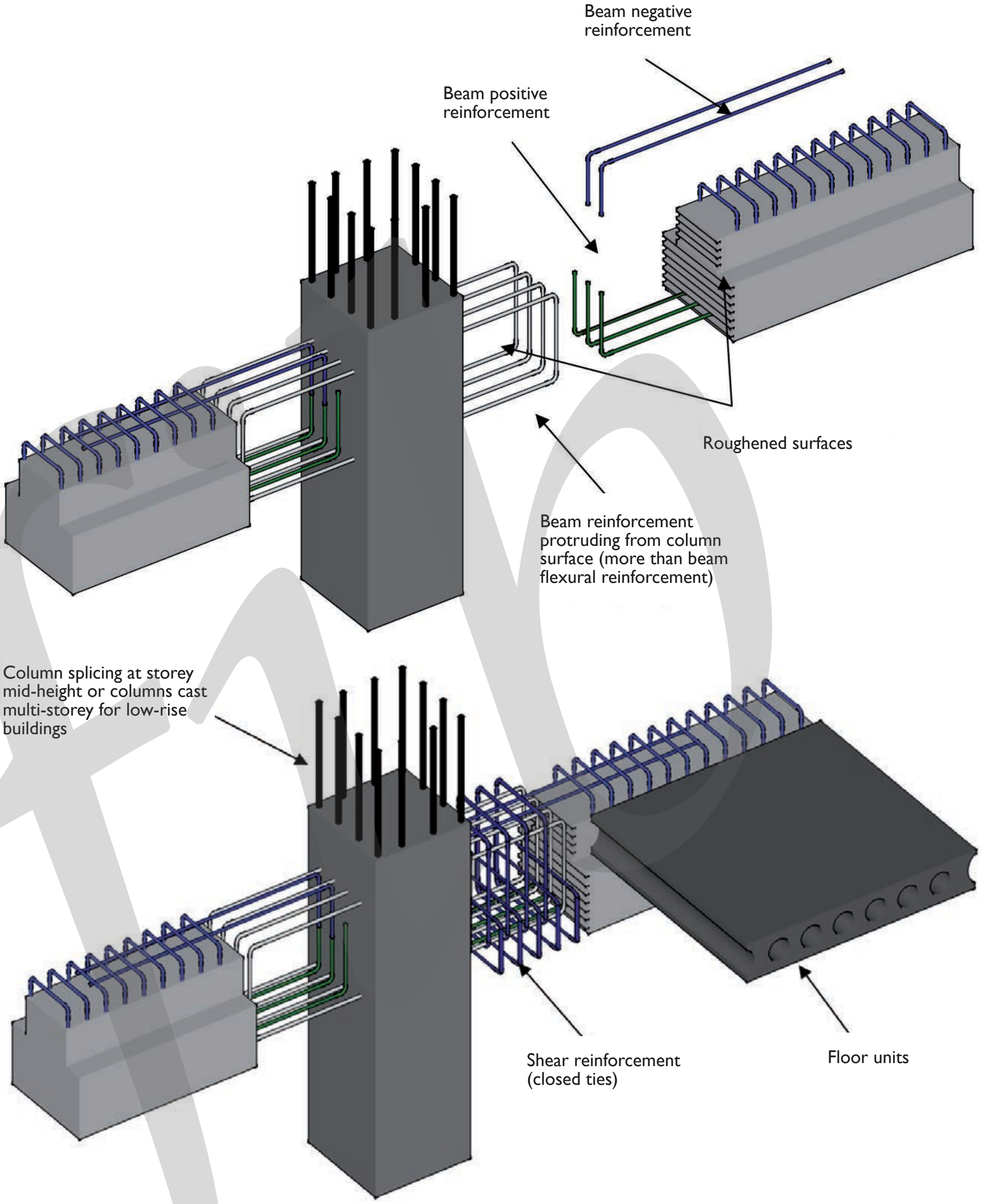


Fig. 5-41 Moment-resisting wet-in-beam connection with inverted T-shaped beams (indicative reinforcements)
(Other details not shown for clarity)

- The protruding bars found either in the column and the beam may be correctly added on site with couplers instead of the structural members with their protruding reinforcement having to be produced and transported. This may reduce the risk of reinforcement damage and breakage during transfer and ensure ease of casting, hence reducing the cost of molding and demolding.

- During a nonlinear analysis, the connection should be modelled accordingly with higher capacity at the wet-in-beam region so as to capture the rotational ductility demand of the precast beam and the overall performance of the structure.

- The lap-splice length of the flexural reinforcement should be checked and high quality control should be carried out on the concrete poured in the connection region.

Note: This type of connection may not be allowed in some countries due to the fact that the lap of primary longitudinal reinforcements takes place in the plastic hinge region.

5.3.2.6 System S6

Figures 5-42 and 5-43 depict a bolted moment-resisting beam-column connection consisting of:

- a precast rectangular column with through openings and metal demountable corbels;
- two inverted T-shaped precast beams with through openings and open channels on upper and lower faces;
- high-quality rods or post-tensioned bars;
- hollow-core or double-tee slabs; and
- cast-in-situ topping concrete.

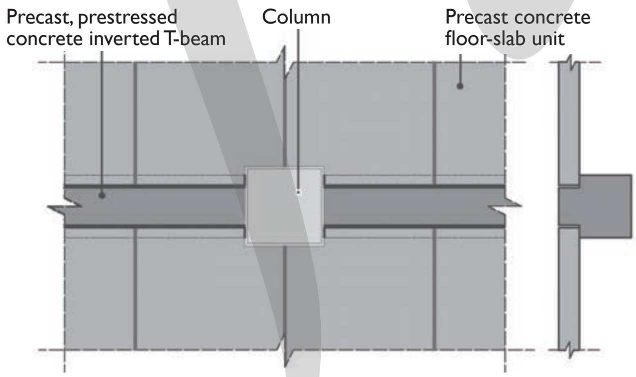


Fig. 5-42
System S6 plan view and cross section of a moment-resisting bolted connection using inverted T-shaped beams (other details not shown for clarity)

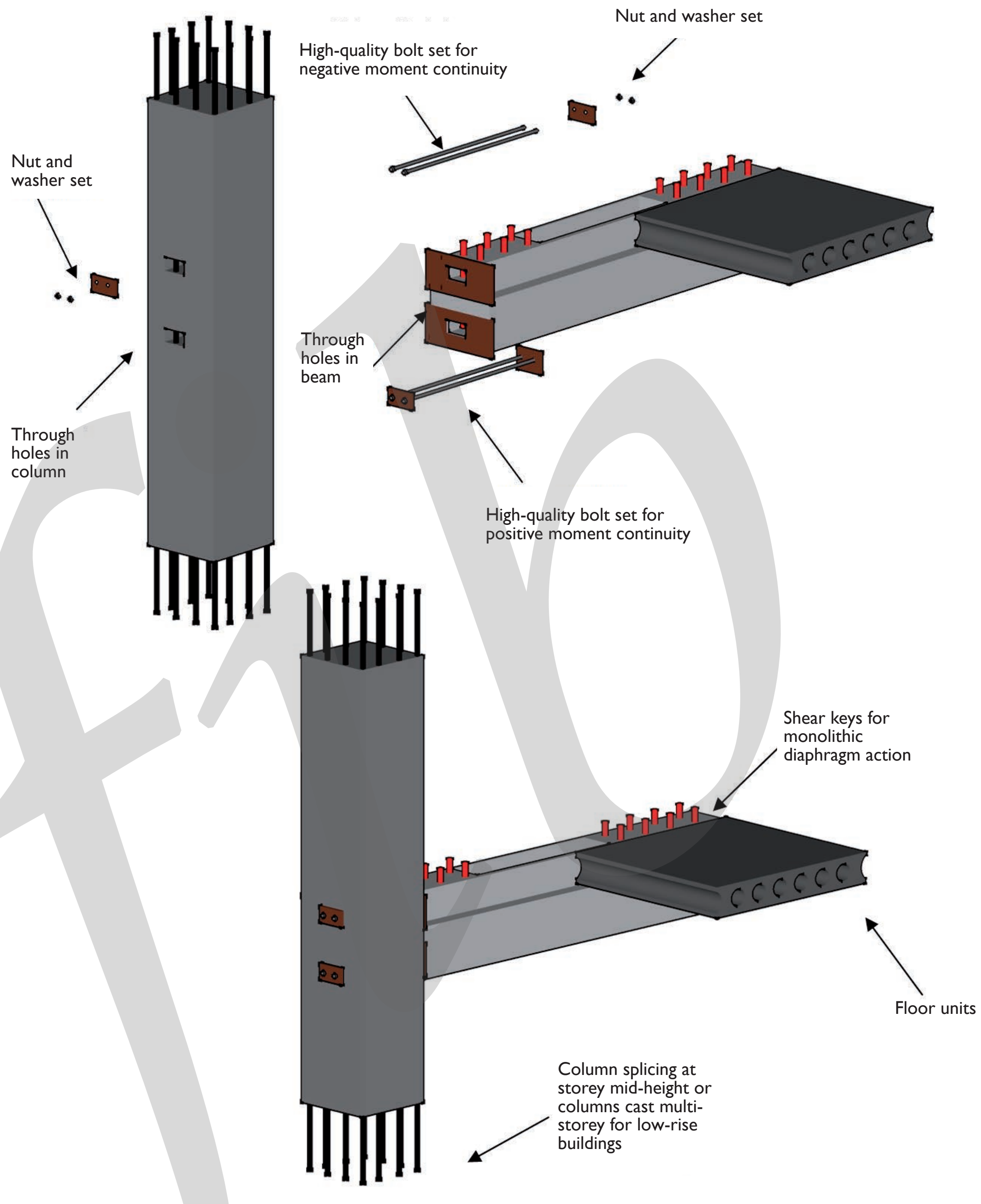


Fig. 5-43 Moment-resisting bolted connection with inverted T-shaped beams (indicative reinforcements)
(Other details not shown for clarity)

The negative and positive moment continuity in the bolted moment-resisting connections is made with high-quality bolts. The 'high-quality' mention may either refer to ductile bolts/rods with a finite yield plateau or to high-strength bolts/rods with a very high yield value and a relatively short yield plateau. The connections with either bolt/rod types have different design philosophies and hinging locations. The fixing of the bolts/rods, either with or without grouting the through-holes, also changes the connection response.

The through-holes in the connections with ductile bolts/rods (or mild steel with treaded ends) are generally grouted with high-strength grouts and such connections respond very closely to that of equivalent cast-in-situ connections. It is worth noting that the flexural mechanism is fundamentally similar to that of a tension-compression yielding (TCY) system, which was conceived and developed as part of the family of jointed ductile connections (see Priestley, 1996).

The metal duct that makes up the through-hole has two sliding surfaces: one with respect to the grout, the other with respect to the beam concrete itself. Special ribs and shear keys should be used to allow for proper anchorage and should prevent premature sliding failures over these surfaces. In other terms, the grout and the beam concrete should not have any relative displacement under gravity and/or seismic loadings.

The next key design parameter of such connections is the de-bonding and unbonded lengths of the bolt/rods reinforcement.

When ductile deformed/ribbed reinforcing bars are used for better bonding, the bond between the bar and the high strength grout can negatively affect the rotational ductility capacity of the section. In such cases, the de-bonding length of the re-bar under seismic hysteresis loops must be calculated and checked against the deformation demand. In other terms, the steel strain demand at the ultimate drift value must be satisfied. The section capacity may be calculated by a traditional section analysis, taking into account that strain compatibility is not respected at a section level, while initial re-bar unbonded lengths may need to be incorporated right at the beam column interface to keep the steel strains below the rupture limits. It should be noted that the bigger the reinforcement diameter, the bigger the bond stresses and, thus, the inherent de-bonding ability (strain penetration) of the re-bar under cyclic loading. However, the increase in de-bonding length at every connection-rotation step reduces the re-bar strain and delays the re-bar rupture. In case the de-bonding length at any hysteresis cycle turns out to be insufficient and leads to re-bar breakage, the precast producer should supply an initial unbonded length (via plastic tape or tube, for example, or by the milling of the reinforcement ribs). In this way the sum of the unbonded and de-bonded lengths keep the reinforcement from rupturing.

In this type of connection and grouting configuration, the plastic hinge is generally located and concreted in the beam-to-column interface. The hysteresis loops of the connections are very sensitive and depend on the quality of the detailing implemented by the precast producer and/or contractor.

Contrary to the ductile bolts/rods, the high-strength bolts/rods are not grouted in the through-holes and the design philosophy is mainly influenced by the design of hybrid post-tensioned connections (see Section 5.3.3), the main difference being the use of unbonded (non-prestressed) high-strength bolts that will remain in the elastic range throughout the hysteresis response. The behaviour of the connection occurs mostly via the opening and closing of the beam-to-column interface. In such connections, the unbonded length of the high-strength bolts should be calculated in such a way that the bolt stress will not go beyond the material proportional limit (yielding) at the ultimate inter-storey drift level.

Figure 5-44 depicts the reinforcement and through-hole detailing in beams and columns.

The **construction steps** are the following:

- The columns are moulded with the steel boxes constituting the through holes.
- The columns are produced either with or without reservations to mount steel corbels. In case steel corbels are mounted onto the column faces, the precast/prestressed beams are seated on these corbels. In the case of free column surfaces, the beams are to be seated on steel or wooden props until the rods are attached and tightened.
- The system becomes statically indeterminate right after the tightening of the bolts.
- In the case of ductile bolts, the through-holes are grouted with cementitious material. In the case of high-strength rods, the through-holes are better filled with an anti-corrosive agent.

The following point deals with **system requirements**.

- The capacity calculation and moment-curvature response for the grouted case can be made with the conventional theories of section analysis as long as the de-bonded length is accounted for when the strain demand in the steel rebars is estimated, which will thus affect stresses and overall section capacity. However, when unbonded high-strength steel rods are adopted, moment-rotation connection analysis/design procedures that can overcome the lack of section strain compatibility typical of jointed ductile rocking/dissipating connections should be adopted (Pampanin et al., 2001; NZCS, 2009).

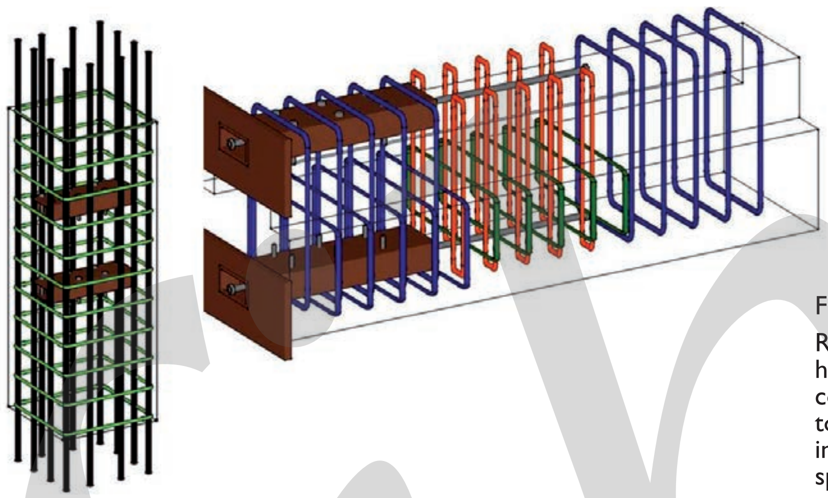


Fig. 5-44
Reinforcement and through-hole detailing in beams and columns (beam is overscaled to highlight changing shapes in shear reinforcement over span)

5.3.2.7 System S7

This system consists of multi-storey, precast-concrete columns that have open gaps at each floor level. Typically, columns are fabricated in one piece (with continuous longitudinal bars through the gaps) and extend over a number of storeys (see Fig. 5-45 and 5-46).

To form the gaps, concrete placement is interrupted at each floor level. These gaps allow for the arrangement of the bottom and top reinforcements of the beams along with the stirrups inside the joint core. Precast beams, either prestressed or conventionally reinforced, are placed between columns, seated on the cover concrete of the columns or on column corbels. Usually a precast hollow-core floor system is used, which is placed on the top of the inverted precast T-beam elements and span between them. After the placement of all the reinforcements to create a moment-resisting, beam-to-column connection and the reinforcement in the topping over the floor system, the concrete for the joint region and for the floor topping is poured. To promote the escape of the entrapped air in the joint concrete during casting and compacting, an inverted pyramid shape is formed in the precast column above the joint. A pipe of about 13 millimetres (0.51 inches) in diameter, which runs from the pyramid vertex up to one side of the precast column, is left embedded.

The following points cover **system requirements** and **additional information**.

- One-way or two-way frames can be built with this system
- Two main difficulties are encountered during the construction of these types of frames:

  - The first is the demoulding and lifting procedure of the rather heavy columns, which have non-monolithic behaviour because of the presence of the gaps (usually two or even three) along their height.

  - Figure 5-47 shows lifting after the demoulding of a column with two gaps. Depending on the geometry and the height of the columns, their main reinforcements and the sizes of the gaps, an additional temporary steel bracing system may be needed around the gaps to increase the rigidity of the columns during lifting. During erection and handling the exposed column bars should be strong enough to support the column and any floors erected overhead before the column closure pour is made. Care should also be taken to enter columns vertically in their final position and to accommodate tolerances based on design. Figure 5-48 shows a column in its final position, with the column-to-footing connection procedure taking place.

  - The second is the complexity of erecting and placing the beams in their final position between columns due to the protruded beam bottom reinforcements which have to enter the column gaps. In this respect, as in System S5, mechanical couplers, such as those shown in Figures 5-49 and 5-50, may be used to facilitate the erection procedure. These couplers should be of Type 2, according to Section 21.1.6 of ACI 318 (2011). Beam-to-column connections of this type have been used in Mexico for low-rise commercial buildings of up to 25 metres (82 feet) in height, in Spain and in India, among other countries. Alcocer et al. (2002) report experimental results on full-scale tests of these types of beam-to-column connections under uni-directional and bi-directional cyclic loading simulating earthquake motions. Figures 5-49 and 5-50 show the connection details of this type of system.

  - Figure 5-49 shows a horizontal section (plan view) of a two-way beam-to-column connection. Attention should be paid to the proper arrangement of the beam bottom reinforcement inside the joint core. Figure 5-50 shows the vertical cross section A-A of Figure 5-49. Figure 5-50 also depicts the gap between the lower and upper column, which is a bit larger than the total depth of the floor (beams + hollow-core slabs + topping) to facilitate the positioning of the beams during the erection procedure.

Fig. 5-45 Schmersal precast project in Pune, India, implementing System S7 (photo credit - Nagesh Kole; photo courtesy of Precast India Infrastructures Pvt. Ltd)

Fig. 5-46 Cummins precast project (Cummins) in Pune, India, implementing System S7 (photo credit - Nagesh Kole; photo courtesy of Precast India Infrastructures Pvt. Ltd)

Fig. 5-47
Lifting of column with two gaps after demoulding for transport, Cummins project, Pune, India

Fig. 5-48
Final position of column with two gaps, Cummins project, Pune, India

(Credit for above photos - Nagesh Kole; photos courtesy of Precast India Infrastructures Pvt. Ltd)

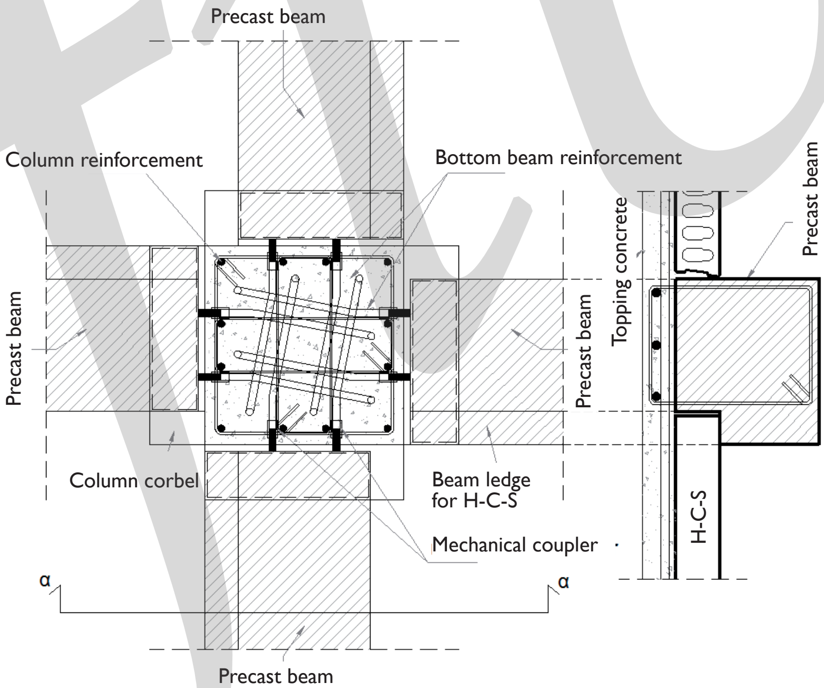


Fig. 5-49
Plan view and cross section of arrangement of bottom reinforcement inside joint core
(Other details not shown for clarity)

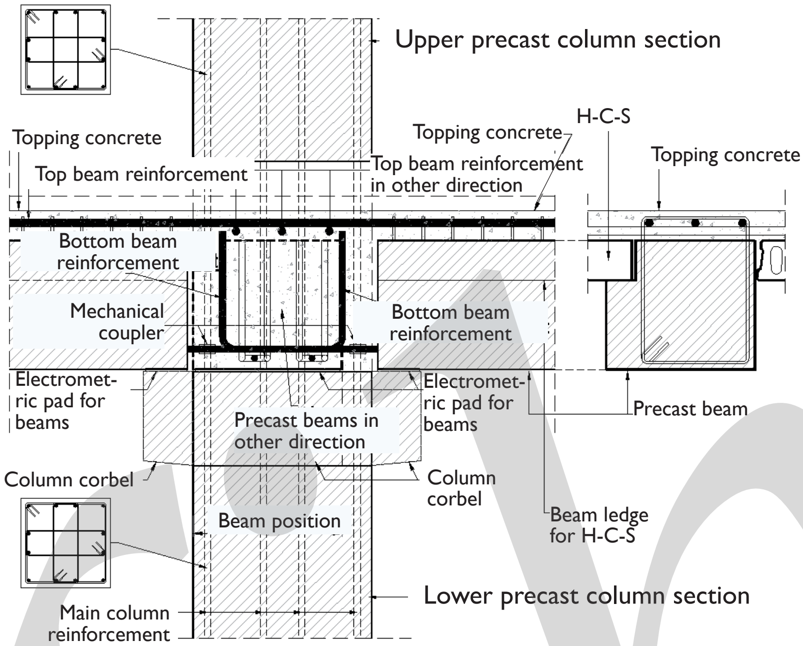


Fig. 5-50
Vertical cross section A-A of Fig. 5-50

5.3.3 General information on jointed systems (H1, H2 and H3)

Figure 5-51 depicts a post-tensioned moment-resisting beam-to-column connection (System H1) that consists of:

- a precast rectangular column with through-ducts for the post-tensioning strands;
- an inverted T-shaped precast beam with central through-ducts;
- post-tensioning (low-relaxation) strands or threaded bars;
- hollow-core or double-tee slabs; and
- cast-in-situ concrete topping.

Figure 5-52, on the other hand, depicts a hybrid moment-resisting beam-to-column connection (System H2) that consists of:

- a precast rectangular column with through-ducts both for the post-tensioning strands/bars and mild steel;
- an inverted precast T-shaped beam with central through-openings and open on the upper and lower faces (or, alternatively, on the lateral faces);
- post-tensioned (low relaxation) strands, or bars and energy dissipating mild steel;

- hollow-core or double-tee slabs; and
- cast-in-situ concrete topping.

The ductile and energy dissipating behaviour of the moment-resisting precast connections listed in the previous sections, which incorporate an emulative cast-in-situ approach, would inherently lead to extensive damage, often beyond repairability, in the plastic-hinge regions (see Fig. 5-53).

Although this has been the philosophy for earthquake-resistant design over quite a long period of time, with life safety the main target, property owners who have opted for this solution have faced substantial repair costs and severe disruption of business (downtime) after design-level earthquakes, and have even be forced to undertake controlled demolition.

The damage to modern buildings after recent earthquakes in well-developed countries, such as New Zealand (2010 and 2011) and Italy (2012), has provided a tough reality check for the performance-based design approach, confirming the current mismatch between societal expectations and the actual performance of modern buildings during seismic events, which suggests that an urgent paradigm shift in damage-control design philosophy and technologies needs to occur (Pampanin, 2012) (see Fig. 5-53).

From the late 1990s onwards, new precast construction systems based on dry jointed ductile connections, also referred to as PRESSS technology (for either frames and walls or dual system), have been conceived, extensively developed and applied in practice as alternative (low-damage) solutions in any seismic regions to traditional wet and/or strong connections following the 'emulative' cast-in-situ approach (Priestley, 1991, 1996, 1999; Pampanin, 2005, 2012; NZCS, 2010).

In PRESSS frame or wall systems - System H3 (see Fig. 5-54) - precast elements are jointed together through unbonded post-tensioning tendons/strands or bars creating moment-resisting connections, with all the advantages associated with such a robust structural scheme.

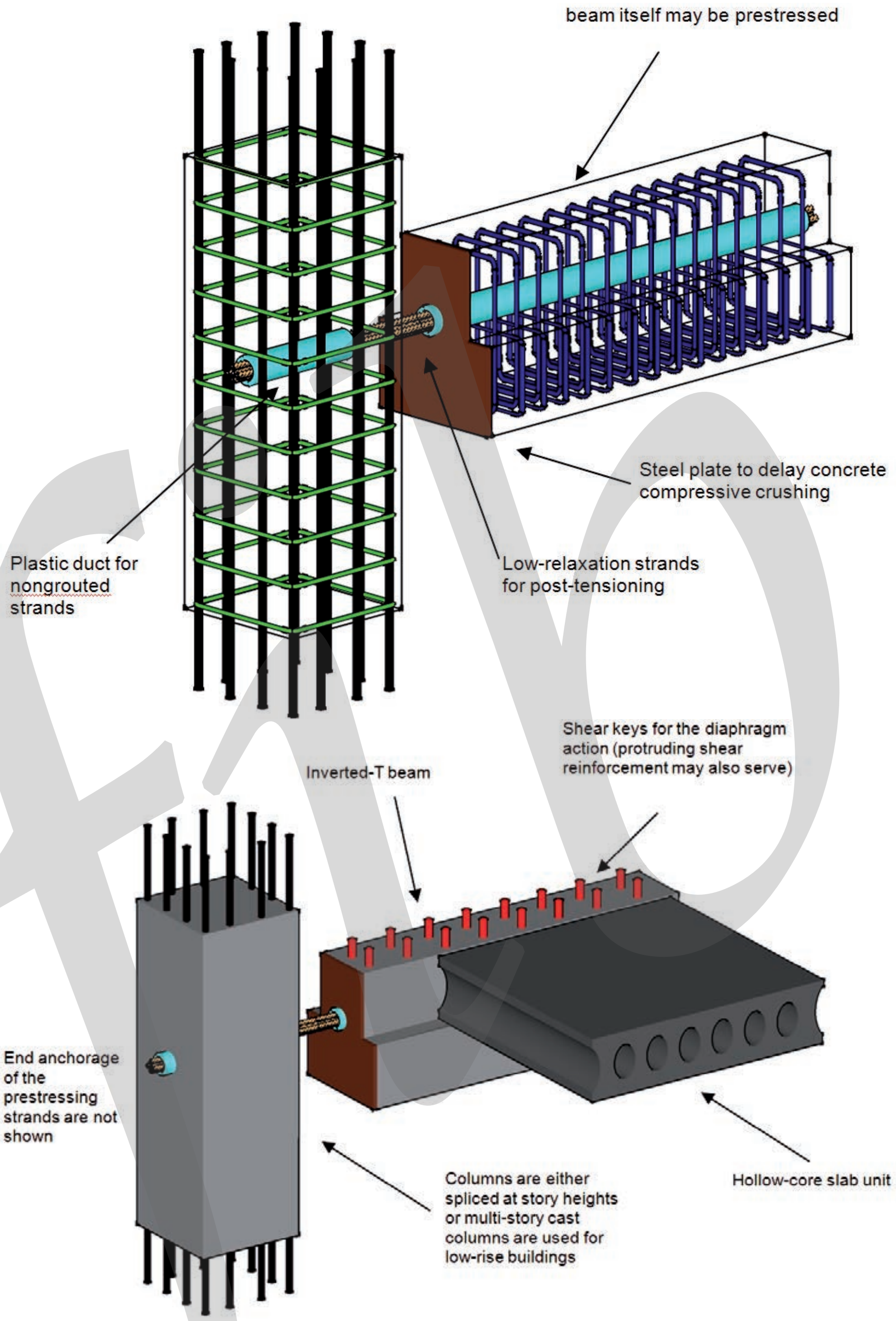


Fig. 5-51 Post-tensioned moment-resisting connection, (indicative) reinforcement and slab units

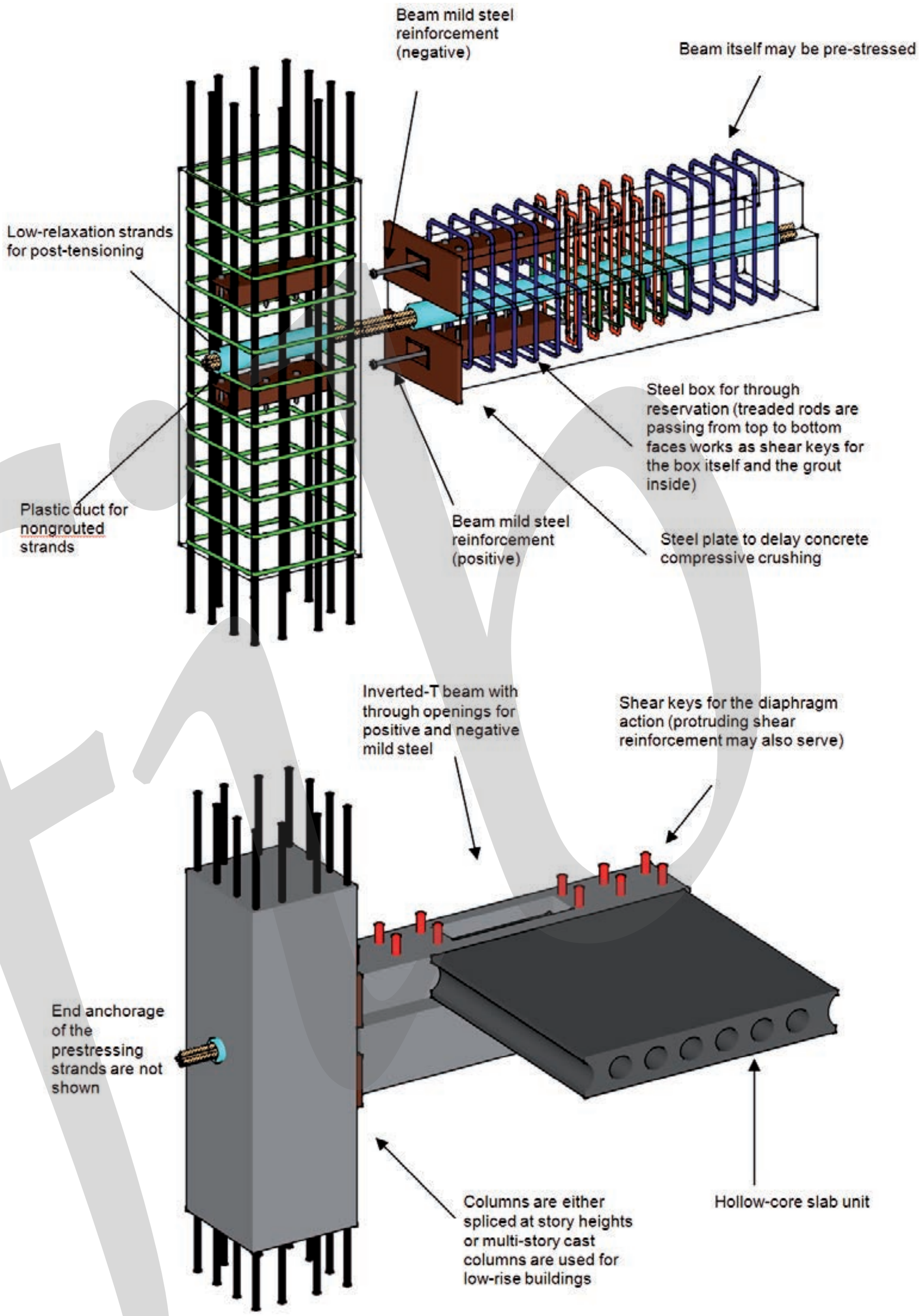


Fig. 5-52 Hybrid moment-resisting connection, (indicative) reinforcement and slab units

Fig. 5-53 Beam plastic hinges in a mid-1980s 22-story reinforced-concrete building based on System 2 and demolished after the Canterbury earthquakes of 2010 to 2011
Left and centre: Building under construction (photos courtesy of J. Restrepo)
Right: Damage to plastic hinge after 22 February 2011 quake and aftershocks and prior to demolition (photo courtesy of S. Pampanin)

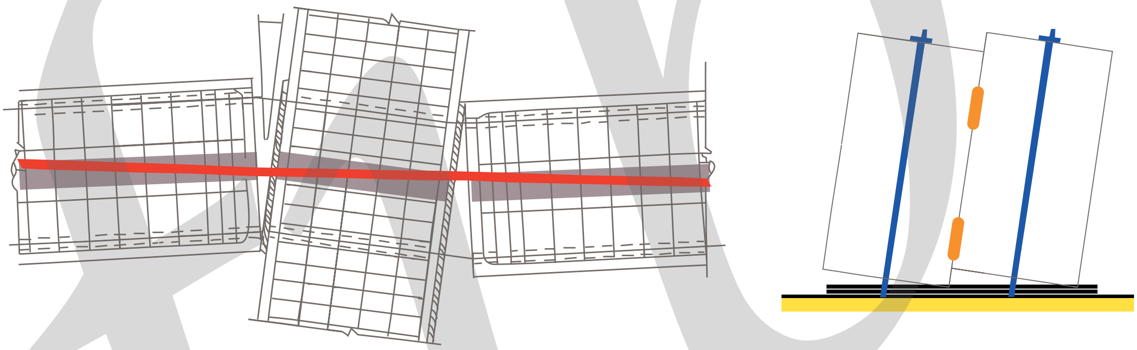


Fig. 5-54 Jointed precast 'hybrid' frame and wall systems developed in PRESSS Research Program (based on *fib* Bulletin 27, 2003, and NZS3101:2006)

In the case of shaking from an earthquake, the inelastic demand is accommodated within the connection itself (in the beam-to-column, column-to-foundation or wall-to-foundation critical interface) through the opening and closing of an existing gap (rocking motion). As a result the gap opening or rocking acts as a fuse or isolation system with no damage accumulating in the structural elements which are basically maintained in the elastic range. The basic structural skeleton of the building would thus remain undamaged after a major design-level earthquake without any need for repair intervention (see Fig. 5-55).

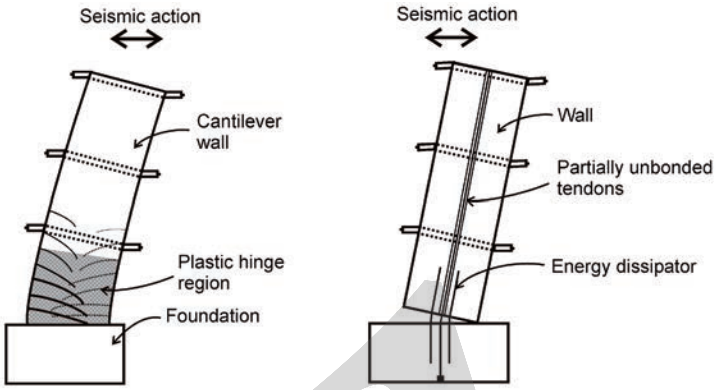


Fig. 5-55
Comparative response of traditional monolithic system (damage in plastic hinge) and jointed precast (hybrid) solution (no damage due to rocking mechanism and negligible residual deformations)
(based on *fib* Bulletin 27, 2003)

This is a major difference and improvement when compared to cast-in-situ solutions where, as mentioned, damage has to be expected and is actually accepted to occur in the plastic hinge regions, leading to substantial costs in repair and business interruptions.

Moreover, the tendons are unbonded so as to elongate within the duct without yielding. They can thus act as re-centring 'springs', guaranteeing that the structure arrives back at its original at-rest position at the end off the shaking, thus with negligible residual or permanent deformations (offset or lining of the building), the repair operations of which would be very costly and often complicated.

A particularly promising and efficient solution within the family of jointed ductile (or PRESSS-technology) connections is given by the **hybrid system** (Stanton et al., 1997) (see Fig. 5-54), where the connection reinforcement is given by the combination of unbonded post-tensioned bars or tendons and nonprestressed mild steel (or similarly additional external dissipation devices, as discussed in the next sections), inserted in corrugated metallic ducts and grouted to fully achieve bond conditions.

Under static loading considerations (no or low-seismic actions), these two types of reinforcement can guarantee a high level of connection strength and stiffness, with reduced congestion of the joint-connection region and an easy installation process.

Clearly, as per any structural system that relies on moment-resisting connections at the beam-to-column interface and takes advantage of the benefits of prestressing, longer span length can be achieved with reduced beam depths.

Under wind-loading and low-seismic actions the clamping action of the post-tensioned bars/tendons guarantee a very stiff initial condition (gross section) when compared to typical cast-in-situ solutions (cracked sections), thus resulting in immediate benefits under serviceability loading conditions.

Under moderate to high seismic actions, the traditional plastic-hinge mechanism is substituted with this controlled rocking mechanism (gap-opening and closing) at the critical interface, with negligible damage (see Fig. 5-55) in the structural elements.

While the tendons provide self-centring and restoring actions, the mild steel bars or other similar devices (such as U-shaped coupling steel plates, and rollers, between adjacent rocking walls) act as energy dissipaters and shock absorber for the structure under the seismic loading. This peculiar dissipative and re-centring mechanism is described by 'flag-shaped' hysteresis behaviour (force-displacement or moment-rotation cyclic behaviour - see Fig. 5-56). As opposed to what can be done for traditional structural connections, the properties and shape of the hysteresis can be modified by varying the (moment) contribution between re-centring and dissipation behaviour.

A damage-control limit state can thus be achieved under a design-level earthquake (typically set at an approximately 500-year return period) leading to an intrinsically high-seismic-performance system almost regardless of the seismic intensity.

Examples of the hysteresis response of a post-tensioned-only (nonlinear elastic behaviour) and a hybrid connection with internally grouted mild steel rebars (dissipative and re-centring behaviour or flag-shaped hysteresis) are shown in Figure 5-57.

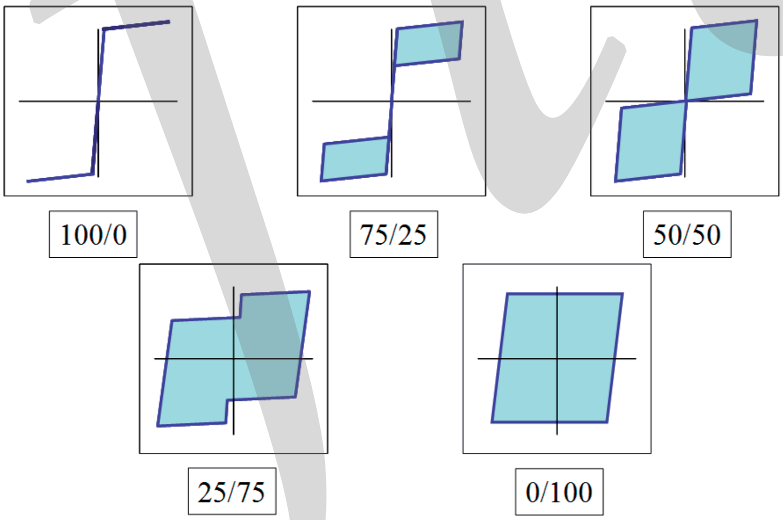
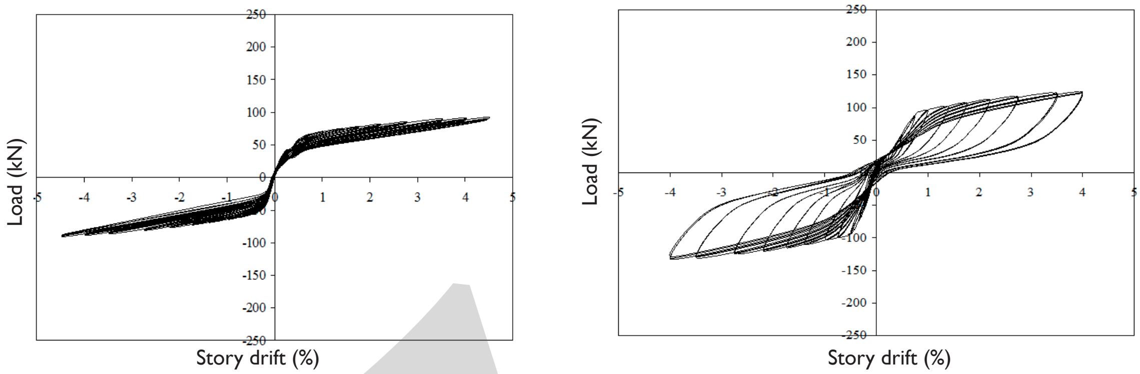


Fig. 5-56
Effects of varying the ratio between re-centring (nominator: post-tensioning and axial load) vs. dissipative (denominator: mild steel and dissipaters) contribution to the flag-shaped hysteresis loop



a. b.

Fig. 5-57 Cyclic response of:
a. post-tensioned-only connection
b. hybrid connection with 30% mild steel (internally grouted rebars) moment contribution

Further details on the modelling, analysis and design of such connections and systems, including full design examples of multi-storey buildings, can be found in *fib* Bulletin 27, 2003, and NZCS, 2010.

The **construction steps** are described in the following paragraphs.

The continuous and rapid development of jointed ductile connections or PRESSS technology for seismic-resisting systems has resulted in the validation and implementation of a wide range of alternative arrangements currently available to designers and contractors for practical applications on a case-by-case basis (cost-benefit analysis). An overview of such developments, design criteria and examples of implementations can be found in Pampanin et al. (2005) and the PRESSS Design Handbook (NZCS, 2010).

The structural systems and individual elements composing the skeleton can either be fully precast or semi-precast, such as when a half beam is concreted as per the previous emulative systems S1 to S4.

Furthermore, alternative solutions can be implemented (depending on the current code provisions) to provide a shear-transfer mechanism at the critical-section interface, namely:

- the friction caused by the contribution of the post-tensioned tendons;
- the dowel actions of the mild steel;

- a shear key, concrete or steel corbel/bracket/cleat; and
- combination of the above systems.

As a result, partial, total or no propping might be required, reflecting significant differences in the construction sequence, which lead to overall wide flexibility in terms of design and construction options.

Based on the previous considerations, it might be argued that a hybrid system could be obtained with minor amendments to traditional existing and well-known construction practices, in order to:

- allocate and anchor the post-tensioning (either with straight or draped tendons)
- develop a reliable rocking interface between beams and columns (using a grout pad and/or shimming), as well as introducing minor steel-plate/angle protection (armouring) to the cover concrete in the rocking beams (and column/wall compression regions).

An example of a general construction sequence can be given as follows:

- The precast columns with tight tolerances typical of other precast connections are placed onto the foundation or on top of each other when they are spliced.

- Notably the column-to-foundation connection can be based on either internally grouted (or spliced sleeves) or externally mounted rebars, as well as on any other column-foundation connections as discussed in Section 5.3.4. Typically the columns are not required to be post-tensioned as gravity loads might guarantee the self-centring of the base-column connection. However, when, as per any other typical moment-resisting frame, particularly in multi-story building, tension forces are expected to develop in the column, tie-down in the form of post-tensioning bars/strands can be applied.

- If permanent concrete or steel corbels are not adopted, mountable steel corbels (cleats) are placed as beam support or appropriate formwork and propping are supplied for the beams.

- The interface spacing between the column and the beam (based on construction tolerances) are grouted (with a grout pad, sometimes containing steel fibre) and/or shimmed.

- The prestressing strands/bars are located in smooth (PVC or smooth metallic) sleeves and a pre-defined tension force is applied. Typically, the post-tensioning operation is carried out in two phases during the construction sequence, at a lower level (50% target post-tensioning) first and then at the full-design level to allow the connections to be set up and to reduce shrinkage/creep effects. Grouting/grease can be applied for durability protection as long as the tendons remain unbonded.

- In the case of a hybrid connection, mild steel bars/rods are installed through the metallic corrugated ducts and the ducts are grouted. Alternatively 'plug and play' external replaceable dissipaters are installed in position.

- The steel demountable corbels and/or formworking can be detached at this stage. If permanent corbels are used, such an operation is not required, but proper detailing is.

- Floor units are seated on the inverted T-beams. Floor topping (mesh and starter bars) reinforcement is applied and the concrete topping is poured. An alternatively pretopped solution with mechanical connectors between floor units and lateral resisting systems may be followed.

The **longitudinal profile of the tendons**/bars can be either straight or draped (see Fig. 5-58), depending on the predominance of gravity vs. lateral-load effects on the frame, as a consequence of:

- a different level of seismicity (target design earthquake), as well as
- the assigned role of the system during the seismic response (namely, a gravity-load carrying system, seismic-resisting system or intermediate solution).

The combination of internally bonded prestressing (as typical of precast solutions with hinged connections, for example) with unbonded post-tensioning, to be implemented at different construction phases, can provide a unique flexibility in the design. Internally prestressed and prefabricated units could be dropped into place on temporary or permanent corbels/brackets without the need for propping or for the casting of the joint connection. If properly designed, the precamber provided by the prestressing can be sufficient to balance the self-weight of the beam and of the incoming floor, allowing for a very competitive speed of construction.

Either **internal** (grouted) mild steel bars or more recently developed **external** and replaceable supplemental damping devices can be adopted (see Fig. 5-59) in hybrid systems, as well as, in general terms in any of the previously presented precast connections to provide or increase their moment-resistance capacity.

The original solution for hybrid connections proposed in the U.S. PRESSS Program relied on the use of grouted, mild steel rebars inserted in corrugated (metallic) ducts. As per the aforementioned tension-compression yielding (TCY) or bolted moment-resisting connection system (S6), a short unbonded length in the mild steel bars would be typically adopted at the connection interface to limit the strain demand in the reinforcing bars and protect them from premature rupturing when the gap opens up to the design level of drift.

The downside of this approach is that following an earthquake, the internal rebars would not be easily accessible or replaceable. The degradation of the bond between concrete and steel during reverse-cyclic loading can also cause a higher deformation of the overall system (stiffness degradation).

More recently, externally located and potentially replaceable dissipaters, described as 'plug and play' (see Fig. 5-59), have been developed and extensively validated (both experimentally and numerically) at the University of Canterbury since 2004 (Pampanin, 2005) on several sub-assembly configurations (beam-to-column, column-to-foundation and wall-to-foundation connections) (see Fig. 5-60), with the aim to further simplify and speed up the constructibility of the structure while improving its repairability after an earthquake event, thus dramatically reducing costs associated with direct repair and downtime. As the name suggests, such a reinforcement/dissipater can be easily mounted, dismounted and replaced after an earthquake event, if required.

This option allows for the conception of a modular construction system with replaceable sacrificial fuses at the rocking connection acting as the 'weakest link in the chain', based on capacity design principles. In its more efficient and practical solution, the 'plug and play' dissipater consists of a mild steel rebar with an axial tension-compression yielding mechanism, machined down to the desired 'fuse' dimension and inserted in a steel tube acting as anti-buckling restrainers (see Fig. 5-61). The tube is then grouted or epoxied to allow for a sound confinement/buckling restraint to occur. Quality-control testing under reversed cyclic loading is carried out to confirm the mechanical behaviour, damping and cycle fatigue.

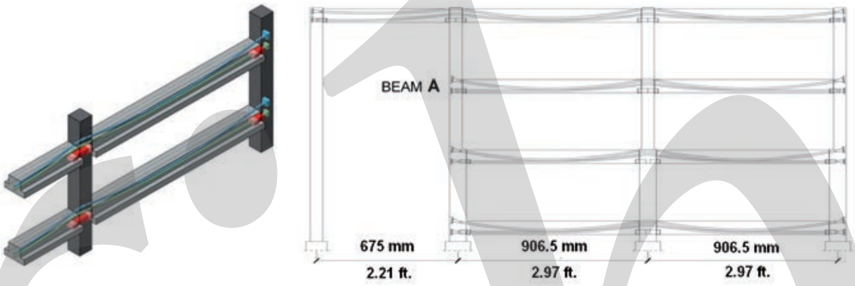


Fig. 5-58
Rendering and application of post-tensioned solution with draped tendons, particular to Brooklyn System, developed by BS Italia, Bergamo, Italy (Pampanin et al., 2004)

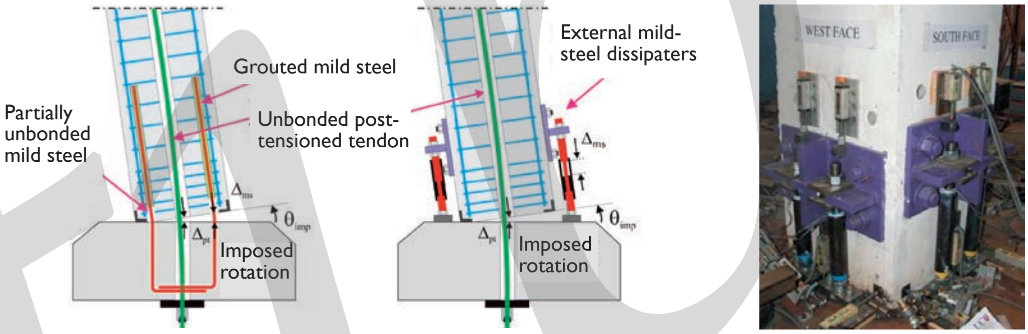


Fig. 5-59 Internal vs. external replaceable dissipaters/fuses at the base-column/pier connection (based on Marriott et al., 2008; NZCS, 2010)

Fig. 5-60 Alternative configurations of external dissipaters for hybrid systems: a. and b. beam-to-column connections, with and without recesses in beam (from Pampanin et al., 2006; NZCS, 2010); c. wall systems combining central hysteretic dampers with two viscous dampers to form advanced flag-shaped system (from Marriott et al., 2008; NZCS, 2010)

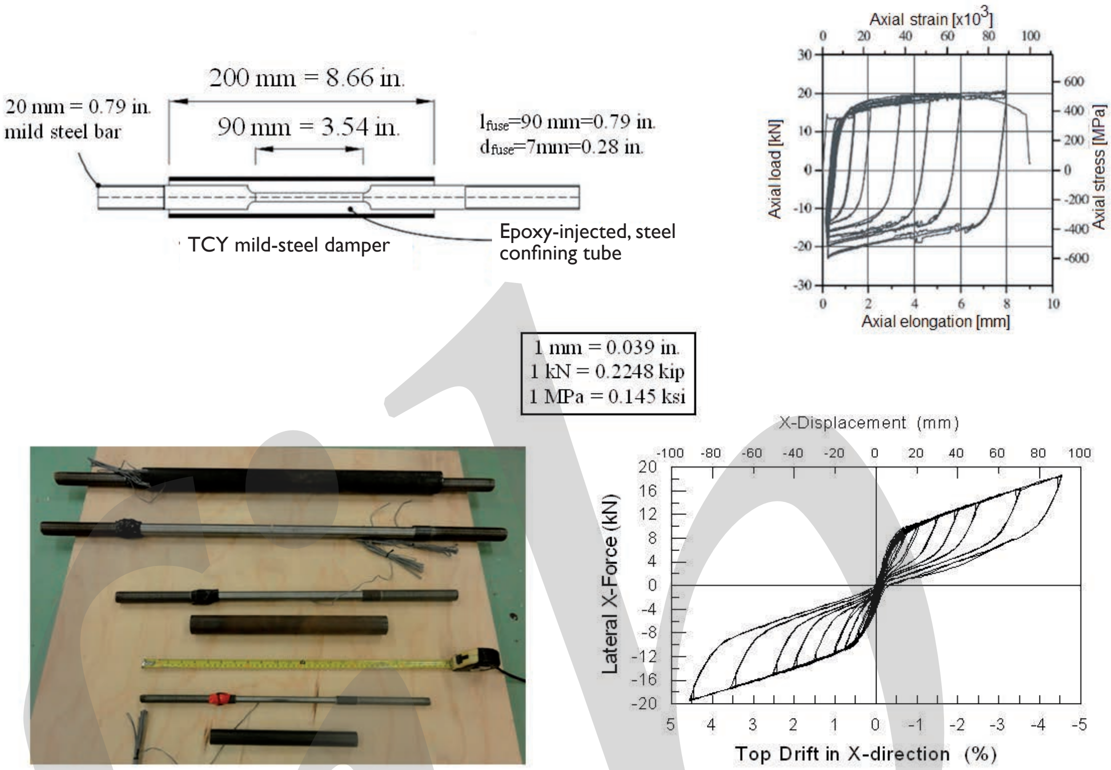


Fig. 5-61 Basic scheme of 'plug and play' device, assembling of different sizes, cyclic behaviour of single dissipater and of hybrid beam-to-column joints with external replaceable dissipaters (Pampanin et al., 2006; Marriott et al., 2008; NZCS, 2010)

The external part of the bar is threaded to allow for easy attachment to a bracket or a similar cleat in the concrete. Depending on the fastening solutions in the concrete, a nut or double nut can be used to tight-fit (by hand) the device during the installation process.

Due to its nature, it is also very useful as an initial bracing system between the precast elements (for example, in column-to-foundation, beams-to-columns, columns-to-roof-beams) during construction, thereby enhancing on-site safety and reducing propping/shoring/vertical-plumbing time.

The cyclic response of a typical dissipater is very stable (see top right chart in Fig. 5-61), allowing for many dissipative cycles prior to reach failure, often caused by low-cycle fatigue (or buckling, if subject to stroke levels well beyond the design level).

An accurate design of the geometry, fuse length, ration between outer and inner diameter, tube size and stiffness, interior grouting element, thickness and so forth can deliver a tailored performance as specified, for example, in terms of maximum stroke (displacement, deformation), yielding and maximum force, number of cycles and equivalent dissipation capacity (Sarti et al., 2013). Moreover, these external and replaceable reinforcements/dissipaters can be adopted for many other arrangements of precast-concrete connection systems for which they were not necessarily originally developed as jointed ductile systems.

For example, a diagonal 'haunch-type' solution consisting of a 'plug and play' dissipater (e.g. Pampanin et al., 2010) can be used to provide additional supplemental damping as well as stiffness and moment-resisting capacity within a hinged beam-to-column connection. In such a solution, originally proposed for the retrofit/strengthening of existing buildings (see Fig. 5-62), the device can take advantage of, while controlling, the relative rotation between beams and columns, thus limiting the drift demand and redistributing stresses and moments as typical of moment-resisting frame systems. In turn, this would limit the overturning moment demand to the column base connection and foundation systems.

The designer needs to be pay attention to capacity design principles so as to guarantee that the structural elements (beams, columns, roof and floor units) will accommodate the newly developed and distributed internal actions that develop a robust load path.

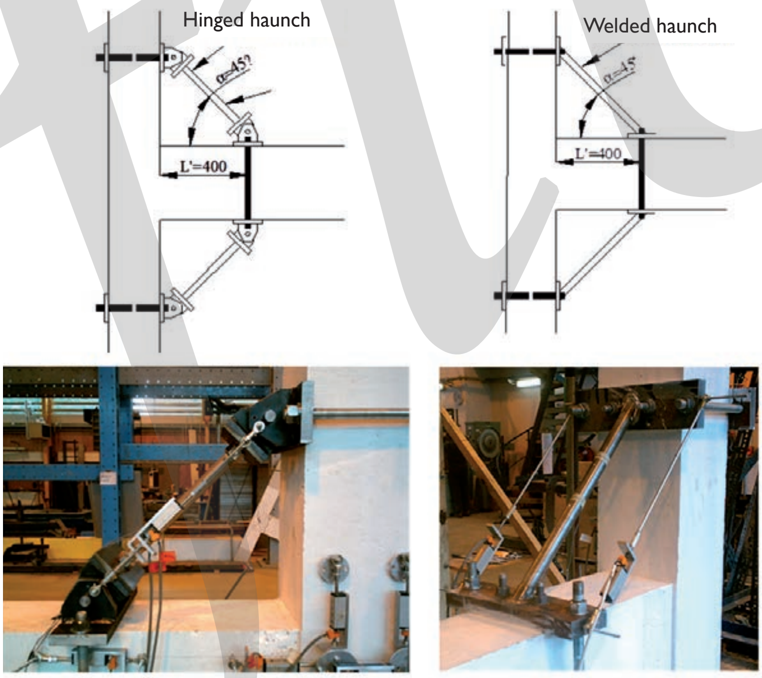


Fig. 5-62
Concept and testing of diagonal haunch retrofit solution (Pampanin et al., 2006)

Several **on-site applications** of PRESSS-technology systems based on jointed ductile connections have been implemented in different seismic-prone countries and regions of the world, including, but not limited to, the United States, Central and South America, Europe and New Zealand. Only a few examples of these will be discussed here. For more information, the reader is referred to other publications (PRESSS Design Handbook, NZCS2010; Pampanin, 2012).

One of the first and most glamorous applications of hybrid systems in high seismic regions is the Paramount Building in San Francisco, California (see Fig. 5-63). The structure is a 39-storey apartment building and the highest precast-concrete structure in a high seismic zone (Englerkirk, 2002). Perimeter seismic-resisting frames were used in both directions. Note the details to the fitting (by hand) of the mild steel rebars through the joint using openings on the side faces of the beams. Corner columns often had to be post-tensioned in two directions.

Fig. 5-63 Left: Completed Paramount Building, San Francisco, California
Centre and right: On-site applications of unbonded post-tensioned hybrid frames in Paramount Building
(Photos courtesy of Charles Pankow Builders, Ltd.)

The Alan MacDiarmid Building (see Fig. 5-64) of the Victoria University of Wellington was the first multi-storey PRESSS building in New Zealand (Cattanach and Pampanin, 2008). The building system, comprising post-tensioned hybrid frames in one direction and post-tensioned hybrid coupled walls in the orthogonal directions, features some of the latest technical solutions described above, such as the external replaceable dissipaters in the frame systems (both at the beam-to-column connections and the base-column connections) and unbonded post-tensioned sandwich walls coupled by steel beams yielding in flexure.

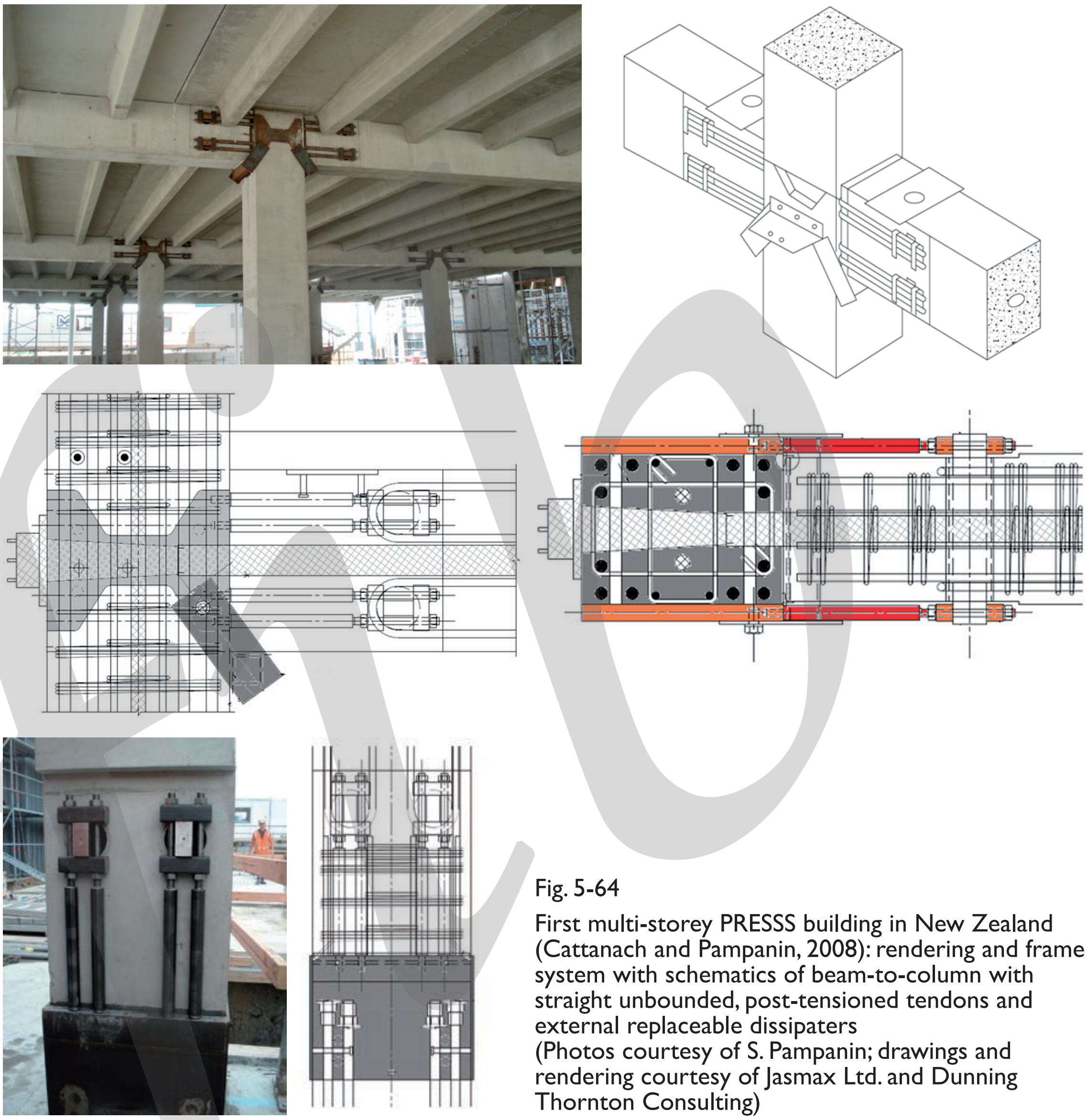


Fig. 5-64
First multi-storey PRESSS building in New Zealand (Cattanach and Pampanin, 2008): rendering and frame system with schematics of beam-to-column with straight unbounded, post-tensioned tendons and external replaceable dissipaters
(Photos courtesy of S. Pampanin; drawings and rendering courtesy of Jasmax Ltd. and Dunning Thornton Consulting)

The second PRESSS building in New Zealand and first in South Island was represented by the Southern Cross Hospital's Endoscopy Consultants Building in Christchurch (see Fig. 5-65 and 5-66).

In this case as well frames and coupled walls were all used in the two orthogonal directions. The structure consisted of four post-tensioned precast concrete frames in the north-south elevation and two sets of post-tensioned coupled precast-concrete walls in the east-west elevation.

The frames incorporated straight post-tensioned tendons and top mild-steel reinforcements (cast with the top part of the semi-precast beam and slab). The post-tensioned concrete walls had a combination of unbonded mild steel reinforcements at the base and U-shaped flexural plates (UFPs) in between the walls for additional strength and energy-dissipation capacity.

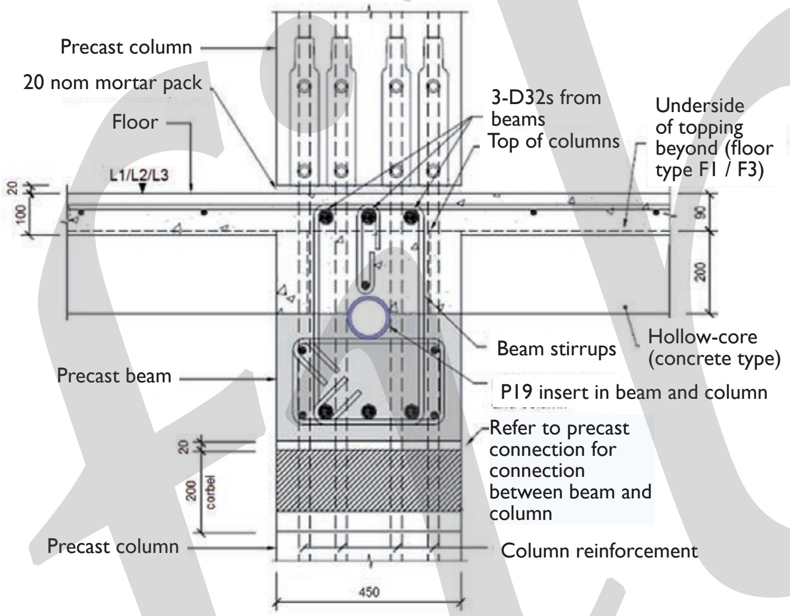


Fig. 5-65

Second multistorey PRESSS building in New Zealand (first in South Island): Endoscopy Consultants' Building for Southern Cross Hospital Ltd., Christchurch (Pampanin et al., 2011, 2012)
Rendering, connection details of frame system (straight tendon, semi-precast beam, and top mild steel bars), and post-tensioned coupled walls (UFP dissipaters coupling)
(Drawing courtesy of Structex Metro Ltd.; Photos courtesy of S. Pampanin)

a.

b.

c.


Fig. 5-66 Detailing of post-tensioned frames:
a. Anchorage block and spiral reinforcing within precast column
b. Precast columns with ducts for post-tensioning tendons and mild-steel reinforcements
c. On-site post-tensioning (Photos courtesy of S. Pampanin)

Fig. 5-67 Practical application of post-tensioned jointed ductile connections for two- and three-story industrial buildings in Costa Rica (Photos courtesy of Productos de Concreto S.A.)

The Southern Cross Hospital PRESSS building in Christchurch passed the severe test of the 2010-2011 earthquake sequence with highly satisfactory results since only minor or cosmetic damage was sustained by the structural systems (Pampanin, 2012). Furthermore, the operating theatres containing sophisticated and expensive machinery were basically operational the day after the largest earthquake. One of the main features in the design of a rocking-dissipative solution is the possibility to adjust the level of floor accelerations (not only drift) to protect both structural and nonstructural elements, including content and acceleration-sensitive equipment. More information on the design concept and performance criteria, modelling and analysis, construction and the observed behaviour of the building can be found in Pampanin et al. (2011).

In Costa Rica, Productos de Concreto S.A. has designed and built a number of two- and three-storey industrial buildings (see Fig. 5-67) using a combination of wet (emulative) connections for frames and dry jointed ductile (post-tensioned) connections for frames, walls or dual systems. As shown in Figure 5-67, the roof beam (double-slope beam) moment-resisting connection with the columns is obtained using an emulation of cast-in-situ connections. Post-tensioned (PRESSS-type) solutions in both directions are used at the beam-to-column connections at the other storeys.

More information on recent developments and proposals for the use of post-tensioned hybrid solutions in precast-concrete industrial buildings can be found in Bonilla et al. (2011).

5.3.4 Column-to-foundation connections

Large actions (N, V, M) may develop in a column-to-foundation connection under vertical or a combination of vertical and horizontal (due to earthquake) loads, in particular when hinged connections are used in the beam-to-column joints of the superstructure and/or when no shear walls are used as primary lateral load-resisting systems. The column-to-foundation connection must be capable of transferring these actions from the column to the foundation and then to the soil.

There are a variety of column-to-foundation connections available. The right choice for a particular application depends upon the magnitude of the actions, soil conditions, efficiency of the connection, targeted damage/performance, required erection time and, last but often not least in the decision-making process, the economic considerations.

Note: In the following only the most common types of column-to-foundation connections will be presented.

For any type of column-to-foundation connection, the foundation should be overdesigned in order to develop, under seismic loading, the plastic hinge in the column base, just above the column-to-foundation connection. This is particularly important if the behaviour of the column-to-foundation connection near failure is not expected to be sufficiently ductile to provide sufficient energy dissipation.

5.3.4.1 Socket foundation

A socket foundation provides a simple way to connect precast-concrete columns to the foundations. It is easy to build, quick to assemble, able to accommodate construction tolerances and is usually economical. However, this type of foundation may not meet code requirements in some jurisdictions due to it having no positive connections.

- Generally, a socket foundation is built by embedding the bottom portion of a column into a cavity of the foundation discrete (plinth) element. Figure 5-68 shows three ways of creating a socket foundation.

  In type A, the upper part is precast, while the foundation base is reinforced and cast in situ. In the bottom part of the precast socket, starter bars are provided with a proper anchorage length to develop monolithic behaviour with the cast-in-situ foundation base.

  In type B, the whole socket base foundation (socket's side walls + foundation base) is precast. The shape of the horizontal section of the socket side walls is usually rectangular and most often comes with external starter bars to receive and connect with tie beams, if required (see Fig. 5-69).

  The side walls of the socket may also come with external vertical side ribs or an extension of the side walls (see Fig. 5-70), whose purpose is to increase the socket rigidity.

  In some cases, depending on local conditions and/or economy, socket foundations may be built entirely on site, mostly with cast-in-situ concrete, as shown for type C.

  In all the above types, the end portion of the precast column is suitably formed to accommodate tolerances between column and socket (see top right image in Fig. 5-68).

In Figure 5-72 the typical shapes and arrangement of the reinforcement in a socket foundation of type A is shown (see also Fig. 5-71a).

Socket foundations of type B are reinforced the same way as type A.

Socket foundations of type C are properly reinforced based on the load effects for cast-in-situ foundations.

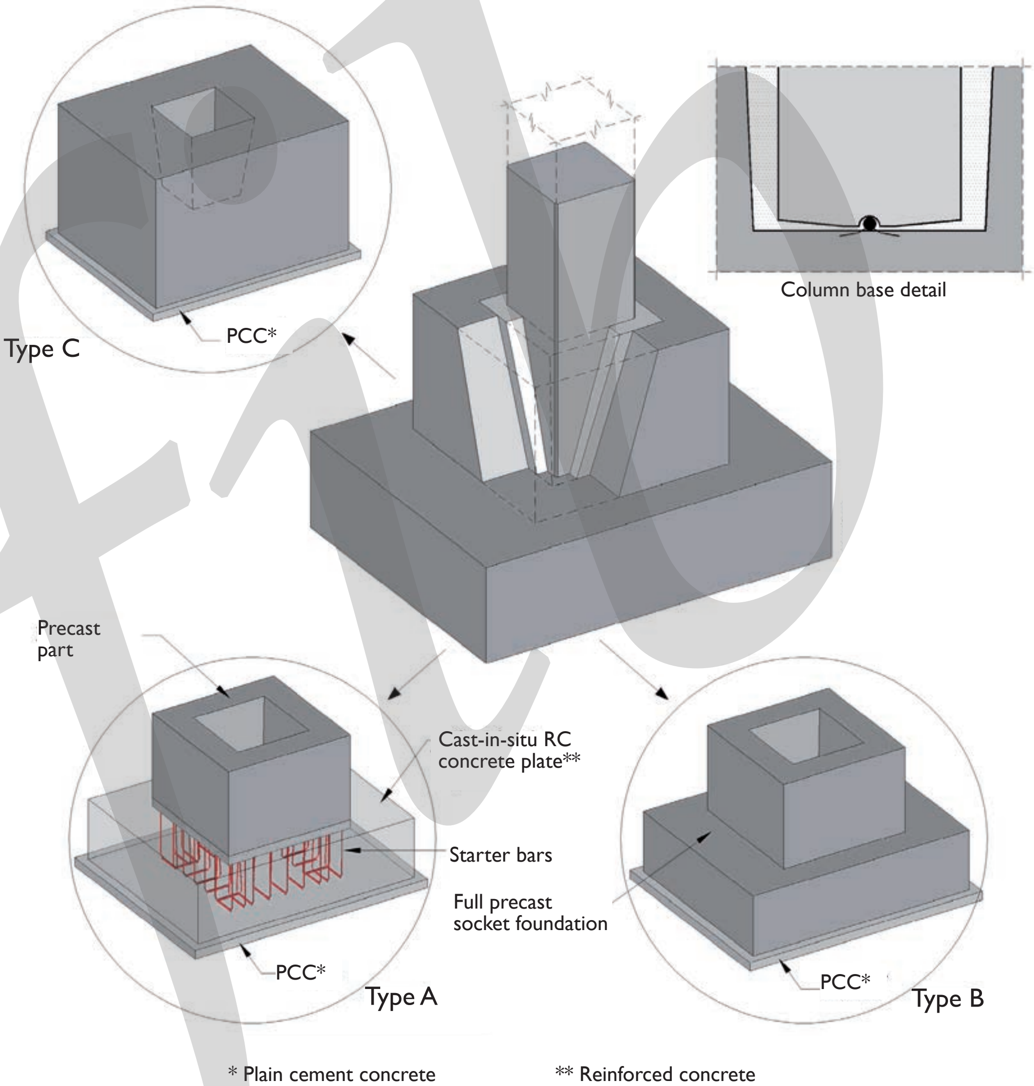
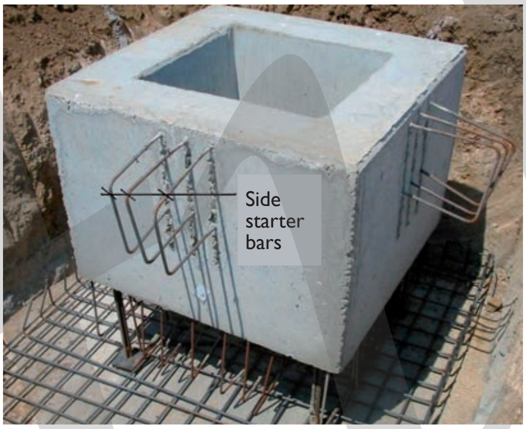


Fig. 5-68 Socket foundation

Figure 5-68 provides **construction steps** for a column-to-foundation connection that uses type-A precast sockets:

- Plain concrete of about 100 mm (3.94 in.) in thickness is placed on suitably treated and levelled ground.
- Reinforcement of the foundation base is placed with suitable spacers.
- The precast socket is positioned. It is placed within permitted tolerances supported by the starter bars that extend down to the level of the reinforcement of the foundation base. In this respect, the starter bars protruding from the four corners of the socket should be strong enough to stabilize the precast socket in place. Vertical steel bars (one in each corner) are sometimes installed during the production of the socket to provide stability (see Fig. 5-69). In this position, the top of the cast-in-situ

a.



b.

Fig. 5-69 Typical socket foundation with side starter bars for connection with tie-beams:
a. before concreting the foundation base; b. after concreting the foundation base
(Photos courtesy of ASPROKAT S.A., Aspropyrgos, Greece)

Fig. 5-70
Special socket foundation with extension of side walls in Portugal
(Photo courtesy of V. Lúcio)

concrete should be marked carefully on the inside and outside of the precast socket. It is crucial that the marking be within the construction tolerances (mm).

- The foundation base is properly formed and the concrete is cast in situ until its upper level reaches the marks described above.

- A centring device for the column is fixed on the foundation base.

- The column is placed vertically in the socket, adjusted in place through the centring device and shimmed to correct elevation. It is then adjusted for verticality and is temporarily supported, usually with timber wedges (see Fig. 5-72b) or by other means.

- After the column has been set up, the space between the column and the socket is filled with suitable grout material.

The **requirements for socket foundations** are the following:

- The following minimum values for concrete grades are recommended:

  - For type A, the concrete grade of the precast part of the socket foundation should not be less than C30 (f_c' = 4350 psi)

  - The concrete grade of the cast-in-situ concrete for the foundation base should not be less than C20 (f_c' = 2900 psi)

  - For type B, C30 (f_c' = 4350 psi) and for type C, C20 (f_c' = 2900 psi) should be used

  - The strength of the concrete/grout for the socket gap infill should not be less than 30MPa (4350 psi) for all types.

- The **dimensions of the socket foundation** depend on the cross section of the column, the magnitude of the internal forces (M, N, V) and soil conditions. In Figure 5-75, recommended ranges are presented for the following: the socket-wall thickness, the gap between the socket-walls and column faces along the height of the socket, and the embedded length of the column.

  In Figure 5-75b the recommended minimum distance between the upper part of the socket foundation and a ground slab is shown.

- The **surfaces of column ends and the inner part of the socket-walls in the embedded area** can vary based on the following criteria. Typically, connections of precast columns with precast socket foundation are made using smooth surfaces in the embedded zone. However, in seismic applications, it is advisable that both the column end and the inner surface of the walls of the socket be suitably roughened (see Fig. 5-72 and 5-73a).

  In some cases columns and footings are built in one piece (see Fig. 5-73b and 5-74).

In international literature, a variety of **design models** may be found for the dimensioning of **column-to-foundation connections by sockets**, mostly with smooth but also with

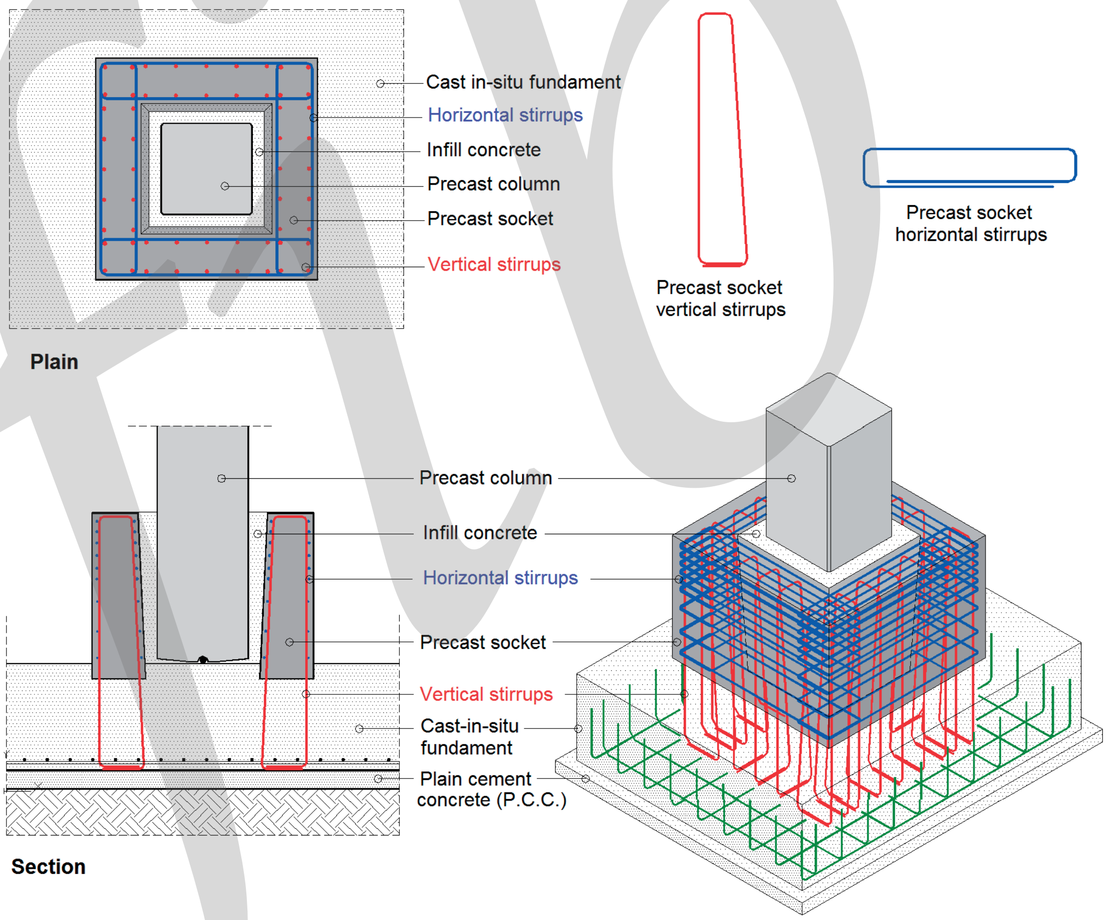


Fig. 5-71 Typical reinforcement in type-A socket foundation

rough interfaces to the column and socket in the embedded area. In almost all models, the basic assumption about the forces and possible flow of stresses inside the embedded area of the socket are qualitatively shown in Figure 5-76 for rather smooth surfaces (small friction coefficient). Simplified models have also been developed under the assumption that no friction develops at the contact surfaces between the column and the socket (see Fig. 5-77a). Such models result in non-conservative forces for checking the possible punching failure of the foundation base (see Fig. 5-77b). When column ends and socket surfaces in the embedded area are roughened or properly keyed, the same qualitative models as for smooth surfaces may be used, which take into account the higher values of the friction coefficient. When interfaces are roughened, friction governs along the length of the embedded area and leads to the better behaviour of the connection and reduces the punching action on the foundation base (see Fig. 5-77c).

Figure 5-78 shows a model (Canha et al., 2009) under full development of friction between column and socket due to its keyed surfaces.

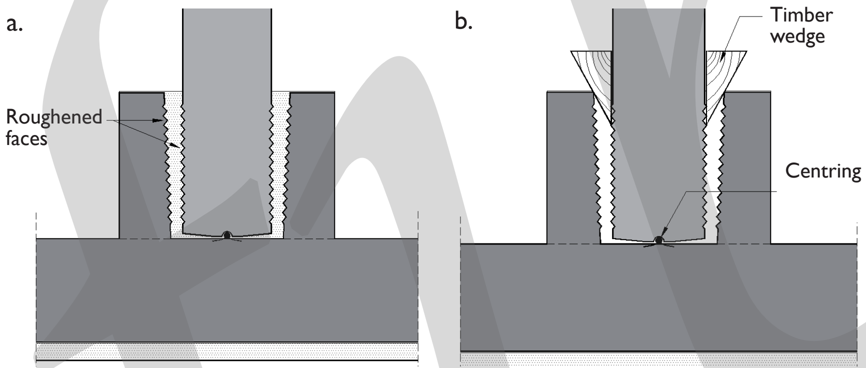


Fig. 5-72
Rough interfaces and timber wedges to control verticality of column

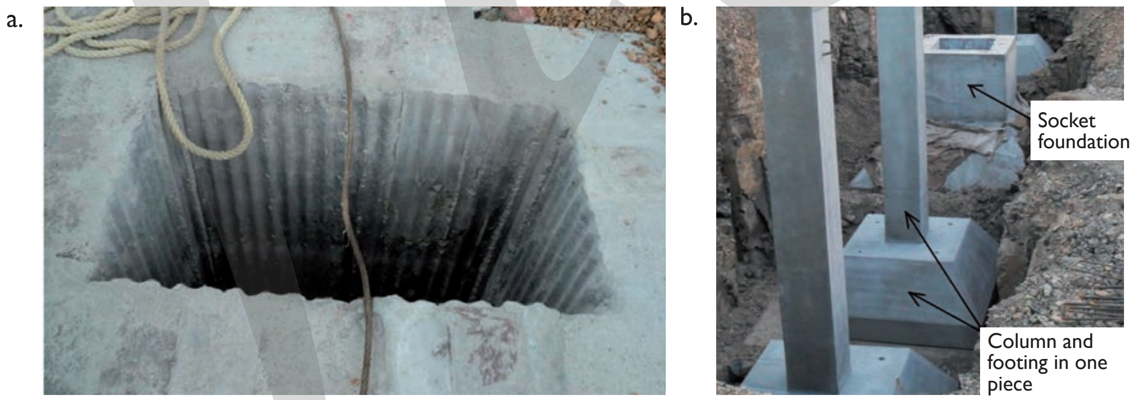


Fig. 5-73 a. Precast socket with roughened inner parts of side walls
b. Column and footing in one piece
(Photos courtesy of Precast India Infrastructure Pvt. Ltd, Pune, India)

Fig. 5-74

Column and footing in one piece after production (Photo courtesy of Precast India Infrastructure Pvt. Ltd, Pune, India)

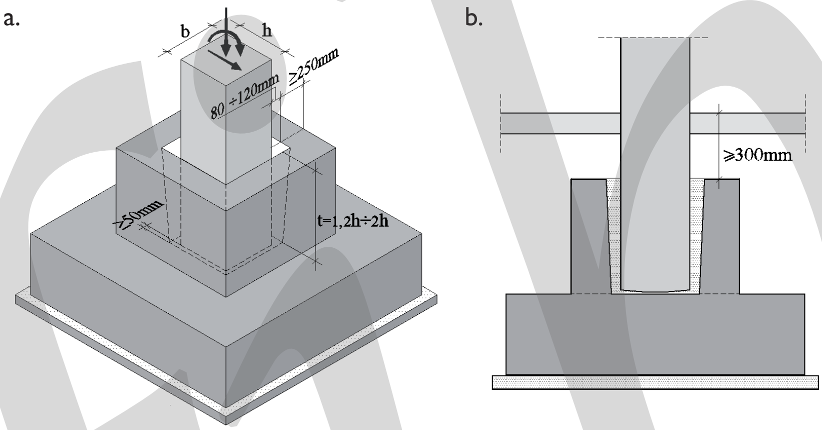


Fig. 5-75

Dimensional limitations of socket foundations

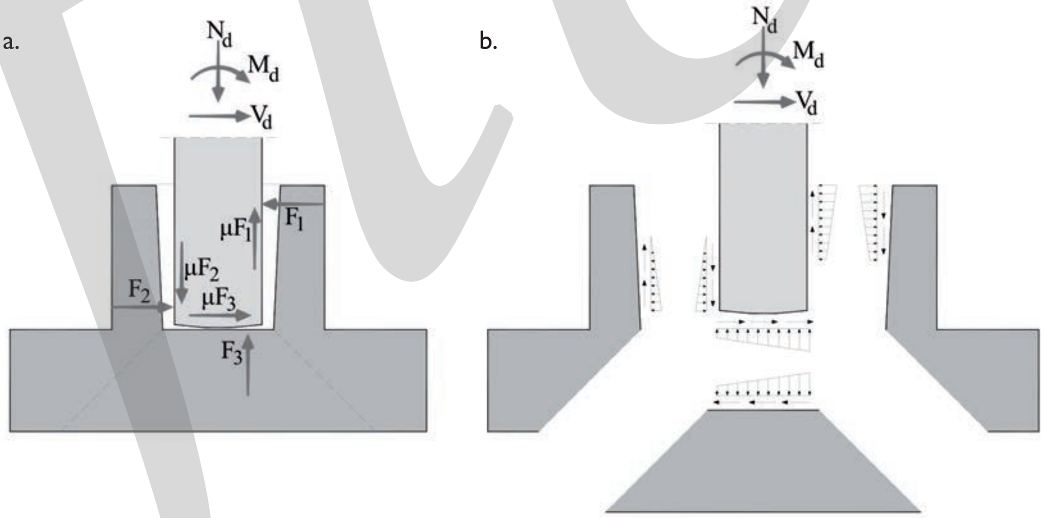


Fig. 5-76 Internal forces in a socket foundation with smooth interface of socket and column (Lúcio and Silva, 2000)

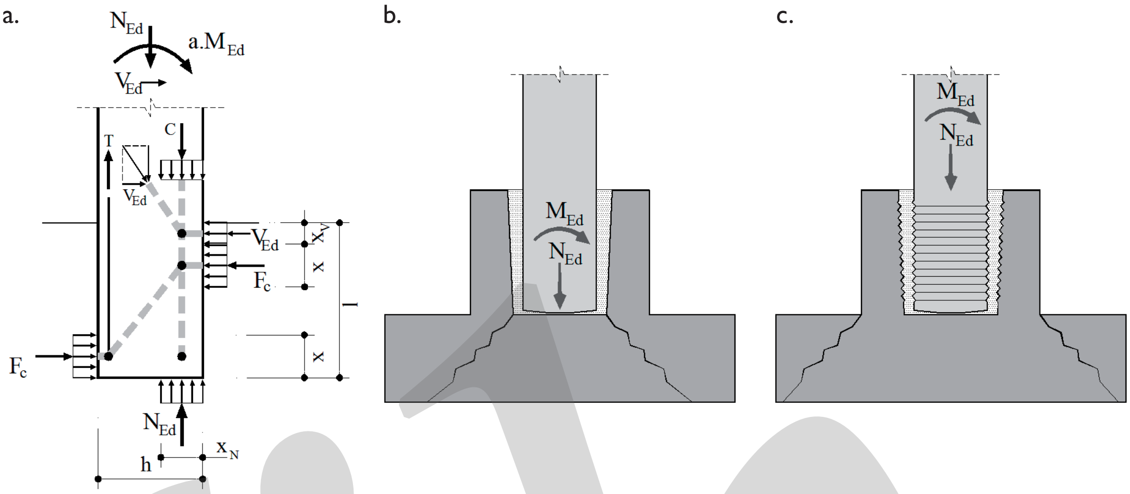


Fig. 5-77 Design of socket foundation (Ebeling et al., 2008)
a. Model under no friction b. Possible punching failure with no friction
c. Possible punching failure with rough interfaces

Notes:
Nearly all the models used for the design of column socket foundations lead to almost the same type and arrangement of reinforcement as shown in Figure 5-72.
A comparison of the design outcomes of different models can be found in Canha and El Debs (2006).

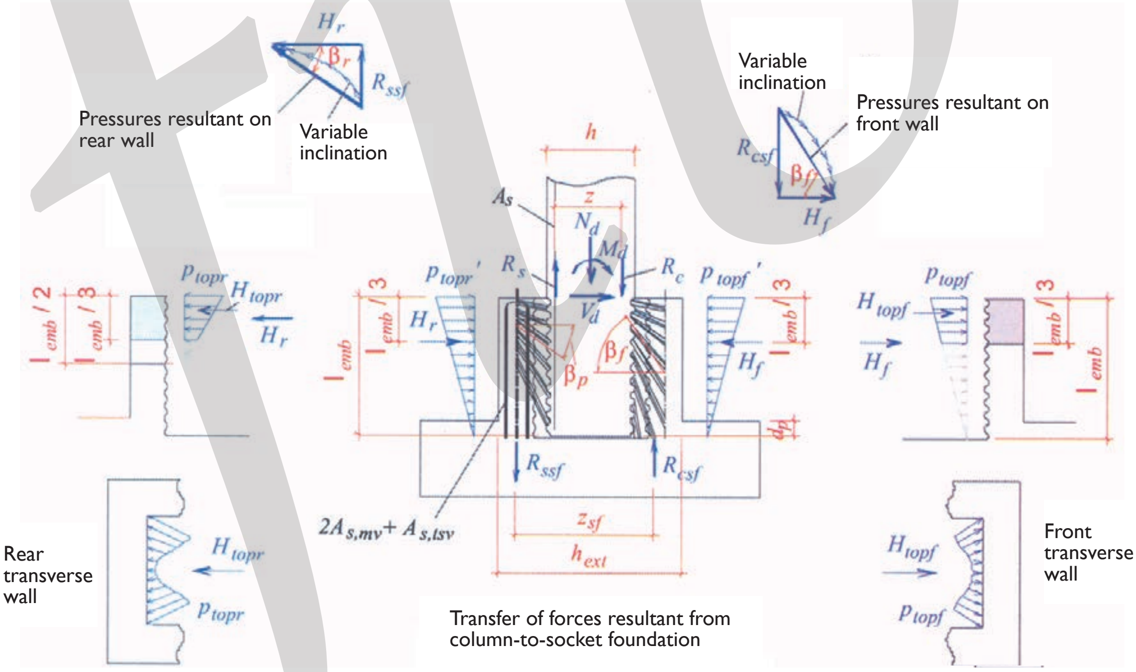


Fig. 5-78 Transfer of forces from column to socket (Canha et al., 2009)

The type-C column-to-socket connection shown in Figure 5-68 may be considered to act like a monolithic connection when the surfaces are intentionally roughened with indentations or keys. In this case, where vertical tension due to moment transfer occurs, careful detailing of the overlap reinforcement allowing for the separation of the lapped bars is needed. The lap length, based on code provisions, should be increased by at least the horizontal distance/spacing s between bars in the column and in the foundation (see Fig. 5-79). Adequate horizontal reinforcement for the lap splice should be provided. The punching shear design may be the same as for monolithic column-to-foundation connections, provided the shear transfer between the column and the footing is verified. Otherwise, the punching shear design should be as it is for sockets with smooth surfaces, namely, with a low friction coefficient.

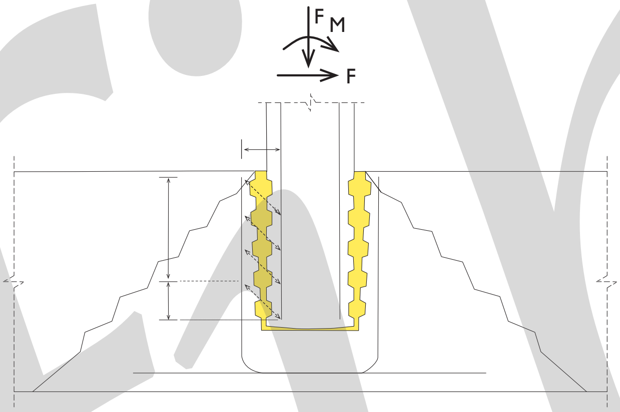


Fig. 5-79
Pocket foundations with keyed joint surface (based on Eurocode 2, 2004) (Other details not shown for clarity)

5.3.4.2 Column-to-foundation connections with corrugated-metal ducts or steel sleeves

Figures 5-80 and 5-81 illustrate alternative options for moment-resisting column-to-foundation connections, based on grouted corrugated metallic ducts or steel splice sleeves, respectively.

The design strategy can vary significantly, with the connector/splicing system being either located in the critical plastic-hinge region above the foundation level and column-to-foundation interface or underneath the critical interface of the foundation level.

In the case shown in Figure 5-80a corrugated metal ducts are embedded in the concrete foundation. The concrete foundation may be either precast or cast-in-situ. The columns have (starter) bars protruding from the bottom. These should align with the centrelines of the metal ducts. The column is properly seated on a bed of nonshrink grout of about

50 millimetres (1.97 inches) in thickness. The metal ducts are then grouted. Measures, such as of steel shims, should be taken on site to keep the column in a vertical position until the hardening of the grout.

In the case shown in Figure 5-80b, the corrugated metal ducts are provided in the bottom part of the column element; these must align with the (starter) bars protruding from the foundation, which may be either precast or cast-in-situ. The placing of the column is followed by a proper grouting operation.

Figure 5-81 shows conceptually similar solutions for column-to-foundation connections with steel splice sleeves. The same considerations as above apply in terms of the construction operation, control of tolerances and alignment/verticality, and grouting operations.

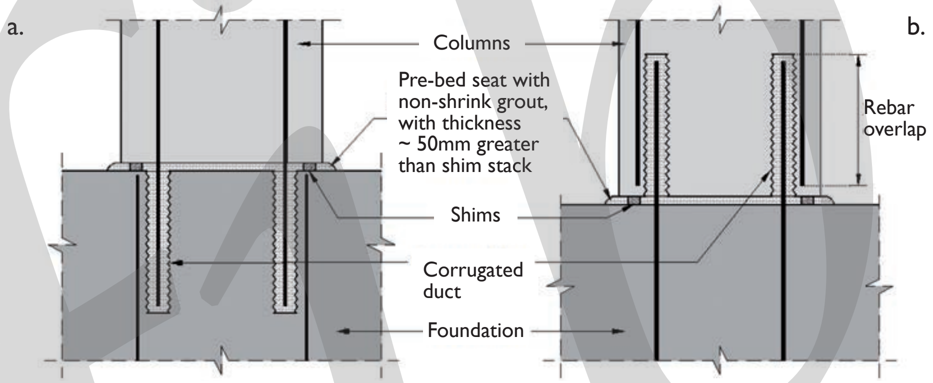


Fig. 5-80 Column-to-foundation connections with corrugated metal ducts (Other details not shown for clarity)

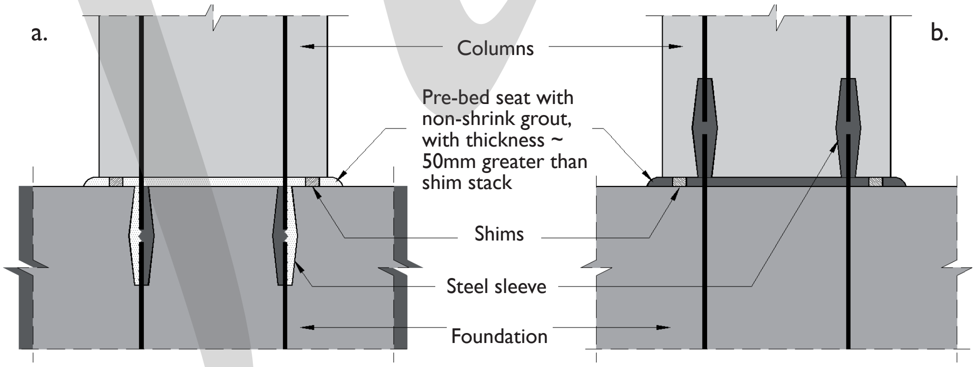


Fig. 5-81 Column-to-foundation connections with steel splice sleeves (Other details not shown for clarity)

Whenever mechanical splices are used a full mechanical splice will generally develop in tension or compression, with a capacity of at least 1.25 f_y of the bar, according to Clause 12.14.3.2 of ACI 318 (2011). Such mechanical splices in ACI 318 (2011) are classified as type 1. However, whenever mechanical splices are used in regions of potential yielding, such as in the plastic-hinge region in Figure 5-82b but also where considerations to strain penetration just above the column-to-foundation interface should be provided in the solution presented in Figure 5-82a, there should be documentation and evidences for the actual strength characteristics of the bars to be spliced, the force-deformation characteristics of the spliced bars and the ability of the splice to meet the specific performance requirements. Such mechanical splices are classified as type 2 (Clause 21.1.6 of ACI 318, 2011). Under IBC (2012), for highly active seismic regions, Type 2 mechanical splices in plastic hinge regions are required to develop at least 160% of the specified bar yield capacity.

Notes:

- *In all the above cases attention should be paid to the quality of the grout and proper grouting operation/procedure.*
  Generally the grout for splice sleeves is of premixed, high-strength, shrinkage- compensating formulation. The grout used in splice sleeves may cost more than the usual bedding materials but it can be installed very quickly, thus reducing labor and crane time.
- *The choice of the type of column-to-foundation connection is more governed by the need to ensure the development of plastic hinges in the columns above the foundation, rather than by matters related to ease of construction and economy.*

5.3.4.3 Column-to-foundation connections with anchor bolts

Figure 5-82 shows an alternative fastening solution based on the use of anchor bolts at the base of the column with the development of a plastic hinge in the column element relocated above the foundation interface.

The column-to-foundation-connection system presented in Figure 5-82 has been developed for consideration as an effective alternative for the use of traditional precast pocket foundations (Bianco et al., 2009). This connection solution is based on the mechanical connection between steel shoes embedded in the column base and protruding anchor bolts anchored in the foundation. Nuts and washers attached to the anchor bolts allow for the vertical position to be controlled to the height of the column as well as for the fixity of the connection. An additional injection of cement mortar bed into the void below the column is required to complete the construction of the system. The stress transfer from the anchor-bolts-steel-shoe system is based on longitudinal bars welded at the top of each shoe and on additional overlapped bars, which represent the steel reinforcement of the column. Further details are to be considered for the effects caused by the eccentricity between the anchor bolt and the longitudinal bars to be avoided, the strength of the bottom part of the column to be improved and the longitudinal bars to be tied effectively. The seismic performances of the system have been evaluated through numerical and experimental research (Bianco et al., 2009).

a. b. c.

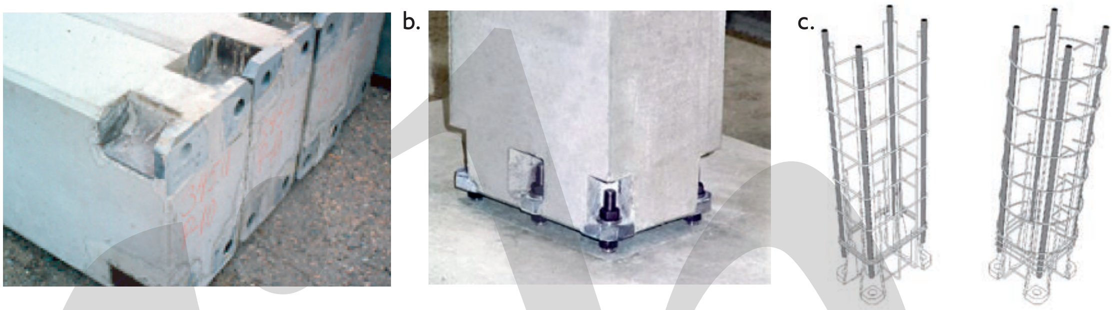


Fig. 5-82 Column-to-foundation connections with anchor bolts

5.3.4.4 Column-to-foundation connections with steel base plates

Unlike socket or grouted-splice sleeve connections, which require time to develop the necessary strength, column-to-foundation solutions with base plate connections can achieve immediate stability, which facilitates the erection procedure. Often the choice of base plates rather than sockets or grouted sleeves is based more on work productivity and economy (when the reduced erection time is taken into account) than on structural requirements.

Figure 5-83a and 5-83c show two commonly used base plate details (PCI, 2010), one where:

- the base plate is larger than the column (5-83a)
- the base plate is flush (5-83c)

Figure 5-84b shows the force distribution and reactions at the column base. These forces are needed for the dimensioning of the grout (under the steel plate), the steel plate and the bolts. The thickness of the high-strength grout should not be less than 5.0 centimetres (1.97 inches). The anchor bolt diameter is determined by the tension or compression in the threaded portion of the bolt. The anchor bolts may be hooked, L-shaped or fitted with steel plates to increase their pullout capacity. Confining reinforcement around the anchor bolts is recommended.

The thickness of the base plate depends on the projection beyond the column face and is subjected to actions such as those shown in Figure 5-83b. The projection length of the plate should be limited in order to reduce the thickness of the steel plate as much as possible. However, a minimum practical limit should be envisaged to permit easy construction. The bending of the plate, for example, around line z-z in Figure 5-83a and 5-83c should also be checked (based on Section 6.12 of PCI, 2010, among others) if the analysis for erection loads or temporary construction loads (before grout is placed under the plate) shows that either all the anchors are in compression or the anchors on one or both sides of the column in tension.

The anchor bolt diameter is determined by the tension or compression on the steel area of the threaded portion of the bolt. Anchor bolts may be ASTM A 307 bolts or threaded rods of ASTM A 36 steel. The anchorage strength of the bolts in tension should also be verified carefully. Holes in the plate are normally oversized to accommodate construction and production tolerances. In general, there should be a gap of between 10 and 15 millimetres (0.39 and 0.59 inches) all around each bolt. Generally, both base plate and anchor bolt stress can be significantly reduced by using properly placed shims during erection (see Fig. 5-83a and 5-83c).

5.3.4.5 Column-to-foundation connections with external mild-steel reinforcement

As discussed in Section 5.3.3 on jointed ductile connections for beam-to-column connections, moment-resisting column-to-foundation connections can be obtained by using external and replaceable devices, referred to as ‘plug and play’, consisting of mild steel rebars machined down to the desired ‘fuse’ dimension and inserted in a steel tube acting as an anti-buckling restrainer (see Fig. 5-59). The tube is then typically grouted or epoxied to allow for good confinement/buckling restraint to occur.

This configuration enhances the constructibility (in both ease and speed) of the structure while improving its repairability after an earthquake, thus dramatically reducing costs associated to direct repair and downtime.

Nuts and washers are used to control the column verticality during erection. A substantial lateral restraint is achieved at an early stage of the construction process.

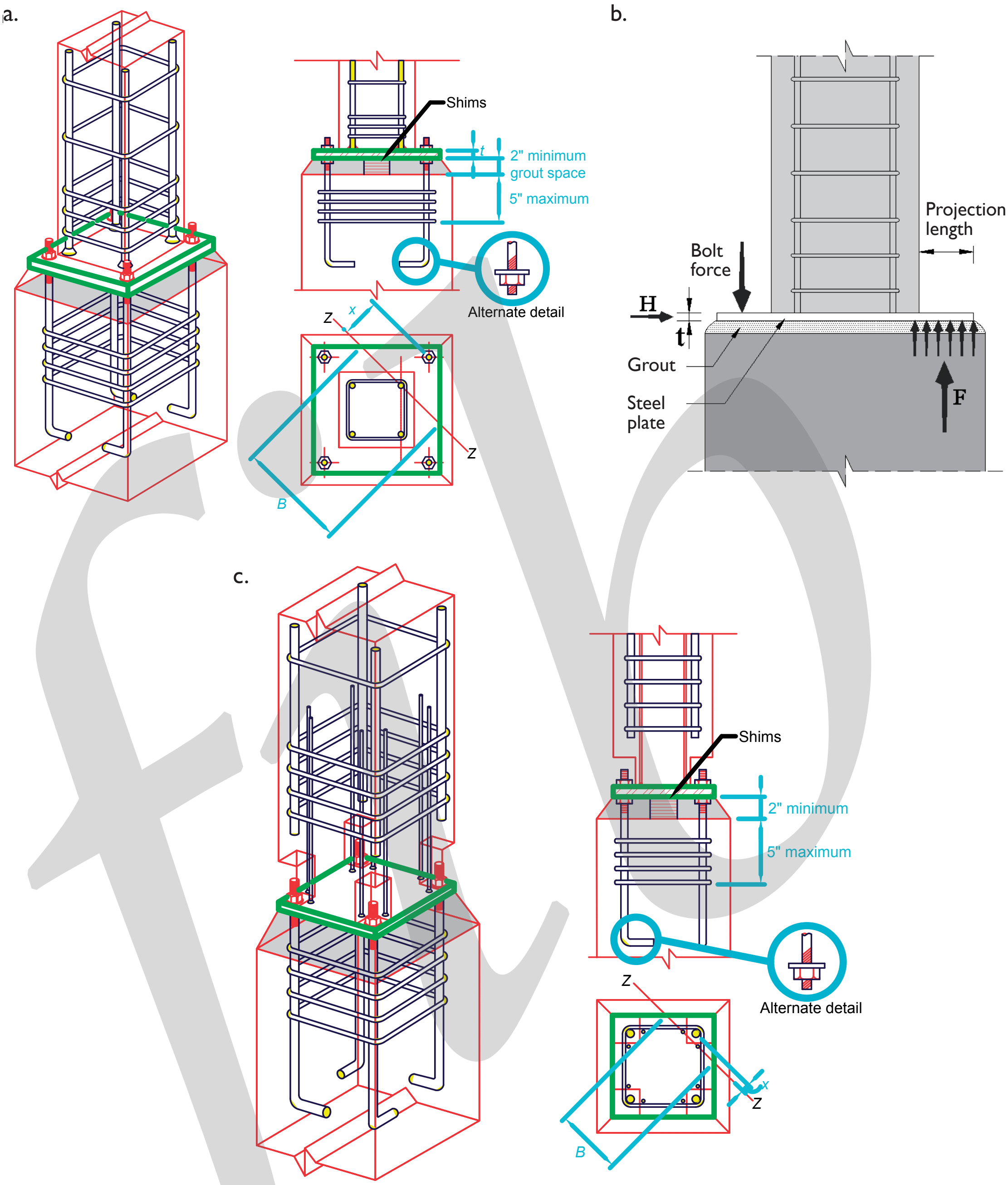
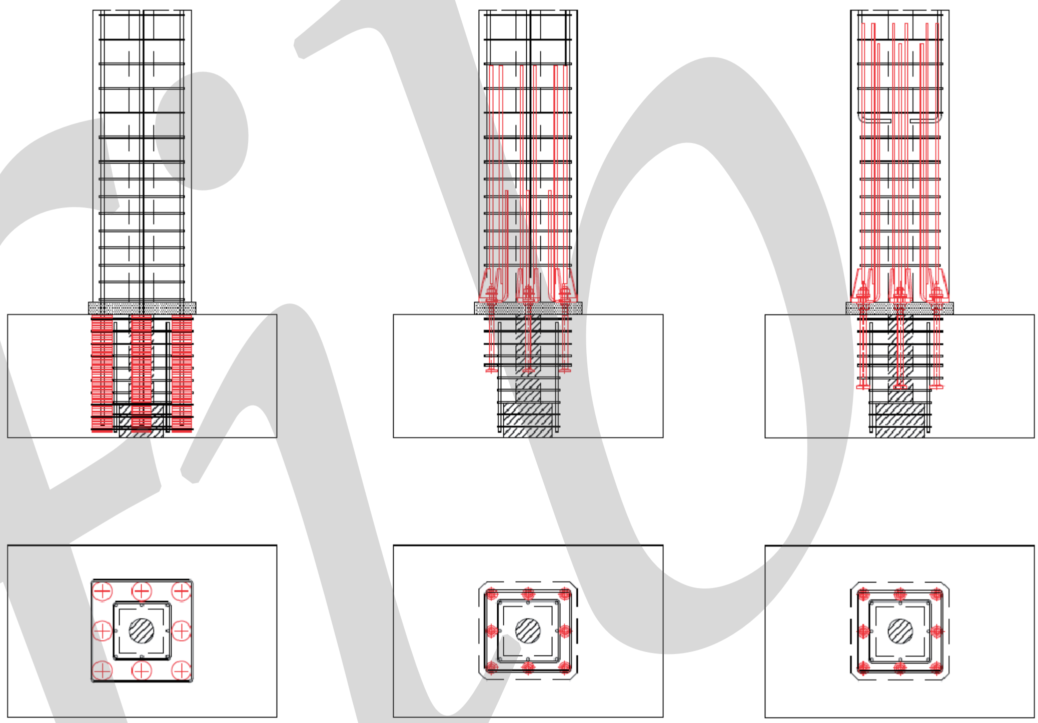


Fig. 5-83 Column-to-foundation connections with steel plates
a. Base plate larger than column (courtesy of the Precast/Prestressed Concrete Institute)
b. Force distribution on base plate larger than column
c. Flush base plate (courtesy of the Precast/Prestressed Concrete Institute)
(Other details not shown for clarity)

5.3.4.6 Other types of column-to-foundation connections

Figures 5-84a and 5-84b show different types of column-to-foundation connections that were experimentally tested at the Technical University of Milan as part of SAFECAST (2012c), a European research project. More information can be found in the project's technical report, *Deliverable 2.4: Experimental behaviour of new/improved connections.*



1st scheme
- Ribbed metallic tubes + mortar in tubes
- Dissipation at the column base Plastic hinge developing along the height of the column
- No stability in transitory phases

2nd scheme
- Columns shoes
- HCC + HAC + mortar at the interface
- Dissipation at the column-foundation interface
- Plastic hinge concentrated in 5cm (1.97 in.) mortar pouring

3rd scheme
- HCC with longer and smaller bars + HAC + mortar at the interface
- Dissipation at the column base
- Plastic hinge developing along height of column

Fig. 5-84a Column-to-foundation connections (based on SAFECAST, 2012c)

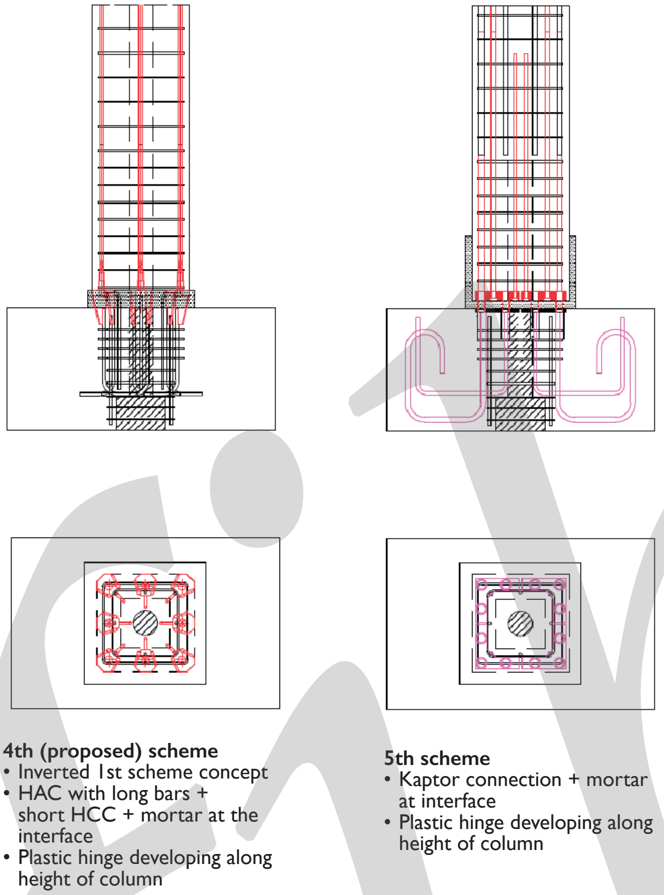


Fig. 5-84b
Column-to-foundation connections (based on SAFECAST, 2012c)

5.3.5 Beam-to-beam connections

Connections between beams might be required in the design and construction of frame systems (see Fig. 5-20, 5-21 and 5-22).

As an example, in the case of the one-way frame discussed in Section 5.3.2.3 (System S3), beams are connected either in their midspan or elsewhere (see step 5 of Fig. 5-35), but always away from their critical sections. In this respect, the protruding bars from the beam elements are lapped in a connection near or at midspan of the beam and then cast with concrete (a wet connection).

There are several approaches to connecting beams together (see Fig. 5-85 to 5-88). Forces are transmitted from one bar to another by lapping bars (with or without the bending of the bars), welding or proper mechanical devices. In all cases, the transmission of forces should be consistent with the local standard code recommendation.

Figure 5-85 illustrates a beam-to-beam connection where straight-bar lap splices are used to connect beams with short aspect ratios. In this case as well as in the case of Figure 5-86, the longitudinal reinforcement at the connection in the mating beams is offset to facilitate the erection procedure. The offset of the longitudinal reinforcement should also comply with local code requirements. According to Eurocode 2 (2004), this offset should not be greater than the lowest of the following results: four times the bar diameter or 50 millimetres (1.97 inches). According to ACI 318 (2011), on the other hand, the offset should not be spaced transversely farther apart than the smaller of one-fifth the required lap splice length or 150 millimetres (5.91 inches).

Figure 5-86 depicts a simple way of connecting precast beams together using a double-straight-bar lap splice. Such beam-to-beam connections are used whenever the span of the beams between the columns is long enough to permit a larger connection length.

Connections between beams may also be achieved either by welding the longitudinal beam bars protruding from the mating beam units (see Fig. 5-87) or by using mechanical couplers (see Fig. 5-88) and cast-in-situ concrete. However, for both the above types of connections, strict construction tolerances are required.

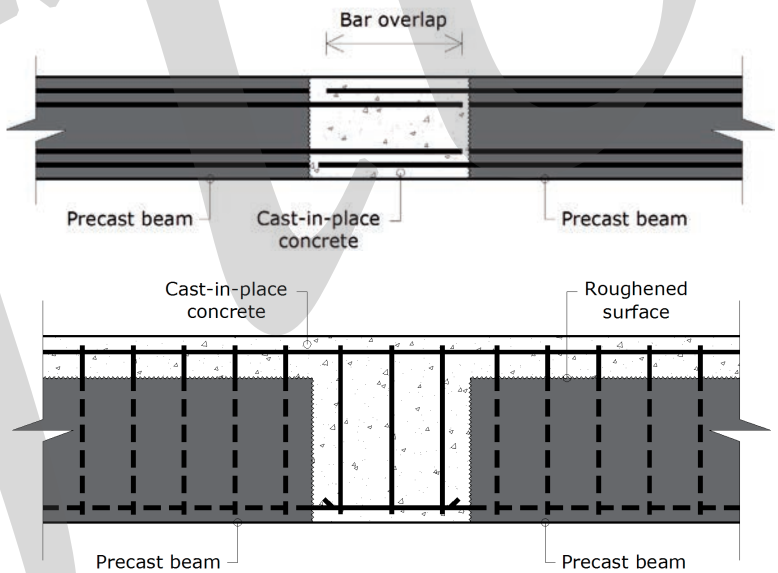


Fig. 5-85 Beam-to-beam connection with noncontact straight-bar laps (Other details not shown for clarity)

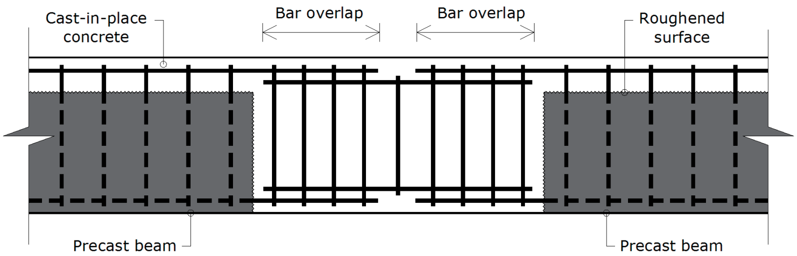


Fig. 5-86 Beam-to-beam connection with double-straight bar laps
(Other details not shown for clarity)

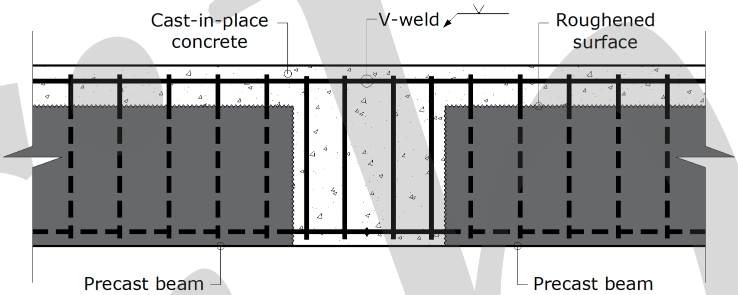


Fig. 5-87 Beam-to-beam connection with welded bars
(Other details not shown for clarity)

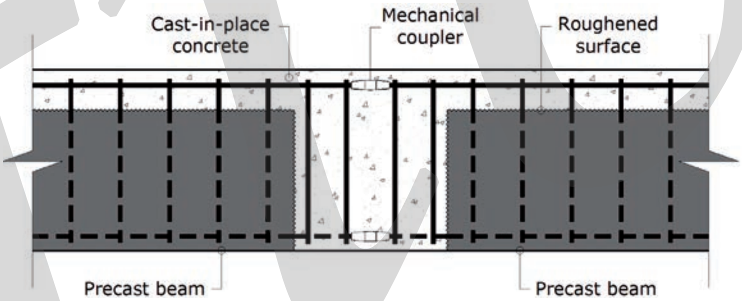


Fig. 5-88 Beam-to-beam connection with mechanical couplers
(Other details not shown for clarity)

Notes:

- *It is recommended that the surface of the precast beam ends be roughened, clean and free of laitance at every connection between bars.*
- *Whenever mechanical couplers are used they should comply with Clause 12.14.3.2. of ACI 318 (2011).*
- *Accurate quality control for welding is required for welded bars.*

- *Restrepo et al. (1995) provide information about design and construction on a variety of beam-to-beam connections.*
- *More information about design and construction on a variety of secondary-beam-to-main-beam connections can be found in a special state-of-the-art report on precast concrete construction developed by the Centre for Advanced Engineering (CAE) in New Zealand, (1999).*

5.3.6 Column-to-column connections

Column-to-column connections might be required in frame systems. Figure 5-22 depicts a number of framed connection systems.

However, for the purposes of prefabrication, erection and transportation, a frame system is often divided into individual beam and column components or sub-assemblies (such as T-shapes and cruciforms). Connections of these individual elements or sub-assemblies can be subject to large forces and need to satisfy the capacity design (overstrength) requirements that state that the strengths of columns at joints must be of a specified percentage. Bending moments are usually transferred through these connections via a force couple formed by the compression resultant in the packed grout or cast-in-situ concrete and by the tension in spliced reinforcing bars.

Two main approaches are usually used for connecting precast columns. In both approaches a tolerance gap is left between the jointing surfaces. The surfaces must be roughened, clean, and free of laitance.

- One approach mostly employed in the United States uses proprietary grouted steel sleeves (see Fig. 5-89).

  A mortar bed is indispensable in the interfaces of the precast members that are to be connected. The mortar to be used for bedding should conform to the engineer's specifications. The use of factory-prepared bedding grouts is highly recommended and the use of mostly uncontrolled field-mixed sand-cement-water materials should be avoided. Bedding may be placed prior to the setting of the precast element (prebedding) or after the setting (post-bedding).

  In all cases, the splicing bars should be aligned inside the steel sleeve and the sleeves should be free of debris before grouting can begin and the sleeves filled. Entrapped air should be removed during grouting.

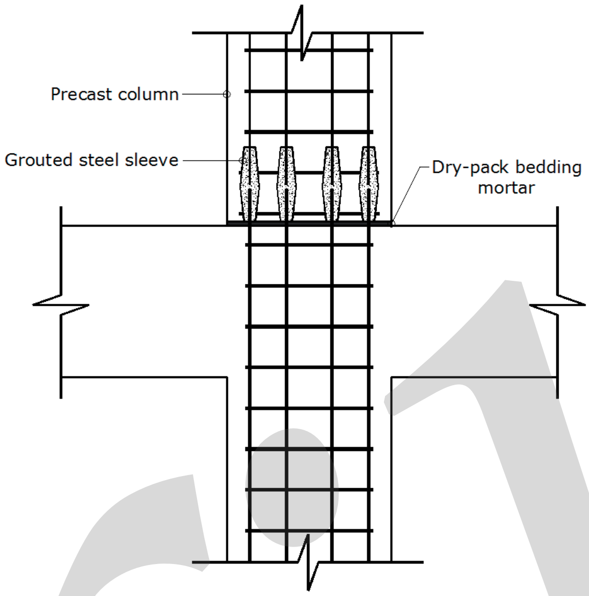


Fig. 5-89
Column-to-column connection through grouted steel sleeves
(Other details not shown for clarity)

It should be noted that the use of steel sleeves requires the oversizing of the transfer reinforcement in the column and its region. This implies that to maintain the minimum concrete cover requirements specified by the design codes, column bars should be placed further towards the centre of the column than in cast-in-situ construction. Sleeves should be type 2, according to Clause 21.1.6.1 of ACI (2011). That is, they should typically be considered to be 160% of the specified yield. The same rules apply for the distance between each grouted steel sleeve as for rebars. The staggering of grouted steel sleeves is usually not required.

- The second approach (see Fig. 5-90) makes use of corrugated metal ducts and grout (grouted sleeve connection). This approach, along with the previous one, is commonly used in Europe.

  Starter bars projecting from the column below pass into the purpose-manufactured corrugated metal ducts of the bottom part of the column above. The duct around the bars is then filled with pressurized, nonshrinkable, flowable grout of a strength in accordance with the design but at least equal to that of the column. The upper ends of the sleeves are open and flush with the concrete face. The duct must be at least 6 millimetres (~0.24 inches) in size and clear on all sides of the bar (Elliott, 2005). The corrugation increases the bond strength through wedging action. Smooth sleeves are not permitted. The design procedure is the same as for equivalent cast-in-situ reinforced concrete columns. The assumption is that the full bond is provided to the starter bars, enabling their full length to be developed. These bars are overlapped with the bars of the other column. Bedding mortar must be applied between the interfaces of the precast members to be connected, as described in the aforementioned approach.

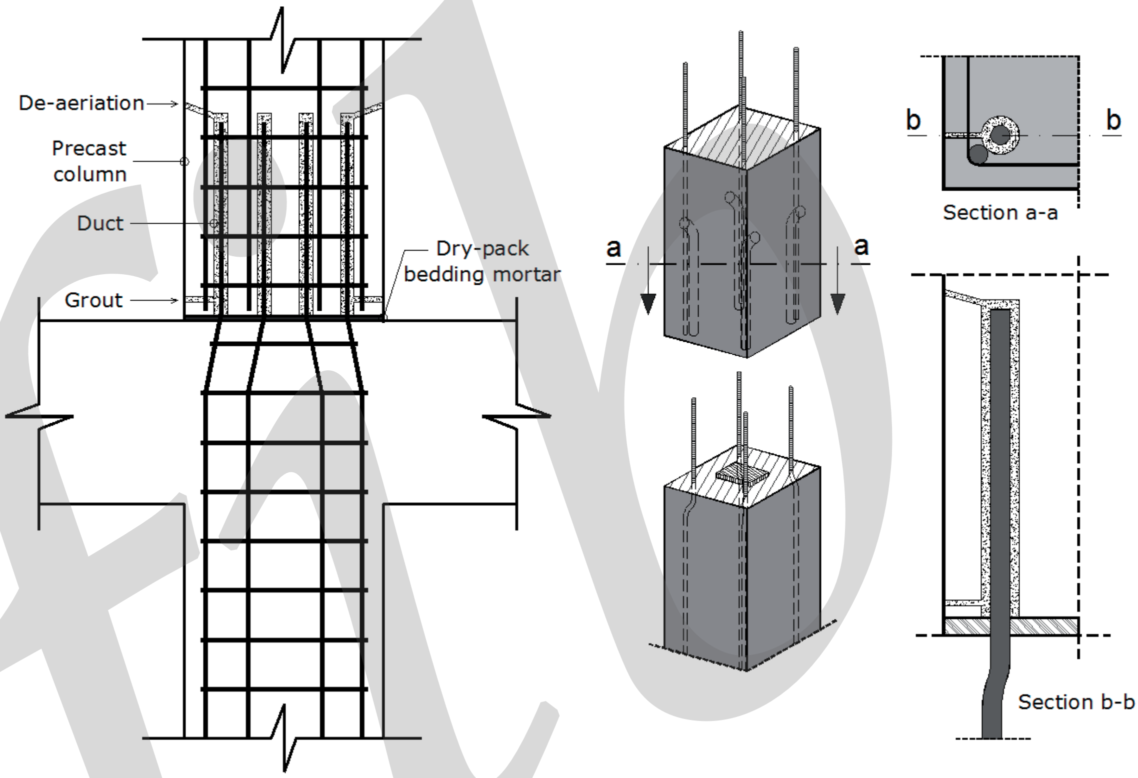


Fig. 5-90 Column-to-column connection with grouted steel sleeves
a. Connection of column-to-beam subassemblies above floor level
b. Column-to-column head connection
(Other details not shown for clarity)

6 Large-panel wall systems

6.1 General

Precast large-panel wall systems are mainly used in cases where there is no need for large open spaces, such as in apartment buildings, office buildings, hotels, housing, educational and administrative buildings. Such systems are composed of precast large-panel load-bearing walls and precast concrete slabs. Usually, walls are a storey high and both walls and slab panels are room size. Walls might also be multi-storey.

Alternatively, floors and roofs can be composed of precast components of other types, such as hollow-core, solid concrete and plank-floor units. In all these cases, the diaphragm action of the floors needs to be mobilized. This can be achieved through proper connections between precast slab elements and their supports or the use of topping of a proper thickness, or a combination of both.

Depending on the arrangement of the bearing walls in plan and the serviceability needs of the building, nonbearing (solid or sandwich-type) walls may also be used for partition or façade walls.

Façades are typically made of sandwich panels comprising a load-bearing inner concrete wythe, an insulating layer of appropriate material and thickness, and a nonbearing external wythe of architectural concrete.

The main advantages of the system are good seismic behaviour, speed of construction, good insulating properties, fire resistance and economy because the whole structure presents paint-ready, relatively smooth surfaces. From a structural point of view, the main advantage is a more or less uniform distribution of the gravity and lateral loads to the various structural elements and the avoidance of stress concentrations.

The possible disadvantages are less flexibility in layout and adaptability of the structure.

6.2 Classification

- Large-panel systems may be classified as follows based on the types of joints (wet or dry) with which panels are connected to each other and to the flooring:
  - large-panel systems incorporating wet connections
  - large-panel systems incorporating dry connections

- Generally, the word 'connection' is used to describe the regions where the elements are connected as well as the way in which they are connected while the word 'joint' refers to the area between the connected elements.

  - 'Wet joints' are made with cast-in-situ concrete. Structural continuity through the joint is achieved via the reinforcing bars protruding from the elements (across the joint) in the form of loops, which are welded or otherwise connected in the joint region before the in-situ concrete is placed (see Fig. 6-10a).

  - 'Dry joints' are made by welding or bolting together steel plates or other steel inserts that were purposely cast into the ends of the elements, thus transferring the actions between the elements at discrete points where the steel inserts are connected (see Fig. 6-1). Such joints are widely used in the United States but also in other countries.

  - The 'platform joint', shown in Figure 6-18, is an alternative to wet joints. This type of joint (see Section 6.8) is commonly used in the United States as well as certain other countries. (Figures are typically referenced in sequence.)

Note: Large-panel buildings incorporating dry joints (Fig. 6-1) are not covered in this document.

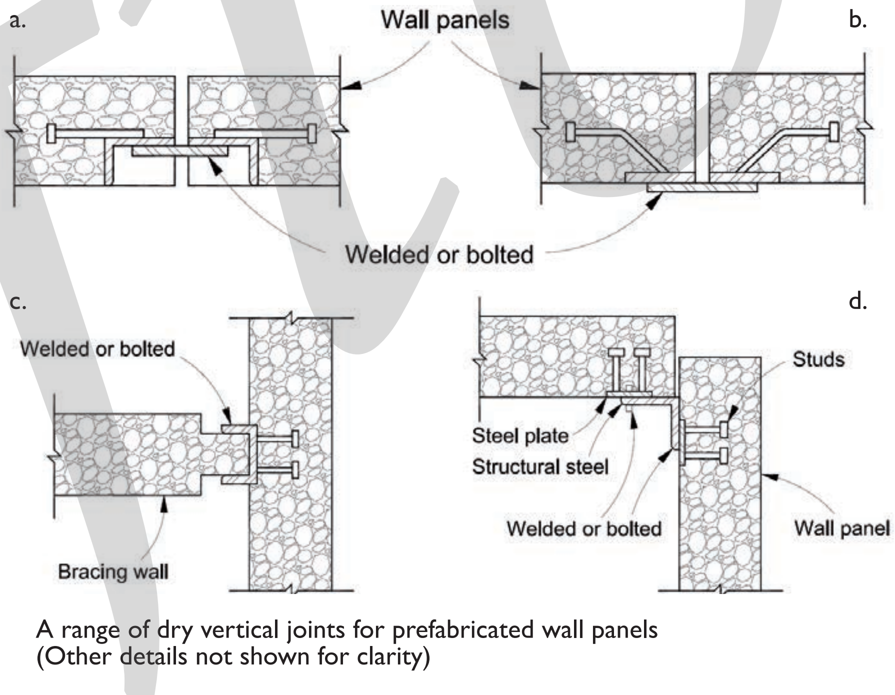


Fig. 6-1 A range of dry vertical joints for prefabricated wall panels
(Other details not shown for clarity)

- Large panel precast buildings may be also classified as follows, according to the orientation of the main load-bearing walls in relation to the longitudinal axis of the building:

  - Cross-wall systems, where the load-bearing walls are provided perpendicular to the longitudinal axis of the building (see Fig. 6-2a).

    Floors are mainly supported by these cross-walls. The longitudinal stability of the whole structure is achieved by cores and/or longitudinal shear walls. Façade walls are, as a rule, non-bearing sandwich walls.

  - Spine wall systems, where the load-bearing walls run parallel to the longitudinal axis of the building (see Fig. 6-2 b).

    In this type of buildings, floors are mainly supported by these longitudinal walls. Transverse stability is achieved by cores and/or transverse shear walls.

  - Two-way systems, in which the bearing walls run both longitudinally and in the transverse direction (see Fig. 6-2 c).

  All the types of systems described above may be used in seismic areas.

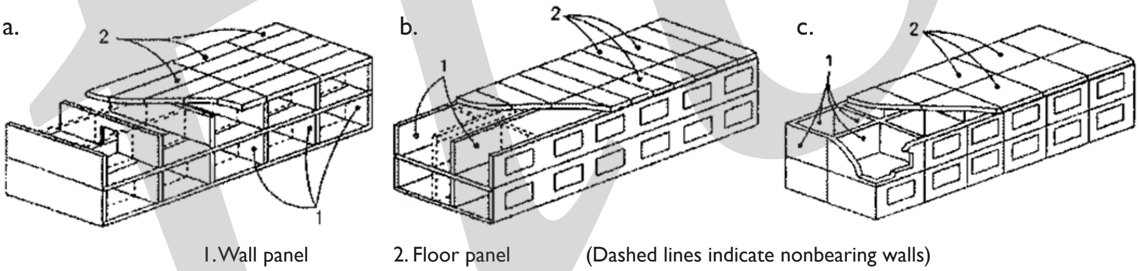


Fig. 6-2 Different configurations of large-panel systems
a. Cross-wall system b. Spine-wall system c. Two-way system

6.3 Seismic behaviour and structural integrity or robustness

Generally, precast buildings with large wall areas are inherently strong against lateral forces. However, such precast-concrete-wall buildings do not have the same characteristics as monolithic cast-in-situ structures. This is because:

- the whole structure is composed of a set of vertical precast walls, horizontal precast members and connections;
- both the elastic and the inelastic responses of the structure depend primarily on the configuration and the properties of the connections;
- the properties and behaviour of the connections, including their strength, stiffness and ductility, are largely controlled by the components of the connection.

The customary form of large-panel concrete construction typically does not emulate monolithic, cast-in-situ construction. This is because, unlike in other precast concrete systems, all the panels in large-panel construction are part of the lateral-force resisting system, which makes strict emulation almost impossible. It is also worth mentioning that strict emulation would remove many of the advantages of precast concrete construction in terms of preferred mechanisms for the dissipation of seismic energy (see Section 6.4).

The ductility and toughness of precast-concrete large-panel walls are less than those of similar configurations built with monolithic, cast-in-situ concrete.

The seismic response of large-panel systems is characterized:

- by energy reduction due to the lower bending stiffness of the walls at the connections and the onset of yielding at lower loads;
- by energy dissipation primarily in the vertical connections and partly in the horizontal connections during major earthquakes.

The connections between individual panels are weak links in large-panel buildings. When connections are properly detailed (for inelastic deformations) to perform in a ductile way, they can undergo cyclic inelastic deformations without failure while protecting the wall panels from damage under large earthquake forces. Such wall assembly behaviour could be regarded as superior compared to the seismic response of monolithic shear-wall structures. This type of behaviour has been verified in numerous studies (Mueller et al., 1979; Schriker et al., 1980; Schultz et al., 1979; Becker, 1983; among others).

Generally, the overall integrity of a precast structural system, which is inherently discontinuous, can be substantially enhanced by providing continuity in tension at connections in both horizontal directions, as well as vertically by means of relatively simple detailing of the reinforcement. The aim is to ensure that all precast elements making up a floor system can effectively interact to transmit diaphragm forces. Moreover, should vertical supports become displaced due to unexpected actions, sufficient continuity should remain to enable catenary and/or cantilever action to be mobilized, thereby minimizing the risk of the total collapse of the precast systems.

- In precast large-panel buildings, horizontal and vertical ties must be suitably incorporated into the structure to keep a local event (caused by abnormal loads) from becoming an overall collapse of the structure.

- For precast-concrete bearing-wall structures three or more storeys in height, ACI 318 (2011) prescribes the minimum number of horizontal and vertical ties that must be incorporated into the structure (see Section 6.8.3). The vertical tie requirements, in fact, apply to all vertical structural members that are precast, except cladding. Although the specific requirements in ACI 318 (2011) are based on the experimental programme for large-panel systems, the integrity tie requirements were generalized to add redundancy for the connections in all precast-concrete systems.

6.4 Possible mechanisms for dissipation of seismic energy

Figures 6-3, 6-4 and 6-5 provide examples of possible behaviour modes of large-panel systems that contribute to ductility. When **emulative design** is followed:

Precast panels and their connections are designed and built in such a way that the entire wall acts monolithically along its height (see Fig. 6-3a). This requires that the connection details of the walls be made intentionally overstrong, thus forcing the inelastic strains to occur away from the connections in regions of the element that can be more readily detailed for ductility. In such walls, plastic hinges are formed at the base, as in monolithic cantilever walls (see Fig. 6-3b). This approach is suitable where the wall panels are solid, with no significant openings.

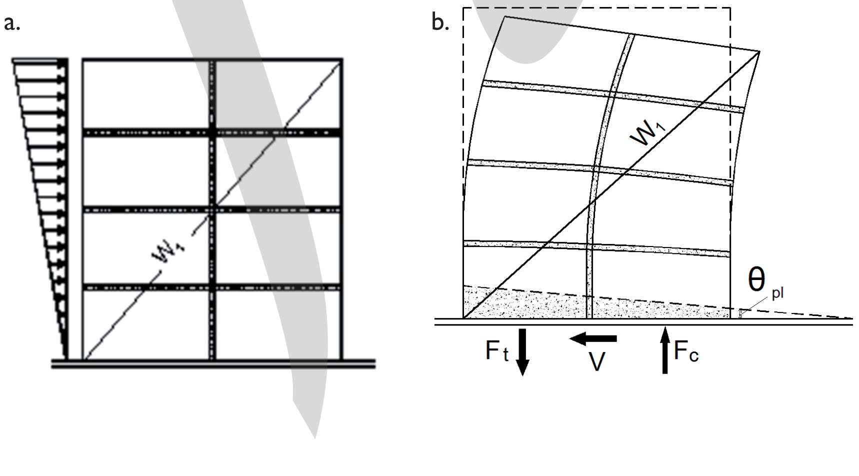


Fig. 6-3
a. Large-panel wall
b. Ductile behaviour at base of wall

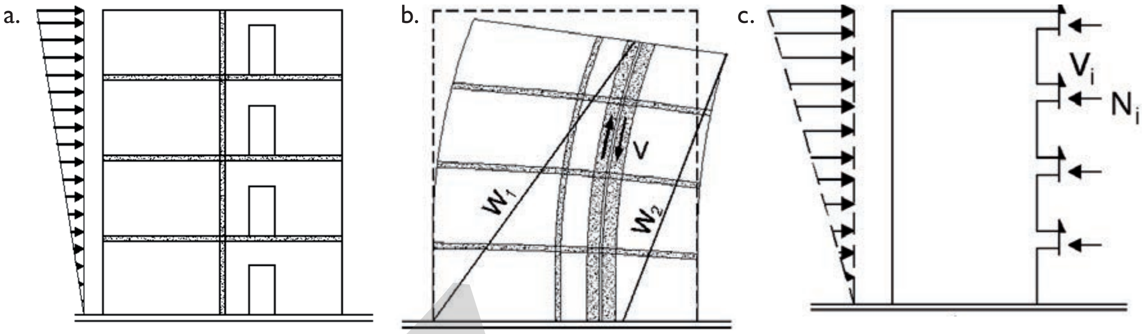


Fig. 6-4 a. Large-panel wall with openings b. Ductile behaviour at coupling beams
c. Midspan actions at lintels

Figure 6-4 shows a wall composed of panels connected together monolithically, that has vertically stacked openings. The reasonable locations for seismic energy dissipation are the coupling beams above the openings. The total wall may be treated as two cantilever monolithic walls (W1, W2 in Fig. 6-4 b) connected together by the coupling beams (see Fig. 6-4 c).

In **jointed large-panel construction** the primary location for energy dissipation is along the vertical joints, depending on the type of joints (see Fig. 6-5).

In the case of wet connections, in which vertical joints are provided with keyed interfaces between the precast panels and the cast-in-situ concrete joint as well as well distributed reinforcement in the form of loops across the joints (see Fig. 6-10a), the preferred method of energy dissipation is through shear sliding at the vertical joints (see Fig. 6-10b). Possible shear sliding may also occur along the horizontal joints (see Fig. 6-5c). However,

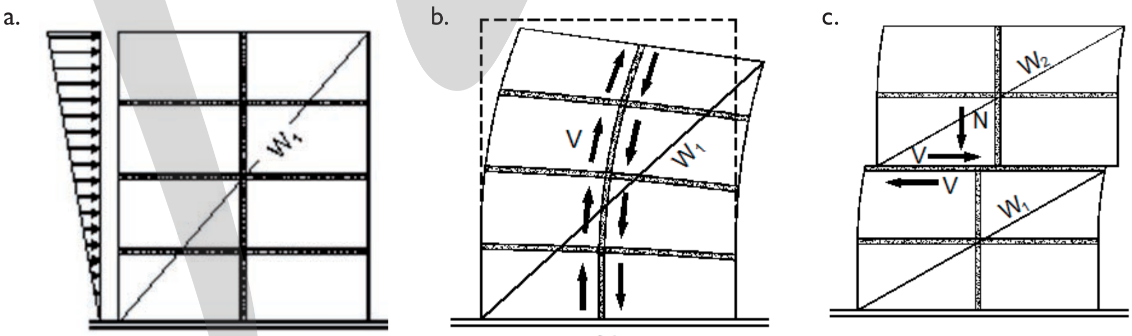


Fig. 6-5 a. Large-panel wall b. Ductile behaviour along vertical joints
c. Sliding along horizontal joints

this latter type of sliding needs to be minimized, if not altogether eliminated, since the stability of the total building may be endangered by it.

Whenever the North American platform-framing system for jointed precast large-panel construction is used (see Section 6.8), primary energy dissipation occurs in major earthquakes due to yielding in vertical-joint connectors and to the tension yielding of vertical ties crossing the horizontal joints.

However, in all cases:

- the principle of weak vertical and strong horizontal connections should be followed, taking into account the unfavorable action of the vertical component of the seismic load (mostly for the horizontal joints);
- combinations of different energy dissipation mechanisms such as those depicted in Figures 6-4 and 6-5 may be used;
- vertical post-tensioning is frequently applied to provide necessary uplift reinforcing and stability.

6.5 Load effects in large-panel connections

In Figure 6-6, the effect of external loads, N, H, vertical loads of the slabs, self weight of the members, and volumetric changes if any, in the horizontal and vertical connections between panels are shown. In this figure the following notations are used:

a. Compression due to upper panels and reaction from vertical loading on the slabs
b. Shear due to lateral loading and diaphragm action of the slabs (in horizontal joints)
c. Horizontal forces acting in the plane of the slabs
d. In-plane moment in the wall
e. Out-of-plane moment in the wall
f. Vertical shear in the vertical joints

Figure 6-7 shows the origin of vertical shear stresses and horizontal tensile stresses on the vertical connections between large panels.

Generally, the predominant internal forces on the connections caused by the seismic response of a wall (see Fig. 6-8) are:

- in the vertical connections, shear forces;
- in the horizontal connections, compression for the entire length of the connection, accompanied by shear forces (see Fig. 6.8b) and compression for a part of the length of the connection, accompanied by shear forces (see Fig. 6.8c).

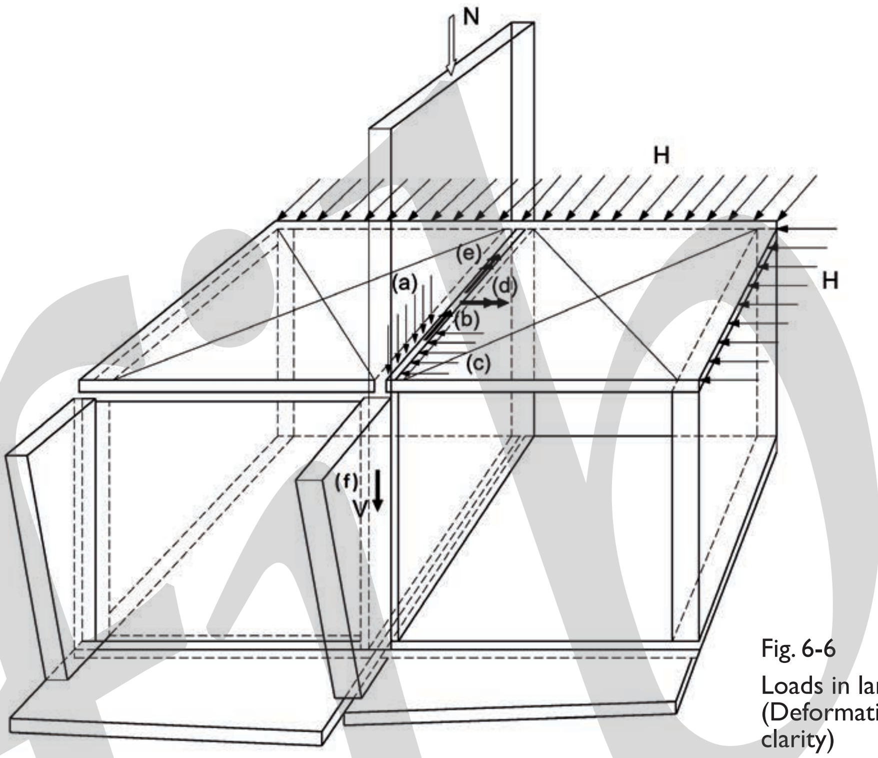


Fig. 6-6
Loads in large-panel connections (Deformations are not shown for clarity)

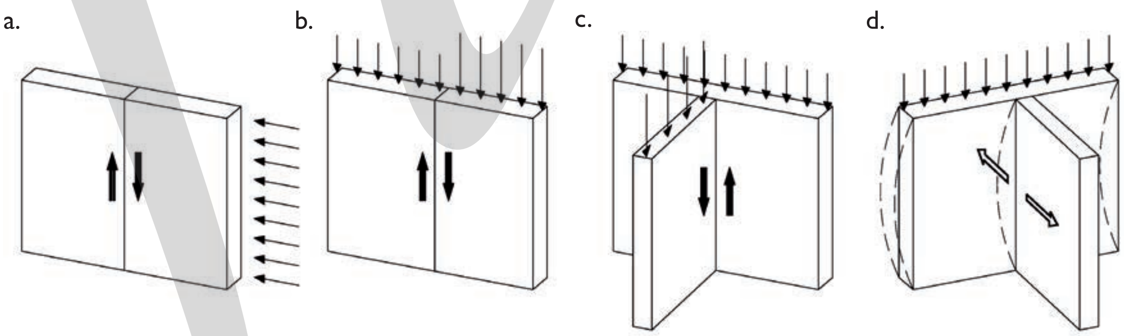


Fig. 6-7 Internal forces in vertical connections
a. Vertical shear caused by in-plane lateral loads of walls, differential shortening, or differential settlement of walls; b. and c. Vertical shear due to uneven loading on walls; d. Horizontal shear due to out-of-plane bending of adjoining walls

6.6 Configuration and structural behaviour of wet joints made with cast-in-situ concrete and loop reinforcements

Shear connections along vertical joints commonly have keyed interfaces between the precast panels and the cast-in-situ concrete in the joint as well as well distributed reinforcement in the form of loops across the joint. Joints without shear keys may also be used. Generally, the shear resistance of the joint (without perpendicular external compression) is characterized by the roughness of the interfaces of the precast members, the concrete grade of the cast-in-situ concrete in the joint, the possible value of the coefficient of friction (<1.0) that will develop under shear sliding, the percentage of the transverse reinforcement and also by the type of loading (monotonic or cyclic). When joints without shear keys are used, much higher percentages of transverse steel are expected to be required than for keyed shear joints. This steel should be uniformly distributed along the joint with proper spacing.

Longitudinal reinforcement along the joint is also required to reduce the widths of cracks developing in the concrete of the joint. Figure 6-9 depicts the configuration of keyed shear connections and indicates the possible development of cracks along the joint in the ultimate limit state for monotonic and cyclic loading, as based on experimental research (Tassios and Tsoukantas, 1983). The strength of the connections under cyclic loading may be assumed to be 50% lower than the strength monotonic loading.

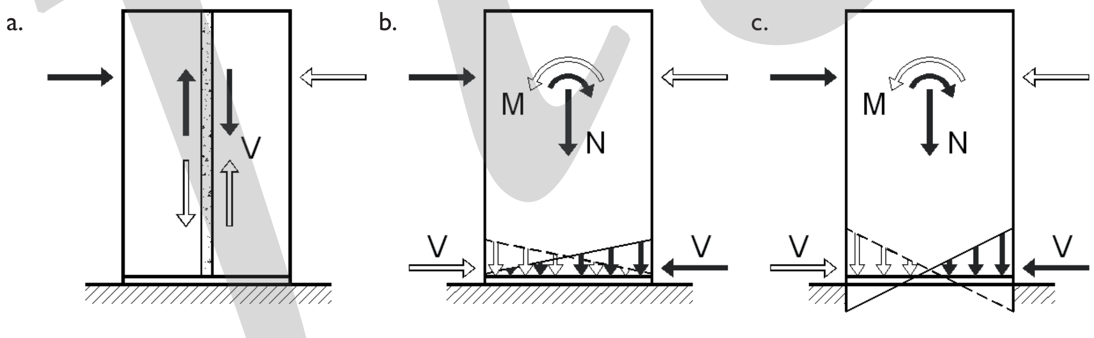


Fig. 6-8 Predominant internal forces in vertical and horizontal connections caused by seismic response of a wall
a. Shear in vertical connections
b. Compression along entire length of horizontal connection, accompanied by shear
c. Compression over part of length of horizontal connection with simultaneous shear

Figure 6-10 shows the typical transverse reinforcement in a closed vertical shear connection, together with shear stress-shear slip relationships (under monotonic shear loads) that are dependent on the configuration of the interface between precast panels and the cast-in-place concrete in the joint.

Figure 6-11 presents different configurations of such shear connections as covered in NZS 3101 (2006). The configuration chosen for a particular application depends on the seismicity of the area, economic considerations and available experience.

Figure 6-12 shows vertical-wall joints used in high seismic zones in Japan (ACI 550.1R-09, 2009).

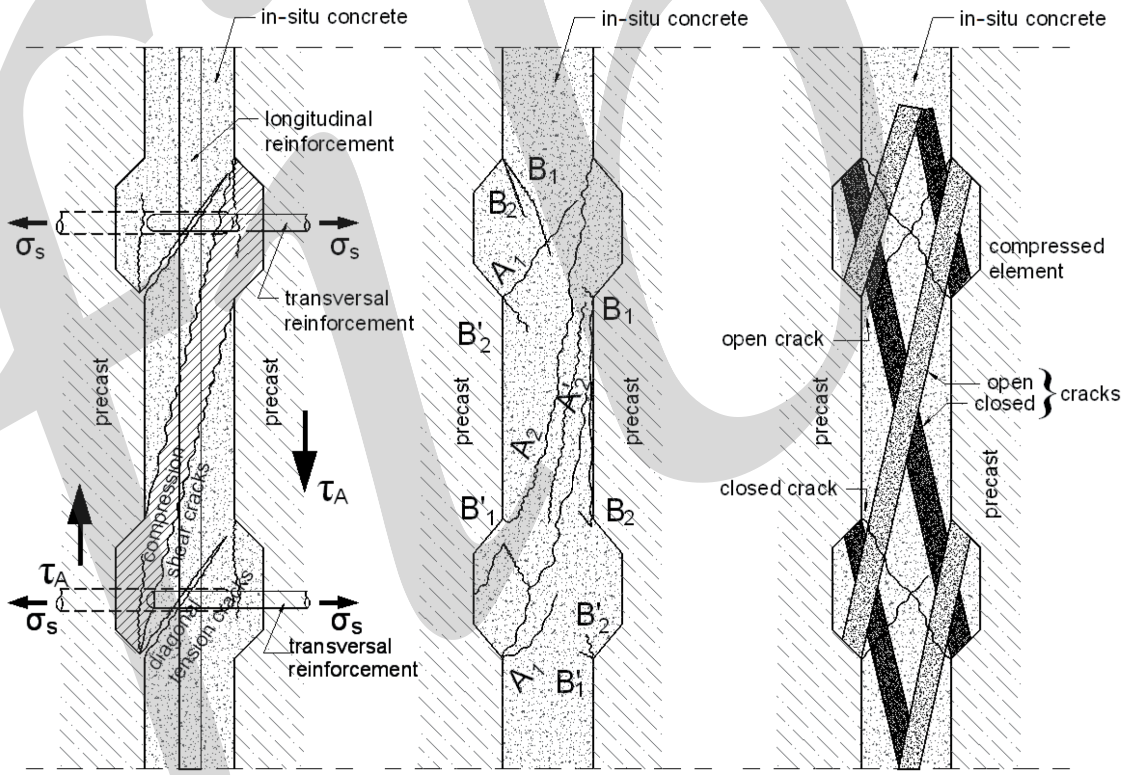
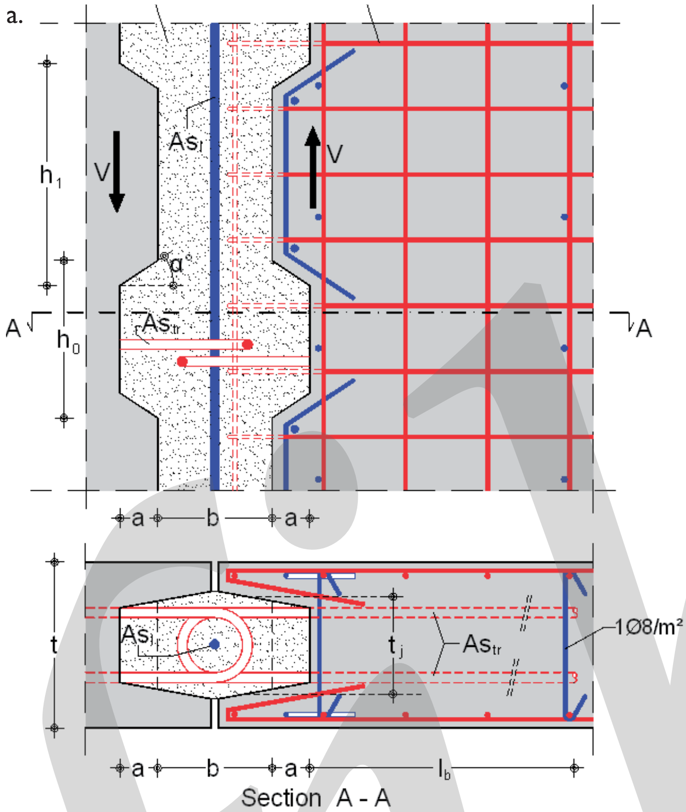


Fig. 6-9 Configuration of shear connections with indication of possible development of cracks for
a. Monotonic loading
b. Cyclic loading
c. Possible development of compression struts under cyclic loading
In all cases reinforcement is as shown in a.



l_j = length of vertical joint between floors

t = thickness of precast panel

t_j = average thickness of the joint concrete

α°= slope of key (45° to 60° recommended values)

n = number of keys along l_j

a = thickness of key inside panel (≥ 5 mm, 0.2 in.)

b = width of joint, b ≥ t

h_o = length of key inside precast panel

h_l = distance between keys

$\frac{n \times h_o}{l_j}$ = density of the keys (≤ 0.5)

As_{tr} = cross-sectional area of transverse reinforcement (loops)

As_l = cross-sectional area of longitudinal reinforcement

l_b = required anchorage length of loops

main reinforcement of connection and precast panels

additional local reinforcement

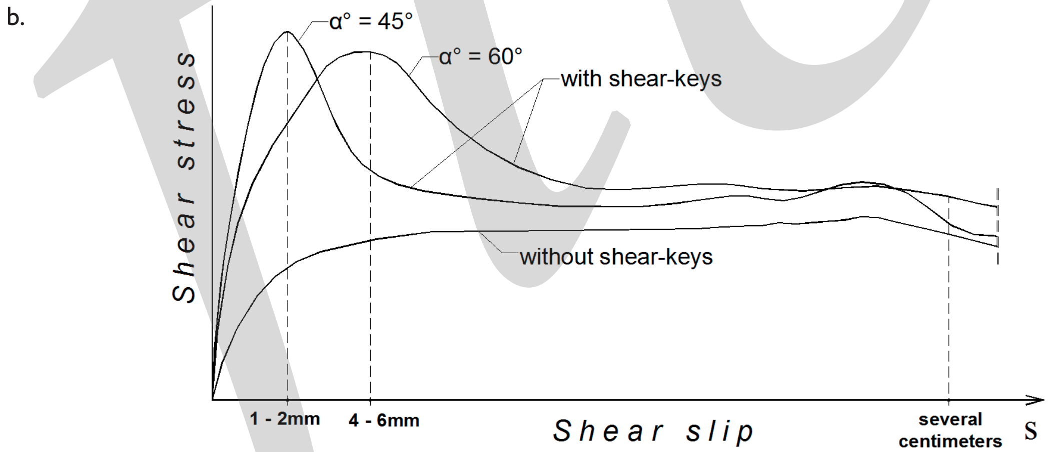


Fig. 6-10 a. Typical transverse reinforcement in closed vertical shear connection and typical reinforcement of precast panels (shown in one panel only)
b. Shear stress - shear slip relationship under monotonic loading as a function of slope of keys for keyed shear joints (based on Eriksson et al., 1978)

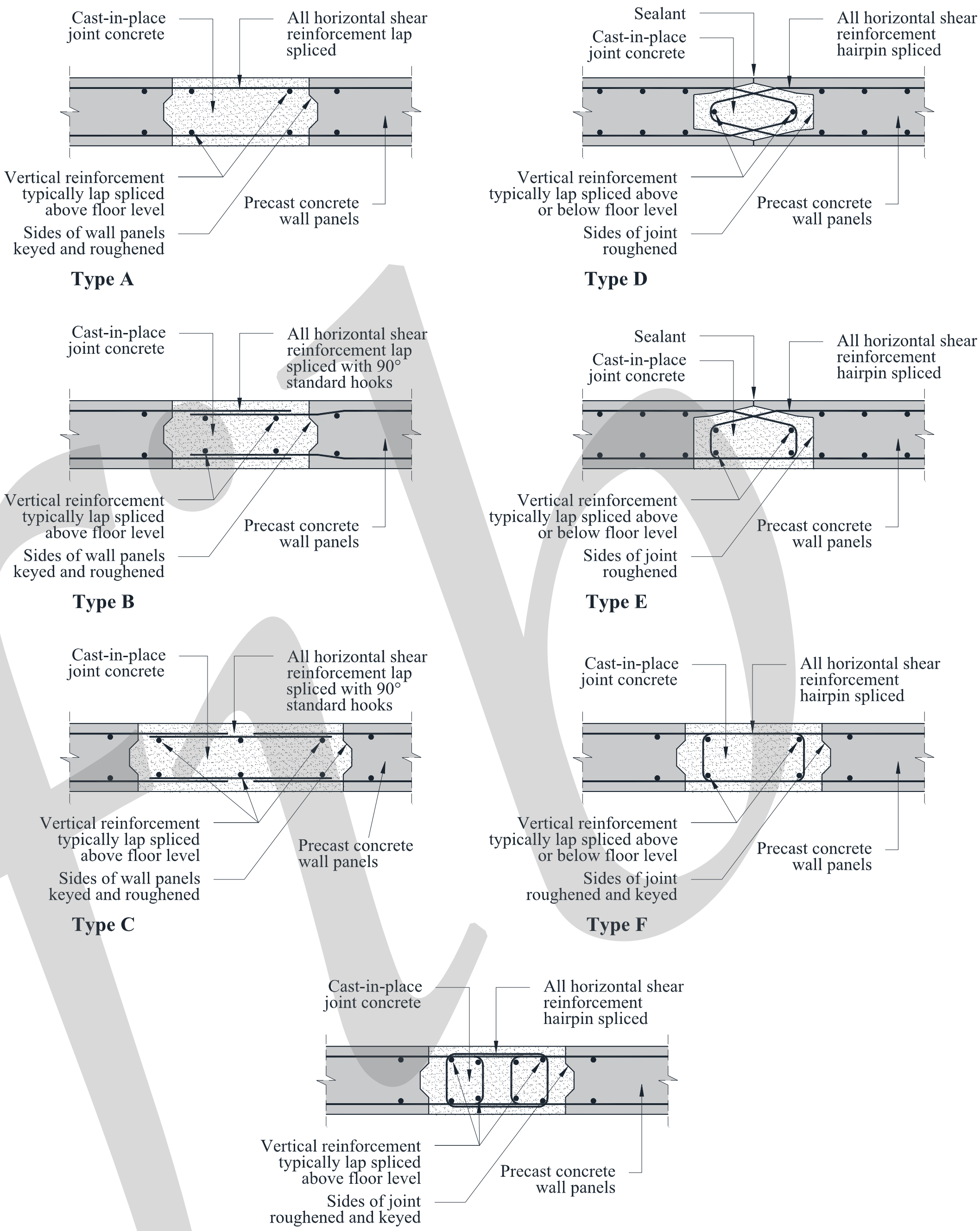


Fig. 6-11 Monolithic precast concrete wall construction joints (based on NZS 3101: 2006)

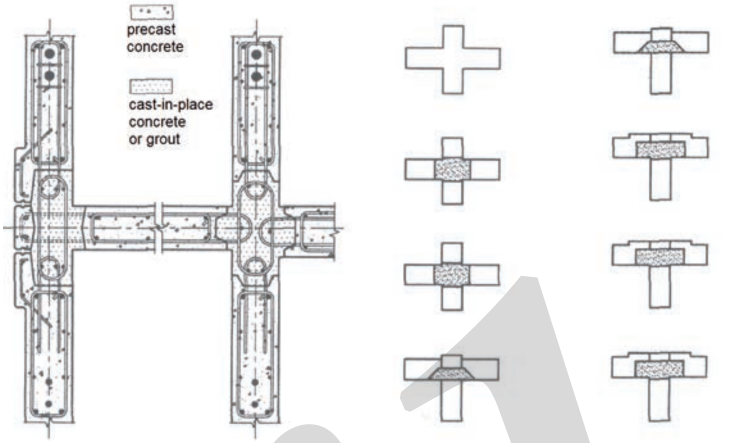


Fig. 6-12
a. Plan view of typical grouted or cast-in-situ vertical joints in shear wall panels reinforced for high seismic loading
b. Variations of vertical wall-to-wall connections
(Authorized reprint from ACI 550.1R-09, 2009)

The next paragraphs deal with **compression connection along horizontal joints**. The configuration of the horizontal connection is largely the same as for the vertical connections (see Fig. 6-10a), especially for rather tall buildings in areas of high seismicity. Generally, the design goal is to keep horizontal joints fully under compression. For this purpose, the use of vertical post-tensioning is recommended. If prestress is used or when it can be shown by analysis under the design earthquake that the entire length of the horizontal joint is under compression (see Fig. 6-8b) when the unfavourable effect of the vertical component of the earthquake ground motion is taken into account, the interfaces between precast and cast-in-situ concrete in the joint may be without shear keys, since compression drastically increases shear resistance and reduces shear sliding. In cases where the horizontal connections are partly under compression and partly under tension (see Fig. 6-8c) under the design earthquake and vertical post-tensioning is not used to bring the horizontal joint fully under compression, the total tension force should be resisted by continuous vertical tension reinforcement fully anchored in the body of the upper and the lower panel. The continuity of this reinforcement can be secured by welding (following prescribed rules) within the horizontal joint or, preferably, within special keys (Eurocode 8, 2004) provided for this purpose (see Fig. 6-13). An alternative solution is provided by the use of bars protruding from the lower panel and that are embedded in the grout within the corrugated ducts in the upper panels.

In all cases of horizontal compression joints, horizontal reinforcement should be provided as required for adequate shear resistance. In the case of joints that are designed to be partly under compression, it must be recognized that shear can be resisted only along the part of the joint under compression. Vertical reinforcement across the horizontal joint is required to prevent shear sliding when the compression area is not sufficient to prevent sliding. Such vertical reinforcement is also needed to prevent the out-of-plane detachment of the walls (due to possible abnormal loading) that may lead to progressive collapse. The vertical reinforcement used in parts of Europe is in the form of overlapping loops or hairpins that are well anchored in the body of the upper and lower walls. In some codes, a certain amount of minimum vertical reinforcement continuous along the height of the wall is required (see Section 6.8.3c).

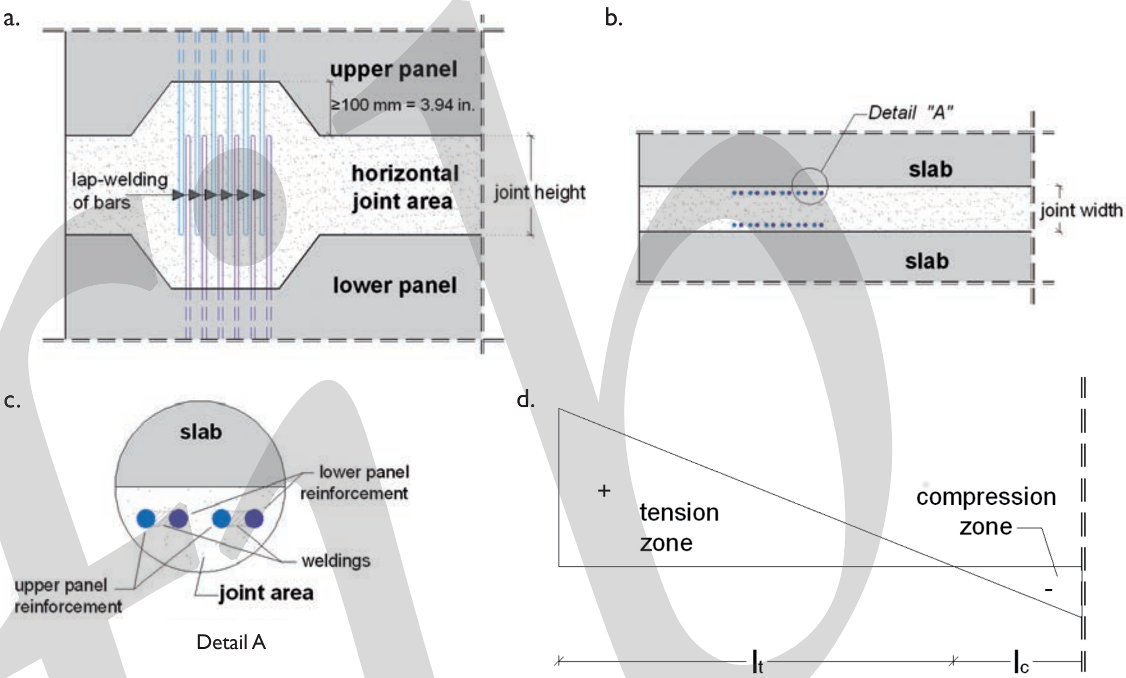


Fig. 6-13 Horizontal joint with special keys
a. Vertical section of joint
b. Horizontal section across joint
c. Detail of reinforcements to be welded at joint
d. Tension and compression zones along joint

6.7 Construction details for large-panel buildings with wet joints (concrete and reinforcement)

Figures 6-14 to 6-17 show construction details of large panels and their vertical wet connections. In these pictures the horizontal joints are assumed to be entirely under compression and are provided with minimum shear keys and transverse reinforcement.

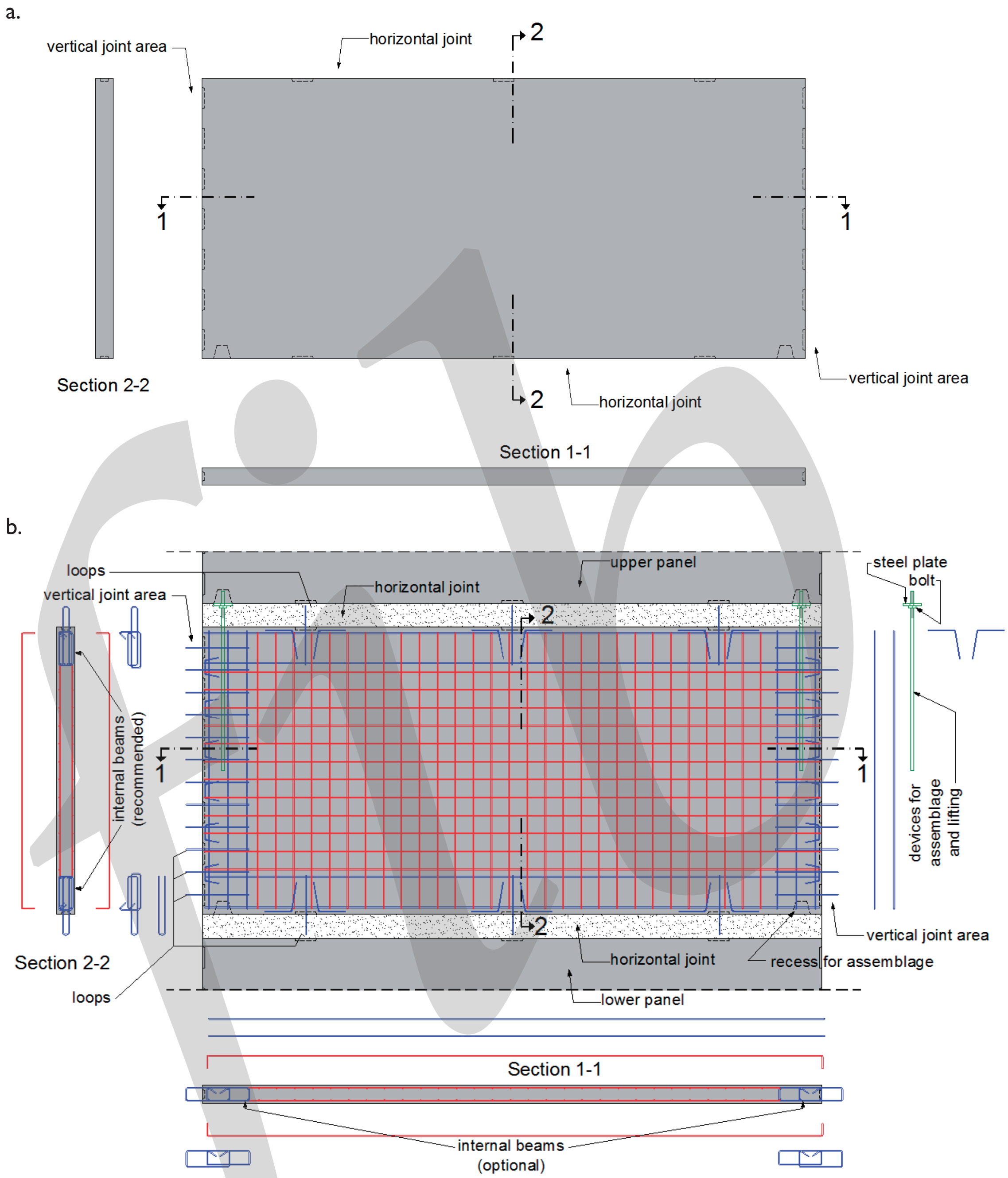


Fig. 6-14 a. Geometry of a solid wall panel where key density along vertical joint is about 50%
b. Main (▬) and additional reinforcement (▬) in wall panel
Illustration b. only shows reinforcement in wall panel between horizontal joints (keys and reinforcements in horizontal joints not shown)

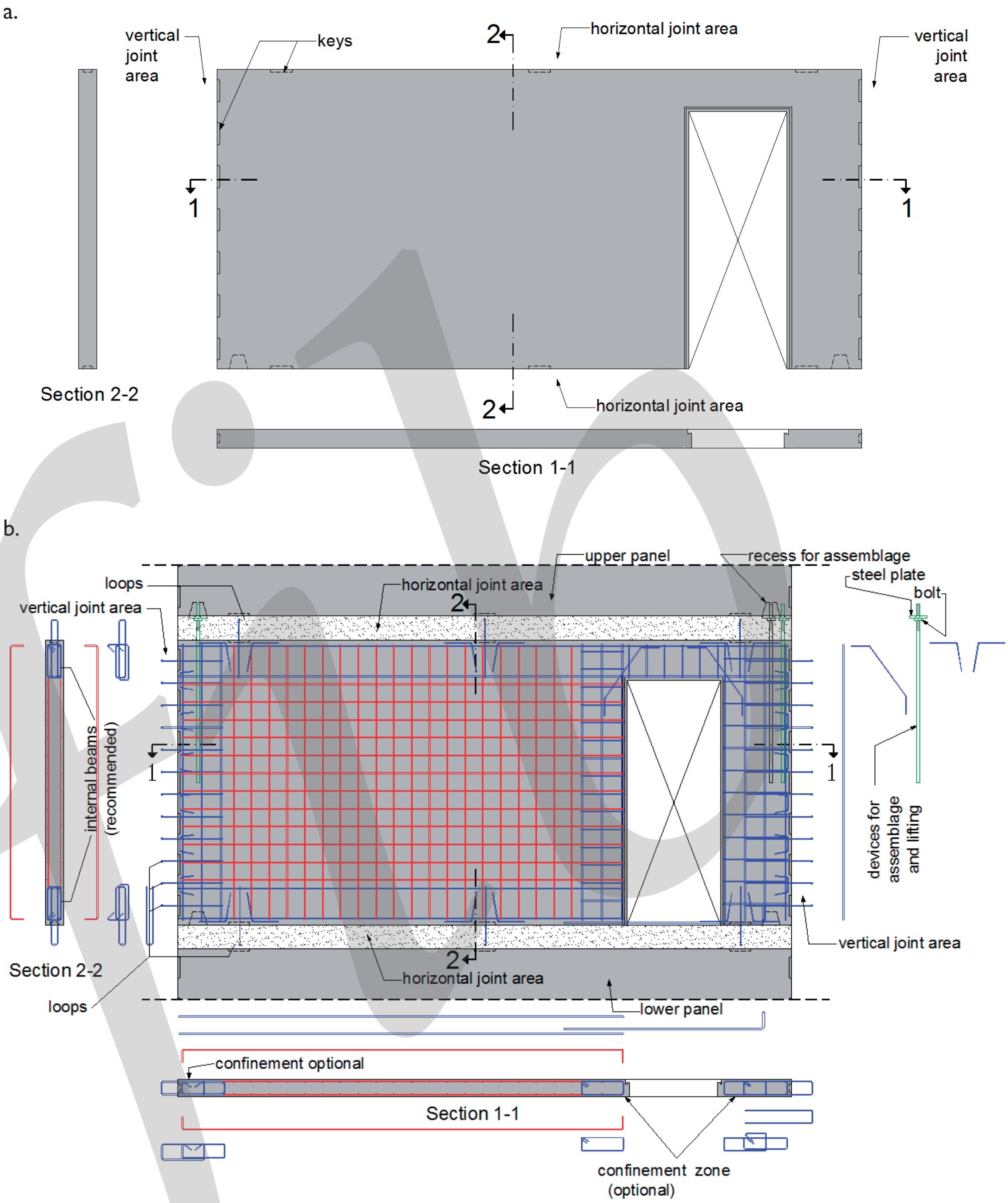


Fig. 6-15 a. Geometry of wall panel with opening where key density along vertical joint is about 50%
b. Main (▬) and additional reinforcement (▬) in wall panel
Illustration b. only shows reinforcement in wall panel between horizontal joints (keys and reinforcements in horizontal joints not shown)

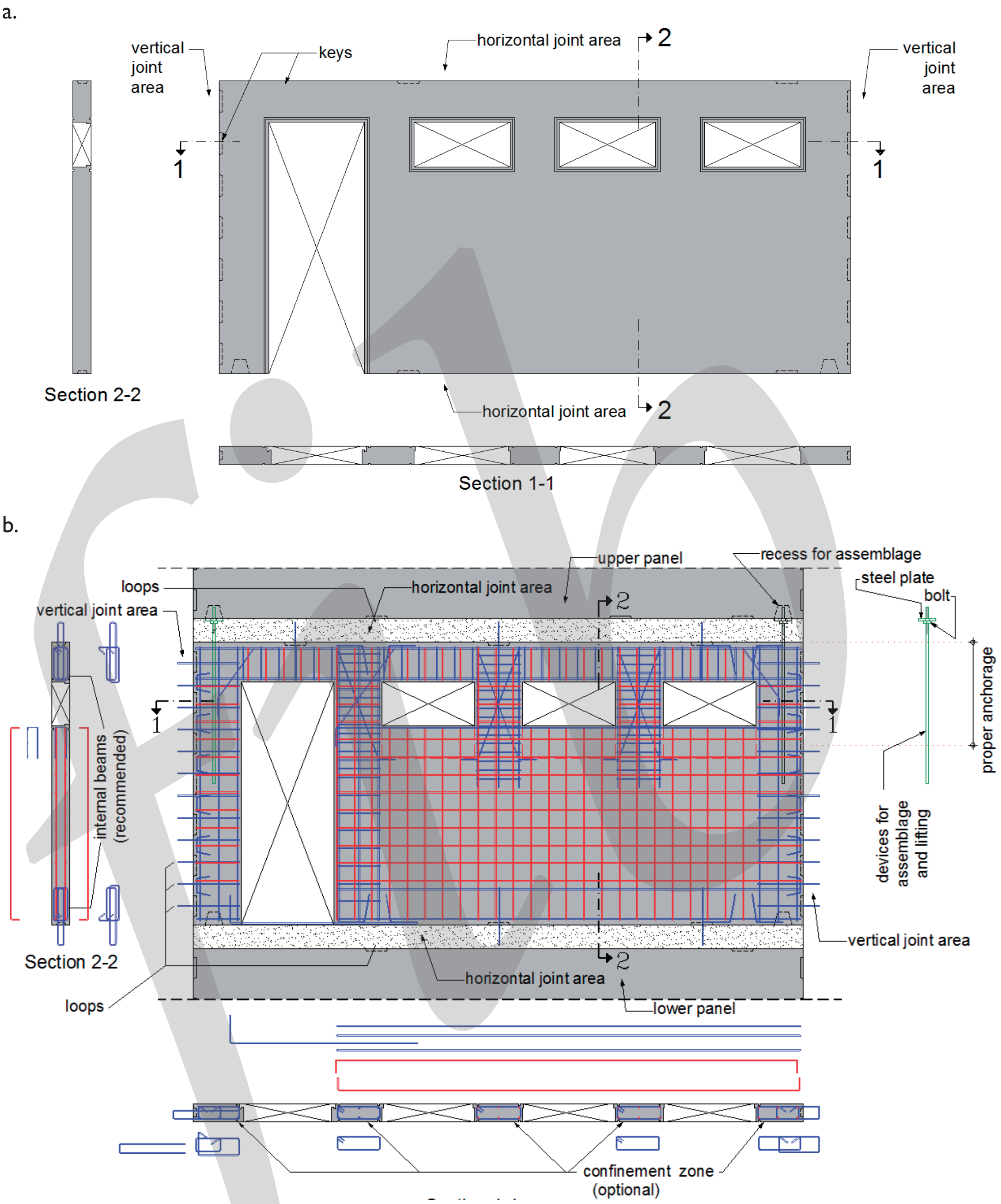


Fig. 6-16 a. Geometry of wall panel with 4 openings where key density along vertical joint is about 50%
b. Main (▬) and additional reinforcement (▬) in wall panel
Illustration b. only shows reinforcement in wall panel between horizontal joints (keys and reinforcements in horizontal joints not shown)

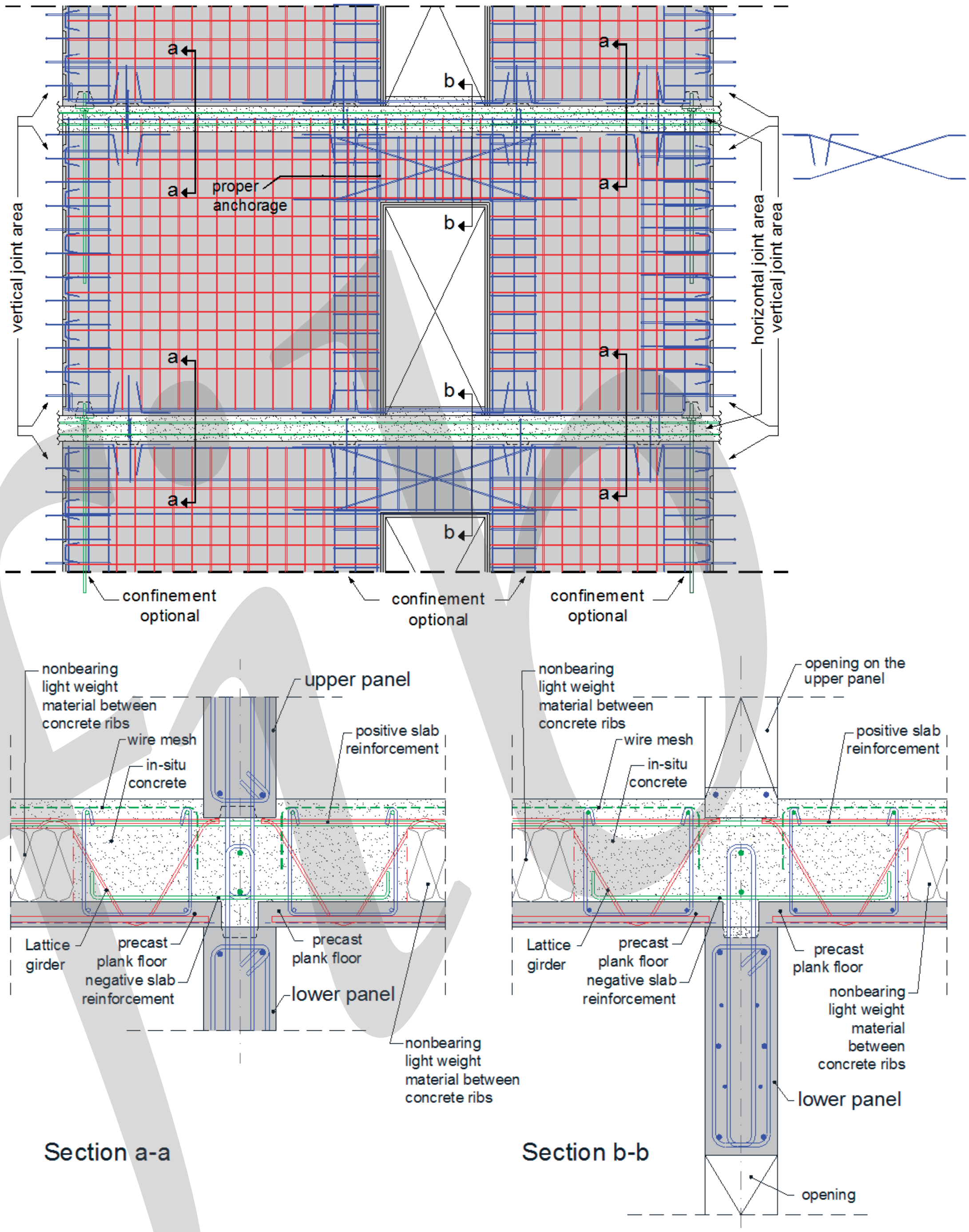


Fig. 6-17 Detailing of wall composed of precast panels, with vertically stacked openings, connected together by means of vertical and horizontal joints
Particular attention should be paid to detailing of coupling beams

6.8 Configuration and structural behaviour of North American platform-framing connections

6.8.1 Horizontal connections

The typical horizontal joint in a load-bearing panel includes a hollow-core floor unit bearing on plastic shim strips on the top edges of a lower wall. The space between the slab and the voided ends of the hollow-core cores are grouted to form a grout column for the transfer of the vertical loads from the above wall (see Fig. 6-18).

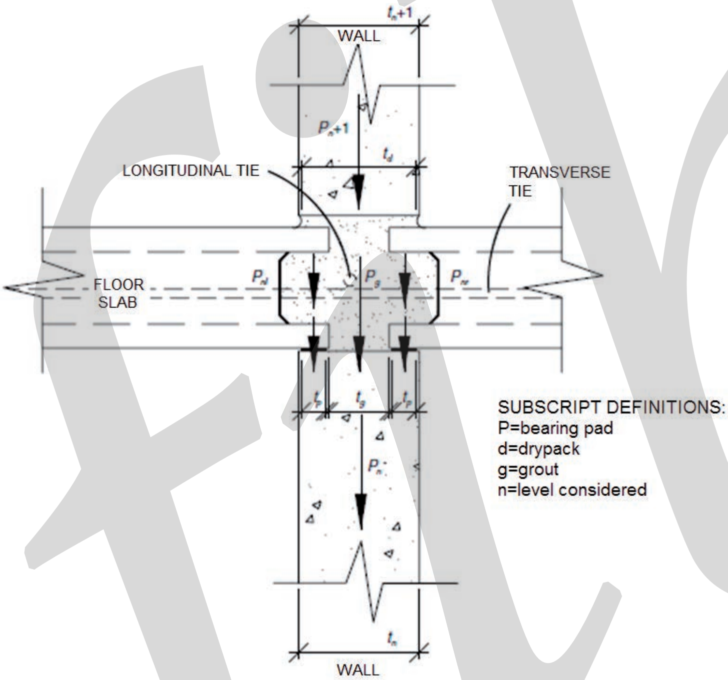


Fig. 6-18
North American platform-framing connection

The horizontal joint achieves compression-force transfer through a complex path formed by the grout, the slab and the bearing pads. Tension transfer across the joint is created with spliced wall bars or post-tensioning, as required by the design forces. This results in a lower reinforcement ratio in the connection region, which in turn tends to result in joint opening instead of flexural-tension cracking in the panels. This has an effect on shear transfer in the joint. Unless shear keys or connectors are used, a uniform diagonal compression field through connections, such as that found in monolithic walls, will not develop in the precast walls through the opening of the connections. Therefore the shear forces need to be transferred through a different force path.

The characteristics and behaviour of horizontal platform connections were the subject of considerable research (Schultz et al., 1979; ACI, 2005). The behaviour is not amenable to accurate analysis due to the multiple load paths through the grout, the slab and the bearing pads. Variations in the geometry and material properties were investigated. Notable results from the aforementioned research include the following observations:

- If the grout strength is less than 80% of the wall concrete strength, the joint strength is governed by grout crushing
- If the grout strength is greater than 80% of the wall concrete strength, the joint strength may be controlled by wall splitting, although reinforcement detailing can reduce the tendency for wall splitting
- With a grout column, any variation of the concrete strength in the slab has no appreciable effect on joint strength
- The grout strength should be at least 3,000 psi (21 MPa), but not more than 125% of the wall concrete strength
- The grout column between the slab should be at least 3 inches (76 mm) wide

The research resulted in detailed design procedures and charts to aid in the design for axial loads and in-plane moments.

Compression develops in the walls as a result of the accumulated gravity loads from the load-bearing walls. The compression component of the flexural couple from the overturning moment also contributes to this compressive load. For structures in areas of high seismicity, ACI 318 (2011) requires the evaluation of structural walls for the possible need of specially confined boundary elements. Section 21.9.6.2 requires boundary elements when the neutral-axis depth c exceeds a limit based on the length of the wall, the height of the wall and the design displacement at the top of the wall. The neutral-axis depth is to be calculated for the factored axial force produced by the additive seismic load combination and the corresponding nominal moment strength at the critical section at the wall base. Alternatively, section 21.9.6.3 requires boundary elements when the extreme fibre compressive stress exceeds 0.2 f'_c, where f'_c is the specified compressive strength of the concrete. When required, these boundary elements must include special transverse reinforcement based on the provisions for special moment-frame columns.

The design of effective connections or the continuity of longitudinal reinforcement across the horizontal joints has also been the subject of significant research. Schriker et al. (1980) describe an interesting extensive multi-phase experimental programme that studied the behaviour of horizontal connections for precast concrete shear wall panels subjected to large reversed cycles of inelastic deformations. The programme evaluated currently used and proposed connections, using 17 specimens. The tests included connections made with mild-steel reinforcement, post-tensioning bars, post-tensioning strand, multiple shear keys and the debonding of the continuity elements.

6.8.2 Vertical-shear wall-to-wall connection

According to Schultz (1979), vertical shear wall-to-wall connections in multi-panelled wall assemblies, as well as in linked wall assemblies with return walls, are built with:

- dry pack provided in the bottom quarter of each vertical joint between adjacent panels, as shown in Figure 6-19; and
- mechanical connections (wet or dry) proportioned to resist the vertical shear according to design.

(Return walls are vertical elements used as a continuation of a load-bearing wall element at a right angle to it.)

The mortar used to pack vertical joints should be a dry portland cement mortar with a compressive strength (measured on 50 mm cubes or 1.97 inches) equal to or greater than the compressive strength of the concrete in the wall panels.

The mechanical connectors should facilitate a horizontal movement (opening) in the vertical joints according to design and should accommodate this movement while maintaining the strength to resist the vertical shear loads.

6.8.3 Structural integrity

According to Section 16 of ACI 318 (2011), horizontal and vertical ties must be incorporated into a precast-bearing-wall structure of three or more storeys in height for reasons of structural integrity. A typical arrangement of tensile ties is shown in Figure 6-20. Minimum horizontal and vertical ties also serve as essential elements of an indirect approach

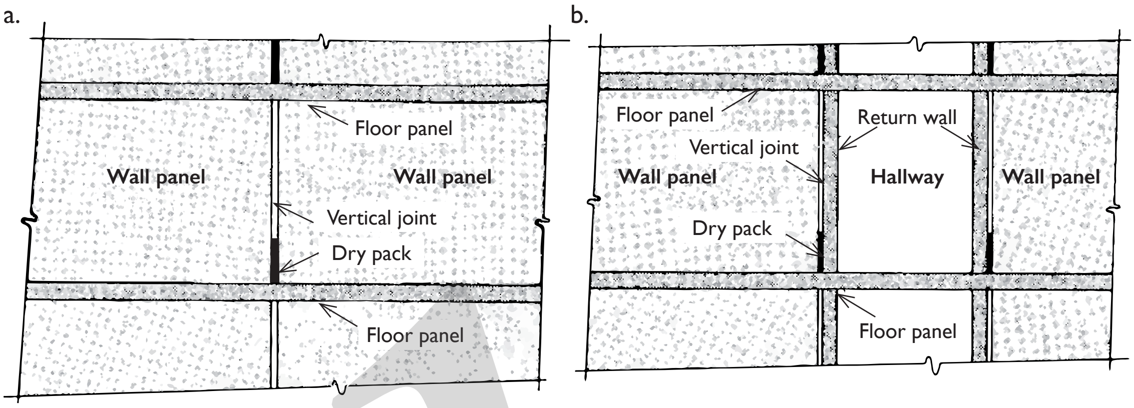


Fig. 6-19 a. Multi-panelled wall assembly
b. Linked-wall assembly (returned walls are vertical elements used as continuation of load-bearing wall element at right angle to it)

to providing minimum detailing in a structure to prevent progressive collapse, that is, to keep a local event (due to abnormal loads) from becoming an overall collapse.

In Cleland and Ghosh (2012) a, b, c and d below are reported for the minimum horizontal and vertical tie requirements according to ACI 318 (2011):

a. Transverse ties to develop cantilever and beam action in wall panels

   The provision for transverse ties is prescribed in ACI 318 Section 16.5.2.1: 'Longitudinal and transverse ties shall be provided in floor and roof systems to provide a nominal strength of 1500 lb (680 kg) per foot (30.5 cm) of width or length. Ties shall be provided over interior wall supports and between members and exterior walls. Ties shall be positioned in or within 2 ft (0.61 m) of the plane of the floor or roof system.' Additionally, Section 16.5.2.3 requires that 'transverse ties perpendicular to floor or roof slab spans shall be spaced not greater than the bearing wall spacing.'

   These provisions essentially locate the transverse ties in the horizontal joints between wall panels at the level of the floor. The intent of these transverse ties is to create cantilever action in the wall stack in the event of the severe damage or loss of a load-bearing wall. This cantilever action will transfer vertical shear from the walls above the damage to adjacent walls in the line of the damaged wall. To accomplish this load transfer there must be tensile continuity in the horizontal connections. The detailing of the floor and the inclusion of transverse walls aid in maintaining the load path. The shear strength in the horizontal joint must be sufficient to prevent horizontal panels from sliding at the joint. The tie force has been determined empirically from the load tests conducted as part of research conducted at the Portland Cement Association, so that this cantilever action can be mobilized by either a stack of cantilevered walls or by individual floor cantilevers. This cantilever action is the primary mechanism that creates a load path around a damaged wall.

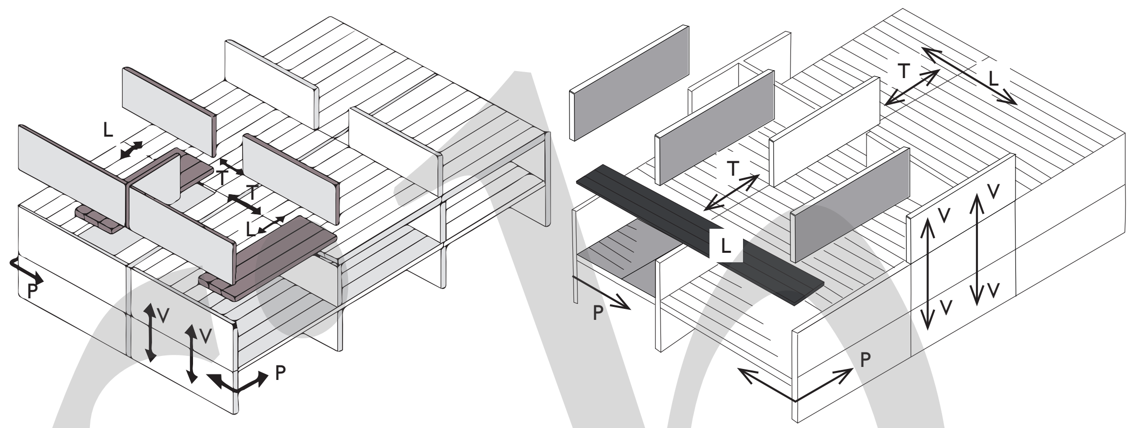


Fig. 6-20 Location of tensile ties in large panel structures (based on ACI 318, 2011)
a. Cross-wall structure b. Spine-wall structure
T = transversal L = longitudinal V = vertical P = peripheral

b. Longitudinal ties for membrane action in floor

ACI 318 Section 16.5.2.1 also provides for longitudinal ties, which are ties that run in the direction of the floor component span. Section 16.5.2.2 adds that 'Longitudinal ties parallel to floor or roof slab spans shall be spaced no more than 10 ft (3.05 m) on centers. Provisions shall be made to transfer forces around openings.' These longitudinal ties are proportioned to act as a catenary between walls on either side of the ineffective wall only to the extent of supporting loads from the local debris. The empirical forces used are not sufficient for the catenary to carry the wall above the floor and floor loads to those adjacent walls. The research found that provision for that level of force was not feasible and not required.

c. Vertical ties to develop suspension action

Vertical ties are prescribed by ACI 318 Section 16.5.2.5: 'Vertical tension ties shall be provided in all walls and shall be continuous over the height of the building. They shall provide a nominal tensile strength not less than 3000 lb (1361 kg) per horizontal foot (30.5 cm) of wall. Not less than two ties shall be provided for each precast panel.' Section 16.5.1.3.b also prescribes minimum vertical tension ties: 'Precast panels shall have a minimum of two ties per panel, with a nominal tensile strength not less than 10.000 lb (45 kN) per tie.' These ties provide for the vertical suspension of ineffective walls to limit debris load. Since the alternative support mechanism develops through the cantilever action of walls above, these ties need only hold the damaged wall and floor that it supported to limit the loads on lower walls and floors. They provide resistance against a wall kicking out, eliminating the need to remove the wall. They also provide clamping action for shear friction capacity in the joint. Walls on the perimeter are more vulnerable and should be detailed with more ties. These vertical ties may also be needed for overturning strength in resisting system loads.

d. Peripheral ties to develop diaphragm action

ACI 318 Section 16.5.2.4 adds provisions for perimeter ties: 'Ties around the perimeter of each floor and roof, within 4 ft (1.22 m) of the edge, shall provide a nominal strength in tension of not less than 16,000 lb (7260 kg).' These perimeter ties provide for a minimum strength in the floor diaphragm.

Through the detailing of the minimum ties, a level of acceptable integrity, established from the research, is achieved. 'This provision of General Structural Integrity eliminates the need to design for any particular abnormal load.' In earthquake design, the anticipated damage is from the post-elastic behaviour of the structure. Elements in the system must have integrity in the event of such damage, including the potential for the partial loss of load-bearing function.

6.8.4 Further information about ties, based on Schultz, 1979

Figures 6-21 to 6-23 show the location and extent of the following types of ties for both cross-wall and spine-wall systems:

- transverse
- longitudinal
- peripheral

Figures 6-24 and 6-25 respectively show an interior longitudinal tie detailing and a terminal longitudinal tie detailing is shown for both cross wall and spine wall systems.

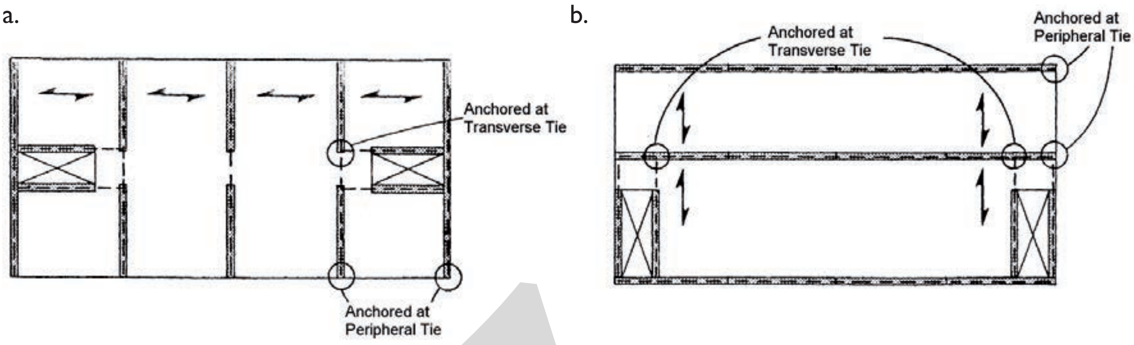


Fig. 6-21 Location and extent of transverse ties
a. Cross-wall structure b. Spine-wall structure

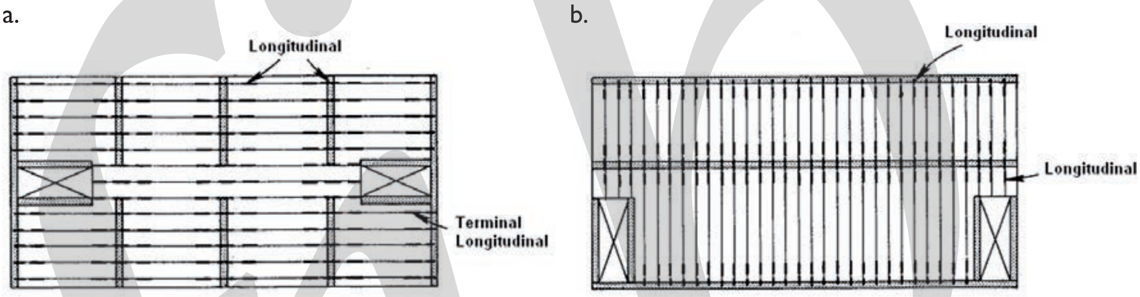


Fig. 6-22 Location and extent of longitudinal ties
a. Cross-wall structure b. Spine-wall structure

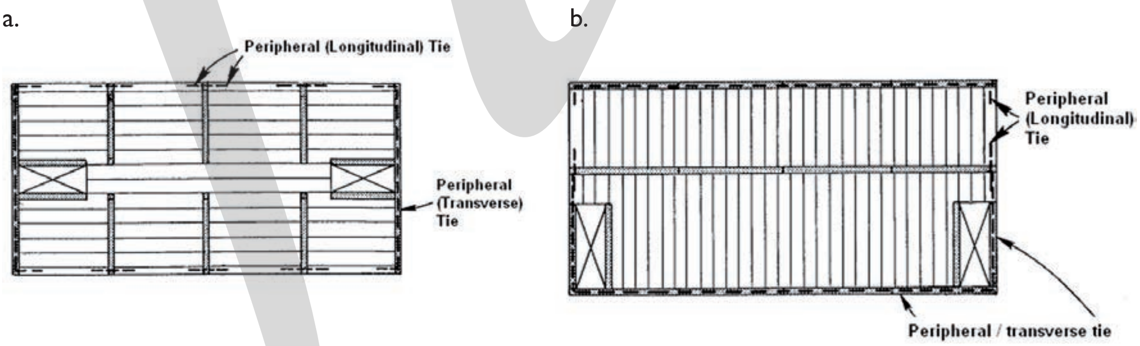


Fig. 6-23 Location and extent of peripheral ties
a. Cross-wall structure b. Spine-wall structure

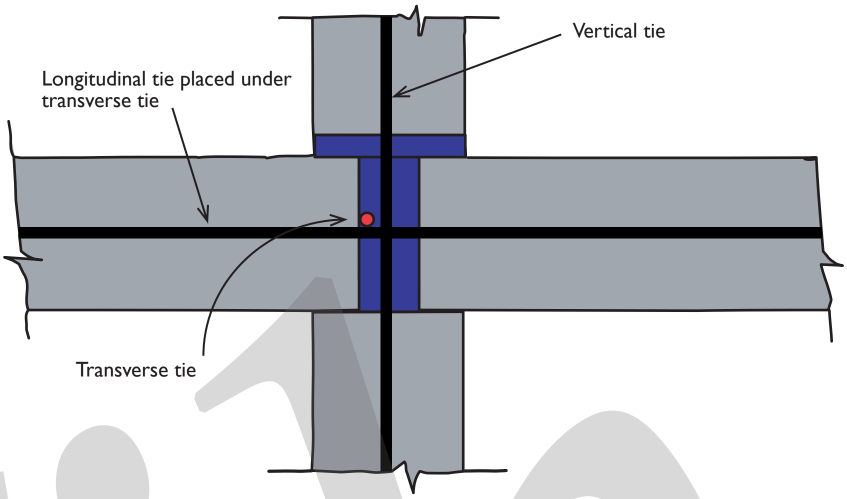


Fig. 6-24 Location and extent of peripheral ties
a. Cross-wall structure b. Spine-wall structure

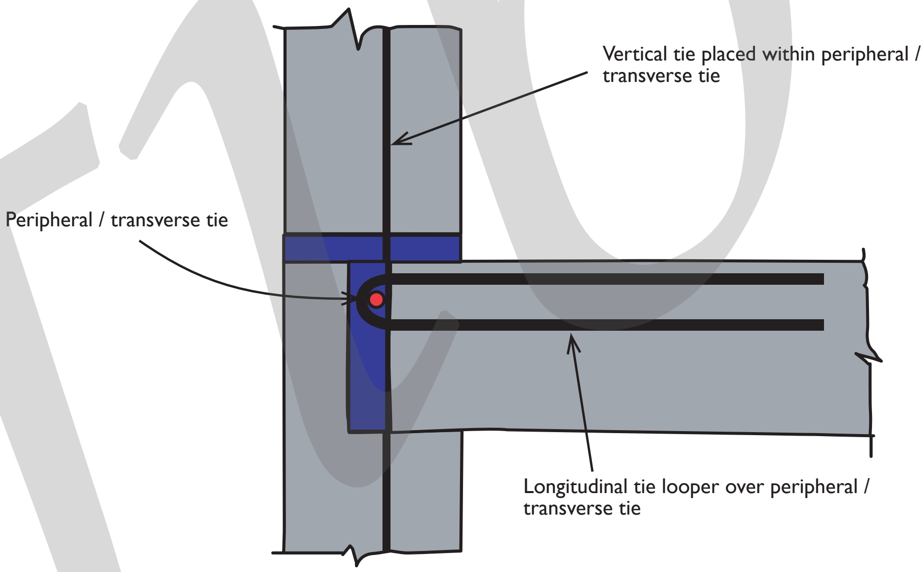


Fig. 6-25 Location and extent of peripheral ties
a. Cross-wall structure b. Spine-wall structure

7 Wall-frame systems (dual systems)

7.1 General

Dual systems consist of a combination of shear walls and moment frames. A dual system is commonly used when the moment-resisting frames alone do not provide desirable lateral stiffness.

However, probable lack of deformation compatibility in both elastic and inelastic modes between walls and frames should be visualized during the design; and this because walls and frames do not deform equally under normal or severe lateral loads. On the other hand, for the design of a lateral-force-resisting system for a precast/prestressed concrete building (consisting of precast walls and precast frames) it is important that the characteristics of the connections between walls and frames should be such as to accommodate the different behaviour of the two systems (walls and frames).

7.2 Shear walls and moment frames in dual systems

Precast shear walls that are used in dual systems follow the same rules as precast shear walls in terms of code.

For walls, simplified analysis methods that rely on the relative shear and flexural stiffness of the walls may be applied (as specified, for example, in PCI, 2010).

According to ACI 550.1R-09 (2009), the analysis should consider the effects of shear deformations for walls with aspect ratios lower that 3-to-1. The effects of the eccentricity of the centre of mass differing from the centre of stiffness of the wall system should be taken into consideration along with the code requirement to include 5% eccentricity for accidental torsion.

Typically, the desired primary ductile behaviour of precast shear walls emulating cast-in-situ detailing is flexural yielding at the wall and wall joints (see Fig. 7-1).

Because a small rotation in a wall will create a large amount of damage through bar elongation, the ductility at the base is important. Ductility can be increased significantly by debonding bars into and out of the foundation, so that they can deform inelastically over a longer length (Soudki, Rizkall and LeBlanc, 1995), thus resulting in greater nonlinear elongation and rotation ductility (see Fig. 7-2).

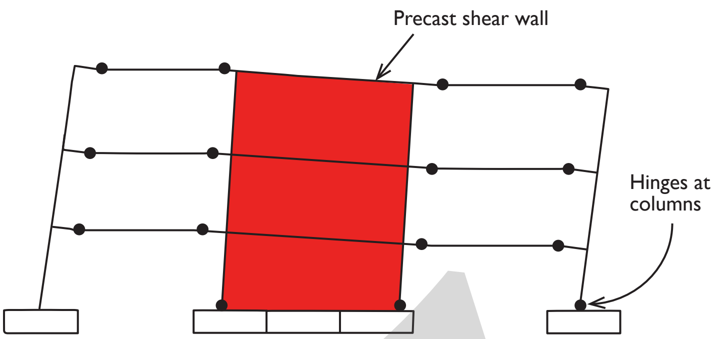


Fig. 7-1
Dual building with rotation of shear wall at each floor (based on ACI 550.1R-09)

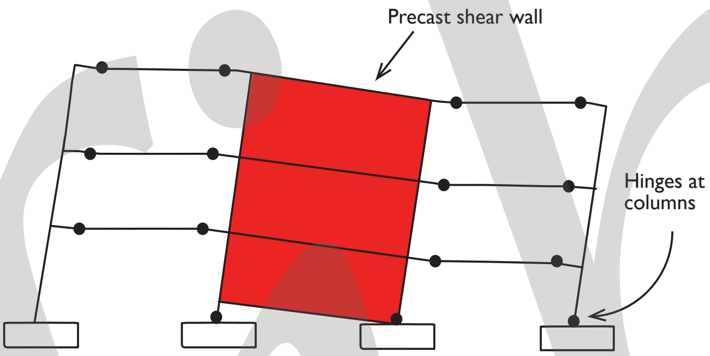


Fig. 7-2
Dual building with ductile yielding of partially debonded bars between foundation and shear wall boundary elements (based on ACI 550.1R-09)

Reinforcing steel specified for special walls should be ductile and have controlled strength properties.

In the **United States** codes now use a seismic design category and not a seismic performance category. The following distinctions are common for structural walls and moment frames:

- **Ordinary shear walls** are walls designed in accordance with ACI 318-11, Chapters 1 through 18. This includes Chapter 16 on precast concrete with provisions for structural integrity. Ordinary precast concrete shear walls are not permitted in seismic design category (SDC) C.

- **Intermediate precast structural walls** are walls complying with all applicable requirements of ACI 318-11, that is, Chapters 1 through 18 in addition to 21.4. Intermediate structural walls are required for (SDC) C and are permitted to heights of up to 12.2 metres (40 feet) [or 13.72 metres (45 feet) in framing only one level] in (SDC) D, E and F.

- **Special shear walls** are walls which meet the requirements for ductile detailing included in ACI 318-11, Section 21.9 (special structural walls and coupling beams) and Section 21.10 (special structural walls constructed using precast concrete). According to ACI 318-11, such walls are used in buildings in seismic performance categories D, E and F.

- **Ordinary moment frames** are precast concrete frames that comply with the requirements of ACI 318-11 Chapters 1 through 18, and, in the case of ordinary moment frames assigned to seismic design category B, also comply with 21.2.

- **Intermediate moment frames** are cast-in-situ frames that comply with the requirements of ACI 318-11 Clause 21.3, in addition to requirements for ordinary moment frames.

- **Special moment frames** are precast frames that comply with the requirements of ACI 318-11 Clause 21.1.3 through 21.1.7 and 21.5 through 21.8. In addition, the requirements for ordinary moment frames should be satisfied.

According to ASCE/SEI 7-05 (Section 12.2.5.1), for a dual system, the moment frames should be capable of resisting at least 25% of the design seismic forces. The total seismic force resistance is to be provided by the combination of moment frames and shear walls or braced frames in proportion to their rigidities.

In **Europe**, as specified in Eurocode 8 (1998), the following distinctions are common:

- **Wall systems** are structural systems in which both vertical and lateral loads are mainly resisted by either coupled or uncoupled vertical structural walls whose shear resistance at the building base exceeds 65% of the total shear resistance of the whole structural system.

- **Frame systems** are structural systems in which both the vertical and lateral loads are mainly resisted by spatial frames whose shear resistance at the building base exceeds 65% of the total shear resistance of the whole structural system.

- **Dual systems** are structural systems in which support for the vertical loads is mainly provided by a spatial frame and resistance to lateral loads is contributed to in part by the frame system and in part by coupled or uncoupled structural walls.

- **Frame-equivalent dual systems** are dual systems in which the shear resistance of the frame system at the building base is greater than 50% of the total shear resistance of the whole structural system.

- **Wall-equivalent dual systems** are dual systems in which the shear resistance of the walls at the building base is higher that 50% of the total seismic resistance of the whole structural system.

From the point of view of displacement and ductility compatibility in buildings with dual systems, the overall response is obtained as the superposition of the response of the walls and frames. In this case it is reasonable to expect that the lateral stiffness of the walls lead to good lateral displacement control during earthquakes, eliminating the possibility of column sideway mechanism in the frame, particularly in the lower stories of a building. Frames on the other hand, help to control inter-storey drifts in the upper storey of the building.

Figure 7-3 is a simple model that depicts the idealized lateral force, the displacement response of the walls and the frames (which constitute the dual system), and the overall displacement response of the dual system.

This simple model shows that in accordance with the overall response of the dual system, the frames remain elastic when the walls reach their ultimate displacement. This means that in spite of the dual system being fully ductile, the frames do not necessarily need to be designed and detailed for full ductility.

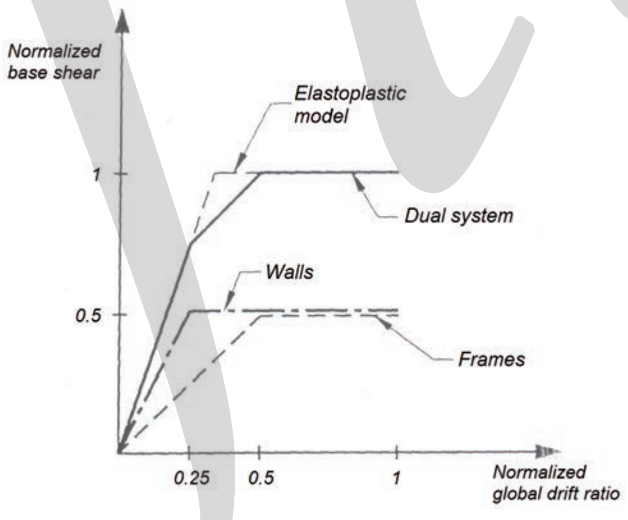


Fig. 7-3
Normalized lateral force-lateral displacement response in dual system and individual sub-systems (*fib* Bulletin 27, 2003)

This concept has been experimentally evaluated (Rodriguez et al., 2001) on a two-storey precast building incorporating a dual system of lateral-load resistance. An evaluation of specimen damage at the end of testing revealed that most of the inelastic response of the system was concentrated at the wall's bases, with only a small inelastic response of the precast frame elements and their connections.

Force-response factors in dual-systems are also related to the displacement ductility capacity of the system. Generally, the displacement ductility demand for a dual system would be less than the individual value of walls and larger than the individual values corresponding to frames

The evaluation of diaphragm forces is an additional issue that needs to be taken into consideration in the seismic design of dual systems. This issue is discussed in detail in Chapter 6.10 of *fib* Bulletin 27, 2003.

7.3 Typical connections in structural wall systems

Figures 7-4 to 7-9 show typical details of wall-to-wall and wall-to-floor connections. The wall-to-wall connections are achieved with a combination of grout and spliced vertical reinforcing bars. The grout provides the continuity of the compressive forces across the joints while the bars provide the continuity for the tensile forces. Both the grout under compression and the vertical bars (mostly those which exist in the compression zone along the joints) provide resistance under the shear sliding of the walls to be connected.

If a rapid field erection is required, high-strength joints can be achieved by means of high-strength non shrink grout and vertical reinforcement that are spliced and grouted with specially designed and code-approved sleeve connectors, as shown in Figure 7-4.

Figures 7-5 and 7-6 show typical wall-to-wall and wall-to-floor connections. The floor system is untopped.

Figure 7-5 illustrates how a wall-to-wall connection is obtained with lapped splices in a large conduit. Overlapping bars in a grout-filled conduit are extended full-height through the structural element. Welded and lapped splices must be located more than 2*h* (where *h* is the floor thickness) from the face of the wall. Mechanical splices must be type 2, according to Clause 21.1.6 of ACI 318-11 (2011), if less than 2*h* from the face of the wall.

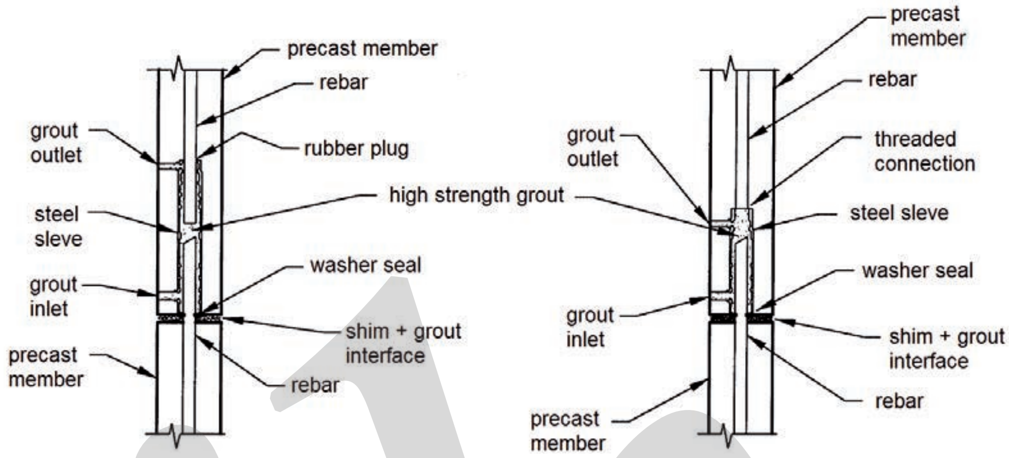


Fig. 7-4 Typical types of mechanical splices using high-strength nonshrinking grout (Authorized reprint from ACI 550.1R-09, 2009)

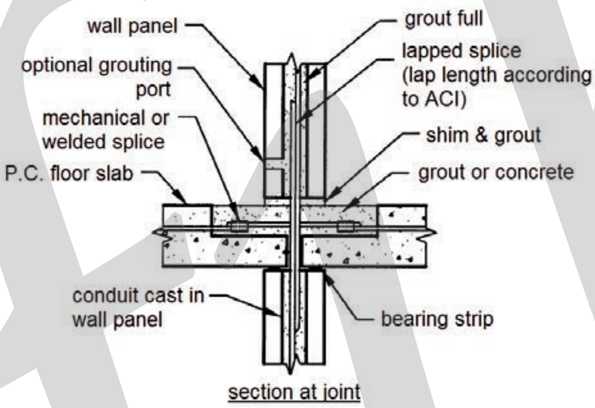


Fig. 7-5

Lapped splices in large conduit (Authorized reprint from ACI 550.1R-09, 2009)

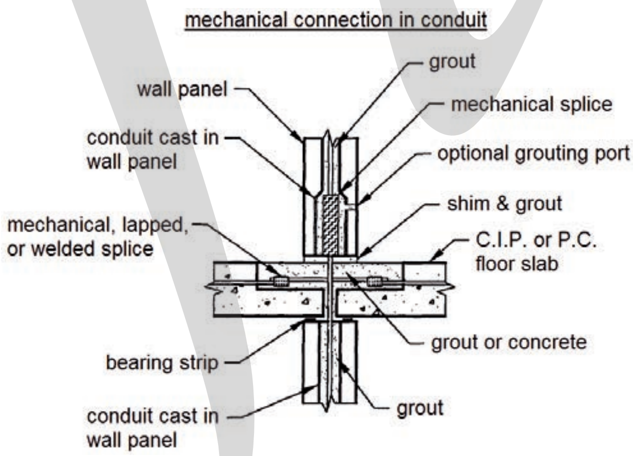


Fig. 7-6

Vertical bars in conduit are spliced and system is grouted (Authorized reprint from ACI 550.1R-09, 2009)

Fig. 7-6 shows a wall-to-wall connection in which vertical bars in a normal conduit are spliced and the system is grouted. The construction procedure is as follows:

- The wall panel is erected
- Loose vertical bars in the panel being erected are spliced to protruding bars from below
- The panel is lowered to the correct elevation
- The conduit is grouted by gravity flow from the top or through an optional grouting port at the bottom of the panel

Welded and lapped splices must be located more than 2*h* (where h is the floor thickness) from the face of wall. Mechanical splices must be type 2 if less than 2*h* from the face of the wall, in accordance with ACI 318-11 (2011).

Figure 7-7, shows two types of wall-to-wall and wall-to-topped-floor connections.

Figure 7-8 illustrates the technique of steel arrangement in a wall-to-floor connection, by which slab bottom bars extended diagonally across the joint provide structural reinforcement continuity of the diaphragm across the wall and partially cover the demand of the shear reinforcement.

Figure 7-9 shows the end detail of a monolithic connection between a precast-concrete floor element and a precast-concrete wall.

Figure 7-10 shows variations of splices and cast-in-situ closure placements in creating vertical joints between precast-concrete elements. Reinforcing steel may be lapped,

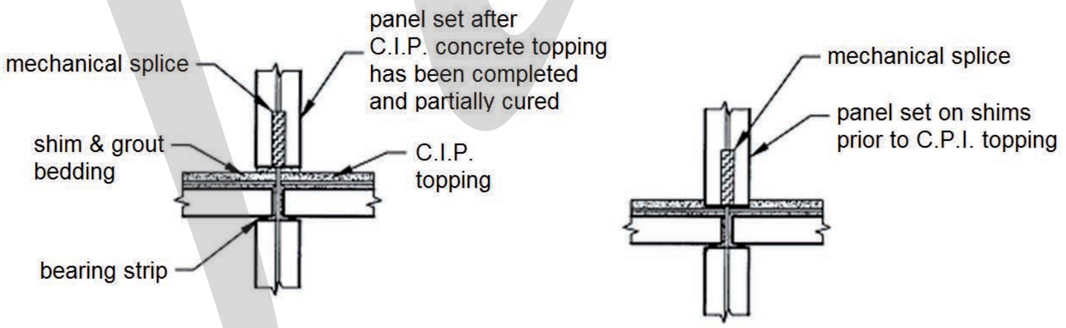


Fig. 7-7 Mechanical splice for connection of two configurations of precast walls and floors. Welded and lapped splices must be located more than 2*h* (where *h* is the floor thickness) from the face of the wall. Mechanical splices must be Type 2 if less than 2*h* according to ACI 318-11 (2011) from face of wall (Authorized reprint from ACI 550.1R-09, 2009)

welded or spliced mechanically. The variation in Figure 7-10a may be used when there is no architectural concern for appearance while the one in 7-10b is mainly used when an architectural face is exposed.

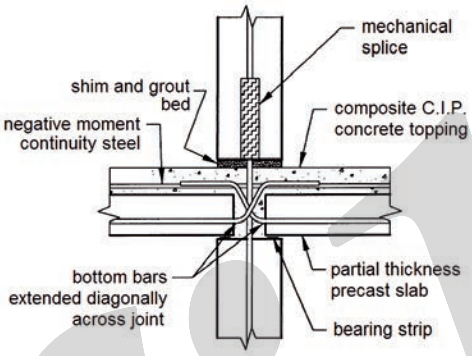


Fig. 7-8

Floor slab-to-wall detail where diagonal dowels cross wall joint into opposite floor (Authorized reprint from ACI 550.1R-09, 2009)

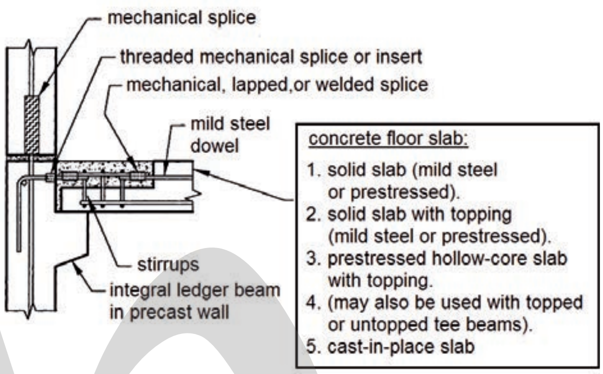


Fig. 7-9

End detail of a monolithic connection between precast-concrete floor element and precast-concrete wall (Authorized reprint from ACI 550.1R-09, 2009)

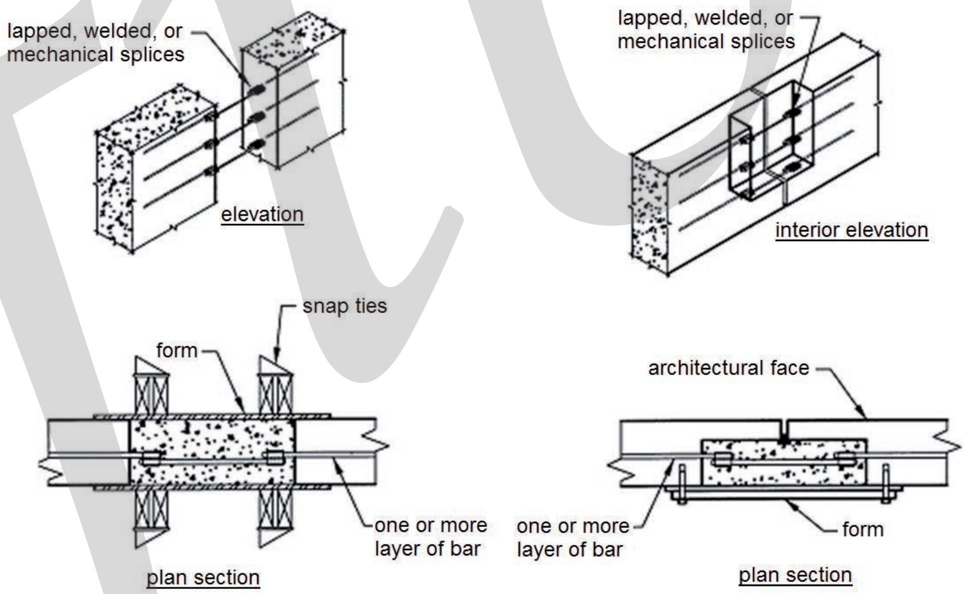


Fig. 7-10 Variations of splices and cast-in-situ closure placements to create vertical joints between precast concrete elements
(Authorized reprint from ACI 550.1R-09, 2009)

8 Floor-framing systems

8.1 General

Floor systems and their constituent precast units are designed in accordance with codes of practice and available literature for all combinations and types of loads (vertical, horizontal or induced displacements) and spans.

Floor systems play a key role in the lateral resistance of precast structures by providing diaphragm action, which serves to:

- transfer lateral loads at each level to the lateral force-resisting system (walls, frames, dual systems); and,
- combine individual lateral force-resisting elements into a single lateral force-resisting system.

Precast floor slabs and roofs are often designed as simply-supported elements because of the lower cost of providing positive capacity in pretensioned floor units.

Figure 8-1 depicts the most common types of floor and roof systems consisting of various types of precast elements.

Generally speaking, the design and construction of floor systems in precast constructions should meet the basic requirements of serviceability and strength typical of any construction system. Serviceability refers primarily to limitations on flexural deformation. Strength requires verification of the following:

- The diaphragm action, which should be effective and in accordance with the design assumptions

- The adequacy of end supports of the slab units, which must accommodate the earthquake-induced displacements (compatibilities), including beam elongations in frame systems and uplifting in wall lateral-resisting systems, or a combination of the above

- The slab action (flexural and shear strength), which should be maintained under gravity and seismic (horizontal and vertical) loads

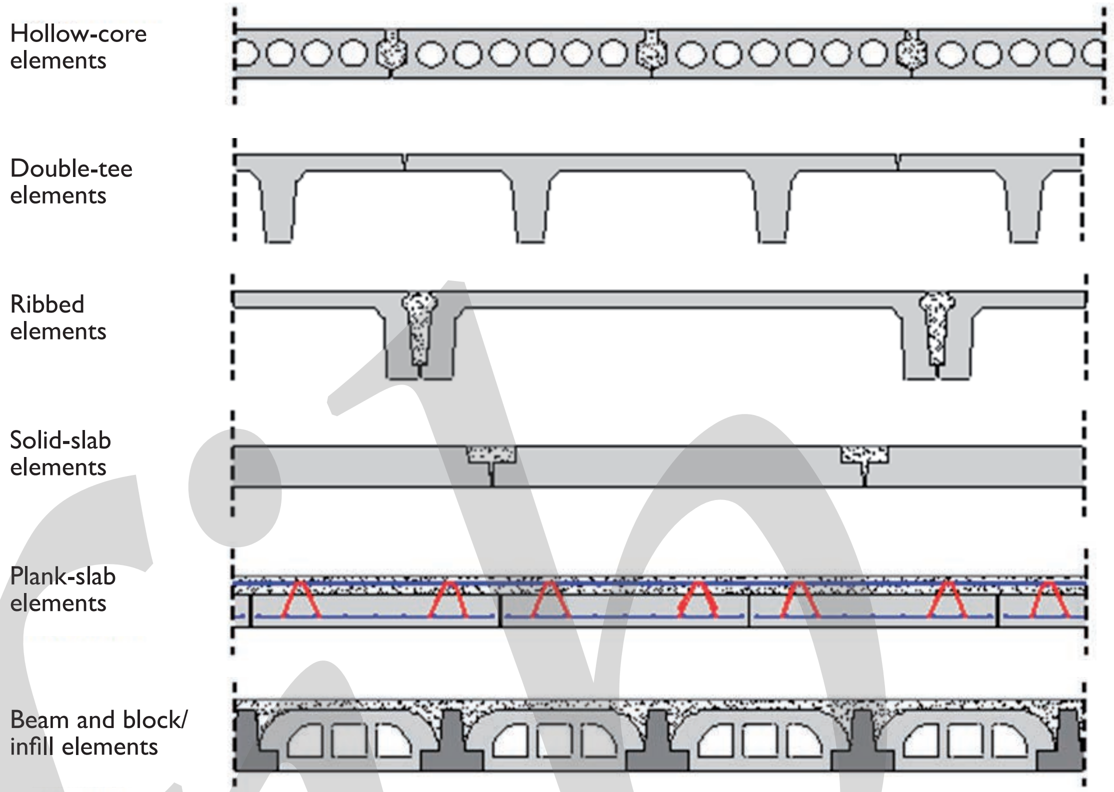


Fig. 8-1 Most common types of precast floor and roof systems

8.2 Aspects of diaphragm behaviour in precast-floor systems

The behaviour of precast floor systems under vertical and horizontal loads depends on the construction method as well as the detailing, the type of the precast units used, the geometry of the floor system, its rigidity, and so forth.

Precast concrete diaphragm construction can be divided into two main categories:

- Diaphragms with topping
- Diaphragms without topping

In all cases, the horizontal shear transfer through diaphragms to the lateral-force-resisting system should be ensured by well-detailed and dimensioned reinforcement.

8.2.1 Diaphragms with topping

Since the floors are built using precast elements, whether prestressed or not, the presence of numerous joints in the floor plan is inevitable. Hence, it is essential to assure continuity in the transfer of internal forces over the entire floor under in-plane horizontal actions. To this end, when it is difficult or expensive to rely on appropriately detailed and constructed joints between precast elements, it is preferable to rely entirely on a cast-in-situ reinforced-concrete topping, provided that the concrete topping is adequately bonded to the precast units.

Figures 8-2 and 8-3 show details of the most common types of solutions for precast floors with topping.

The sectional properties of the composite section can be used to determine the structural performance of the floors. Additionally, topping also improves vibration and the thermal and acoustic performance of the floor. In most precast codes, the absolute minimum thickness of the topping is about 40-50 millimetres (1.8-1.97 inches). The requirement of cover to reinforcement and splices may necessitate a slightly larger topping thickness. Similarly the thickness of the concrete topping should be designed to accord with the diaphragm actions caused by seismic loading and can typically be, in a region of moderate-high-seismicity, in the order of 75-90 millimetres (2.95-3.54 inches). In the aftermath of the Canterbury Earthquake sequence, a minimum thickness of 75 millimetres (2.95 inches) was recommended as a good practice (SESOC, 2011).

The minimum thickness value applies in the midspan region of pretensioned elements. In view of the precamber, the topping thickness towards the ends of the spans may be significantly higher. In diaphragms with topping, the full transfer of longitudinal shear between precast units must be achieved by interface bond stresses and, if required, properly anchored dowels. The contact surfaces should be clean, free of laitance, and adequately roughened in order to develop proper bond. Welded wire fabric provided within the topping will contribute to crack control and also to shear resistance through shear friction across the joints of the precast floor units.

Following the numerous failures of welded wire meshes as diaphragm topping reinforcement during the Christchurch earthquake (22 February 2011) as a result of the high concentration of inelastic demand associated with beam-elongation effects (Kam et al., 2011; CERC, 2011; SESOC, 2011) (see Fig. 8-4) the use of ductile meshes or deformed bars has been strongly recommended for precast floor systems (CERC, 2011; SESOC, 2011) instead of hard-drawn or any other nonductile mesh, in general.

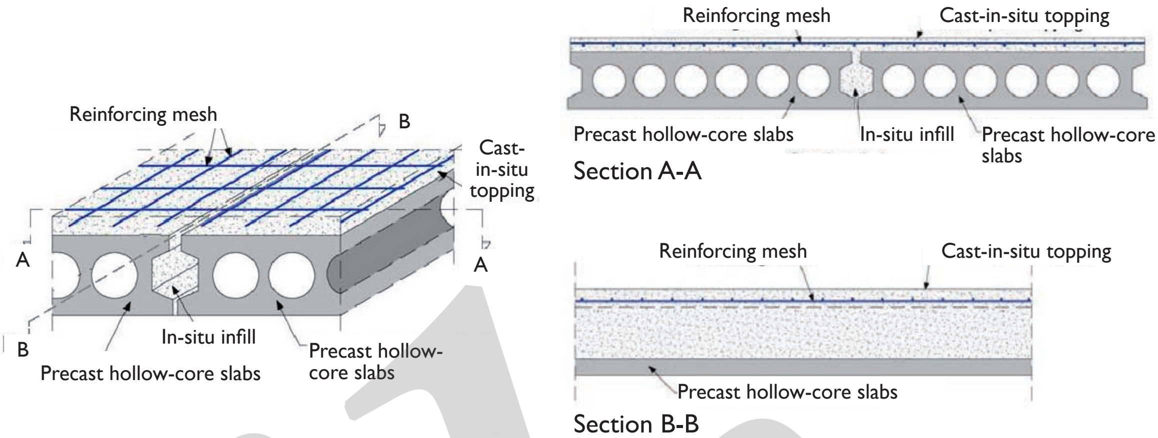


Fig. 8-2 Typical details for topped hollow-core slab to ensure full transfer of longitudinal shear across interface of precast units

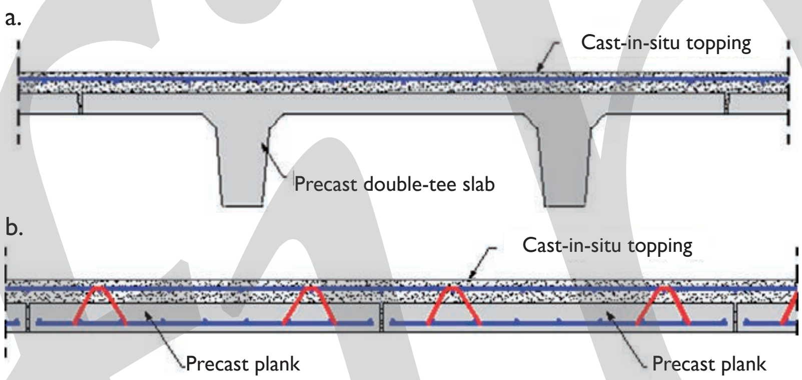


Fig. 8-3
a. Topped double-tee floor
b. Topped plank floor

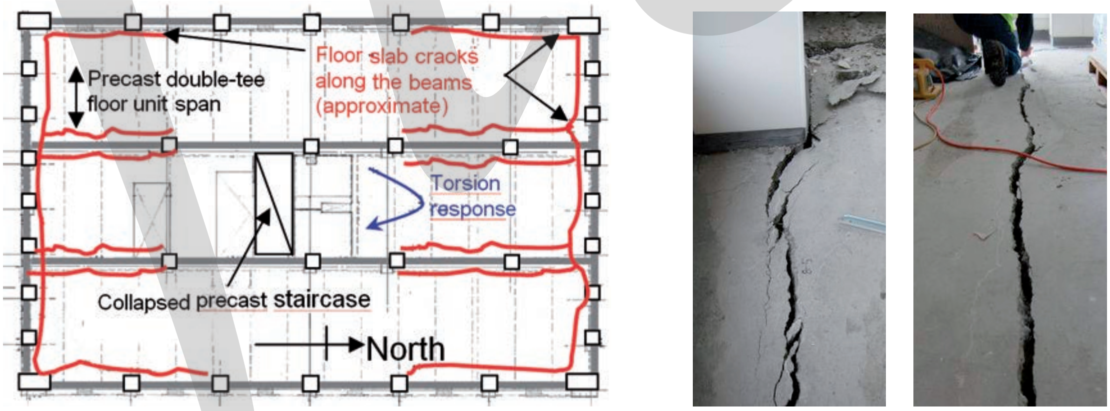


Fig. 8-4 Extensive damage to floor diaphragm and loss of floor support in ductile RC multi-storey frame building caused by beam-elongation effect during Christchurch earthquake of 22 February 2011 (Pampanin et al., 2012; Kam et al., 2011)

8.2.2 Floors without topping

Floors without topping have an advantage over floors with topping in that they have lower weight and depth. Generally, floors without topping should not be used in areas of high seismicity and should be used with caution in areas of moderate seismicity. In cases where topping is not used in a hollow-core floor, hollow-core units should best have mechanical connectors to provide the required shear transfer mechanisms and/or special profiles along their edges, as shown in Figure 8-5. It has been shown experimentally (Menegotto et al., 2005) that this special feature of joints with waved shear keys between hollow-core units mobilizes shear friction. However, untopped solutions of such types necessitate a higher level of quality control in grouting the joints and executing the details than what it is required in topped solutions. In areas of high seismicity, floors without topping may be used provided that they are connected together to form a full diaphragm action by means of connection systems whose efficiency is verified through experimental testing. There are various types of connections depending on the required strength and strain capacity to accommodate expected joint movement. A possible flange-weld connection between double-tee units is shown in Figure 8-6 as an example. In such a connection some ductility must be developed to be able to sustain the shear and tension forces along with the deterioration caused by cyclic loads. Therefore, the connection embedments should be oversized in relation to the connecting plate, which will lead the connecting plate to act as a fuse.

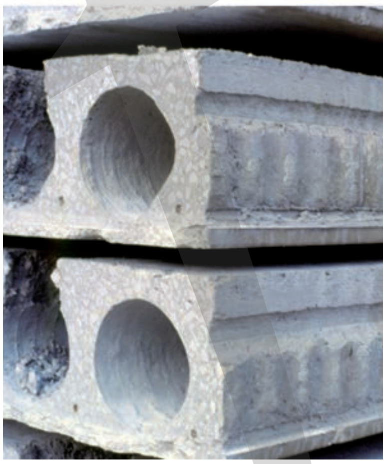


Fig. 8-5
Edge of a hollow-core slab with waved shear keys (*fib* Bulletin 74, 2014)

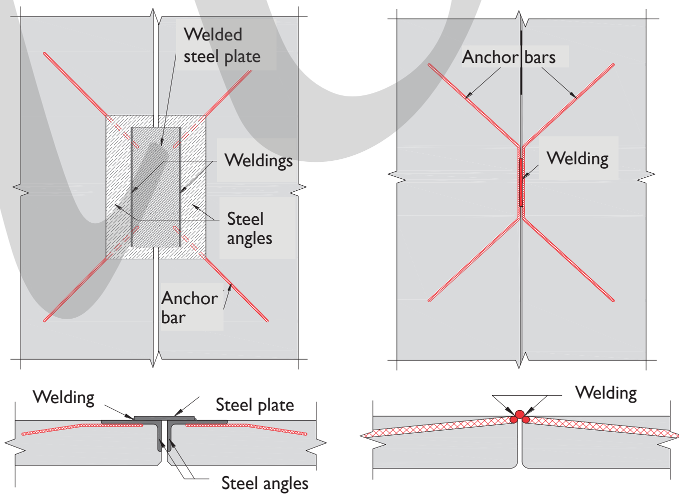


Fig. 8-6
Typical flange weld connection (*fib* Bulletin 74, 2014)

8.2.3 Rigid versus flexible diaphragms

Diaphragms serve to collect, transmit and distribute horizontal forces caused by earthquakes and wind over the vertical elements of the lateral force-resisting system. The distribution of the horizontal forces depends on many factors, such as:

- the spans of the diaphragm between the vertical elements of the lateral force-resisting systems;
- the geometry and position of the openings (if they exist) within the diaphragms;
- whether the diaphragms are topped or untopped and their aspect ratios;
- the reinforcing details along and across the diaphragm joints;
- the in-plane stiffness of the diaphragms; and
- the extent of dissimilarities and/or discontinuities of the vertical elements of the lateral force-resisting system.

In general terms, the distribution of the seismic design storey shear to the various vertical elements of the lateral force-resisting system is affected by their rigidity and by the rigidity of the diaphragm relative to that of the vertical system. The following definitions of diaphragm rigidity are given by the PCI Design Handbook and Eurocode 8:

- 'A diaphragm is classified as rigid, if it can distribute the horizontal forces to the vertical lateral force resisting elements in close proportion to their relative stiffness. However, close examination of the effective properties of diaphragms along with long-span applications suggest that many precast diaphragms may in fact be flexible.' (PCI Design Handbook, 2010)

- 'A diaphragm is flexible for the purpose of distribution of storey shear when the lateral deflection of the diaphragm under lateral load is more than twice the average storey drift of adjoining vertical elements of the lateral force resisting system under equivalent tributary lateral loads.' (PCI Design Handbook, 2010)

- 'The diaphragm is taken as being rigid, if, when it is modelled with its actual in-plane flexibility, its horizontal displacements nowhere exceed those resulting from the rigid diaphragm assumption by more than 10% of the corresponding absolute horizontal displacements in the seismic design situation.' (Eurocode 8, 2004)

In US practice, as delineated in ASCE 7-05 (2005) and ASCE 7-10 (2010), some diaphragms may be defined as flexible without any deflection calculation being required.

These are typically wood or untopped metal-deck diaphragms that are supported by rigid lateral force-resisting systems. Certain other diaphragms may be defined as rigid without any deflection calculation being required. They are concrete or topped metal-deck diaphragms with an aspect ratio of 3:1 or lower that have no horizontal irregularities. If a diaphragm is neither prescriptively flexible nor prescriptively rigid, it is defined as flexible if the maximum in-plane deflection of the diaphragm under any lateral load is more than twice the average inter-storey drift of the supporting lateral force-resisting systems under equivalent tributary lateral loads. If a diaphragm is not flexible by this definition, it is regarded as semi-rigid by ASCE 7-10 (2010), which then requires a semi-rigid diaphragm analysis for the purposes of distributing lateral loads to the lateral force-resisting elements. The IBC (2012), however, regards a diaphragm that is not flexible based on the above deflection criterion as rigid.

8.2.4 Internal diaphragm actions

Generally, the magnitude of the inertia force at each level is estimated by taking into account the distribution of the design base shear along the height of the building in the equivalent lateral force procedure (which accounts primarily for fundamental mode response) but by also making an allowance for higher mode effects (see, for instance, the diaphragm design force formula in ASCE 7-10, 2010). Recent research in the United States, Mexico and New Zealand (Rodriguez et al., 2002; Fleischman et al., 2002; Farrow et al., 2003a; Farrow et al., 2003b; Gardiner, 2011), among other countries, has shown that the diaphragm design force levels in most modern codes, design guidelines and code-like documents such as ASCE 7 (2010) are insufficient to ensure elastic behaviour during the design-level earthquake (typically set as having a 10% probability of exceedance in 50 years or return period of approximately 500 years).

In order to design and detail a diaphragm, it is necessary to transform the diaphragm design forces into internal forces within the diaphragm. Figure 8-7 shows the internal forces in a diaphragm supported by two parallel walls.

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

- For rather simple, regular floor plans, horizontal beam analogy is usually employed
- For more complicated cases, such as when the presence of large openings may interfere, a strut and tie model is recommended

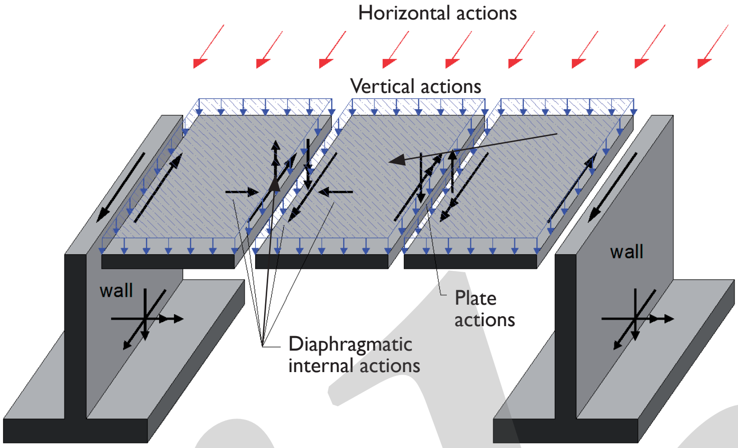


Fig. 8-7
Internal diaphragm forces and plate action of precast floor

In **horizontal-beam analogy**, the floor is considered a horizontal 'beam' analogous to a plate girder to analyse the diaphragm.

Figure 8-8 illustrates the internal actions on a simple diaphragm using horizontal beam analogy. The compression and tension forces are developed along the edges of the diaphragm because of the diaphragm moment, while shear forces develop along:

- the joints of the precast members in the direction of the horizontal action;
- the joints between precast members and their supports (beams) perpendicular to the direction of the horizontal action; and
- the supports of the diaphragm due to the horizontal actions.

The **strut-and-tie method** (STM) may be considered a rational analysis procedure for irregular or complex (though not necessarily limited to these cases) diaphragms and is recommended by several researches, designers and building codes. This method may also be used as a tool to identify and describe the flow of forces in a cracked concrete continuum in any reinforced or prestressed concrete member, as well as a quantitative tool for the development of a complete design.

However, when using this method particular attention should be paid to regions where wide cracks, typically arising from displacement incompatibilities between the lateral resisting system and the floor diaphragm, may disrupt the potential path of compression strut forces (see Section 8.2.5). Sound design examples using strut-and-tie models may be found in *fib* Bulletin 61 (2011).

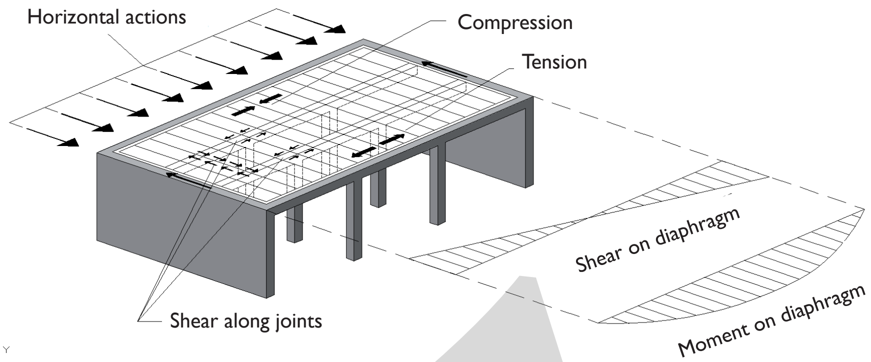


Fig. 8-8
Shear and moment on diaphragm; compression and tension forces due to moment; shear along precast member joints; shear along supports of diaphragm due to horizontal actions

8.2.5 Behaviour of precast-floor diaphragms under seismic action

Floor diaphragms serve mainly to distribute horizontal forces to the lateral force-resisting systems and to tie the structure together so as to ensure its robustness.

Though there are several causes of the differential expansion of floors relative to the lateral force-resisting system, in the event of earthquakes a major concern is the expansion of the supporting structure due to the elongation of plastic hinges in beams and its effects on the supported floor diaphragm.

Any differential expansion or contraction of the floor relative to the supporting structure can open up wide cracks at the support locations of the floor, as shown in Figure 8-9. Such wide cracks, which cross reinforcements that have yielded, may adversely impact the ability of the floor to tie the structure together and to distribute the lateral forces to the lateral force-resisting elements of the building.

Where the strut-and-tie method has been used in the design of the floor diaphragm, the formation of such wide cracks between columns (or walls) may prevent the development of a number of critical struts in the assumed strut-and-tie mechanisms.

Where the precast concrete floor systems (such as hollow-core units and double tees) run parallel to a lateral-resisting frame system, the units and the topping restrain the elongation of the plastic hinges in those beams. In cases where there are gaps between the edges of the precast floor units and the parallel beams, the topping over the precast units, connected to the beams, still exercises some restraint on the elongation of the plastic hinges in those beams. Such restraint may result in cracks that open up in the topping and that are parallel to the span of the precast floor units.

In practice, such regions of potential cracking in the floors should be carefully detailed. For example, Figure 8-9 illustrates how:

- in a corner of the structure, the floor slab should be properly tied with the beams around the column, as shown in detail D1;
- when precast floor units span past beam plastic hinges, the relevant region of the floor should be properly tied to the beam and the column, or preferably, as suggested in the case of hollow-core units but extendible to other types of precast flooring systems, a more flexible 'linking slab' (NZS3101:2006) should be provided between the beams and the precast floor units (in detail D3).

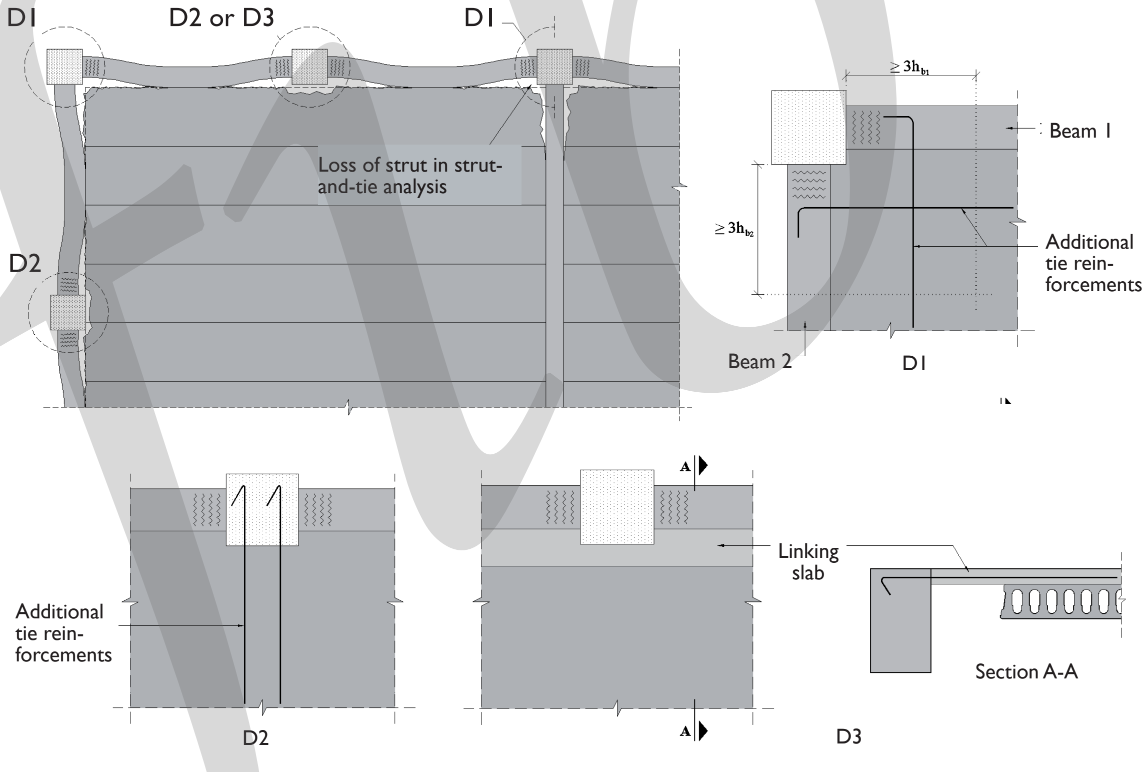


Fig. 8-9 Cracks due to diaphragm action of floor; measures to be taken (Based on Fenwick et al., 2010-02) (Other details not shown for clarity)

8.3 Displacement incompatibility issues between lateral resisting systems and precast floor diaphragms

8.3.1 General

When (pretensioned) precast units are used in floors, attention should be paid to the following issues.

Depending on the arrangement of the structural elements (columns, beams and walls) and the arrangement and type of the precast floor units sitting on the beams, there are a number of situations where seismic deformations may induce an appreciable differential deformation/displacement demand (either horizontal or vertical) between adjacent precast floor units or between a precast unit and other structural elements belonging to the lateral load-resisting system or the gravity system.

This phenomenon, which is a displacement incompatibility between lateral-load-resisting systems and 'gravity' elements, such as a precast floor, gravity elements and transfer beams, has been recognized, in recent research and after earthquake events, as a potentially critical structural weakness, with the engineering community paying ever greater attention to the issue.

- Damage to the floor diaphragm can compromise the structural performance of the whole building. Examples of undesired effects are:

- Damage to or unseating and collapse of the floor system

- Beam overstrength (with associated issues in terms of the capacity design and the hierarchy of strength) due to the interaction with the floor (flange effects), as well as to the beam axial load induced by the restraining action of the floor and columns

- An increase in curvature, moment and shear demand in the exterior columns

An example of vertical displacement incompatibility between the floor and a frame system is shown schematically in Figure 8-10. The frame system tends to deform in double curvature, while the floor system, acting mainly as a slab, tends to deform in single curvature. As a result, a significant vertical relative deformation/displacement demand is developed, which could lead to concentrated damage to the concrete topping and the starter bars, impairing the shear transfer capacity of the lateral forces between the floor and the lateral resisting system.

An example of vertical displacement incompatibility between floor units and a wall or a coupled wall system is shown schematically in Figure 8-11. The uplifting of the wall due to the formation of a plastic hinge at the base causes an uplifting of the floor units.

This mechanism can have two adverse effects:

- It could result in damage at and the potential failure of the wall-floor connection, impairing the shear transfer mechanisms of the seismic loads, if not leading to the collapse of the floor and/or gravity resisting systems

- Alternatively, the out-of-plane stiffness of the floor systems can resist the wall uplifting by introducing a significant axial load in the wall. This could lead to undesired overstrength effects (with consequent capacity design issues in the foundation system and other elements of the superstructure) and/or to the brittle failure of the wall under local or global buckling mechanisms due to the high axial-shear loads (Canterbury Earthquake Royal Commission, CERC, 2011-2012).

Horizontal displacement incompatibilities can develop as a result of the beam elongation effects associated with the development of plastic hinges as part of a 'strong column-weak beam' (beam sideway) mechanism (see left-hand image in Fig. 8-12). Beam elongation effects are studied in Douglas (1992), Fenwick and Megget (1993) and *fib* Bulletin 27 (2003).

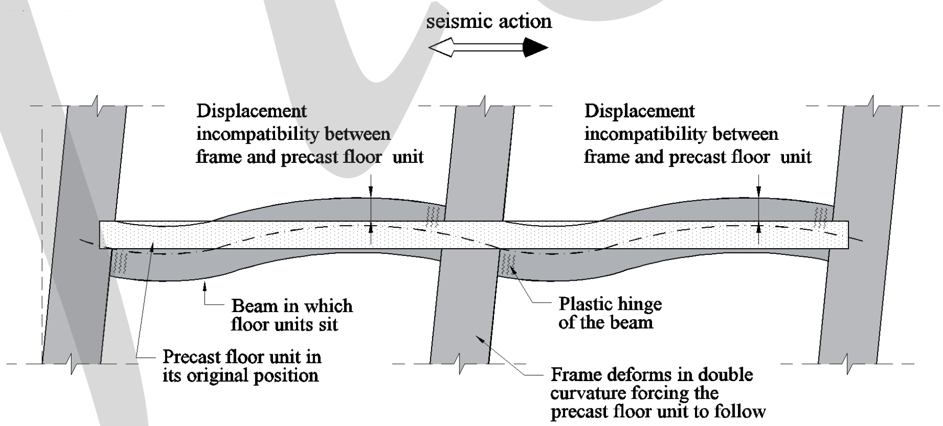


Fig. 8-10 Example of vertical displacement incapability between floor and frame systems (based on Matthews et al., 2003; NZS3101:2006)

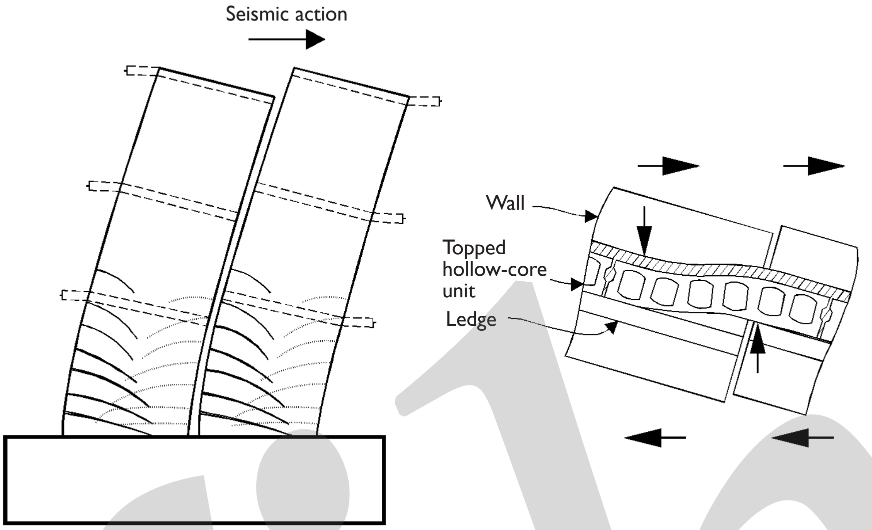


Fig. 8-11
Vertical displacement incapacity between floor and (single or coupled) wall systems due to uplift wall (fib Bulletin 27, 2003)

a. Relative vertical displacement in adjacent walls

b. Coupling effect at floor level

The beam elongation phenomenon comprises two types of contribution:

- Geometric, due to the physical shift of the neutral axis position
- Material-based, due to the cumulative residual strain in the steel
  Monolithic connections would cover both types of contribution, while a recentring jointed ductile connection would cover only the first type (geometric).

Generally, an elongation to the order of 2 to 4% of the beam depth h_b is expected to occur in each plastic hinge when expansion is free to develop (Park, 2002; Fenwick, 2010). A lower value, associated with the geometrical component only (the neutral axis position), would lead to the recentring of the jointed ductile connections. An example of the effects of beam elongation is given in Section 8.3.5.

The combination of vertical and horizontal incompatible displacements may adversely affect the entire structural system (see the right-hand illustration in Fig. 8-11).

It is generally difficult to determine the overall effect of earthquakes on the structural strength and safety of precast floors when we take the floor details as well as the induced displacements and induced or applied loads (including the vertical component of the seismic loads) into consideration.

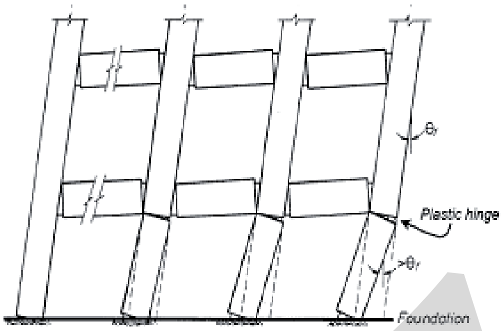


Fig. 8-12 Left: horizontal displacement incapability between floor and frame systems due to beam elongation (*fib* Bulletin 27, 2003)
Right: failure of precast hollow-core floor during large scale tests of 3D precast super-assmeblage at University of Canterbury (Matthews et al., 2003)

8.3.2 Strength enhancement of beams due to interaction with precast floors

Beams designed to sustain inelastic deformation in the event of a major earthquake may gain in flexural strength due to the interaction with the precast floor slabs. As a result, the assumed hierarchy of strength between the beam and the column, namely, the strong-column-weak-beam mechanism that is assumed in accordance with the capacity design considerations for the design of a beam-sway mechanism, may be violated.

Such a situation is presented in Figure 8-13, where the precast units of a hollow-core floor span pass the beam plastic hinges. This arrangement of beams and precast units may have an adverse effect during a seismic event for the following reasons:

- The seismic forces cause the building to sway and the frame systems respond by developing a 'beam sideway mechanism' as a result of 'strong column-weak beam' design.
- In the above situation, plastic hinges form in the beams and elongation occurs in these hinges.
- The elongation of the hinges is partially restrained by the floor (precast units and topping).
- As a result of this restraining action, the floor slab develops a tension force (see

Fig. 8-13).

- This tension force (or part of it) acts with the tension force in the top reinforcement of the beam (in the beam-to-column connection region) and increases its negative flexural strength.

- Overall, the restraint (due also to column stiffness and so forth) on beam elongation causes in the beam a compression resultant at both ends, which increases the moment-resisting capacity of the ends. In this situation, if no specific measures are taken, plastic hinges may form in the column prior to the formation of a hinge in the beam.

It is worth noting that the increase in strength in the beam plastic hinges can also occur in the more typical case of a precast floor (of any type) spanning only one beam bay length. The effects will be reduced but still quite significant, depending on the structural detailing of the floor-to-beam connections and the ability of the system to partly accommodate the beam elongation or 'beam growth'.

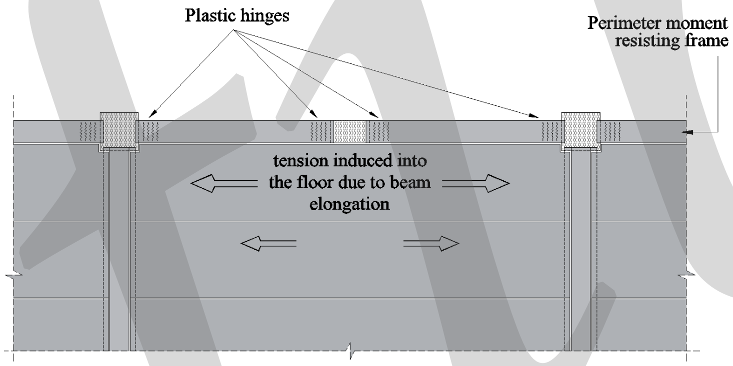


Fig. 8-13
Interaction of floors and beam where precast floor units span past the beam plastic hinges (based on Fenwick et al., 2010-02)

8.3.3 Other examples of displacement incompatibility effects

Figure 8-14 shows the plan view of a portion of a perimeter frame in a building where the cantilever beam parallel to the precast units is stronger than the beam in the orthogonal direction on which the precast floor units sit. This situation of a cantilever beam in the corner of a building should be avoided (see section on conceptual design). The expected mechanism is shown in Figure 8-14 to demonstrate the consequences of the vertical displacement incompatibility between precast floors and a lateral-resisting system (a frame in this case) under seismic actions (Fenwick et al., 2010-02).

During a seismic event, the building sways, the frame deforms, as shown in Figure 8-14b, and the corner rises. The rather stiff precast units attempt to remain straight and vertical displacements develop between the beam and the adjacent precast floor units (see Fig. 8-14b and c). These induced vertical deflections coupled with torsional effects in the precast floor units can cause significant disturbance to the precast floor, such as:

- Severe damage to the support of the units, depending on the type of unit, the support details and the magnitude of the seismic forces

- Possible failures of the webs between the cores of the hollow-core units due to extensive cracking

  This may lead to a separation of the tension flange from the compression flange of the units, resulting in the premature collapse of the floor (see Fig. 8-15c).

Experimental tests on a large-scale frame building super-assembly (Matthews, 2004) have shown that even if the precast units were supported on transverse beams framing into columns, the situation would not improve unless a flexible 'linking' slab were used between the beam and the adjacent precast unit (see Fig. 8-15) to accommodate these vertical relative displacements. This linking slab should have a minimum clear span equal to or greater than 600 millimetres (23.62 inches) or six times its thickness. This allows differential displacements to develop between the floor and beam without endangering the hollow-core units.

Note: More details about linking slabs may be found in NZS 3101:2006, Part 1, Clause 18.6.7.2 and Part 2, Clause C9.4.1.6.2)

Figure 8-15 shows the transfer of horizontal and vertical shear forces across a linking slab, based on Figure C9.13 of Clause C9.4.1.6.2 in Part 2 of the New Zealand Concrete Standard NZS3101:2006.

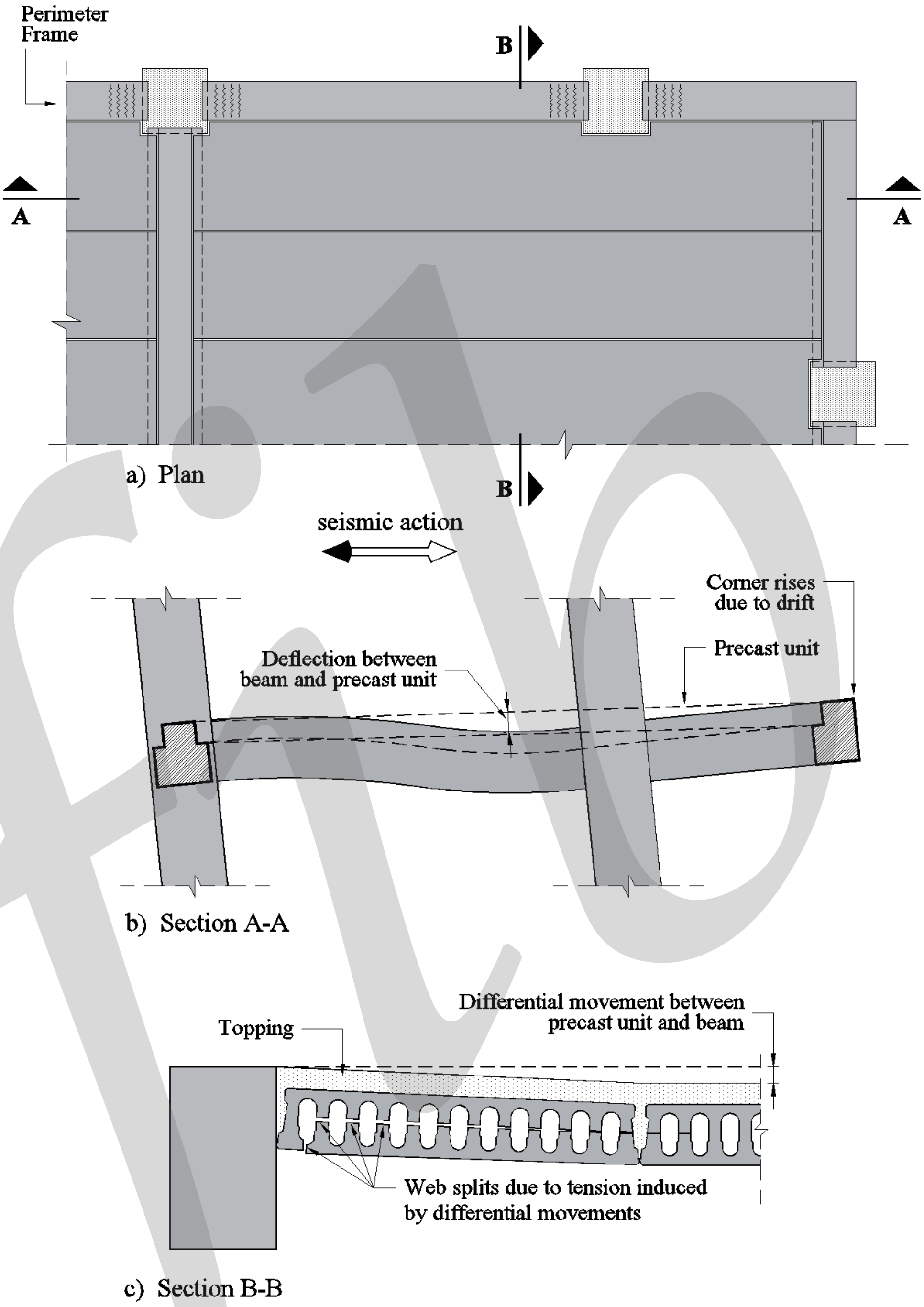


Fig. 8-14 Differential movements between beams and floor during earthquakes (Fenwick, 2010-02)

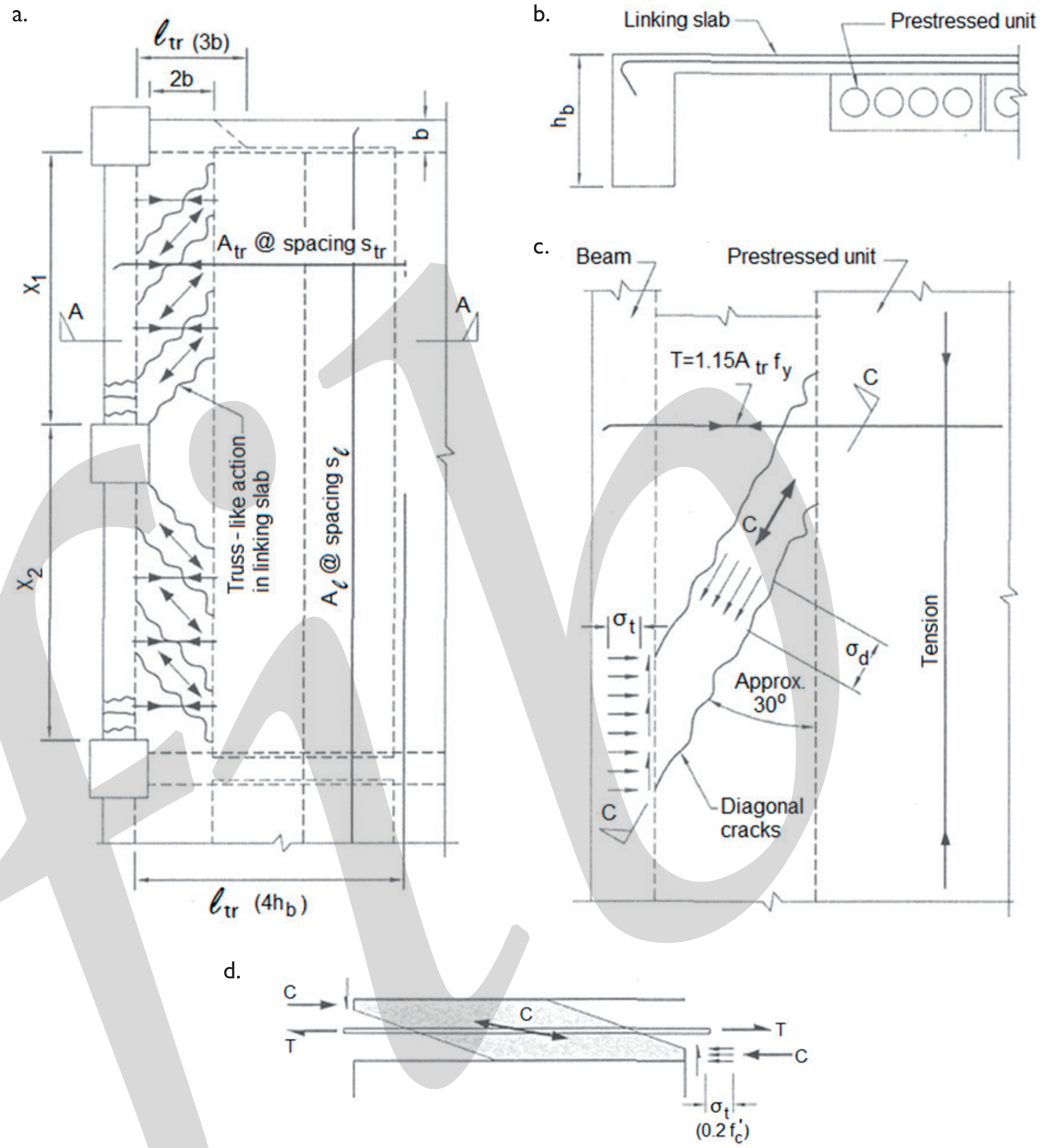


Fig. 8-15 Transfer of horizontal and vertical shear forces across linking slab (based on NZS 3101, 2006)
a. Plan on floor b. Section A-A
c. Strut-and-tie action in linking slab d. Elevation C-C on diagonal strut

8.3.4 Design guidelines for hollow-core-floor-to-lateral-resisting-system connections

A comprehensive draft document on the seismic performance of hollow-core floor systems, which includes guidelines for design, assessment and retrofitting, is available. The findings are based on experimental tests and/or analytical considerations and the document is a joint venture between the New Zealand Society for Earthquake Engineering (NZSEE), Structural Engineering Society New Zealand (SESOC) and the New Zealand Concrete Society (NZCS), with the endorsement of the Department of Building and Housing (NZSEE/SESOC/NZCS, 2009). Further refinements and integrated information can be found in a series of more recent publications (Fenwick et al., 2010-02; Fenwick et al., 2006; MackPherson et al., 2005; Lindsay et al., 2004; Matthews, 2004; Matthews et al., 2003 [a and b]; NZS3101, 2006), some of which includes lessons learned from the Canterbury earthquakes sequence. These publications provide an overview of alternative damage and failure mechanisms in hollow-core flooring systems and describes conceptual design/remedial strategies.

Once again, the proper detailing of the connections is proven to be critical for a robust seismic performance.

Figure 8-16 illustrates the potential failure mode under negative moment due to the relative rotation between the floor and the supporting beam.

Although hollow-core units are normally designed to resist gravity loading as simply supported members, the detailing of the support connections (such as starter bars used as continuity reinforcement) often results in unintended restraining effects.

The magnitude of the bending-moment demand, developed as a consequence of this rotation, depends on the amount and strength of the reinforcement (starter bars) in the beam-to-slab connection. In these cases, care should be taken to avoid brittle negative-moment flexural failure where the starter bars are lapped to the mesh of the topping. The flexural strength provided by the mesh may be lower than that provided by the starter bars and the reinforcement, if any, in the in-filled cells. For these reasons, attention should be paid to the amount and type of the reinforcement of the mesh and the location of the lap so that brittle failures in the slab can be eliminated.

Figure 8-17 shows the potential failure mode under positive moment at or close to the support, which is caused by the relative rotation between the floor and the supporting beam.

In these cases, care should be taken while detailing the beam-to-slab connection. When hollow-core units are mounted on mortar (and there is no evidence of slip) the mortar together with any infill concrete at the end of the units may allow significant flexural tension to be transmitted into the soffit of the unit, possibly generating flexural tension cracks. At these locations close to the support the prestressing strands are not fully developed and, therefore, there is little positive moment capacity.

The situation becomes worse when these units run parallel to and are close to the main beams, which could elongate due to the development of plastic hinges. Additional tension forces then develop along the length of the units (see Fig. 8-13).

In order to prevent such failure mechanisms under positive moment, it is recommended to break out cells at the end of the hollow-core units, to reinforce them suitably (see Fig. 8-17b as an example) and fill them with concrete. The resulting additional positive moment strength, which will be developed by the reinforcement in the cells, may prevent a positive moment failure at the soffit of the hollow-core units or near its support.

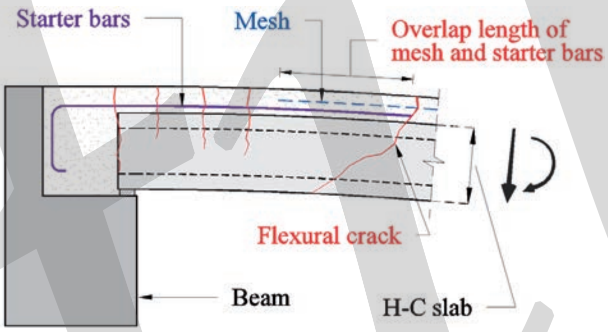


Fig. 8-16
Negative moment failure in beam-to-floor connection due to inadequate position of lap of starter bars and topping mesh and amount and type of mesh reinforcement (based on Fenwick et al., 2010-02)

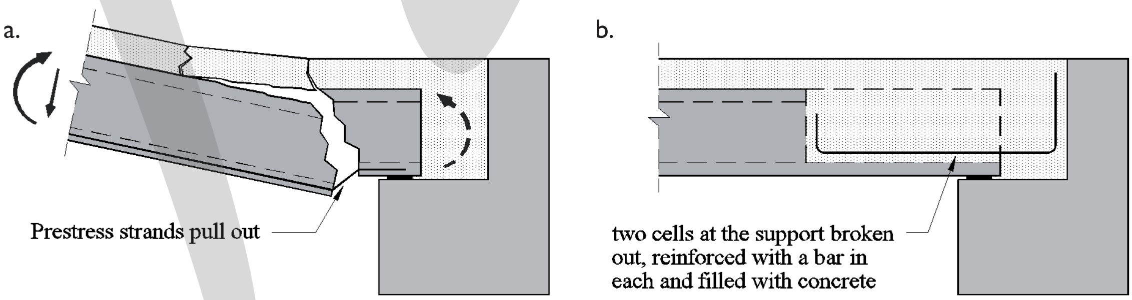


Fig. 8-17 Positive moment failure near a support (Fenwick et al., 2010-02)
a. Positive moment failure b. Reinforcement against positive moment failure

8.3.5 Support length of precast floor units for prevention of unseating in seismic situations

Generally, the support length is defined as the width of the total overlapping area between a precast floor unit and the supporting member. The nominal support length α is the value specified in design and aimed at during construction. The actual support length is the value achieved through construction. Allowable deviations (tolerances) are specified in various codes.

For example, according to Eurocode 2 (2004) (see Fig. 8-19) the nominal support length α is determined as:

$$\alpha = \alpha_1 + \alpha_2 + \alpha_3 + \sqrt{\Delta\alpha_2^2 + \Delta\alpha_3^2}$$

where

α_1 is the net bearing length;

b_1 is the net bearing width (see Fig. 8-19);

α_2 is the distance from the outer edge of the bearing area to the outer edge of the supporting member;

α_3 is the distance from the inner edge of the bearing area to the edge of the supported member;

$\Delta\alpha_2$ is an allowance for deviations in the distance between supporting members;

$\Delta\alpha_3$ is an allowance for deviations in the length of the supported member, ($\Delta\alpha_3 = l_n/2500$, l_n is the length of the member).

Minimum values of α_i are also given for various types of members (supports and precast units) in Eurocode 2 (2004).

It is very important to note that these EC2 formulas do not include other effects caused by seismic actions, such as beam elongation, and should thus not be used as is for design in seismic regions; they should be integrated to account for additional relative displacements caused by seismic actions.

The recent amendment of NZS3101:2016, which became available for public comments at the end of 2015, incorporates more detailed suggestions on how to account for the effects of beam elongation caused by seismic loading when designing the seating length of a precast floor system.

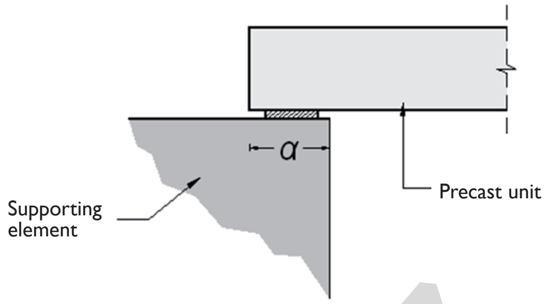


Fig. 8-18
Support length

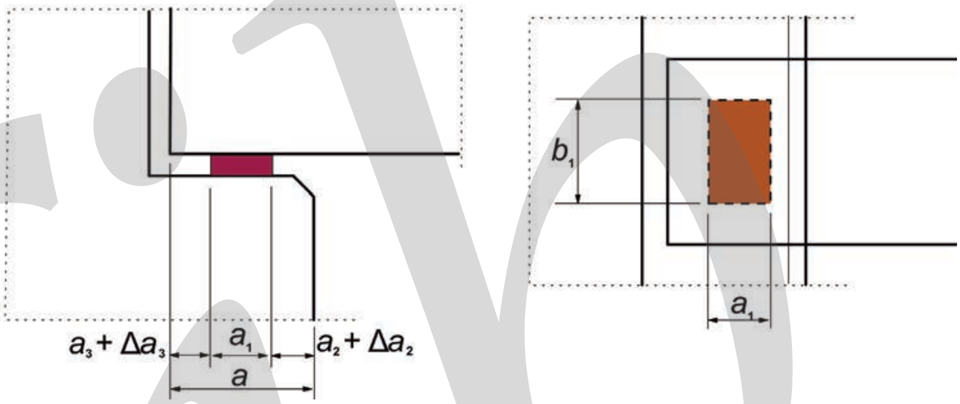


Fig. 8-19 Example of seating length of precast units (based on Eurocode 2, 2004, for nonseismic regions)

In floors containing precast units in seismic regions, the length of the supporting ledge, which corresponds above with Eurocode 2, may not be sufficient to support the floor. This may happen because during an earthquake, as the building sways forwards and backwards, the beam supporting precast units will rotate. This rotation between the supporting beam and the floor develops alternately negative and positive moments at or close to the supports of the precast floors. Positive moments, causing tension in the soffit of the hollow-core units (see Fig. 8-17) may reduce the length of the contact between the floor and the supporting beam. Furthermore, beam-elongation effects caused by the development of plastic hinges in adjacent beams (see Fig. 8-13) should be accounted for. Such effects tend to push the supporting beam away from the floor, potentially leading to the full unseating of the floor. Matters can get worse when some of the precast units of the floor sit directly on the plastic hinge regions of a beam. Typically, when plastic hinges develop, the cover concrete on the sides of the beam along the length of potential plastic hinge zones break down and spall out. Thus, precast units must not be supported on the cover concrete in these regions and alternative support systems are required.

Loss of support may occur for several reasons, namely:

- Inadequate allowance for construction tolerances
  Generally a construction tolerance of 20 millimetres (0.79 inches) is recommended.

- Creep, shrinkage, and thermal movement of the floor
  For practical purposes, the loss in support length due to creep, shrinkage, and thermal strains may be assumed as 0.6 mm (0.024 in.) per metre length of the precast unit.

- Crushing of concrete at support locations

- Spalling of concrete from front face of support ledge and back face of precast unit
  For practical applications, in hollow-core precast units the loss of the support length due to spalling may be taken as half the initial contact length, not to exceed 35 millimetres (1.38 inches). When a low-friction bearing strip is used the above values may be reduced by 25%.

- Failure of support
  Such failures mostly occur if the supported member is seated on cover concrete that is not reinforced.

- Movement of precast units relative to ledge providing support caused by elongation and rotation of beam of which ledge is a part
  This occurs mainly under earthquake loading.

The movement of the support relative to the precast units under seismic excitation depends on the elongation of the plastic hinge (related to the mid-height of the beam containing the plastic hinge), which is restrained by the floor, and the rotation of the beam on which the floor unit sits. The magnitude of elongation in a plastic hinge depends on the degree of restraint (to beam elongation) provided by the precast floor slabs.

Figure 8-20 shows potential plastic hinge locations according to Fenwick et al. (2010-02) (for the case of a hollow-core slab floor), which are marked with the capital letters H, L and M to indicate the following:

- High restraint (H)
  This occurs when precast, prestressed units span past these plastic hinges.

- Low restraint (L)
  This occurs where the plastic hinges are located in beams that support the floor units or the plastic hinges are close to the ends of the beam parallel to the floor units.

- Moderate restraint (M)
  This is the case when plastic-hinge zones are located next to a column with a transverse beam framing into it.

To estimate possible movements of precast units relative to the ledge of the supporting beam for practical applications, Fenwick et al. (2010-02) propose the following assumptions for the estimation of the expected maximum elongation at the mid-depth of the beam, based on the type of potential plastic hinge:

- for type H, elongation ≤ 0.02 h_b
- for type L, elongation ≤ 0.037 h_b
- for type M, elongation between ≤ 0.02 hb and 0.037 h_b
  where h_b is the beam depth

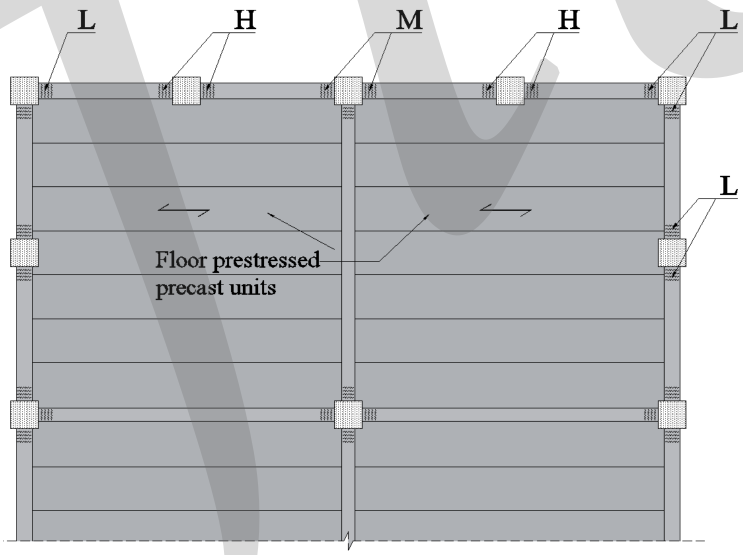


Fig. 8-20
Moment-resisting frames with precast floors showing different types of plastic-hinge elongation: high (H), low (L) and moderate (M) restraint (based on Fenwick et al., 2010-02)

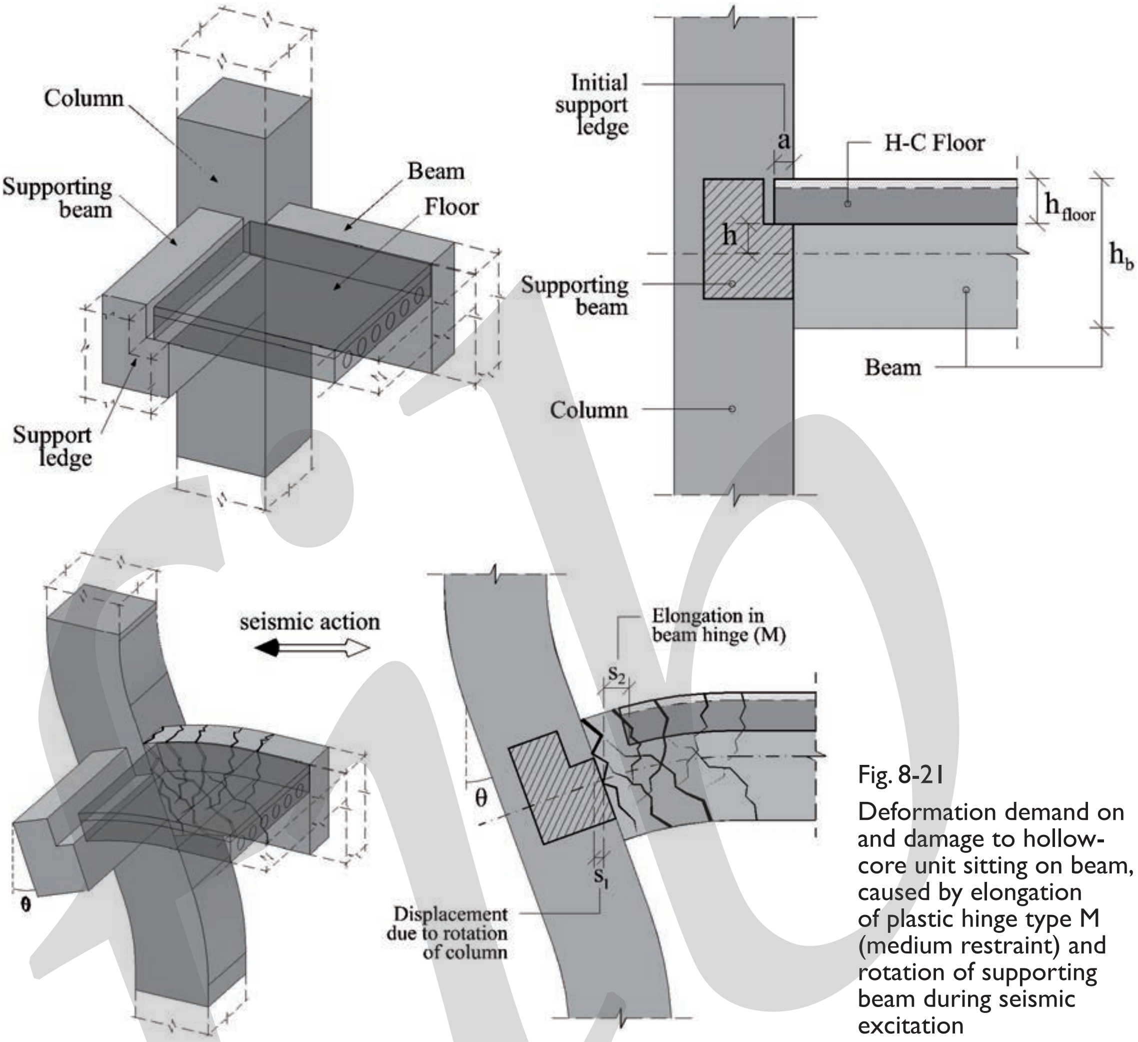


Fig. 8-21
Deformation demand on and damage to hollow-core unit sitting on beam, caused by elongation of plastic hinge type M (medium restraint) and rotation of supporting beam during seismic excitation

Figure 8-21 depicts the possible loss of support of a hollow-core floor seated on a beam caused by the following events occurring during seismic excitation:

- The rotation $\theta°$ at a beam-to-column connection, which causes a displacement S_1 between the hollow-core units and the supporting beam
- The displacement S_2 of the hollow-core unit caused by the possible maximum plastic-hinge elongation of the adjacent beam, which pushes the hollow-core unit away from its support

Thus, in seismic situations, the required support length of a hollow-core unit seated on a beam should also include the above displacements S_1 and S_2.

For example, in Figure 8-21 the depth of the beam where the plastic hinge (of type M) forms is assumed to be h_b = 1000 mm (39.37 in.), the depth of the floor h_{floor} = 200 millimetres (7.874 inches) and the drift rotation θ =0.035 (3.5%). The displacements S_1 and S_2 of the slab caused by the above would be:

$$S_1 = \theta \times h = 0.035\left(\frac{1{,}000}{2} - 200\right) = 10.5 \text{ mm } (0.4134 \text{ in.})$$

$S_1 \approx 0.03 \times h_b$ =0.03 × 1,000 = 30 mm (1.18 in.) (assuming an elongation equal to 3% h_b)

Thus $S_1 + S_2 \approx$ 40.5 millimetres or 1.59 inches (per beam edge connection), should be added to the values of minimum seating length calculated with non-seismic considerations (as per EC2).

Note: More details on the assessment of the elongation at the mid-depth of a beam (depending on the type of potential plastic hinge) can be found in Fenwick et al. (2010-02) and the amended version of NZ3101:2006.

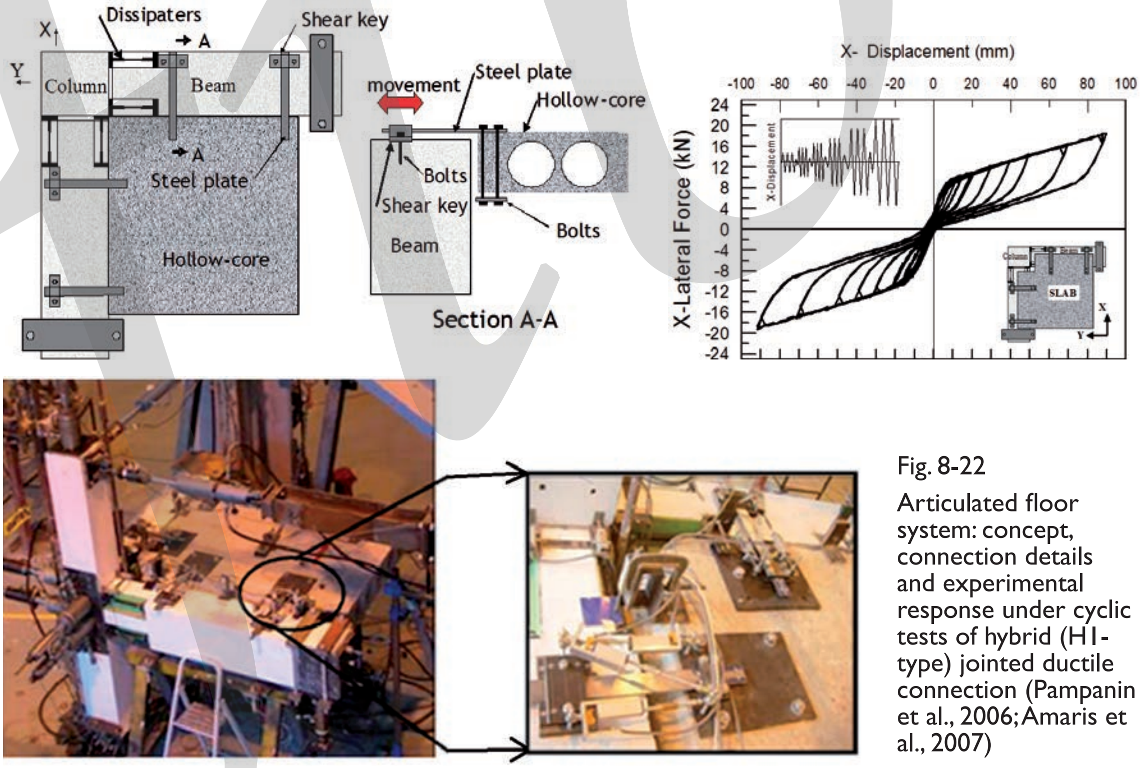


Fig. 8-22
Articulated floor system: concept, connection details and experimental response under cyclic tests of hybrid (H1-type) jointed ductile connection (Pampanin et al., 2006; Amaris et al., 2007)

8.4 Controlling and reducing damage to floor diaphragm

Alternative innovative solutions have recently been developed, following experimental and analytical/numerical investigations proposed in literature and/or already implemented in practice to minimize the damage to the floor system while guaranteeing reliable diaphragm action.

8.4.1 Jointed 'articulated' floor system

The first approach would consist of combining standard precast rocking/dissipative frame connections with an articulated or jointed floor system (Amaris et al., 2007). Based on this proposed solution, which was developed from the original concept of discrete X-plated mechanical connectors implemented in the five-storey PRESSS building tested at UCSD (Priestley et al., 1999), the either untopped or topped floor-system unit is connected to the beams with discrete mechanical connectors that like shear keys when the floor moves orthogonally to the beam and like sliders when the floor moves parallel to the beam (see Fig. 8-22).

As a result, the system is able to accommodate the displacement-compatibility demand between the floor and the frame by creating an articulated or jointed mechanism, which is effectively decoupled in two directions. Due to the low flexural stiffness of the shear key connectors in the out-of-plane directions, the torsion of the beam elements caused by the pulling out of the floor or the relative rotation of the floor and the edge support can also be limited.

Note that a relatively simple design option for reducing the extent of floor damage caused by beam elongation is to use a combination of walls and frames to resist lateral loads, with walls in one direction and frames in the orthogonal direction. If the precast one-way floors run parallel to the walls and orthogonal to the frame, the elongation effects of the frame to the floor are reduced. This approach can be combined with the partial debonding of the reinforcing bars (starter bars) in the concrete topping, as well as the use of a thin cast-in-situ slab or timber infilled slab (linking slab) in the critical areas to further increase the deformation compatibility.

A conceptually similar solution based on an articulated connection can be implemented for wall systems.

As discussed in Figure 8-11, the development of the plastic hinge at the base of the shear walls can lead to an equivalent elongation effect, with the uplifting of the wall and a potentially significant interaction with the floors, possibly leading to the damage and failure of the connection, which can impair the diaphragm action and might lead to the failure of the gravity system.

Given the length of the base section of typical walls when compared to beams, the geometric component of the beam elongation may already be significant at a low level of drift. The inelastic deformation associated with the development of the plastic hinge would then further increase the uplifting and vertical displacement incompatibilities between the lateral-resisting system and the floor.

In this case, the allowance for movement should be given in the vertical direction, to accommodate the vertical displacement incompatibility. Shear keys or connectors with vertical slots could thus allow the shear transfer mechanism to be decoupled from the uplifting effects, as shown in Figure 8-23 (*fib* Bulletin 27, 2003), in relation to the uplifting mechanism or a rocking system.

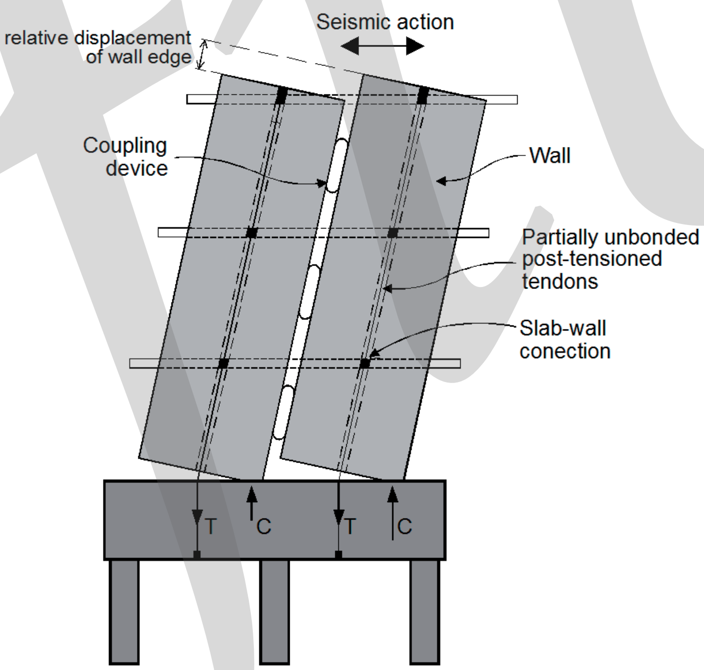


Fig. 8-23
Articulated floor system for wall-to-floor connection with vertical slotted shear connectors (*fib* Bulletin 27, 2003)

8.4.2 Top-hinge and slotted solution

An alternative method for preventing floor damage caused by beam elongation is the use of a newly developed 'top-hinge' or 'top-hung' beam-to-column connection in the frame system, to be used with a standard floor solution (namely, with topping and continuous starter bars). In its general concept (see Fig. 8-24), the top hinge allows the relative rotation between the beams and column and a yield in tension and compression of the bottom reinforcement. The presence of a slot or gap on the bottom part of the beam will prevent direct contact between beams and columns from occurring, thus preventing the elongation of the beam and the tearing of the floor. A debonded length is adopted in the bottom steel rebars to prevent premature buckling, as would typical PRESSS jointed ductile connections.

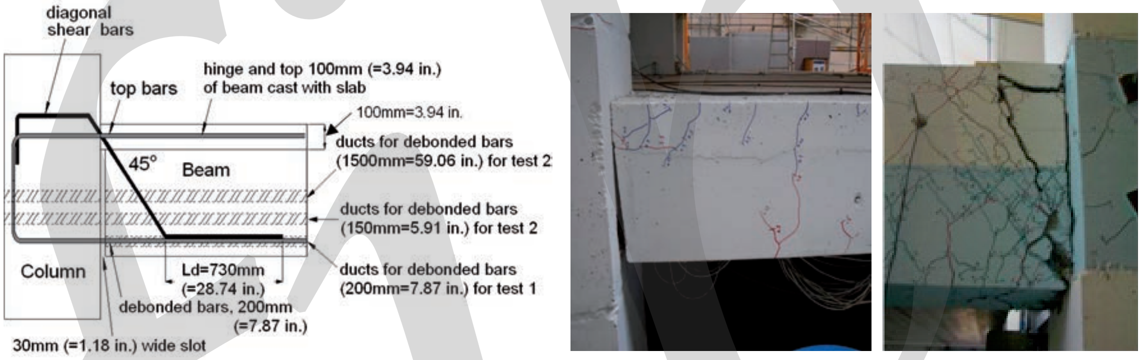


Fig. 8-24 Top-hinge-slotted non-tearing floor system
Left: Concept and connection details
Centre: Experimental response under cyclic loading (from Muir et al., 2012)
Right: Moment-resisting monolithic (emulative approach) connection (from MacPherson et al., 2004

This concept originates in the tension-compression yield-gap (TCY-Gap) connection, developed during the PRESSS programme as one of the jointed ductile connection that use internally grouted mild-steel bars on the top, unbonded post-tensioned tendons at the bottom and a slot or gap at the interface between the column and the beam. This solution prevents beam elongation but not the tearing action in the floor caused by the opening of the gap at the top of the beam. An intermediate, improved version would consist of an inverted TCY-Gap solution based on a single top hinge with the gap and the grouted internal mild-steel bars placed in the bottom part of the beam. A connection like this, similar to the 'slotted beam' connection proposed by Ohkubo and Hamamoto

(2004) for use in cast-in-situ frames (without post-tensioning), would prevent both elongation and tearing effects in the floor but would not yet be able to provide re-centring because of the location and the straight profile of the tendons.

Further development of the concept and refinement of the details at the University of Canterbury, New Zealand, have led to the creation of a 'non-tearing-floor' beam-to-column connection that can be combined with any traditional floor system (Amaris et al., 2007; Eu et al., 2009; Muir et. al., 2012). Based on a series of experimental tests on interior and exterior beam-column sub-assemblies as well as on 2D and 3D frame building specimens, a number of solutions, with or without post-tensioning, have been developed, ranging from semi- to fully precast connections.

9 Double-wall systems

9.1 General

Double-wall precast systems are used for both low and high-rise buildings, such as residential and office buildings, housing, hotels educational and administrative buildings.

Double precast walls are also used in lateral-force-resisting systems, sometimes in combination with other types of precast systems, such as elevator shafts and walls around staircases and basements.

Double-wall systems are normally built using an industrialized automatic production process. These are walls composed of two concrete layers, each usually 5 to 7 centimetres (1.97 to 2.75 inches) thick, with a gap of about 8 to 20 centimetres (3.5 to 7.87 inches) (see Fig. 9-1).

The two concrete layers are internally connected by means of reinforcement in the form of lattice girders at a spacing of about 40 to 80 centimetres (15.75 to 31.5 inches). One layer of welded wire reinforcement is typically provided in each layer of the double wall and acts as the main wall reinforcement. The gap between the two precast concrete layers is filled with cast-in-situ concrete during construction, after the placement of additional connection reinforcements where needed and of the installations.

The double walls described above are not traditional precast 'sandwich panels' since, as a rule, the whole gap between the two concrete layers is filled with cast-in-situ concrete.

The slabs are usually of the plank type. These slabs are made up of a bottom concrete layer that includes the main slab reinforcement and lattice girders at appropriate spacing, which increases the slab rigidity during erection and also ensures monolithic behaviour with cast-in-situ concrete poured on top of the precast planks (see Fig. 9-2).

The advantages of the double-wall system are speed of construction, fire resistance and economy. From a structural point of view, the main advantage is monolithic behaviour under vertical and horizontal loads, which results in a more or less uniform distribution of the seismic loading in all directions of the building, thus minimizing storey drifts.

Compared with monolithic cast-in-situ concrete walls, less visible cracking is observed in double precast walls due to their production process (Hegger et al., 2006). The ease of the on-site placement of the hydraulic and electrical installations that precedes that of the cast-in-situ infill concrete is also worth mentioning.

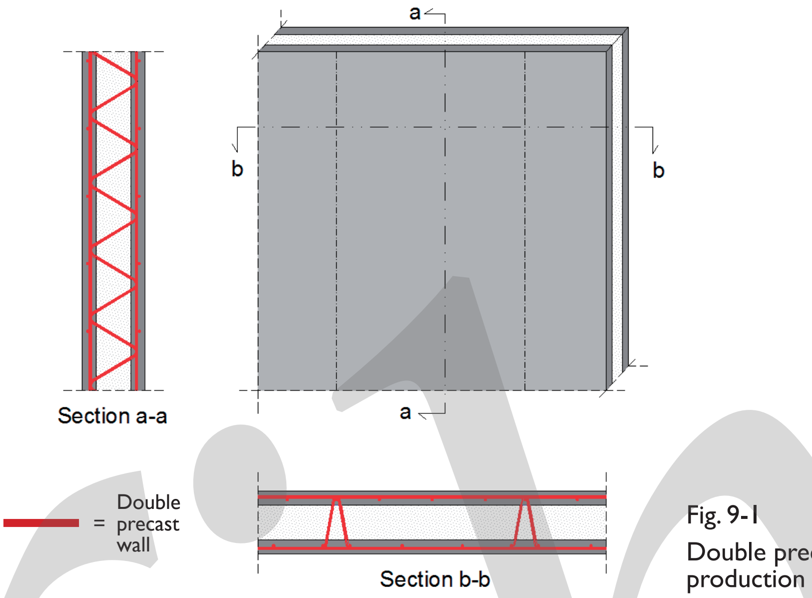


Fig. 9-1
Double precast wall reinforcement during production

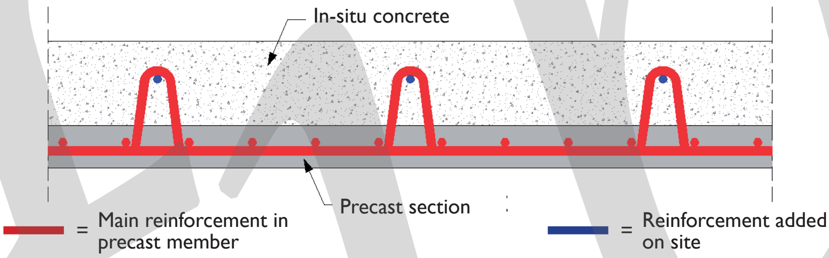
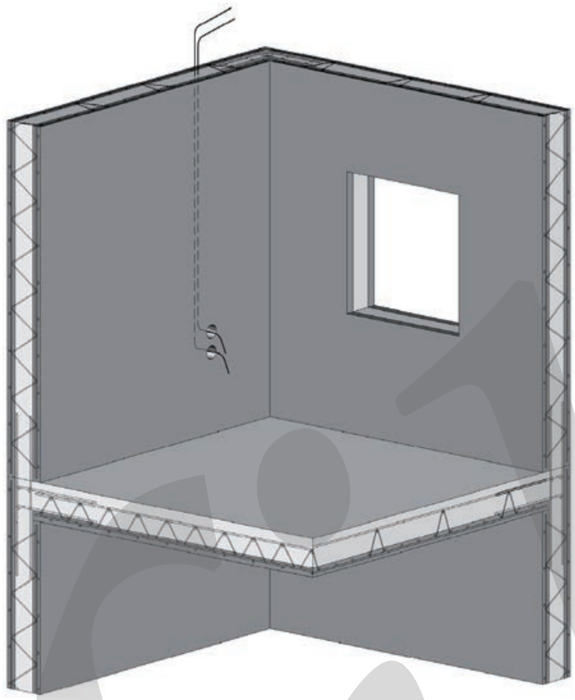


Fig. 9-2
Typical plank-type slab

9.2 On-site construction technique

After the completion of the cast-in-situ foundation, the walls of the lowest storey are placed in the positions designated in the drawings using cranes and they are held in their final position with suitable lateral bracings. The precast slabs are placed at the top of these walls and the additional reinforcement of the slabs and the walls are installed. Temporary propping of the precast slab, based on the latter's thickness, ultimate use and span, may be required in order to minimize construction deformations. Thereafter, all installations are placed in accordance with the drawings, at designated locations in the gap between the vertical walls and on the precast slab. The cast-in-situ concrete is placed on top of the precast slab as well as in the gaps of the vertical walls. After the hardening of the concrete, the same procedure is followed for the next storey and so on. Figure 9-3 illustrates these construction steps.



1. Placing of lower walls
2. Placing of precast planks
3. Placing of additional reinforcement and installations
4. Concreting
5. Placing of upper walls after hardening of concrete

Fig. 9-3
Constructions steps

9.3 Ductile behaviour

Generally, double-wall systems are rigid and produce buildings that are rigid in all directions. Elastic analysis may, therefore, be appropriate, if the low value of the behaviour factor is taken into consideration. This does not necessarily produce un-economical designs.

In cases where the architectural design introduces rows of openings in the walls in both directions, the ductility can be mobilized in the coupling beams above the openings. Higher behaviour factors can then be assumed in analysis. These higher values must be used in conjunction with proper detailing for the coupling beams.

9.4 Numerical models for structural analysis

Since in a double-wall system almost all the walls are concrete bearing walls, and there are many walls, the proper modelling of the walls is essential in the structural analysis.

The use of finite elements seem to be most suitable for the modelling and the estimation of the internal forces that act horizontally and vertically on wall-to-wall, wall-to-slab and slab-to-slab connections as well as on areas around the openings in the walls (such as the doors and windows).

9.5 Structural connections and other structural details

Some structural details of double-wall systems are shown in Figures 9-5 to 9-16 (based on the Syspro Handbook, 1997, and Tsoukantas and Topintzis, 2006). When construction is undertaken as illustrated in these figures, particular attention should be paid to:

- the installation of the additional reinforcement in the cast-in-situ concrete of the walls in relation to the position of the lattice girders; and
- the anchorage of the additional reinforcement.

Generally, the thermal insulation of the walls is achieved with a suitable treatment of the exterior surface of the concrete wall.

In some cases, an insulating layer of proper material is provided in the gap between the two precast layers of the walls, as shown in Figure 9-4. This alternative is difficult to execute. Particular attention needs to be paid to the proper placement of the insulating layers.

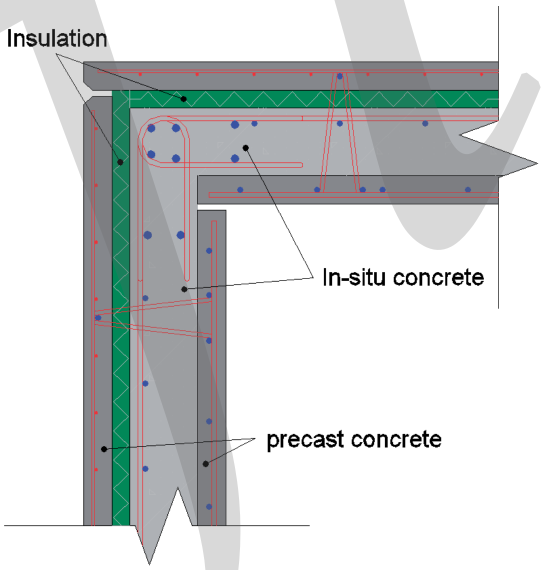


Fig. 9-4
Horizontal section of double walls with internal insulation (Tsoukantas/Topintzis, 2006)

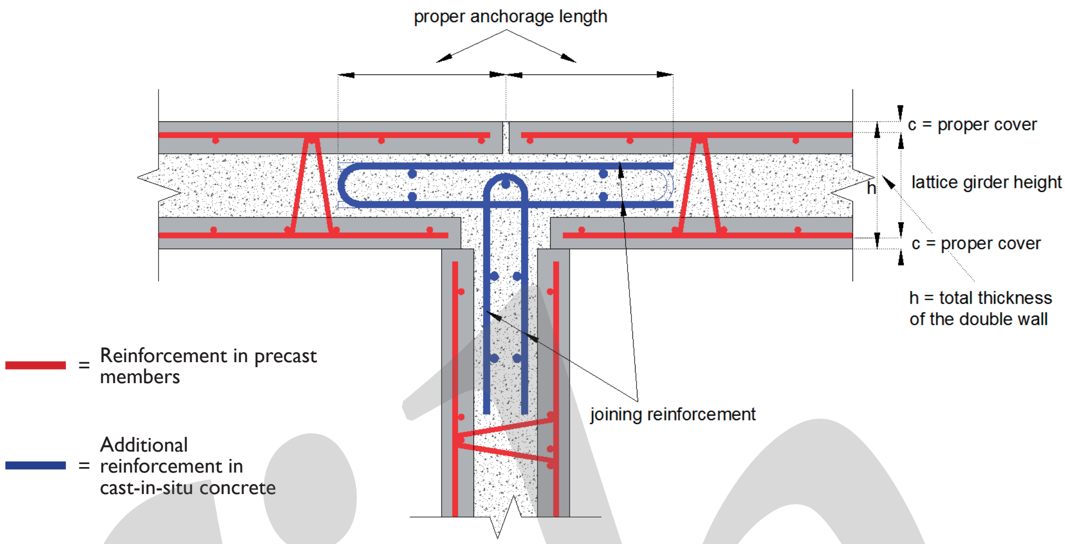


Fig. 9-5 Horizontal section of connection of exterior and interior walls

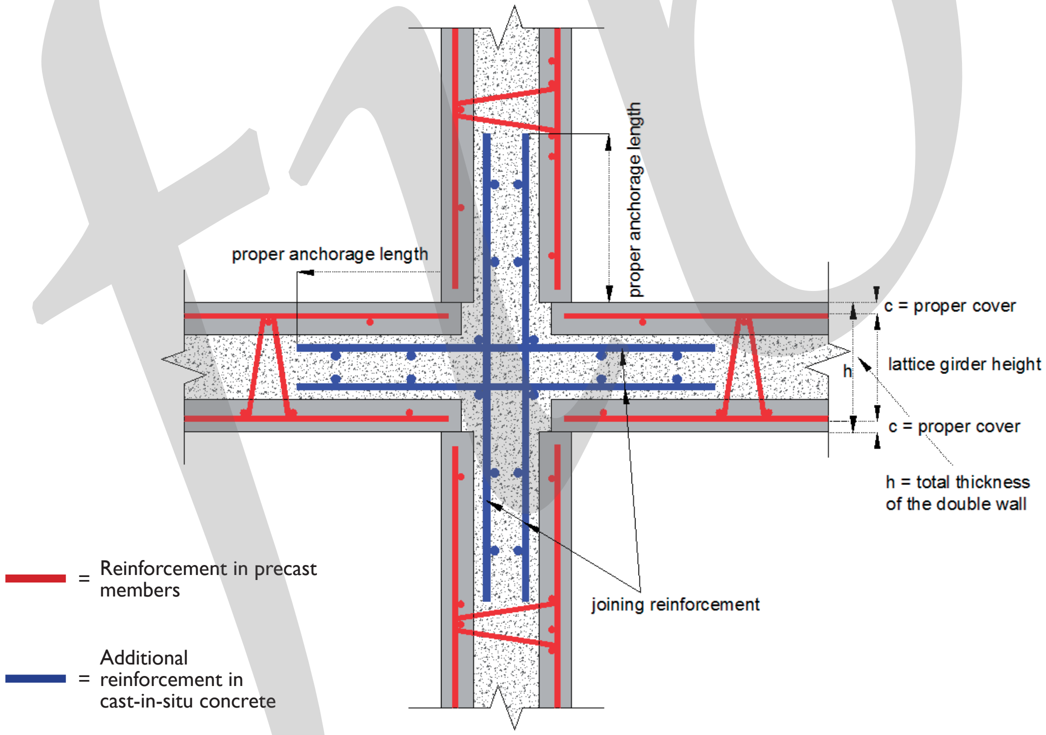


Fig. 9-6 Horizontal section of connection of interior walls

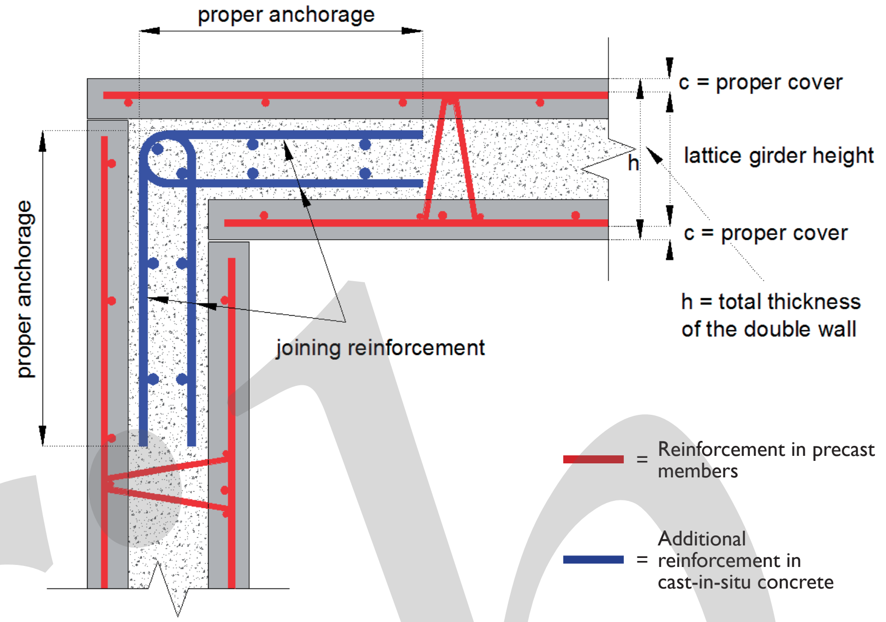


Fig. 9-7 Horizontal section of typical corner connection of double walls

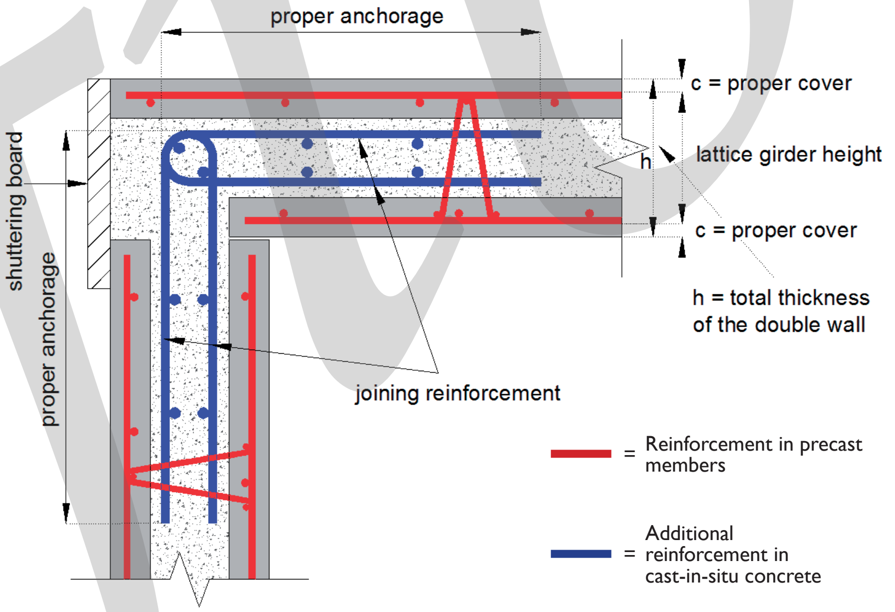


Fig. 9-8 Horizontal section of connection detail for corner connection of double walls to satisfy direct anchorage lengths of additional reinforcement

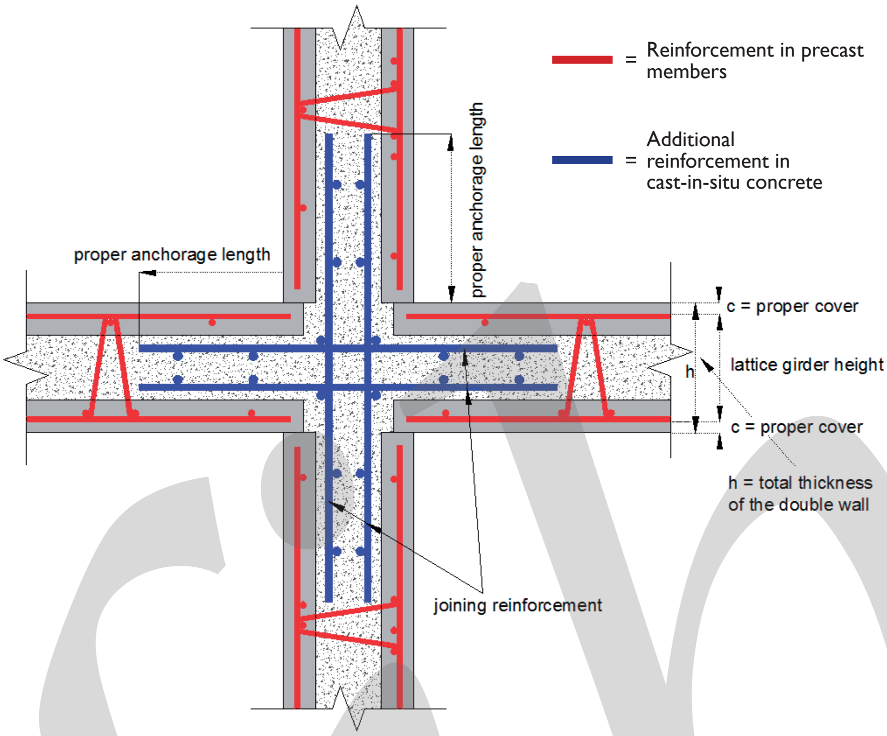


Fig. 9-9
Vertical section of interior wall and floor connection

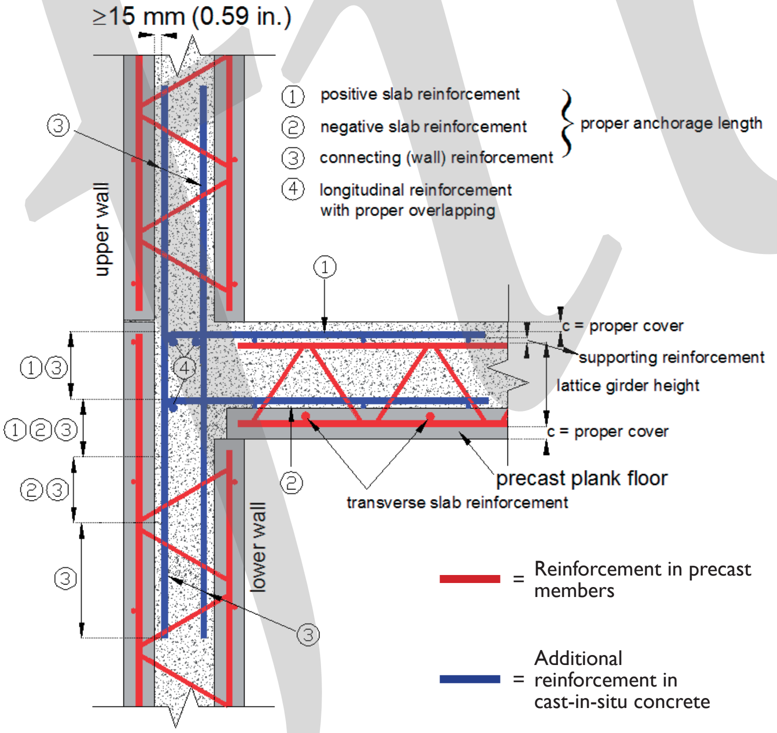


Fig. 9-10
Vertical section of exterior wall and floor connection

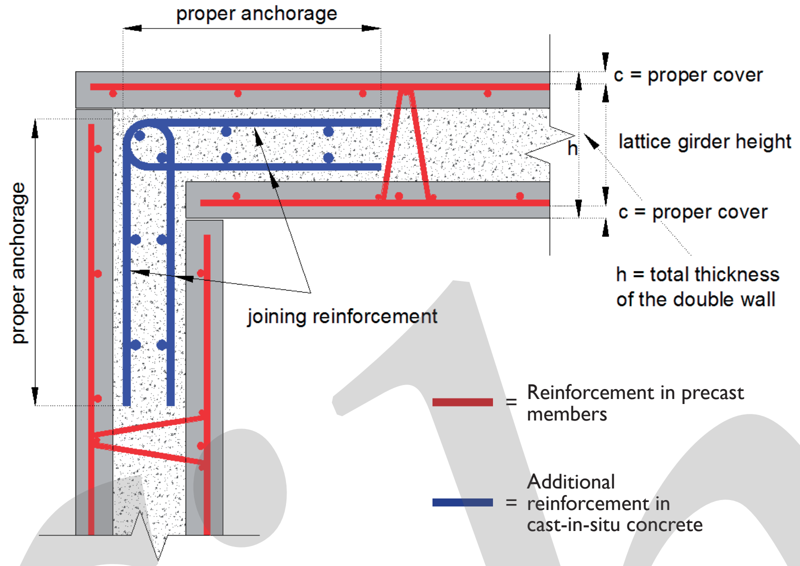


Fig. 9-11
Vertical section of exterior wall and roof connection

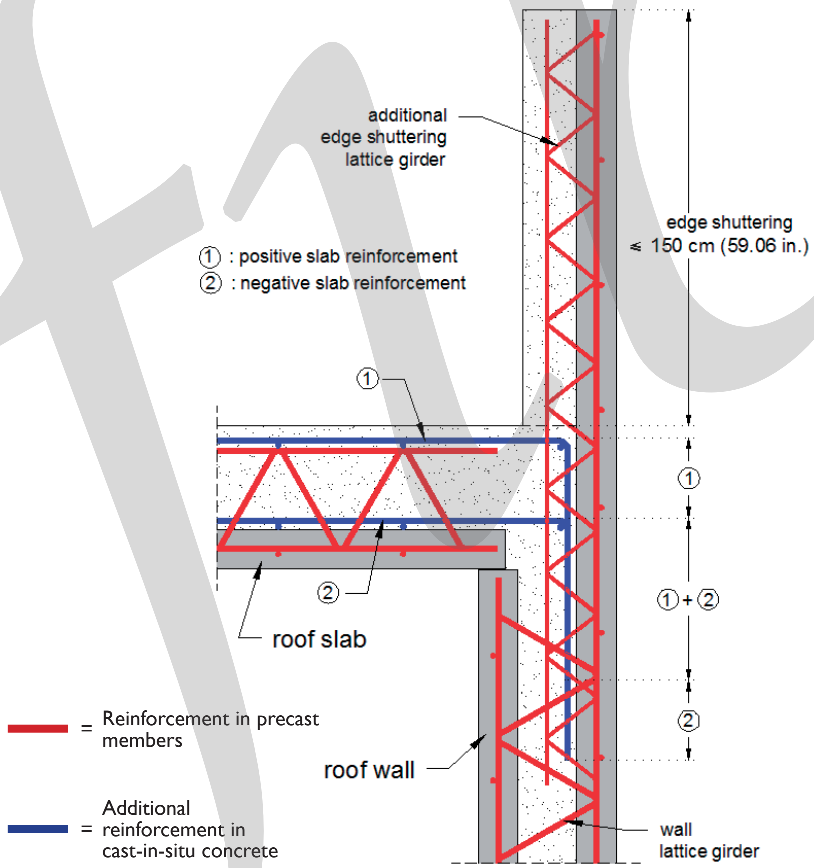


Fig. 9-12
Vertical section of exterior wall and roof connection with parapet

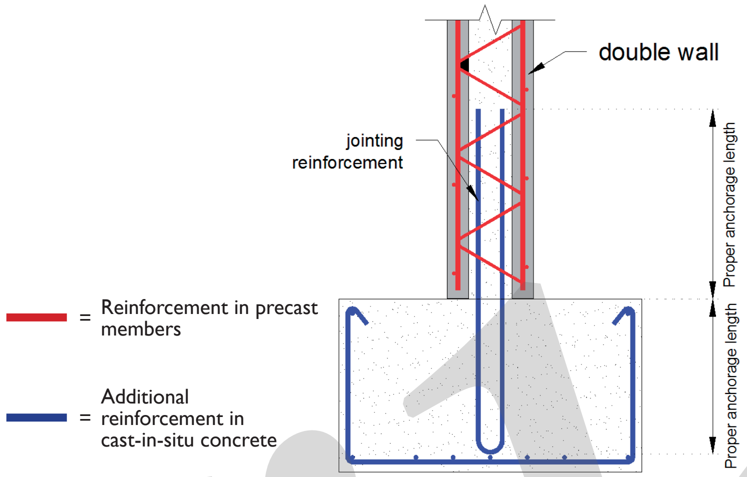


Fig. 9-13
Connection of a wall to strip footing

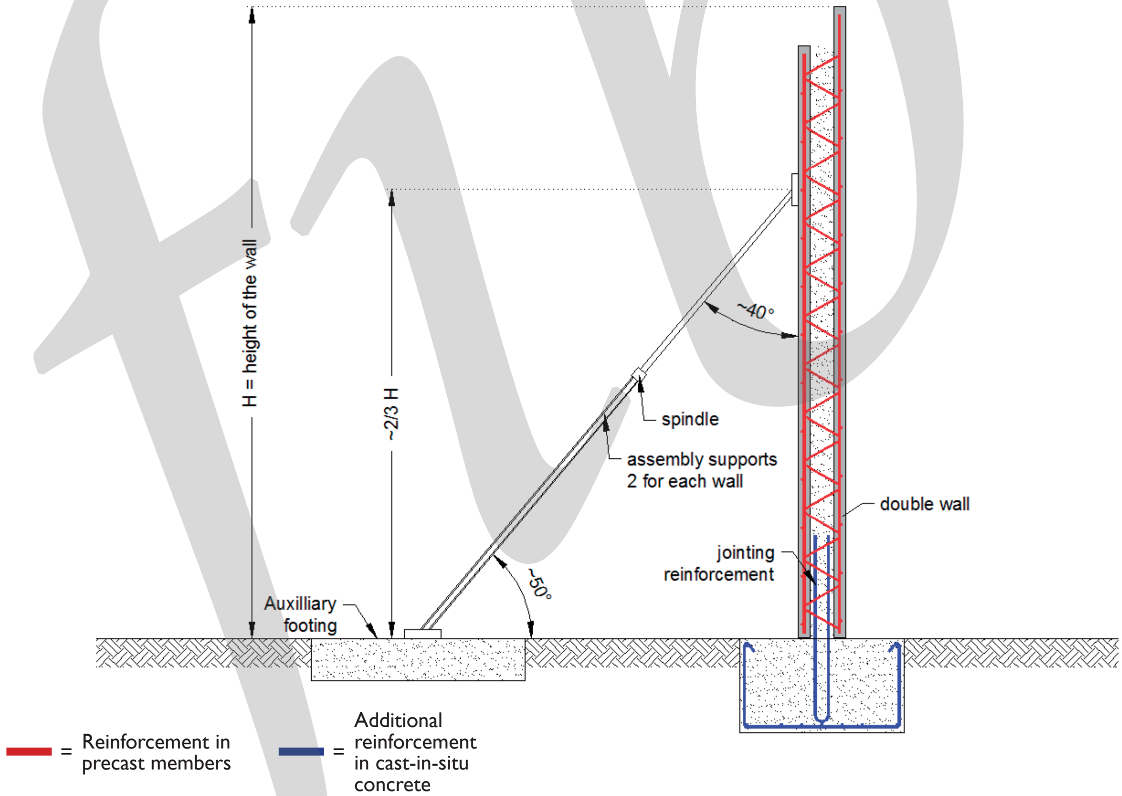


Fig. 9-14 Assembly on strip footing without a base plate

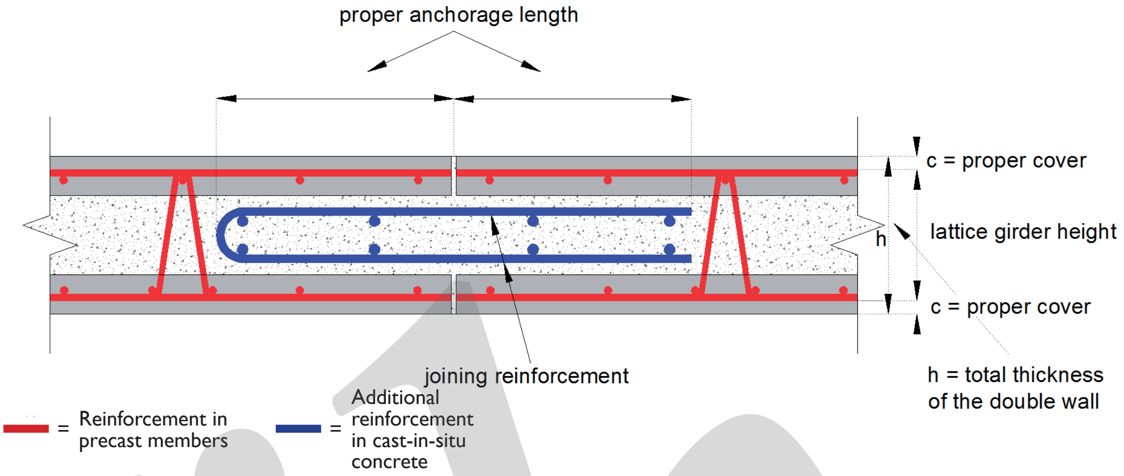


Fig. 9-15 Horizontal section of connection of double walls in a straight line

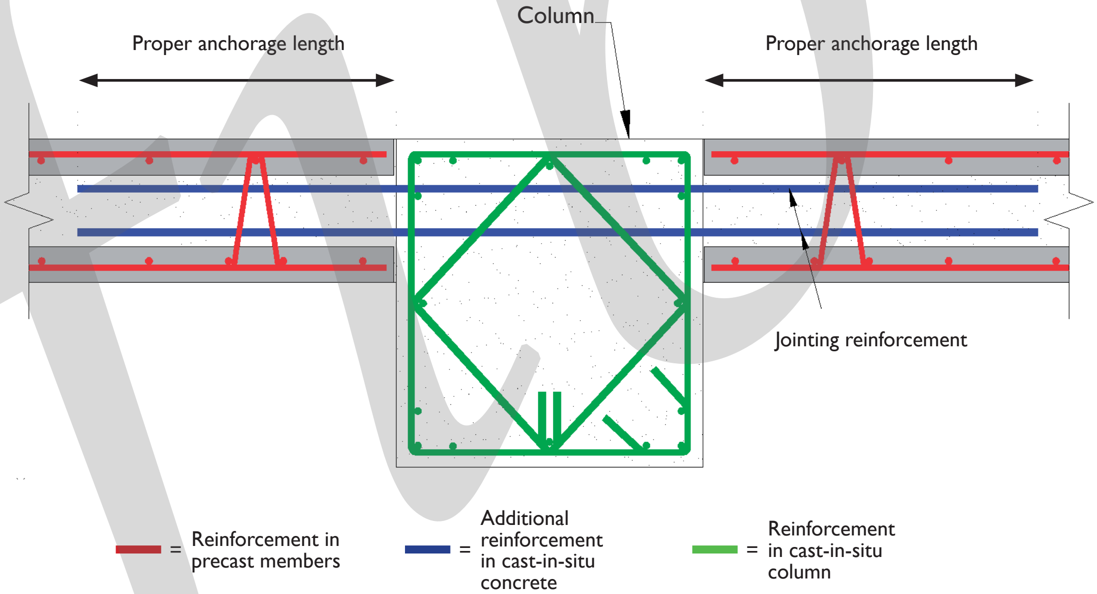


Fig. 9-16 Horizontal section of connection of cast-in-situ column with double walls

10 Precast-cell systems

10.1 General

Precast cells are typically used as parts of buildings, for example bathrooms, kitchen blocks, garages and staircase modules. They are industrially produced, completely finished and fitted out at the precasting plant, delivered to the building site and installed in the building. They are typically placed in one vertical line and usually form a self-supporting tower when properly connected together.

Precast cell systems are also used for the construction of other types of buildings, such as residential buildings, office buildings, hotels, educational facilities and correctional facilities. The cell units are built entirely in the precast plant and ready for use after on-site assembly. They satisfy structural as well as serviceability requirements and incorporate finishing such as plumbing, electrical and mechanical installations.

These systems are very suitable in cases where a building has to be dismantled and rebuilt in another location.

Prestocked, precast cell units are sometimes used to accommodate people after natural disasters.

The advantages of cell systems are:

- Their rapid construction and high quality (since the cells are finished and equipped at the factory)
- The structural integrity of the final building if cell units are properly connected to each other horizontally and vertically
- A minimum risk of progressive collapse under accidental loading
- Good insulating properties
- Cost savings due to the industrialization of the production

The main disadvantages are:

- The restriction on the sizes of the cell units, mostly on their width, which is limited to about 3.6 metres (11.81 feet) since they have to be transported from the precast plant to the site
- The high self weight of the cell units, which may cause problems during transportation and erection
- Their limited architectural flexibility

10.2 Classification

Precast cell systems may be classified according to:

- the structural and architectural characteristics of the cell units;
- the way in which cell units are combined to form the final structure;
- the degree of industrialization of the cell units;
- the production and erection process of the units; and
- the combination of cell units and large panel systems.

Precast cell units are characterized according to their structural and architectural features as follows:

- **Closed cell units** have bearing walls on all four faces; the walls may have openings (see Fig. 10-1)

- **Semi-closed cell units** have one of the four faces or the ceiling open to offer flexibility in choosing the final room size (see Fig. 10-2)

- **Ring cell units** have two opposite sides open, may be completely or partly finished at the factory and offer higher architectural flexibility when combined with closed or semi-closed cell units (see Fig. 10-3)

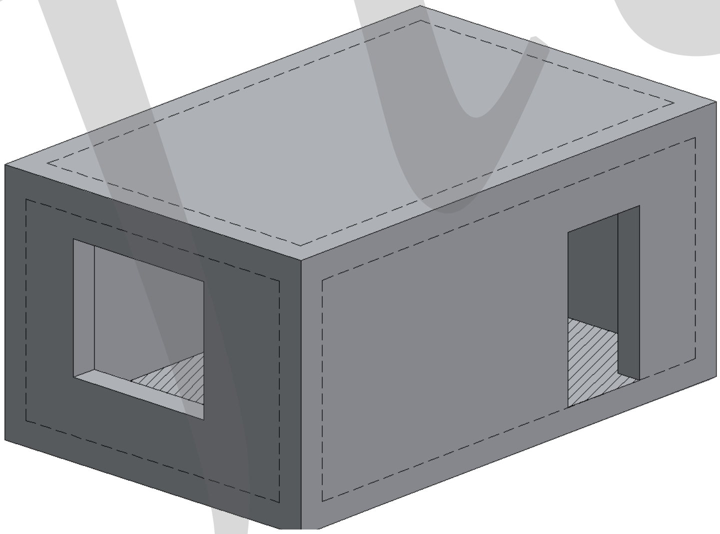


Fig. 10-1
Closed cell unit

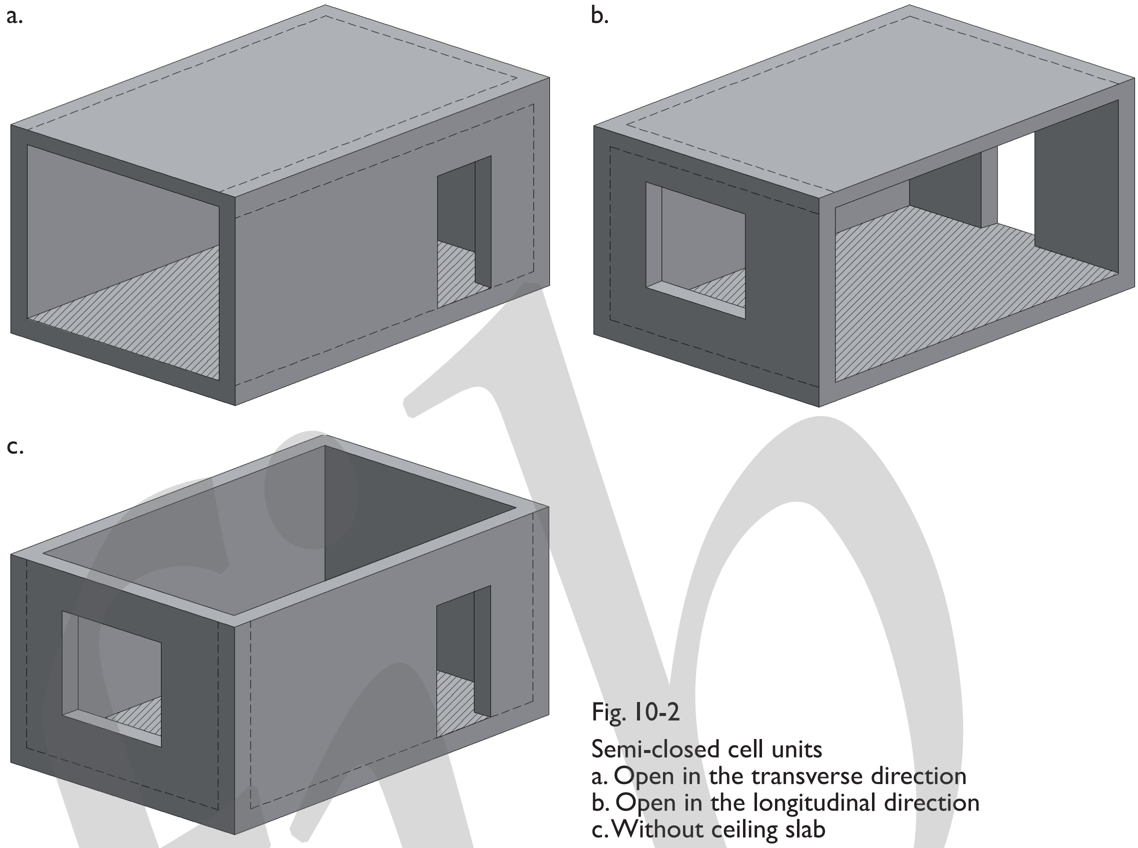


Fig. 10-2

Semi-closed cell units
a. Open in the transverse direction
b. Open in the longitudinal direction
c. Without ceiling slab

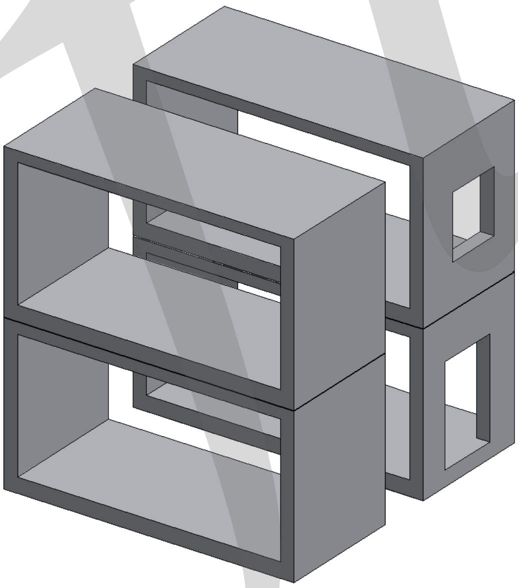


Fig. 10-3
Ring cell units

10.3 Construction aspects

Precast cell units can be used in various ways to construct a building using. The simplest ways are to combine:

- closed cell units (see Fig. 10-4) or
- closed and semi-closed cell units (see Fig. 10-5) or
- closed, semi-closed and ring units (see Fig. 10-6)

Though buildings with such combinations provide excellent acoustics, insulation and increased rigidity, they are relatively uneconomical since twin walls between rooms (horizontally and vertically) are often created.

A better approach (for architectural flexibility) is to combine on site various types of cell units with large, individual, precast panels in such a way as to avoid twin walls between rooms both horizontally and vertically (see Fig. 10-7).

Corridors in buildings may be created through the suitable arrangement of cell units and large precast panels assembled on site (see Fig. 10-8).

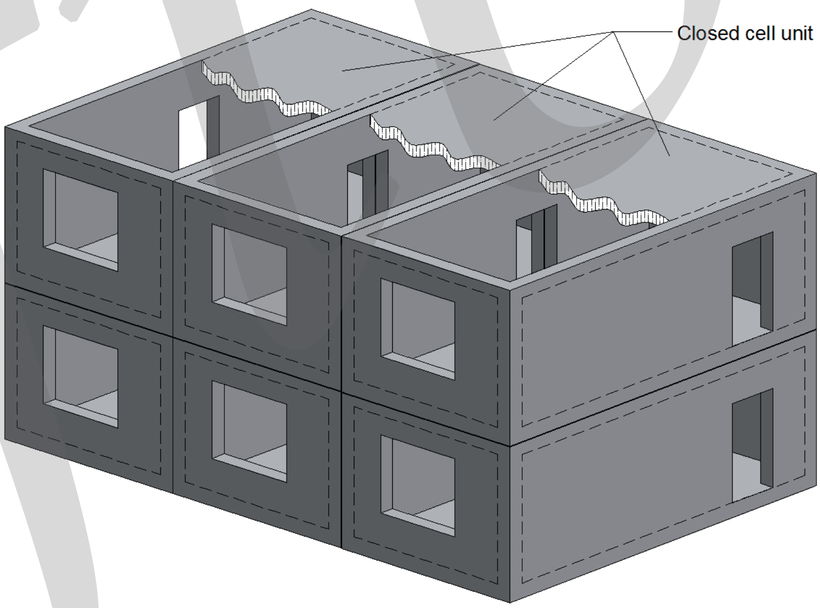


Fig. 10-4 Assembly of closed cell units

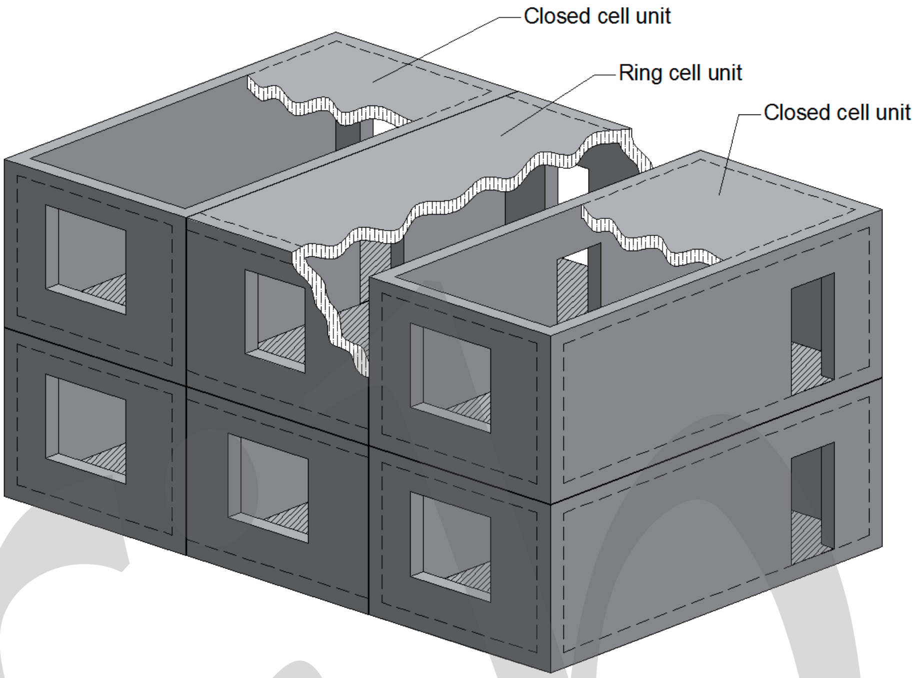


Fig. 10-5 Assembly of closed and ring cell units to reduce twin walls between rooms

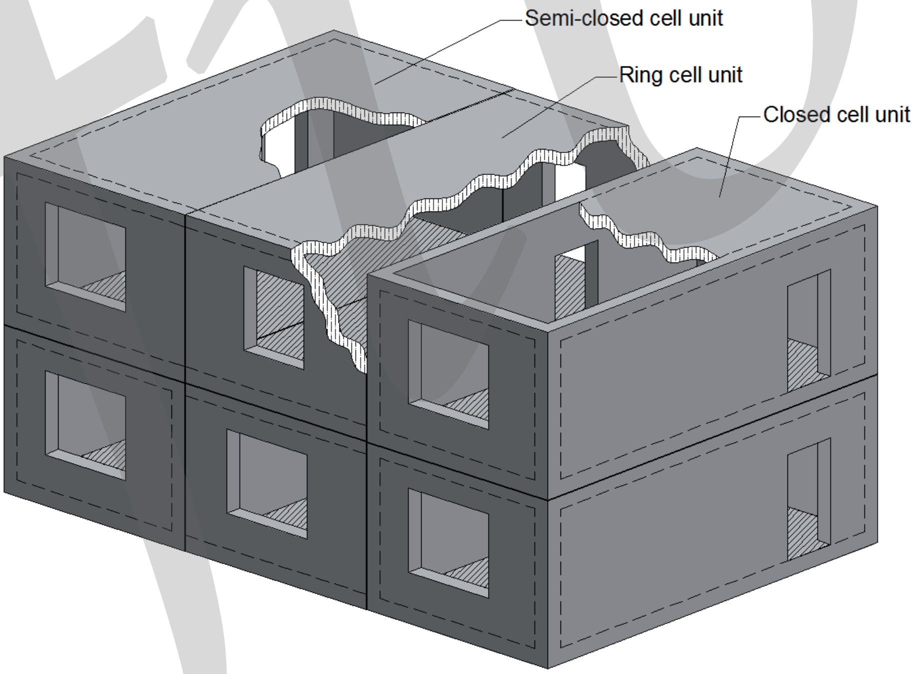


Fig. 10-6 Assembly of closed, semi-closed and ring units to create larger rooms

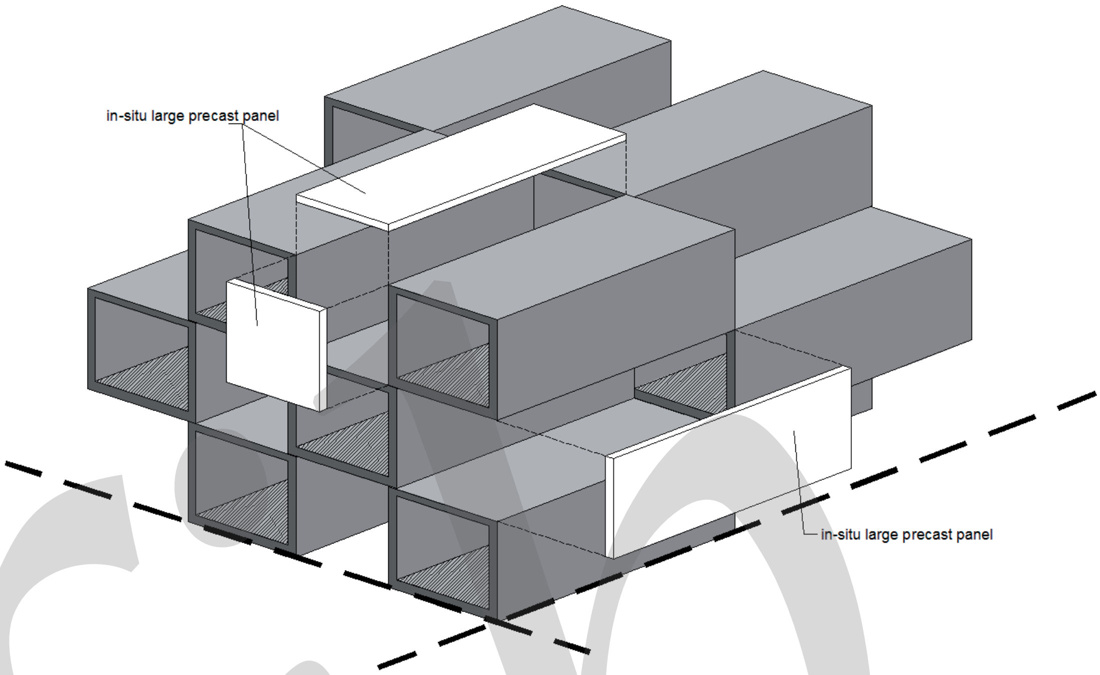


Fig. 10-7 Assembly of cell units of different types and large panels (based on Paschen and Wolff, 1975)

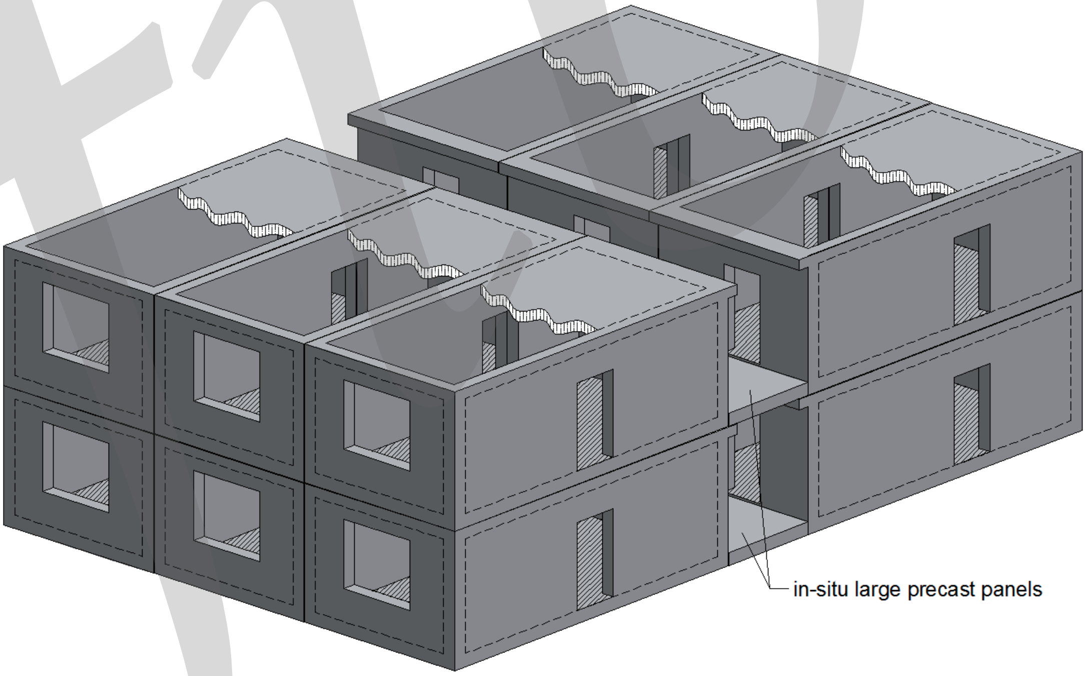


Fig. 10-8 Assembly of cell units to form building with corridors

10.4 Connections

Connections between cell units depend on the type of the cell units to be connected and their arrangement in the structure. Typically, dry connections or grouted tubes are used.

Horizontal connections mostly occur at the top of the slabs of the cell units to be connected and are created using steel plates, mechanical anchors and welding.

For vertical connections, post-tensioning may be used.

Figures 10-9 and 10-10 respectively show horizontal connections between cell units and a cell-unit-to-foundation connection (Manolatos et al., 2003).

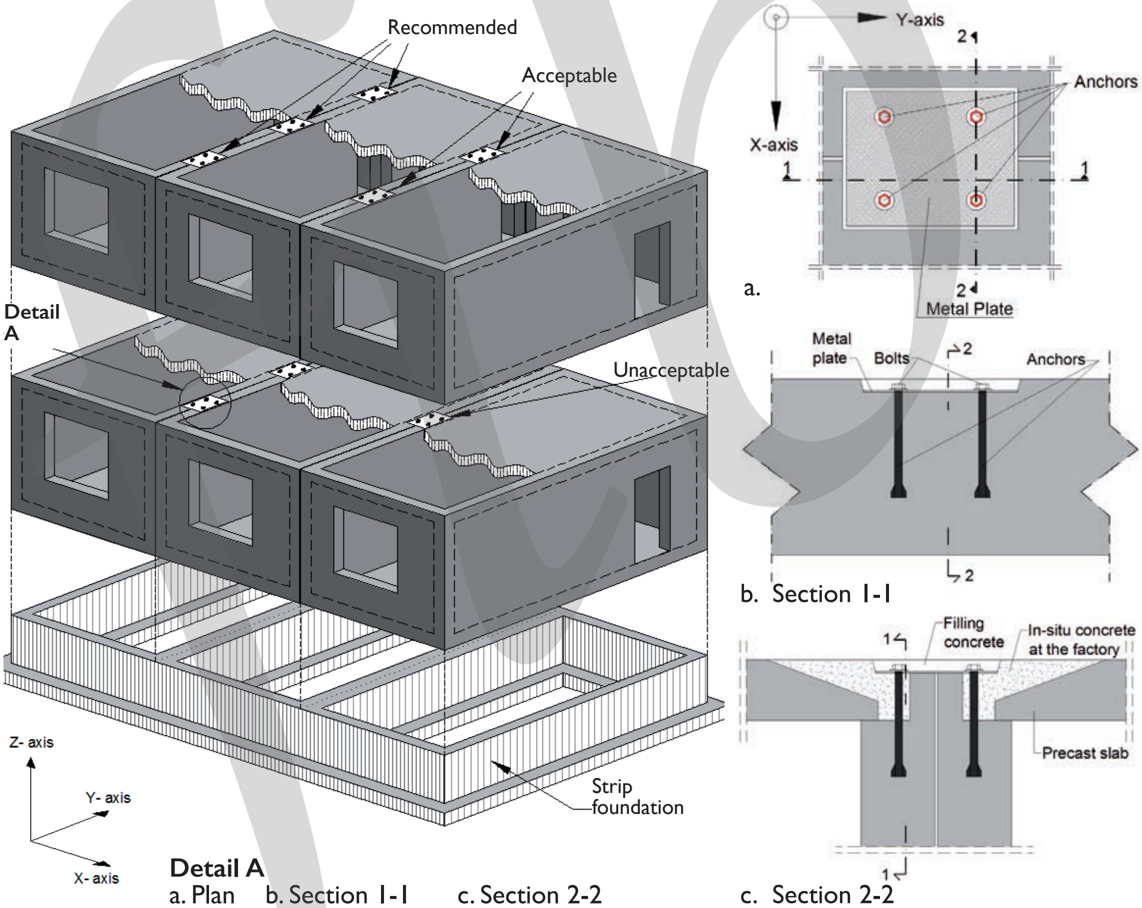


Fig. 10-9 Horizontal connection between closed cell units using dry connections at top of slabs (other reinforcements not shown for clarity) (Manolatos et al., 2003)

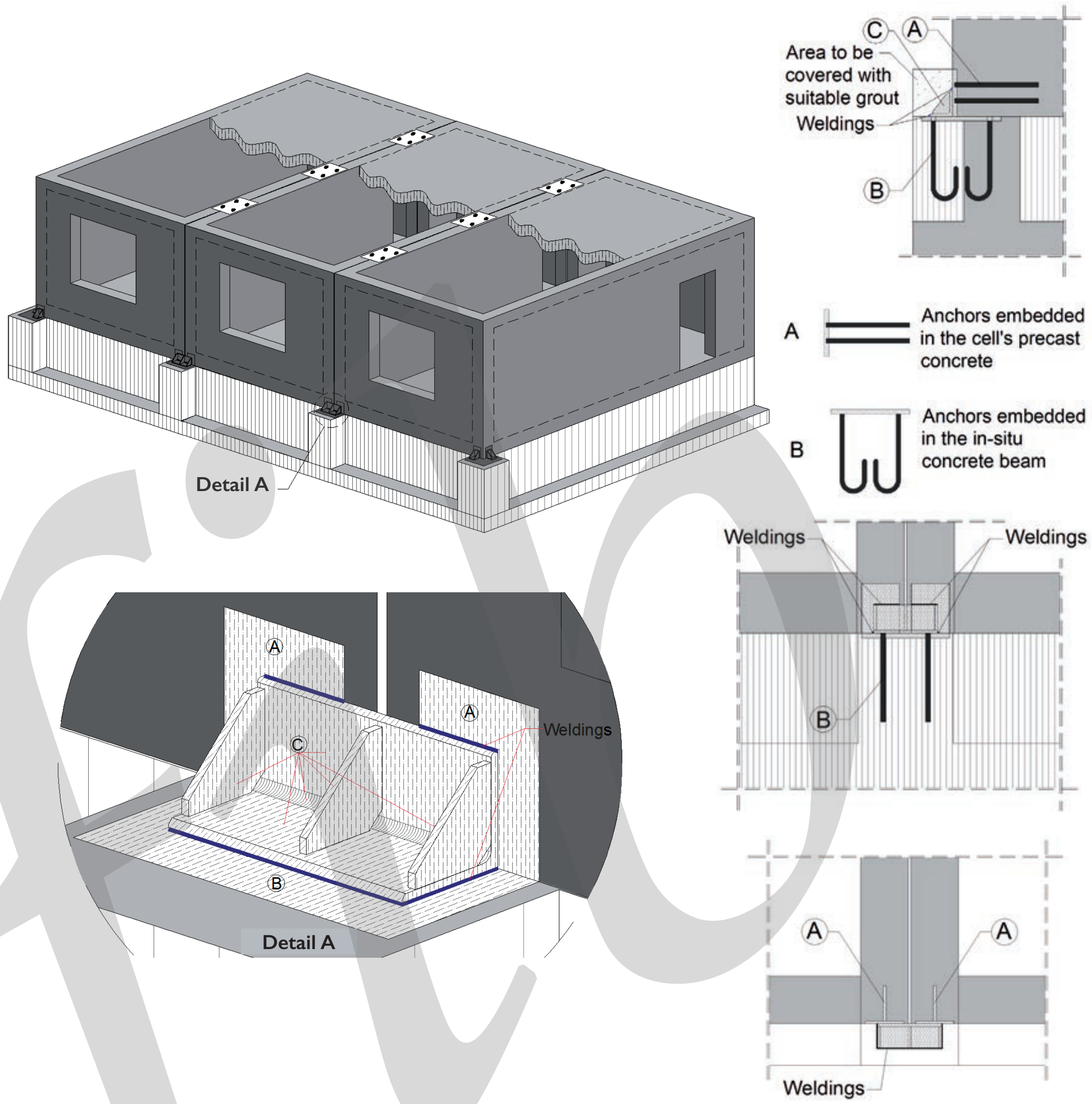


Fig. 10-10 Connections of cell units with strip foundation (other reinforcements not shown for clarity) (Manolatos et al., 2003)

Figure 10-11 shows a simple way to prestress cell units vertically (Manolatos et al., 2003).

Note: The above connections do not necessarily represent the best possible solutions. They simply illustrate methods that have already been implemented. Precasters should develop their own methods based on their experience and on local conditions.

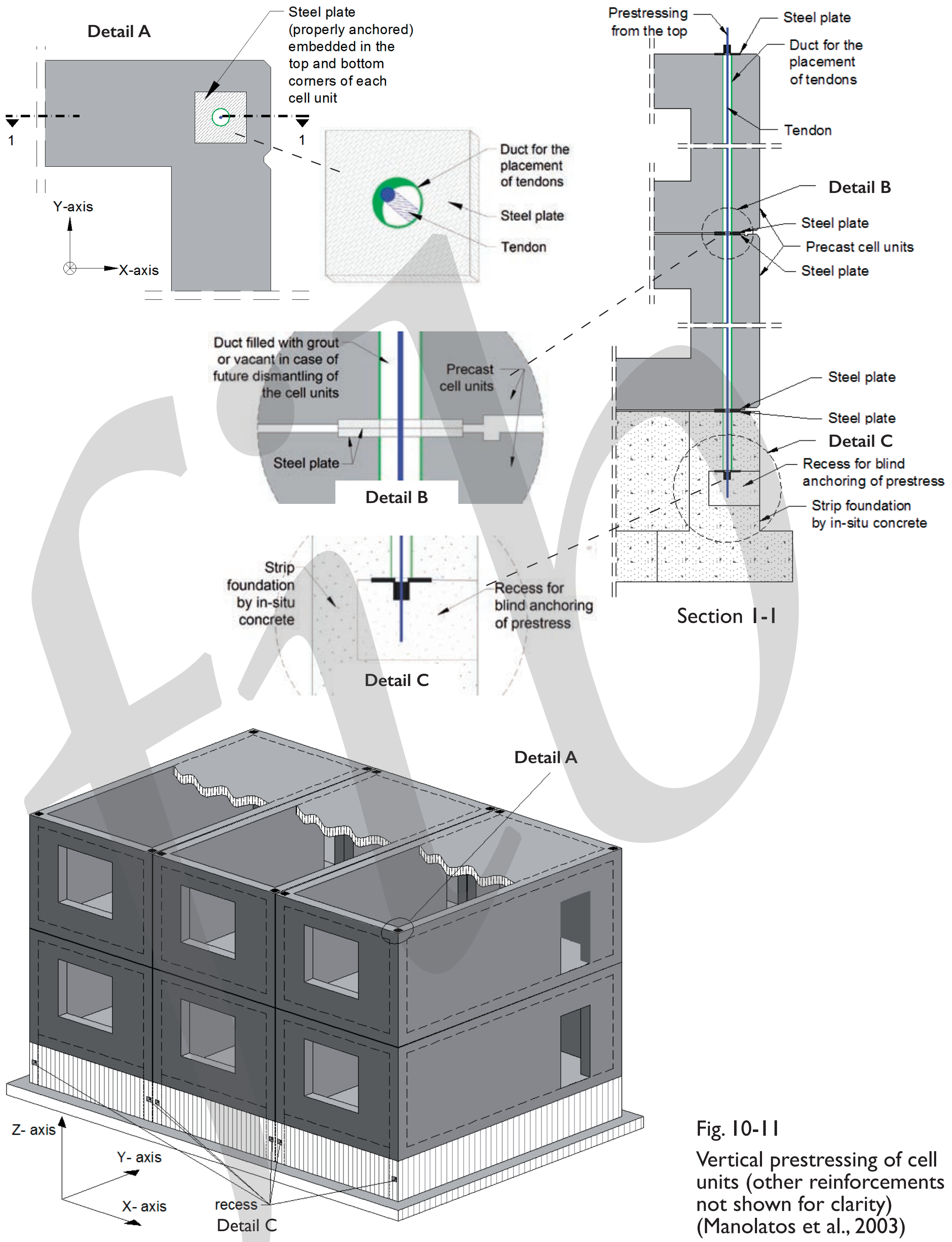


Fig. 10-11
Vertical prestressing of cell units (other reinforcements not shown for clarity) (Manolatos et al., 2003)

10.5 Box structures in the United States

ACI 550.1R-09 maintains that box structures are a special type of building and may fall under the category of walls. Familiar examples of box or cellular structures, shown in Figures 10-12 and 10-13, include stairwells, elevator cores and panel-type multi-storey residential buildings. The overlapping corners shown in Figure 10-13 provide a strong shear component when completed. In particular cases, when the boxes include integral floors or ceilings or both, they are called cells. Even though a large number and variety of buildings that fall in this category have been built in North America, it was primarily the Architectural Institute of Japan (AIJ) that formalized the classification of box structures as a structural system for earthquake-resistant buildings (Suenaga, 1974).

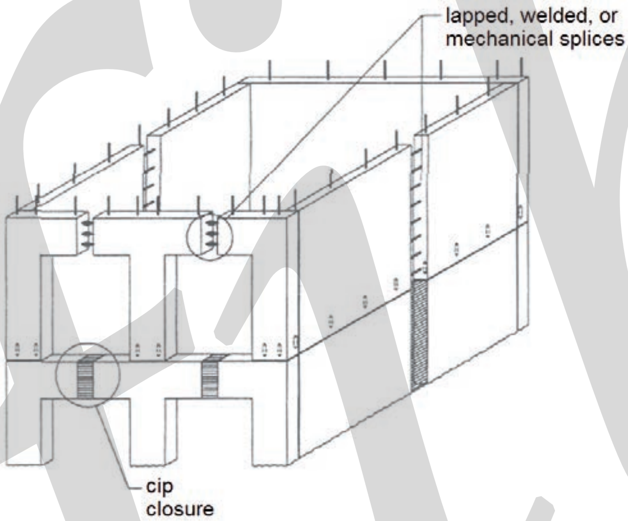


Fig. 10-12
Precast shear tower with mechanical splices and cast-in-situ closure connections between elements
(Authorized reprint from ACI 550.1R-09, 2009)

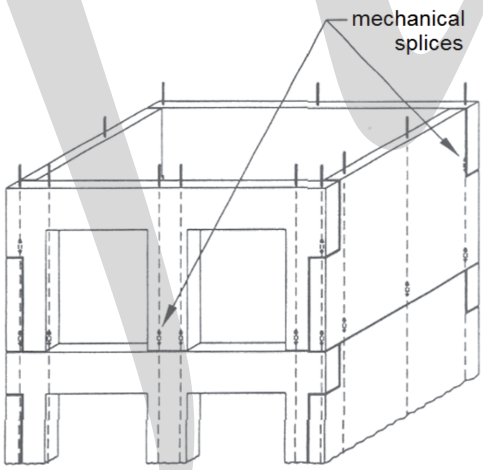


Fig. 10-13
Use of mechanical connections and interlocking precast wall elements to create a monolithic shear tower
Note: Erection sequencing must be coordinated
(Authorized reprint from ACI 550.1R-09, 2009)

Appendix A Structural ductility of precast-frame systems

Appendix A presents basic aspects of the seismic design of precast structures within the wider frame of concrete structures, of which they represent a particular case. The discussion is limited to frame structures made of beams and columns.

The assumption of ductility classes is given only as an example of how to use this resource in the design phase, while the proper definition can be found in the design codes of each country. Similarly, the symbols and nomenclature adopted here are not necessarily identical to those used in design codes.

The energy dissipation capacity, which at the ultimate collapse limit permits the attenuation of the seismic response of the structure with respect to the elastic response, is measured on the basis of its ductility recourses. The ductility displays at three levels:

- local flexural ductility of the critical sections (curvature ductility)
- local traslatory ductility of the single element (a column or beam) (local displacement ductility)
- global traslatory ductility of the entire structure (global displacement ductility)

A.1 Local ductility

The local curvature ductility is measured on the moment-curvature diagram M-χ of the section by the curvature ductility factor μ_x which is the ratio between the ultimate curvature χ_u and the curvature χ_y at the yielding limit (see Fig. A-1):

$$\mu = \frac{\chi_u}{\chi_y} = \frac{\chi_y + \chi_p}{\chi_y} = 1 + \frac{\chi_p}{\chi_y}$$

with χ_p being the plastic curvature of the section.

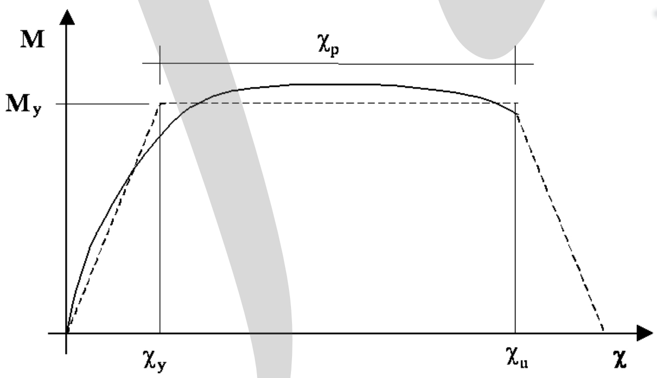


Fig. A-1

The ultimate curvature χ_u , with a sudden fall of the resisting moment, is usually reached with the failure of the confined concrete core, possibly initiated by the failure of some buckled bars at their subsequent straightening. If a relevant strength decay develops before this failure, a conventional early ultimate limit is assumed at 15% decay of the resisting moment, which is the lower decays covered by the γ_s = 1.15 partial factor applied to steel strength in resistance verifications.

For reinforced concrete sections, this factor depends mainly on the steel ductility and reinforcement details that will prevent the early buckling of longitudinal bars and provide the effective confinement of the concrete core. With a uniform steel elongation ε_{uk} of at least 7.5% the following values can be obtained as a function of the spacing s of the stirrups:

$\mu_\varphi \approx 8.0$	for $s = 3.5\ \phi$	$\omega_w \geq 0.16$	enhanced ductility DCE
$\mu_\varphi \approx 6.0$	for $s = 6.0\ \phi$	$\omega_w \geq 0.12$	high ductility DCH
$\mu_\varphi \approx 4.0$	for $s = 8.5\ \phi$	$\omega_w \geq 0.08$	medium ductility DCM

where ϕ is the diameter of the bars and ω_w is the volumetric mechanical ratio of confining reinforcement.

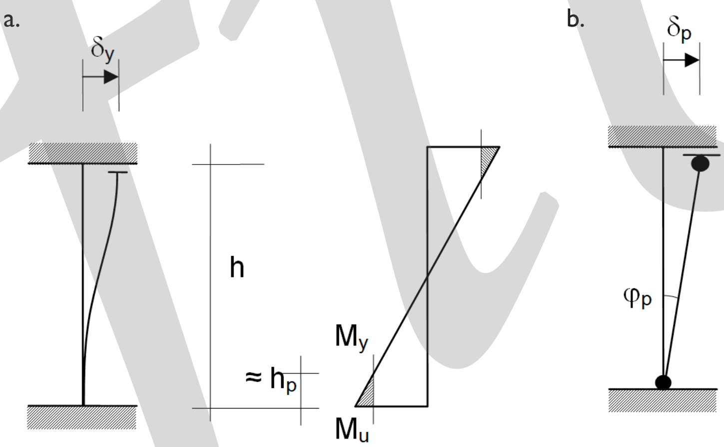


Fig. A-2

The local displacement ductility of a single element is measured by the **displacement ductility factor** μ_δ , which is the ratio between the ultimate displacement δ_u and the displacement δ_y at the yielding limit.

$$\mu_\delta = \frac{\delta_u}{\delta_y} = \frac{\delta_y + \delta_p}{\delta_y} = 1 + \frac{\delta_p}{\delta_y}$$

The column in Figure A-2 being:

$$\delta_y \cong \frac{M_y h^2}{6EI} = \chi_y = \frac{h^2}{6}$$

and assuming a constant distribution of the plastic curvature χ_p over the plastic length h_p inclusive of the tensile strain penetration:

$$\delta_y \cong \varphi_p h = \chi_p h_p h$$

the following is obtained:

$$\mu_\delta = 1 + 6\,\frac{h_p}{h}\,\frac{\chi_p}{\chi_y}$$

This factor depends on the plastic rotation capacity $\varphi_p = \chi_p\, h_p$ of the end regions of the element, as indicated in Figure A-2b. This capacity is proportional to the plastic length h_p as determined by the hardening ratio of the steel f_t / f_y. Taking into account the deeper lever arm of the internal forces at ultimate limit state with respect to the first yielding limit, the relation with the corresponding bending moments can be written as 1.05 f_t / $f_y \approx M_u$ / M_y. For a value of at least f_t / f_y = 1.15, which corresponds to $h_p \approx 0.083h$, the following relation is obtained:

$$\mu_\delta \approx 1 + 0.5\,\chi_p / \chi_y = (1 + \mu_\varphi) / 2 \qquad (\text{with } \chi_p / \chi_y = \mu_\varphi - 1)$$

which leads to the following values:

$\mu_\delta \approx 4.5$	for $\mu_\varphi = 8.0\ \phi$	enhanced ductility DCE
$\mu_\delta \approx 3.5$	for $\mu_\varphi = 6.0\ \phi$	high ductility DCH
$\mu_\delta \approx 2.5$	for $\mu_\varphi = 4.0\ \phi$	medium ductility DCM

A.2 Global ductility

What counts for the seismic capacity is the global ductility of the structure that is measured with the **displacement ductility factor** μ_Δ defined as the ratio between the ultimate displacement d_u and the displacement d_y at the yielding limit:

$$\mu_\Delta = \frac{d_u}{d_y} = \frac{d_y + d_p}{d_y} = 1 + \frac{d_p}{d_y}$$

This factor depends on the ultimate collapse mechanism, as indicated in Figure A-3 for a multi-storey frame. For the more desired mechanism b., which has plastic hinges at the end regions of the beams, the global ductility remains of the same magnitude as the traslatory ductility of the single elements:

$$\mu_\Delta = \frac{n\delta_y + n\delta_p}{n\delta_y} = \mu_\delta$$

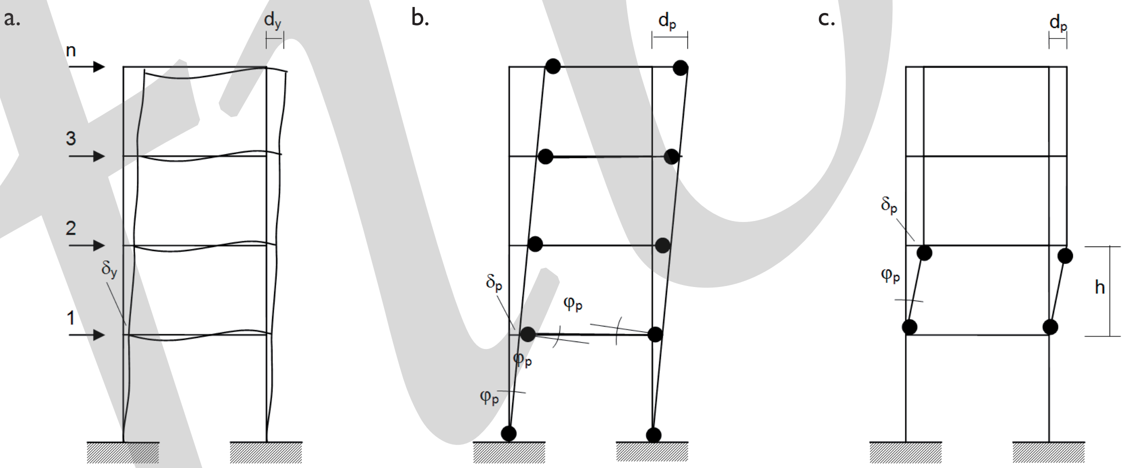


Fig. A-3 a. Deformed shape b. Beam sway
c. Soft-storey or column sway mechanism in multi-storey frame

For the brittle and dangerous mechanism c., with plastic hinges at the end regions of the columns of the 'weak storey', the global ductility decreases drastically with the total number of storeys:

$$\mu_\Delta = \frac{n\delta_y + \delta_p}{n\delta_y} = 1 + \frac{1}{n}\frac{\delta_p}{\delta_y}$$

In the case of the four storeys of Figure A-3 the structural ductility for mechanism c. (with $\delta_p / \delta_y = \mu_\delta - 1$) would decrease to the values:

$\mu_\Delta \approx 1.9$	for $\mu_\delta = 4.5$	enhanced ductility DCE
$\mu_\Delta \approx 1.6$	for $\mu_\delta = 3.5$	high ductility DCH
$\mu_\Delta \approx 1.4$	for $\mu_\delta = 2.5$	medium ductility DCM

This example clearly shows the fundamental importance for multi-storey frames of the capacity design criterion, which is obviously not specific only to precast structures but also applies to cast-in-situ frames. To avoid the disastrous weak storey mechanism, which has led worldwide to many tragic results during earthquakes, the rule of the 'weak-beam-on-strong-column' should be followed. This is obtained by verifying that the strength of the columns is greater than the strength of the beams in the joints of all the intermediate floors.

A.3 One-storey frames

The above rule does not apply for one-storey frames since the two possible mechanisms of Figure A-3 are equivalent, with a global displacement ductility of the structure that is always equal to the local displacement ductility of the elements: $\mu_\Delta = \mu_\delta$

One-storey precast frames with typical hinged connections have a higher elastic flexibility to which, for the same materials, a plastic deformation extended to regions of as higher length is added. The resulting traslatory ductility is the same as for the monolithic frames (see Fig. A-4 and A-5):

$$\delta_y \cong \frac{M_y h^2}{3EI} = \chi_y = \frac{h^2}{3} \qquad \delta_p \cong \varphi'_p h = \chi_p h'_p h$$

$$\mu_\delta = 1 + \frac{\delta_p}{\delta_y} = 1 + 3 \frac{h'_p}{h}\frac{\chi_p}{\chi_y}$$

which, with $h'_p = 2h_p$ and with the same reinforcing steel, leads again to:

$$\mu_\delta = \frac{1}{2} = (1 + \mu)$$

Starting from the same flexural ductilities of the sections, the same values

$$\mu_\delta = \mu_\Delta = 4.5 - 3.5 - 2.5$$

of the local and global displacement ductility factors are obtained respectively for enhanced, high and medium-ductility classes.

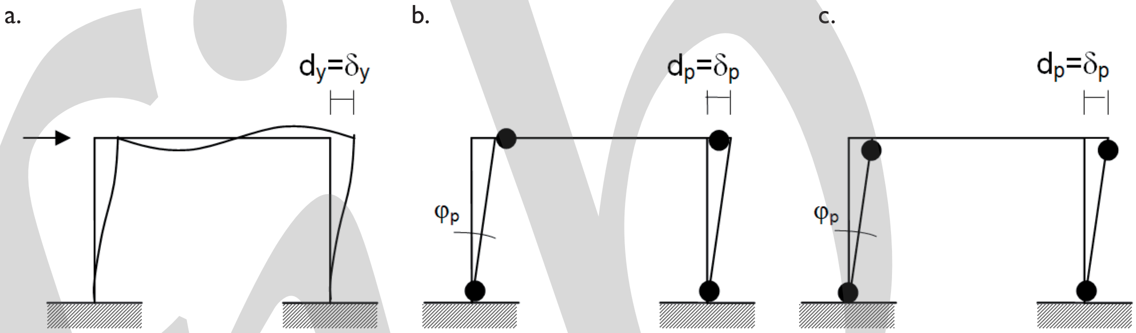


Fig. A-4 a. Deformed shape b. Beam sway
c. Soft-storey or column sway mechanism in one-storey frame

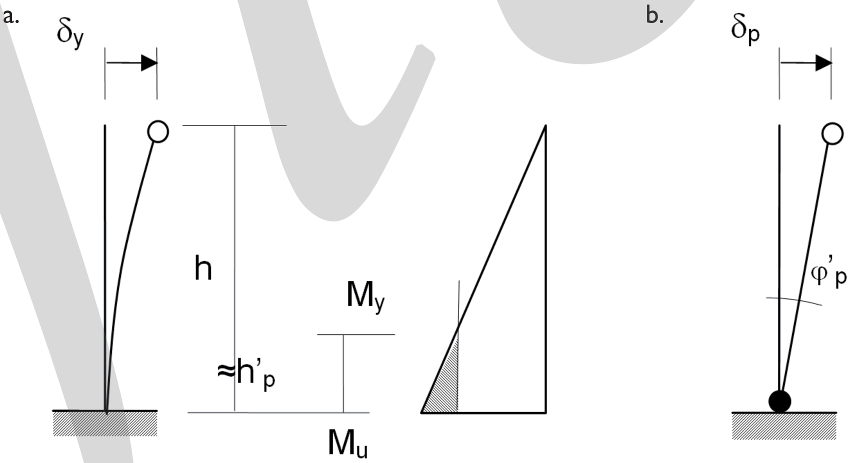


Fig. A-5 Elastic vs. plastic displacement of cantilever system

A.4 Other types of ductility

The discussion presented above refers to the flexural failure of critical sections, with the yielding of reinforcing longitudinal bars, when it is assumed that no early **shear failure** of the elements occurs. In fact, shear failure would be much less ductile and should be avoided in frame structures when following this specific capacity design rule by proportioning the beams and columns in such a way as to allow the shear strength to be greater than the shear force corresponding to the action of the resisting end moments of the element.

The values of the curvature ductility factors here indicated are also conditioned to other requirements like the limitation of **axial compression** forces to low values, indicatively comprised within 0.30 of the corresponding ultimate strength of the section and to 0.30 of the elastic critical load of the element.

The ductile resources of the elements can develop over the structure provided that early brittle failures of the **connections** do not occur. To avoid this possibility the connections of precast frames should be designed in accordance with the following criteria:

- Non-ductile connections located away from the critical regions of the elements should be dimensioned using capacity design with a minimum overstrength factor γ_R = 1.0 - 1.1 - 1.2 for DCM, DCH and DCE respectively for the ultimate action as limited by the critical sections.

- Non-ductile connections located in the critical regions of the elements should be dimensioned using capacity design with a larger overstrength factor γ_R = 1.15 - 1.25 - 1.35 for DCM, DCH and DCE respectively for the ultimate action as limited by the critical sections.

- Ductile connections located anywhere in the structure should possess a stable cyclic deformation capacity with a ductility ratio corresponding to the behaviour factor *q* of the structure and a corresponding energy dissipation capacity.

More specific details on connection type, behaviour and design criteria can be found in the core part of this bulletin.

Appendix B Behaviour factors of precast frame systems

The **behaviour factor** *q*, sometimes referred to as the force reducing factor *R* in other texts, is used in seismic design approaches based on the response spectrum (such as linear static analysis and dynamic modal analysis). It takes into account in an approximate way the attenuation effects of energy dissipation that originates in the ductility resources of the structure. Codes provide various (maximum allowed) values to this factor depending on countries' local traditions and their more or less conservative criteria. Those given in the following clauses are only a mean of the various (maximum allowed) values of different codes. The actual design should of course refer to the code in force in a specific country. Furthermore, as demonstrated in Appendix C, it is important to remember that the actual *q* or *R* factor to adopt depends on the actual ductility demand and would need to be determined through an iterative process or closed-form solution. As a result the actual (design value) behaviour or reduction factor compatible with a specific design can be substantially lower than the maximum value allowed in the code.

B.1 General

Frame systems refer to assemblies of columns and beams that are able to resist load effects mainly by means of bending moments distributed over all the members of the structure. With respect to the action of horizontal forces, sway frames are those that react with the bending moments of their members and the horizontal displacements of their joints; non-sway frames are those that are supported by other structural elements (such as bracing trusses and walls of higher stiffness) able to restrain the horizontal displacements of the joints. For seismic design purposes, the term 'frame system' is used in this appendix to mean a sway frame.

Three main systems are typically used for connections:

- **Dry joints** typically made with mechanical connectors that provide a hinged support able to transfer forces, included those joints completed in situ with mortar for infillings, beddings, and sealings

- **Wet joints** made with reinforcement splices and in-situ concreting that provide a fixed-in support able to transfer forces and moments with a monolithic continuity proper to cast-in-situ structures

- **Bolted joints** made with steel plates and/or sockets that provide a fixed-in support able to transfer forces and moments like the corresponding bolted connections of steel construction.

The three systems of connections lead to different structural arrangements and member sizes. The first system leads to hinged-connection frames (HCFs) with hinged beam-column connections consistent with the features of precast technology (see Fig. B-1). The other systems lead to fixed-connection frames (FCFs) with moment-resisting connections that are similar to monolithic cast-in-situ frames (see Fig. B-2).

Additional and alternative solutions are available, as described in more details in the core of this bulletin, where the concepts of dry joint and ductile connections are combined to provide moment-resistance (fixed-connection solutions), such as dry jointed ductile connections.

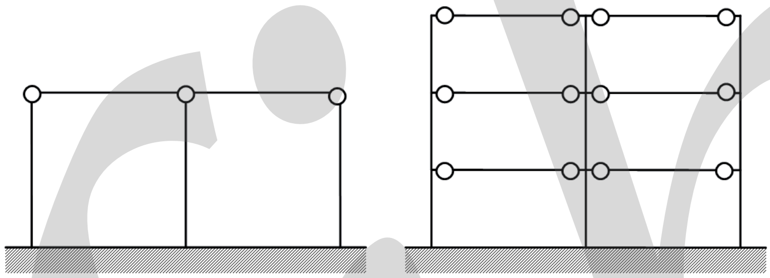


Fig. B-1
Hinged-connection frames (HCFs)

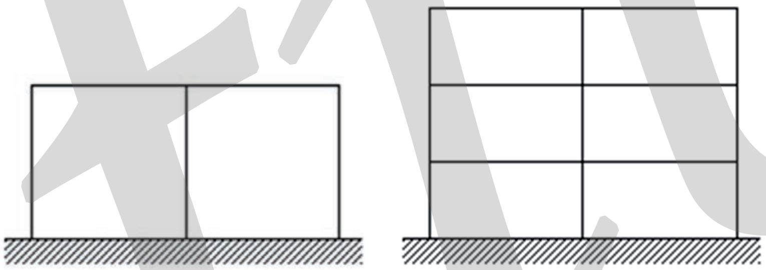


Fig. B-2
Fixed-connection frames (FCFs)

B.2 Ductility-dissipation relation

For the definition of the relation between the ductility resources of the structure and its dissipation capacity under earthquake conditions, different criteria apply depending on the first natural vibration period of the structure.

For flexible structures with a natural vibration period greater than $T_d = cT_C$, (with c = 1.5) the criterion of equal displacement applies, for which, under the same seismic action, a structure q_y times more resistant has an elastic displacement equal to the elastic-plastic displacement of the less resistant structure. Following this criterion, then, the ductility factor would be equal to the behaviour factor (see Fig. B-3a): $q_y = \mu_\Delta$ for $T_1 \geq T_d$

For a perfectly rigid structure, the criterion of equal acceleration applies with no reduction of the seismic action (see Fig. B-3c): $q_y = 1.0$ for $T_1 = 0$

For an intermediate structure, the criterion of equal energy applies (see Fig. B-3b), leading to:

$$q_y = \sqrt{2\mu_\Delta - 1}$$

A comprehensive equation of linear interpolation can be given with:

$$q_y = 1 + (\mu_\Delta - 1)\,\frac{T_1}{T_d} \leq \mu_\Delta$$

For frame systems with $T > T_d$ and with the ductility resources specified in Section A.2 of Appendix A, these criteria lead to a respective behaviour factor $q_y = \mu_\Delta = 4.5 - 3.5 - 2.5$ for enhanced, high and medium-ductility classes, as defined in Appendix A. For T_d the value $c = 1.5$ can be assumed ($T_d = 1.5T_C$).

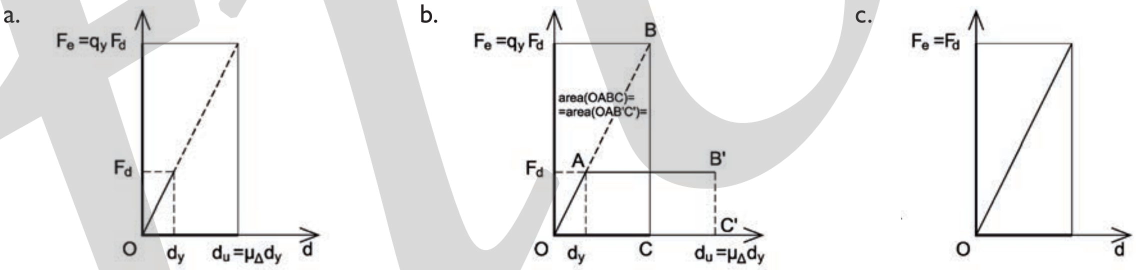


Fig. B-3 a. Equal displacement rule b. Equal energy rule c. Equal acceleration rule

B.3 Standard values

The values of the behaviour factor should be taken from the national regulation valid in the country of construction. When we refer to the prevalent international codes for the behaviour factor of frame precast structures, along with the corresponding codes for cast-in-situ structures, the values in Table B-1 may be used, along with the values of the overstrength factor specified in Table B-2.

Table B-1

All precast frame systems	**DCL**	**DCM**	**DCH**	**DCE**
q_y	1.5	2.5	3.5	4.5

The low ductility class DCL added in Table B-1 applies to structures designed without any reference to the specific criteria and rules of seismic design, except for the class of reinforcing steel.

Table B-2

	α_u / α_1
Frames designed in DCL	1.0
HCFs	1.0
One-storey FCFs	1.1
Multi-storey one-bay FCFs	1.2
Multi-storey multi-bay FCFs	1.3

The force reducing factor to obtain the design response spectrum can be calculated with

$$q = k_p \, q_y$$

where k_p for precast frames is given in Table B-3.

Table B-3

	$\boldsymbol{k_p}$
Frames without roof diaphragm	0.8
Frames with seismic connections	1.0
Frames with no-seismic connections	0.6
Frames designed in DCL	1.0

The seismic connections quoted in Table B-3 are those complying with the requirements of Section A.4 of Appendix A.

Appendix C Design examples of one-storey industrial building

C.1 General

This appendix provides the design example of a single-storey industrial building with:

- a traditional force-based design (FBD) approach without iterating on the initial stiffness assumptions;
- an FBD approach that includes proper iteration on the initial stiffness to account for both:
  - the variation/proportionality of stiffness with strength,
  - the need to define the stiffness as the secant-to-yielding of the equivalent elastoplastic SDOF system;
- a closed-form force-based design (CFBD) approach utilizing a closed-form solution based on the proportionality between a structure's strength and stiffness;
- a displacement-based design (DBD) approach.

The design is carried in accordance with a performance-based design approach that complies with both ULS and SLS requirements as described in Section 3.1 of this document. In this appendix the shortcomings and non-conservative design outcomes caused by improper assumptions on the initial stiffness, or initial period, as well as on the final verification of the displacement demand will be highlighted. The impact of the wide discrepancy between the assumed behaviour factor *q*, or reduction factor *R*, and the actual factor to be used in the design, which represents the actual performance or achieved displacement, will also be considered.

More information about this design example and similar comparative studies can be found in Reyes et al. (2015) and Sporn and Pampanin (2013).

C.1.1 Prototype one-storey industrial building

The prototype building consists of a one-storey industrial building with a plan of 24 × 50 metres (78.74 × 164 feet), comprising five portal frames spaced at 10 metres (32.8 feet) (see Fig. C-1). The portal frame is made up of a 24-metre-long-in-span (78.74 ft.) roof girder with a 7.5-metre-tall (26.6 ft.) precast-concrete column at each end. The base of each precast column is grouted in a socket footing to form a fixed connection. The depth of the roof girder varies along the length, forming a typical triangular or A-shaped element. The height of the single degree of freedom equivalent is taken as 8.22 metres (26.97 feet).

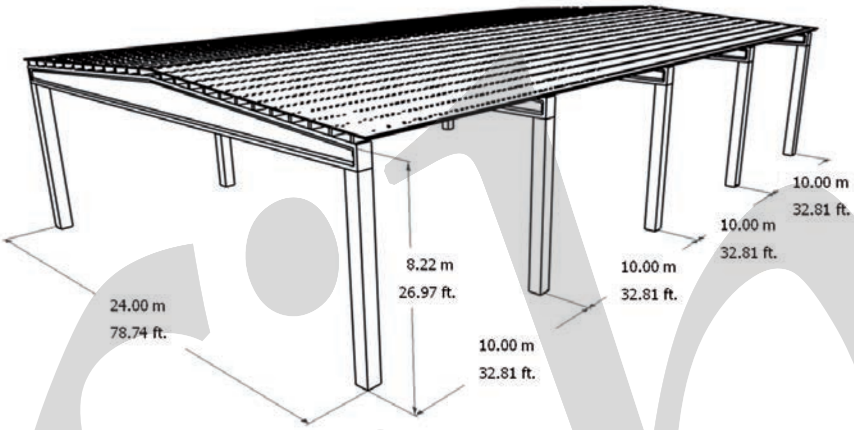


Fig. C-1
Prototype building

To simplify this design example, the roof girder is assumed to be rigid and the total roof mass will be divided evenly between each column, assuming equal lateral displacement. Therefore the design procedure will be conducted for one interior column, as it will correspond identically to the design of one internal portal frame with two columns and double the mass.

To calculate the actions on the structure, permanent (dead) and live loads of 0.2 kN/m2 (4.1 psf) and 0.5 kN/m2 (10.24 psf) respectively have been used. Additional wind loads of q_b = 490 N/m2 (10.1 psf) for terrain category III and snow loads of s = 0.64 kN/m2 (13.11 psf) for zone II in Italy are assumed.

To calculate the seismic masses (see Table C-1) the following load combination is used, where the self-weight of the elements, snow, and permanent loads are included. Approximately 60% of the mass of the vertical façade panels is considered in the seismic load combination.

$$\sum_{j\geq 1} G_{k,j} + P + A_{Ed} + \sum_{j\geq 1} \psi_{2,i} Q_{k,i}$$

Table C-1 Masses per column (internal frame)

Item	Mass (t =1,000 kg = 2.2 kip)	
	t	kip
Gutter beam	2.55	2.55
Roof beam	8.81	8.81
Double-tee beam	9.26	9.26
Columns 500 mm x 500 mm (19.68 in. × 19.68 in.)	2.36	2.36
Panels	21.00	21.00
Snow	3.08	3.08
Permanent load	2.4	2.4
Total	**49.46**	**49.46**

The structural information of the column for design is summarized in the table below.

Table C-2 Prototype column properties

H_e		m		P		f'_c		f_y	
m	ft.	t	kip	kN	kip	MPa	psi	MPa	psi
8.22	26.97	50	110.23	246.6	55.44	35	5076.3	450	65265

C.1.2 Seismic hazard and spectra

Seismic intensity is considered to be associated to a peak ground acceleration of 0.35g. The elastic (5% damped) acceleration and displacement design spectra corresponding to ULS (collapse prevention, 475-year-return period) and SLS (damage control, 95-year-return period) are shown in Figure C-2, which is based on Eurocode 8 (2004) specifications, and soil class B (Table C-3).

Table C-3 Soil class B parameters

Ground type	S	T_B [s]	T_C [s]	T_D [s]
B	1.2	0.15	0.5	2.0

$$0 \leq T \leq T_B : S_\alpha(T) = \alpha_g S\left[1 + \frac{T}{T_B}(2.5\eta - 1)\right] \quad \text{(C-1)}$$

$$T_B \leq T \leq T_C : S_\alpha(T) = \alpha_g S \eta 2.5 \quad \text{(C-2)}$$

$$T_C \leq T \leq T_D : S_\alpha(T) = \alpha_g S \eta 2.5\eta\left(\frac{T_C}{T}\right) \quad \text{(C-3)}$$

$$T_D \leq T \leq 4s : S_\alpha(T) = \alpha_g S \eta 2.5\eta\left(\frac{T_C T_D}{T^2}\right) \quad \text{(C-4)}$$

The (pseudo-)displacement spectra can be directly derived from the acceleration spectra as:

$$S_d(T) = S_\alpha(T)\left[\frac{T}{2\pi}\right]^2 \quad \text{(C-5)}$$

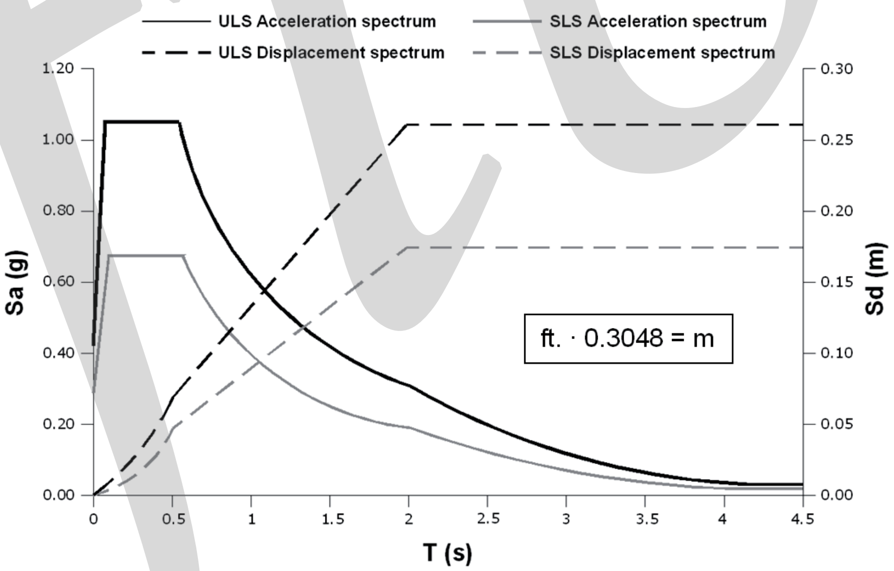


Fig. C-2 Elastic design spectra (acceleration S_α and displacement S_d) for ULS and SLS

The ultimate limit in terms of displacement demand under ULS is set by the sensitivity coefficient θ or a defined maximum allowable displacement. The sensitivity coefficient θ is shown in Equation C-6 below and must be less than 0.1 to meet P-Δ effect requirements.

$$\theta = \frac{P\Delta_d}{M} \leq 0.1 \tag{C-6}$$

A maximum allowable displacement is also set for both the ultimate limit state and the serviceability limit state. The interstorey drift limit under ULS spectrum demand is taken as 2.5% and the limit on SLS spectrum demand is taken as 1% to protect non-structural elements from excessive damage. This gives a ULS maximum displacement of $H_e \times 0.025 = 8.22$ m $\times$ 0.025 m = 0.205 m (0.67 ft.) and an SLS maximum displacement of $H_e \times 0.01 = 8.22$ m $\times$ 0.01 = 0.082 m (3.23 in.).

C.2 Iterative force-based design (FBD) of one-storey industrial building at ULS (collapse prevention)

The design is carried out based on an FBD procedure as it is used in Eurocode 8 (2004) and as it has been integrated where required in the recently adopted Italian Code (NTC, 2009).

C.2.1 Step 1: Evaluating initial period T_i

As mentioned in Section 3.1, when force-based design (FBD) versus displacement-based design (DBD) is discussed in general terms, it is very critical to evaluate the first step of the initial period T_i immediately; different results can be obtained depending on the different allowed methods that are adopted. In this example both code formula approximations (period-height relationships) and estimations from an elastic numerical model (for example, the one created with SAP2000) have been used.

Table C-4 presents a comparison of different results for the estimation of the initial period with three different methodologies and two assumptions on gross (EI) or cracked (EI_{cr}) sections properties. Not surprisingly, the range of variation is very large, between 0.3s to 1.95s, thus a variation of over 600%.

As a starting point, the columns are assumed to have geometrical properties derived from gravity load design, for example, 500 × 500 millimetres (19.68 × 19.68 inches), thus some iterations may be required because of a change in the dimension of the column.

Furthermore, and most importantly, it is worth acknowledging that the base shear and therefore the strength of the column sections are not yet known at the beginning of the design process. This would imply significant variations to the secant-to-yielding stiffness, or in other terms, to the stiffness of the equivalent elastoplastic SDOF system, on which the equivalent displacement rule adopted in a spectrum approach is based.

Table C-4 Different values of estimated initial period T_i depending on method used

	Columns 500 mm x 500 mm (19.68 in. x 19.68 in.)		
	EI **gross sections**	EI_{cr} **cracked sections**	**In accordance with EC8, 2004:**
$C_t\, H^{3/4}$	0.312	0.312	Clause 4.3.3.2.2. (3)
$2\sqrt{(d)}$	1.050	1.483	Clause 4.3.3.2.2. (5)
SAP2000	1.381	1.952	

The model in SAP2000 was created using lumped masses of 36.85 t (81.24 kip) at each column. This mass represents the façade panels, double-tee cover beams, gutter beams, dead load (0.2 kN/m2, 0.028 psi) and snow load. The programme automatically evaluates the mass due to the columns and roof-beam self-weight. The total resulting seismic mass applied to the roof is 50 t (110.23 kip) per column.

C.2.2 Step 2: Calculating spectral ordinates and base shear

Following the equivalent displacement rule assumption, the elastic acceleration spectrum is reduced (divided by) a selected behaviour factor q, or in other codes, reduction factor R (United States) or ductility factor μ (New Zealand).

There is a significant and still controversial debate on what q factor could be used for in precast structures. See Appendixes A and B of this document for a more general discussion, as well as the outcomes of this design process as evidence of the potential non-conservative results of the initial assumptions.

A value of $q = 3$ is adopted as a starting point in this design example. It is fundamental that regardless of the ductility capacity and thus the maximum q factor available, the assumption on the q factor must be checked at the end of the design. As will be shown later, depending on the seismic region either the P-Δ effects or the displacement limits at SLS might end up governing the design. This would effectively require the designer to reiterate as well as discontinue using or reduce the behaviour factor q used in the design. The behaviour factor q is in fact assumed to be either equal to μ, in accordance with the equal displacement rule, or still related (somehow proportionally) to μ, in accordance with the equal energy rule.

Assuming $q = 3$, the base shear and overturning moment are:

$$V_b = S_\alpha\,(T) \times g \times m \tag{C-7}$$

$$S_\alpha\ (1.952\ \mathrm{s}) = 0.0896\ \mathrm{g}$$

$$V_b = 0.0896\ \mathrm{g} \times 9.81\ \mathrm{m/s^2} \times 50\ \mathrm{t} = 44\ \mathrm{kN}\ (9.8\ \mathrm{psi})$$

$$M_b = H_e \times V_b \tag{C-8}$$

$$M_b = 8.22\ \mathrm{m} \times 44\ \mathrm{kN} = 362\ \mathrm{kNm}\ (265\ \mathrm{kip} \times \mathrm{ft})$$

C.2.3 Step 3: Evaluating displacement demand

The evaluation of the inelastic displacement demand is typically carried out by multiplying the evaluated elastic displacement, $d_e(T)$, by the ductility or reduction factor q as shown below:

$$d_m = d_e(T) = 254\ \mathrm{mm} = 0.254\ \mathrm{m}\ (10\ \mathrm{in.})$$

$$d_y = \frac{d_m}{q} = \frac{254\ \mathrm{mm}}{3} = 85\ \mathrm{mm}\ (3.35\ \mathrm{in.})$$

It is important to note that this approximation is correct if the elastic displacement actually corresponds to the yielding displacement of the equivalent SDOF elastoplastic system. In fact, this correspondence is unlikely to occur with the initial assumptions for stiffness and period either when the elastic spectrum or a numerical elastic model subjected to lateral load distribution corresponding to the base shear is used on.

C.2.4 Step 4: Checking ultimate limit state caused by *P-Δ* effects

According to Eurocode 8 (2004), the sensitive coefficient θ, which represents the overturning stability of the structure, should be lower than 0.1 ($\theta \leq 0.1$).

In this case, the excessive lateral displacement of the 500-by-500-millimetre (19.68-by-19.68-inch) column yields to higher values than the limit.

$$\theta = \frac{P_{tot} d_r}{V_{tot} h} \tag{C-9}$$

$$\theta = \frac{246.6 \text{ kN} \times 0.254 \text{ m}}{44 \text{ kN} \times 8.22 \text{ m}} = 0.173$$

To respect $\theta \leq 0.1$ it is necessary to reduce the displacement limit by increasing the base shear (thus directly reducing q), by increasing the base section of the column or by doing both.

In this example the section of the columns have been increased until the criterion is satisfied. Iteration of steps 1, 2, 3 and 4 is therefore required. Tables C-5 and C-6 give the final values obtained using a column section of 600 × 600 millimetres (23.6 × 23.6 inches).

Table C-5 Approximations of elastic period for second iteration

	Columns 600 mm x 600 mm (23.6 in. x 23.6 in.)		
	EI **gross sections**	EI_{cr} **cracked sections**	**In accordance with EC8, 2004:**
$C_t H^{3/4}$	0.312	0.312	Clause 4.3.3.2.2. (3)
$2\sqrt{(d)}$	0.729	1.030	Clause 4.3.3.2.2. (5)
SAP2000	0.971	1.36	

Table C-6 Data for second iteration

Columns 600 mm x 600 mm (23.6 in. x 23.6 in.)									
Seismic zone	V_b		M_b		d_e		d_i		θ
	kN	kip	kNm	kip × ft	mm	in.	mm	in.	
$a_g = 0.35g$	63	13.89	517.86	380	59	2.33	178	7	0.0847

The selection of a column section of 600 × 600 millimetres (23.6 × 23.6 inches) is sufficient to limit the ultimate displacement and associated *P-Δ* effects, thus reducing the sensitive coefficient θ within the limits.

Figure C-3 shows the behaviour of the structure in accordance with the hypotheses used in the FBD approach (including an equal displacement rule). Note that the sections have neither been designed yet nor thus reinforced.

Clearly, if the actual behaviour differs from the assumed one, an iteration process would be required.

a. b.

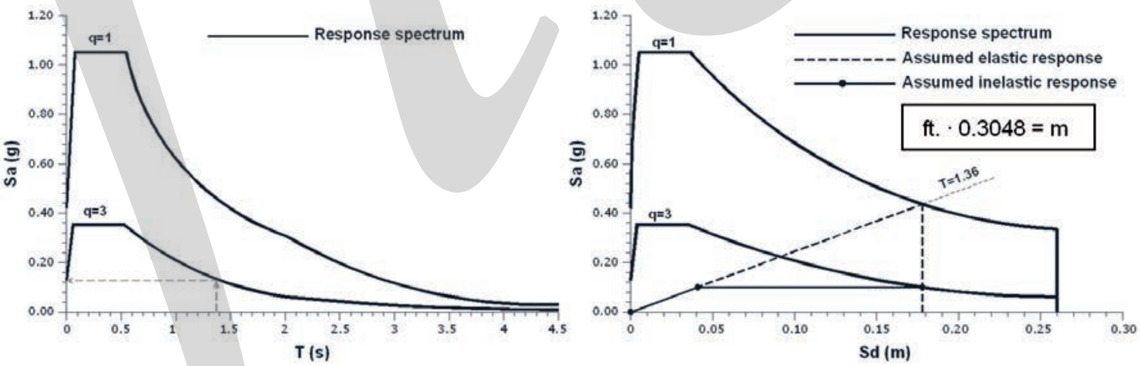


Fig. C-3 Assumed conditions for FBD
a. Acceleration spectrum
b. Acceleration-displacement response spectrum (ADRS)

C.2.5 Step 5: Designing structural elements and critical sections

The column section of 600 × 600 millimetres (23.6 × 23.6 inches) is at this stage designed for required moment and base shear demands. The code limits for transversal and longitudinal steel are:

- a minimum and maximum ratio of longitudinal steel between 1% < ρ < 4%; and
- a minimum spacing of transversal steel, for moderate ductility, equal to $8d_b$ (Eurocode 8, 2004).

The ultimate required capacity (moment and shear) is obtained with a longitudinal steel ratio of ρ = 1.40% and transverse steel bars of 10 millimetres (0.4 inches) in diameter at 80 millimetres (3.15 inches) in spacing, with the layout shown in Figure C-4.

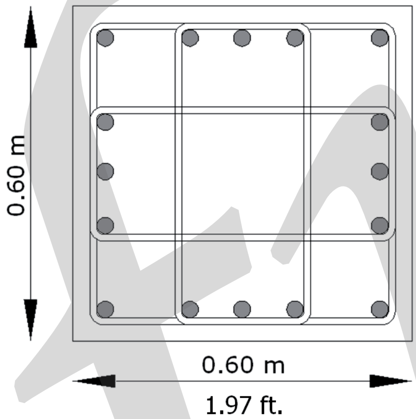


Fig. C-4
Concrete section of 600 × 600 mm (23.6 × 23.6-inch) with rebars 20 mm (0.79 in.) in diameter

C.2.6 Step 6: Checking initial assumptions of stiffness, period, displacement demand and oft-neglected q factor

At this point the force-based design at ULS is too often deemed to have been concluded.

Instead, the actual stiffness (to the yielding of the SDOF elastoplastic system), that is, the actual period to be used when entering the acceleration spectrum should at this stage be checked by using the actual steel reinforcement ratio and by evaluating the secant-stiffness to the yielding of the critical section.

It is worth noting that when assuming, as per traditional approach, a constant initial-yielding stiffness regardless of the strength, this would imply the assumption that the yield curvature is directly proportional to the flexural strength, as shown in Figure C-5a.

However, as shown by Priestley et al. (2007) from detailed analyses and experimental evidence, such an assumption of constant stiffness regardless of strength is invalid. The initial-yielding stiffness of an equivalent bilinear reinforced concrete section is instead proportional to the strength, while the yield curvature is essentially independent of the strength for a given section, as shown in Figure C-5b.

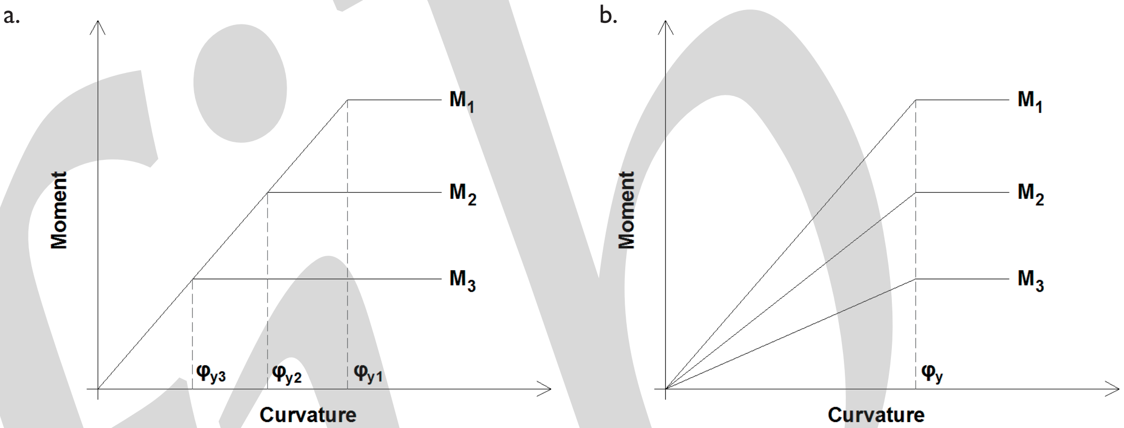


Fig. C-5 Influence of strength on moment-curvature relationships (based on Priestley, 1998, and Priestley et al., 2007)
a. Typical design assumptions
b. More appropriate assumption

The yielding curvature of a reinforced concrete section can be obtained by using any moment-curvature analysis, or more simply, as a valuable approximation, using dimensionless relationships between yielding curvature and section depth. A comprehensive series of analyses (Priestley, 1998; Priestley et al., 2007) demonstrated that the yielding curvature of a section with a varying geometry, reinforcement ratio and axial-load ratio can be approximately estimated as:

$$\Phi_y = k\,(\varepsilon_y / D)$$

where ε_y is the yielding strain of the reinforcing steel, D is the section dimension (depth), and k is a constant varying approximately between 2 and 2.5 and a main function of the section geometry as shown below:

- Circular column $k = 2.25$
- Rectangular column $k = 2.10$
- Rectangular cantilever Wall $k = 2.00$
- T-section beams $k = 1.70$

The yield displacement of a cantilever scheme, such as the column of the prototype building, can be determined using the moment area theorem. If a linear distribution of curvature along the height of the column is assumed, the yielding deflection will equal the area under the curvature diagram times the distance of the centroid to the cantilever end.

$$\Delta_y = \frac{\Phi_y \times H_2}{2} \times \frac{2}{3} H_2 = \frac{\Phi_y \times H_2^2}{3}$$

Alternatively, this could also be obtained by performing a pushover analysis (for one column) to compare against the initial period assumed. Figure C-6 shows a pushover approximation with bilinear elastoplastic behaviour, where the yield deflection is determined as Δ_y = 135 mm (5.32 in.).

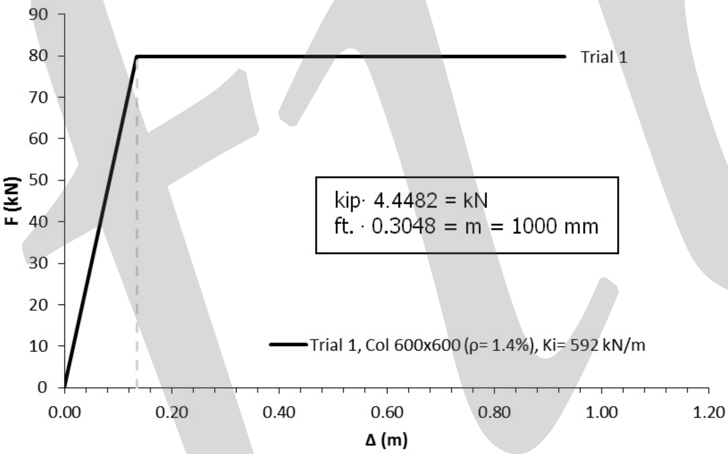


Fig. C-6
Pushover (for one column) of one-storey industrial building with columns 600 mm x 600 mm (23.6 in. x 23.6 in.) in dimension and ρ=1.40%

C.2.7 Step 7: Evaluating actual performance

Figure C-7 (S_a - S_d curve or acceleration-displacement response spectra [ADRS]) shows the comparison of the actual pushover curve and the displacement/ductility demand, with the initial assumptions about the initial stiffness, behaviour factor, displacement and ductility.

The 'actual' initial stiffness is lower (higher period) than that assumed at the beginning of the procedure. The initial period was in fact estimated at T = 1.36s when instead it is T =1.87s. This implies that higher displacements are expected to the point where the ultimate limit-state limits might not be satisfied (sensitivity coefficient greater than 0.1).

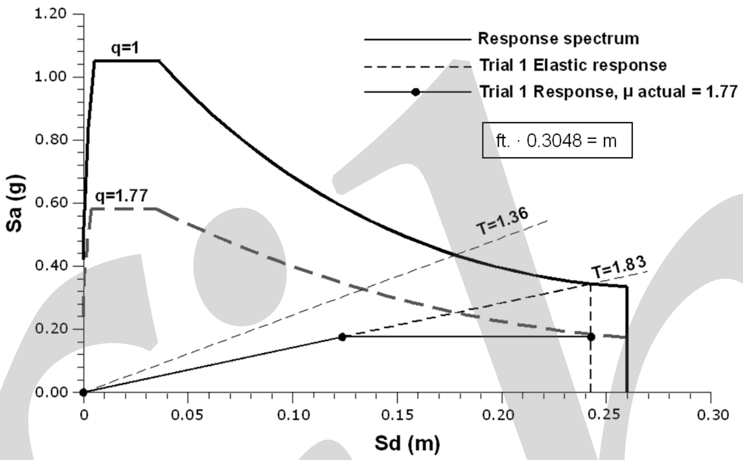


Fig. C-7
Comparison between actual and assumed initial stiffness

According to Calvi et al. (2000), if an inelastic SDOF system with a bilinear force-displacement relationship is given, the (pseudo-inelastic) acceleration spectrum S_a and the displacement spectrum S_d can be calculated as:

$$S_a = \frac{S_{ae}}{\mu}$$

$$S_a = S_{de}$$

This would imply iterating on the q-reduced spectra until the maximum displacement point of the capacity curve intersects at a ductility level $\mu = q$.

As shown in Figure C-7 this process will lead to an estimation of the actual ductility of the structure as approximately μ =1.77. This is very different from the initially assumed ductility of 3 due to the inconsistencies of the strength and stiffness of the initial assumption and would require an iteration to reduce the behaviour factor and, thus, the base shear.

If we know the yield deflection from the pushover analysis and the ultimate displacement from the design spectrum, we can calculate the ductility accurately.

$$\mu = \frac{S_{de}}{\Delta_y} = \frac{0.240}{0.135} = 1.77$$

In this trial, the sensitivity coefficient is calculated at lower than 0.1, meaning the design satisfies the code requirements for *P-Δ* effects. However, the maximum displacement of the structure exceeds the design displacement/drift limit (0.205 m, 8.07 in. or 2.5% drift).

$$\theta = \frac{246.6 \text{ kN} \times 0.240 \text{ m}}{78.10 \text{ kN} \times 8.22 \text{ m}} = 0.09 < 0.1$$

S_{de} = 0.240 m (9.45 in.) (= 2.9% inter-storey drift)

C.2.8 Step 8: Further increase of base shear capacity (column stiffness) until column meets design requirements

An increase in the base shear to reduce the displacement demand and meet the design requirement can be achieved by either increasing the section dimensions or the longitudinal steel ratio, leading in turn to an increase of the section moment capacity and the building base shear. In this case only the longitudinal steel ratio was changed in multiple trials until the design requirements were met. Trials 2 through 4 are shown in Figure C-8.

The elastic period, and therefore the elastic (secant-to-yielding) stiffness, of the second through to the fourth trial is still different from the originally assumed initial period. The higher period of the structure T = 1.54s, or the lower stiffness, still does not match the initial assumptions. However, in Trial 4 the response adequately meets the displacement limit and sensitivity coefficient requirements. The final ductility demand of the structure is μ = 1.33, which is only 44% of the originally assumed ductility demand. It can be noted that, due to the governing displacement limits and the fact that a proper evaluation of the yielding stiffness has been carried out, the actual behaviour factor is q = 1.33, instead of q = 3, as originally assumed. A higher base shear and overturning moment are thus required when compared to the initial assumptions after the stiffness and strength compatibility are met.

Note: This confirms that the initial design, without iterating on the initial stiffness to account for the stiffness-to-strength proportionality, would have led to an non-conservative approach and excessive displacement/deformation/ductility demand in the structural element and overall system.

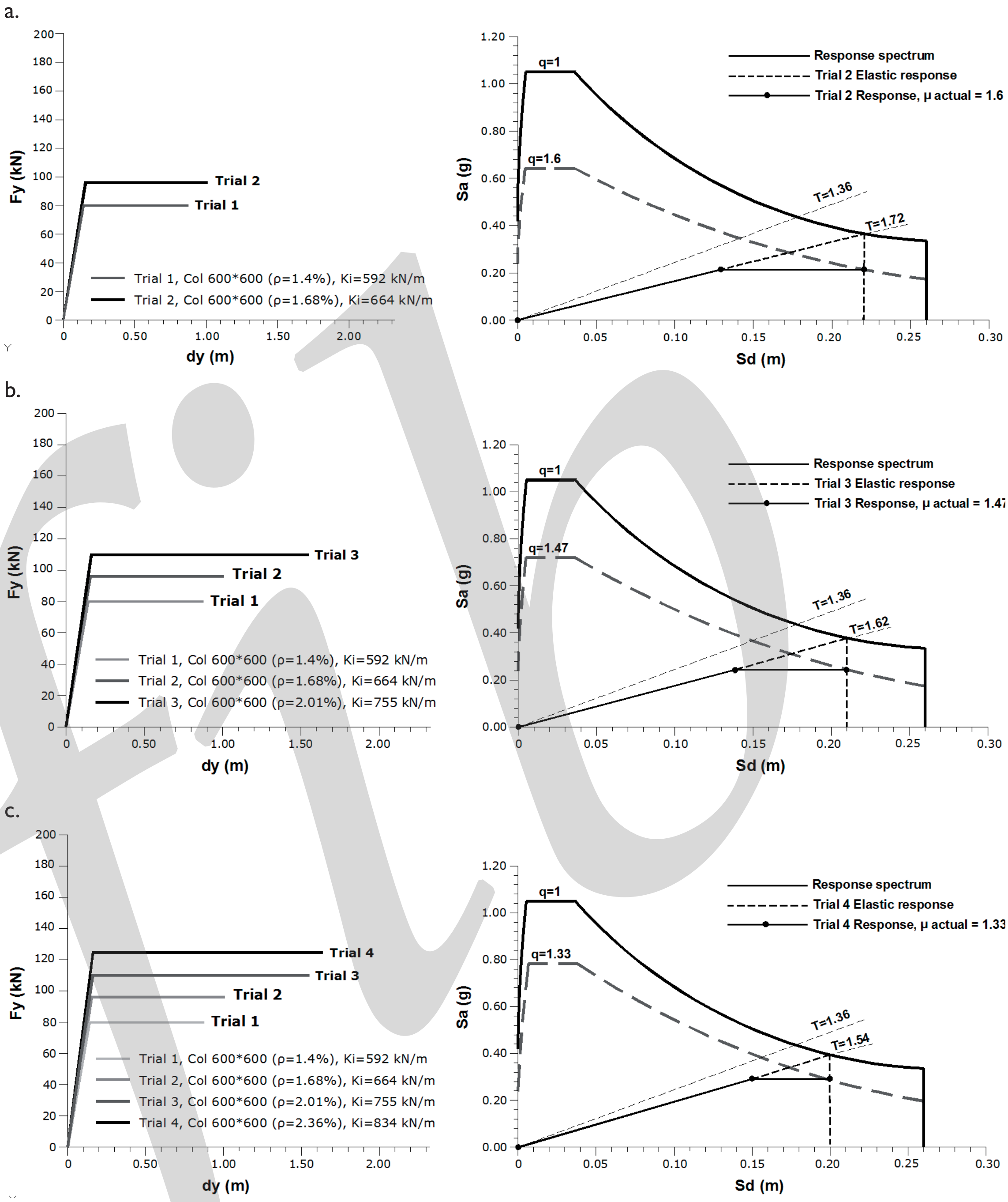


Fig. C-8 Response of Trials 2 to 4 and comparison with initial assumptions
a. Trial 2 pushover results and response in ADRS format
b. Trial 3 pushover result and response in ADRS format
c. Trial 4 pushover result and response in ADRS format

Table C-7 Iteration data

Trial	$F_y = V_b$		d_y		K_i		T_i	M_b		d_m		μ	θ
	kN	kip	m	in.	kN/m	kip/ft	s	kN/m	kip/f	m	in.		
1	80	17.7	0.135	5.31	592	40	1.83	657	486	0.240	9.45	1.77	0.09
2	93	20.5	0.140	5.51	664	45	1.72	764	563	0.224	8.82	1.60	0.07
3	108	24.0	0.144	5.66	755	52	1.62	888	655	0.210	8.27	1.47	0.06
4	125	28.0	0.151	5.95	834	57	1.54	1028	758	0.200	7.87	1.33	0.05

On each column, the base shear and base moment demand are respectively 125 kN (27.6 kip) and 1028 kNm (758 kip × ft). The resultant section design, using this 4th iteration, uses 26-millimetre (1.023-inch) longitudinal bars in the same configuration as shown in Figure C-4.

The overall flow-chart of the force-based design procedure, with iteration on the initial period to account for stiffness and strength compatibility, is shown in Figure C-9 below.

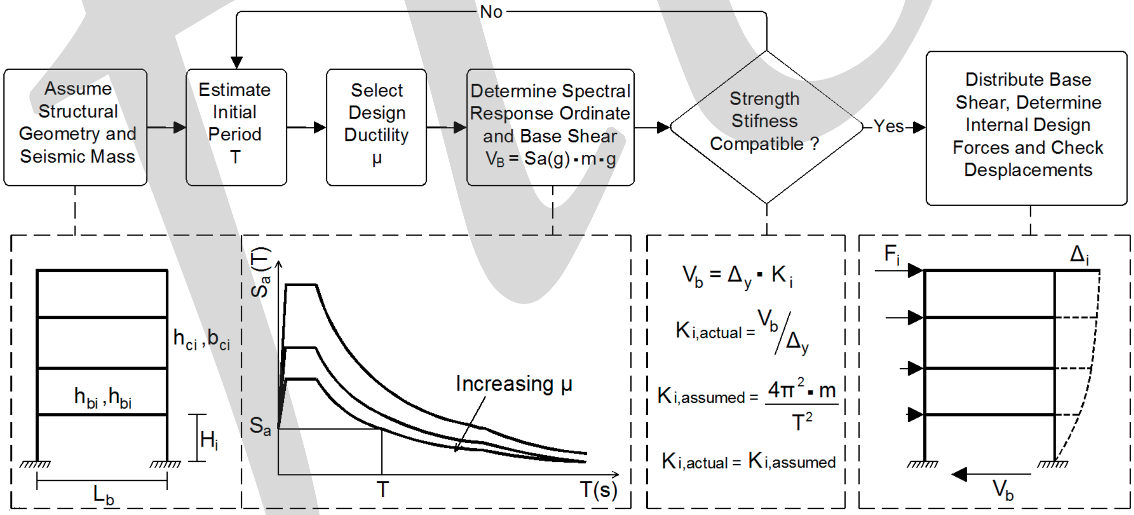


Fig. C-9 Iterative force-based-design method (based on Sporn and Pampanin, 2013)

C.3 Iterative force-based design (FBD) of one-storey industrial building at SLS (damage limitation)

The above process for ULS has to be reproduced for SLS, with the main variation in the process being the maximum displacement allowed at SLS. The Eurocode 8 (2004) case provides an interstorey drift limit of 1% (0.01H).

Figure C-10 shows the interstorey drift limit of 1% and the assumed initial period necessary to satisfy an SLS requirement. Trial 1, which uses the designed section from ULS, does not meet the required displacement limit, which is calculated using the SLS spectrum. Therefore, further iterations are required to obtain a higher yielding stiffness while respecting the maximum allowed reinforcement ratios in the sections. The iterations are shown in Figure C-10 below and the results are summarized in Table C-8.

The column section for each layout is shown in Figure C-11, where each section has 10-millimetre (0.4-inch) stirrups spaced at 80 millimetres (3.15 inches) with a varying section size and longitudinal bar diameter.

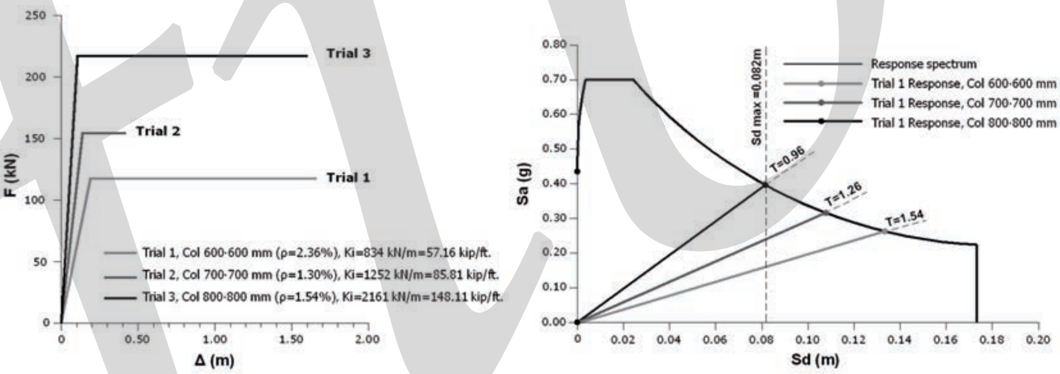


Fig. C-10 SLS trial sections

The final result, obtained once Trial 3 was completed, shows that an 800-by-800-millimetre (31.5-by-31.5-inch) column section with a reinforcement ratio of ρ = 1.54% is required to satisfy the SLS displacement limits. The initial period (secant to yielding of the equivalent elastoplastic system) of the column is T = 0.96 seconds for serviceability limit state requirements and T = 1.54 seconds for ultimate limit state requirements. Therefore, the serviceability limit state controls the overall design.

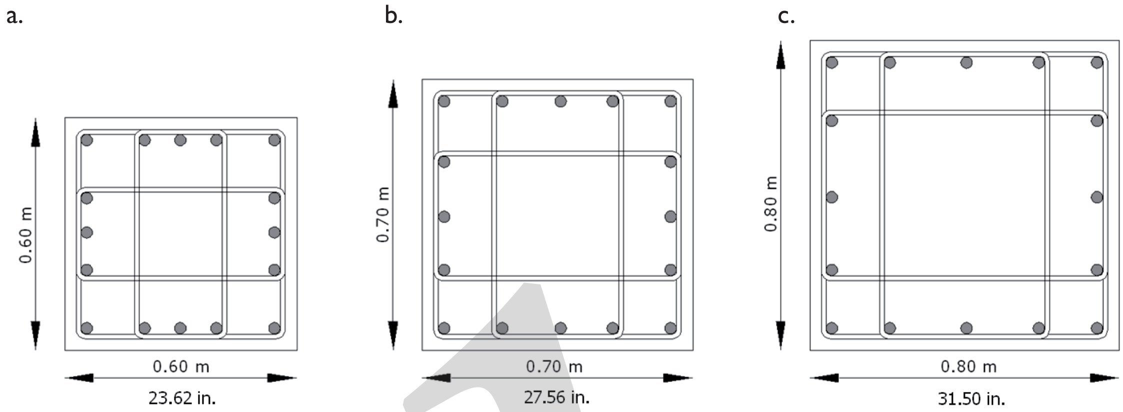


Fig. C-11 SLS trial sections
a. Trial 1: 600 × 600 mm, 10 mm stirrups at 80 mm spacing, 26 mm longitudinal rebars (23.6 × 23.6 in., 0.4 in. stirrups at 3.15 in. spacing, 1.02 in. longitudinal rebars)
b. Trial 2: 700 × 700 mm, 10 mm stirrups at 80 mm spacing, 26 mm longitudinal rebars (27.56 × 27.56 in., 0.4 in. stirrups at 3.15 in. spacing, 1.02 in. longitudinal rebars)
c. Trial 3: 800 × 800 mm, 10 mm stirrups at 80 mm spacing, 28 mm longitudinal rebars (31.5 × 31.5 in., 0.4 in. stirrups at 3.15 in. spacing, 1.1 in. longitudinal rebars)

Table C-8 SLS response iteration

Trial	F_y		d_y		K_i		T_i	V_b		M_b		d_m	
	kN	kip	m	in.	kN/m	kip/ft	s	kN	kip	kN/m	kip×ft	m	in.
1	126	27.78	0.151	5.94	834	57	1.54	69	15.2	567	418	0.135	5.31
2	157	34.61	0.126	4.96	1246	85	1.26	103	22.7	847	624	0.110	4.33
3	214	47.18	0.1	3.94	2161	148	0.96	177	39.0	1736	1280	0.082	3.23

The design must now be analysed or verified for the ultimate limit state with the final design section, as shown in Figure C-12.

The final column design has an initial period T = 0.96 seconds, an initial stiffness K = 2161 kN/m (148 kip/ft.), a ductility demand μ = 1.24, a yield displacement d_y (or Δ_y) = 100 mm (3.94 in.), a maximum displacement d_m (or Δ_d) = 124 mm (4.88 in.), a design base shear V_b = 177 kN (39 kip) and an overturning moment M_b = 1736 KNm (1280 kip × ft.).

In conclusion, after an initial assumption of a behaviour factor q = 3 in accordance with an FBD approach, the design process required a number of iterations to account for the actual stiffness-strength relationship. Moreover, the displacement limits at both ULS (rather than ductility) and SLS governed the design, leading to a significant increase in the section dimensions and of the total reinforcing ratio.

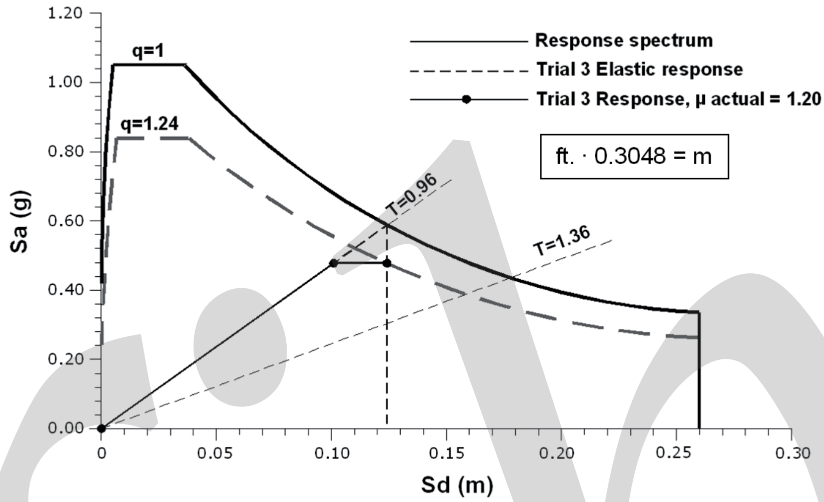


Fig. C-12
ULS final design

C.4 Closed-form force-based design (CFBD) of one-storey industrial building for ULS (collapse prevention)

The same building prototype and seismic hazard assumptions will be used to demonstrate the implementation of a closed-form force-based design (CFBD) approach. The method proposed by Sporn and Pampanin (2013) relies on a closed-form approach to respecting stiffness and strength proportionality, referred to as the compatibility condition, in order to arrive at a feasible design solution that meets the strength and stiffness compatibility of the structure. The method is outlined below and starts from an initial assumption of the design ductility or behaviour factor q = 3, which is typical in an FBD approach.

C.4.1 Step 1: Determining yield deflection of structure

Assuming that the yield curvature ϕ_y is approximately constant, as discussed in Section C.2.6, the yield deflection Δ_y of the structural element, in this case of the SDOF structural system, may also be assumed to be approximately constant (that is, independent of the strength or design base shear). The yield curvature can be calculated as follows:

$$\Delta_y = \frac{\Phi_y \times H_2}{2} \times \frac{2}{3} H_2 = \frac{\Phi_y \times H_2^2}{3}$$

$$\Delta_y = \frac{7.88 \times 10^{-4} \times (8{,}220 \text{ mm})^2}{3} = 177 \text{ mm}$$

$$[\Delta_y = \frac{2 \times 10^{-4} \times \text{in}^{-1} \times (323.62 \text{ in.})^2}{3} = 6.98 \text{ in.}]$$

Note that this value, derived from simplified equations, differs from the result found through a more accurate pushover analysis; however, it is similar to the final design value obtained in the traditional force-based-design section.

C.4.2 Step 2: Determining feasible design solutions using strength-stiffness compatibility-domain curve

The strength-stiffness compatibility domain curve is defined as a function of the yield deflection of the structure, as shown in the relationship below. By plotting this curve, along with the design acceleration spectrum, one can find feasible (compatible) design solutions where compatibility between strength and stiffness is met in the structure.

$$S_a(T) = \frac{4\pi^2}{T^2 g} \Delta_y$$

As shown in Figure C-13, it is apparent that there is no feasible design solution for a ductility value of $q = 3$. The strength-stiffness compatibility-domain curve does not, in fact, intersect the design acceleration spectrum corresponding to $q = 3$ at any location. When plotting different acceleration spectra for a range of design behaviour factors q, it is instead possible to determine a range of feasible (compatible) design solutions.

In this case, in addition to an elastic design ($q = 1$), one feasible design solution is available that corresponds to a design ductility of $q = 1.25$. The compatibility-domain curve intersects the acceleration spectra for $q = 1$ and $q = 1.25$ while never actually crossing the spectrum for $q = 1.5$, which is therefore assumed to be an upper bound of feasible/compatible q factor. In other terms, a design solution for design ductilities of 1.5, 2 or 3 and an associated behaviour factor would not be possible. The solution for $q = 1.25$ is found at a design period T of 1.71 seconds and a design spectral acceleration of $S_a = 0.26$ g. This confirms that a design assumption of behaviour factor $q = 3$ would be, in this case, incorrect and non-conservative.

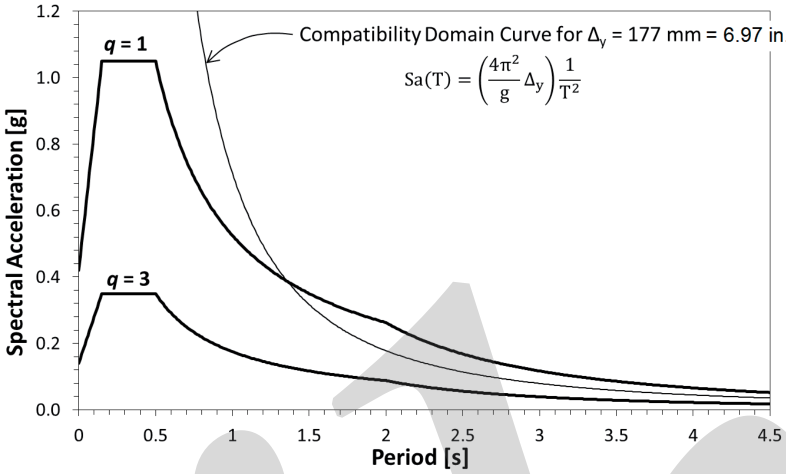


Fig. C-13
Compatibility-domain curve and acceleration design spectra

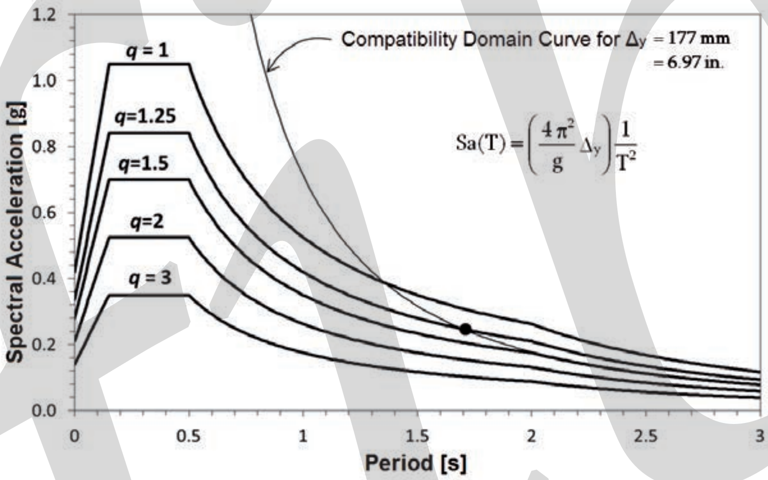


Fig. C-14
Graphic visualization of feasible design solutions (in this case, only one for q = 1.25)

C.4.3 Step 3: Determining design seismic base shear and verifying sensitivity coefficient

The design base shear is then calculated as:

$$V_E = S_a \times g \times \mathrm{m} = 0.26 \times 9.81 \times 50 = 127.53 \text{ kN (28.11 kip)}$$

The maximum design displacement is determined by plotting the design spectra on an acceleration-displacement-response spectral domain.

From the ADRS plot, the maximum displacement for the structure for a design ductility/behaviour factor of q = 1.25 is approximately d_m = 0.21 metres (8.27 in.), which is within an acceptable range for the code displacement/drift limit discussed previously. Next, the sensitivity coefficient must be calculated in order to determine if the design meets P-Δ requirements.

$$\theta = \frac{246.6\ \text{kN} \times 0.22\ \text{m}}{127.5\ \text{kN} \times 8.22\ \text{m}} = 0.051 < 0.1$$

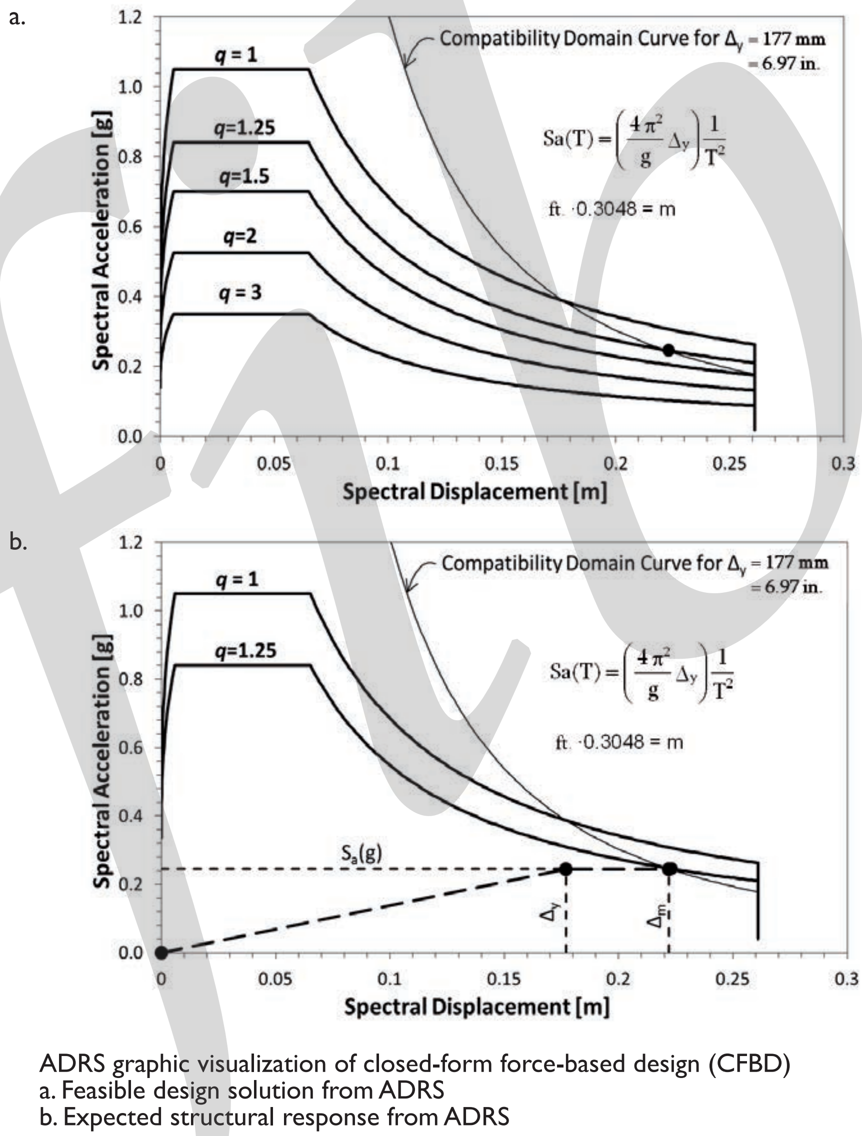


Fig. C-15 ADRS graphic visualization of closed-form force-based design (CFBD)
a. Feasible design solution from ADRS
b. Expected structural response from ADRS

This closed-form design method produces results in satisfactory agreement to those found with an iterative force-based design until the strength and stiffness compatibility condition was met, only in a much more efficient way. Furthermore, graphic visualization can be a valuable aid to designers for controlling their design choice.

Figure C-16 provides the order of the design steps in the CFBD method.

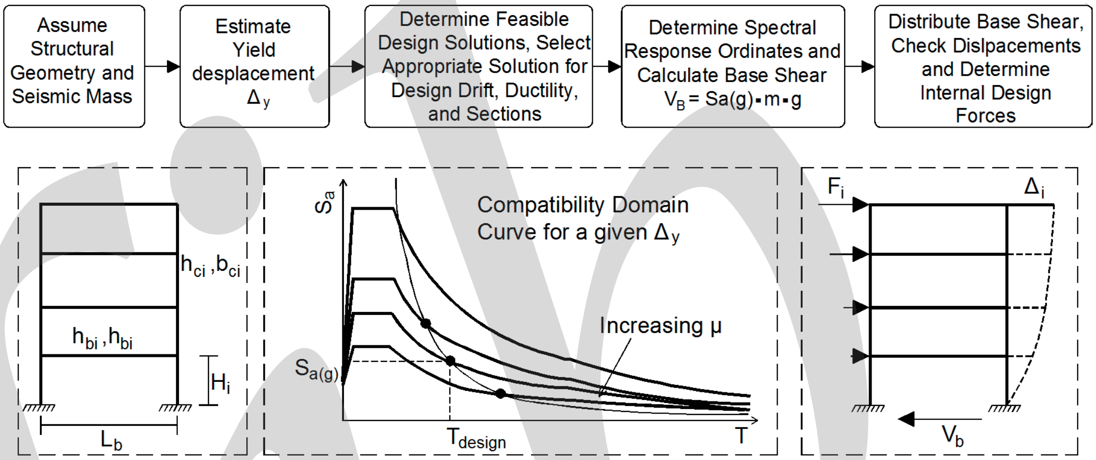


Fig. C-16 Sequence of design steps in closed-form force-based design (CFBD) (Sporn/Pampanin, 2013)

C.5 Closed-form force-based design (CFBD) of one-storey industrial building for SLS (damage limitation)

The same process that was followed above for ULS has to be reproduced for SLS, with the main difference being the maximum displacement allowed at SLS. In the Eurocode 8 (2004) case, an interstorey drift limit of 1% (0.01H) is considered. At this interstorey drift level the maximum displacement at serviceability must be less than 82 millimetres (3.2 inches).

The serviceability limit state can easily be checked by plotting the serviceability acceleration spectrum on top of the acceleration-displacement response spectrum developed in the ULS design approach.

It is apparent from the ADRS in Figure C-17 that the structure will reach a displacement of approximately 150 millimetres (5.9 inches) for the SLS design. This is approximately double the maximum deflection allowed by code, therefore the serviceability limit state will govern and the design process will need to be repeated.

An 800-by-800-millimetre (31.5-by-31.5-inch) column with a yield deflection of 100 millimetres (3.94 inches), determined in the pushover analysis, was evaluated next. Similarly to the previous ULS design, the acceleration spectra were plotted for multiple design ductilities or behaviour factors in order to determine a feasible design solution. Both ULS and SLS spectra were plotted in ADRS format to provide a synoptic view of the design at two levels. This will allow designers to simultaneously determine feasible design solutions that respect both SLS and ULS requirements.

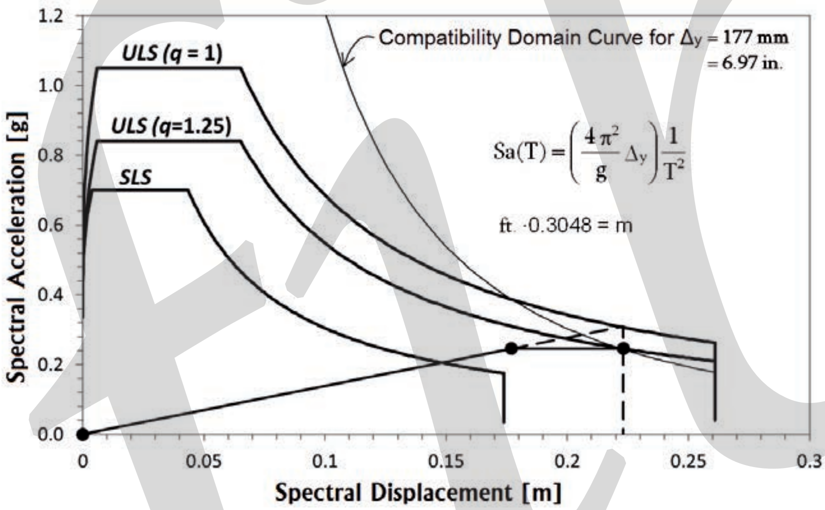


Fig. C-17
Serviceability limit state check for CFBD

It is apparent in Figure C-18 that feasible design solutions are available for behaviour factors of q = 1.25, 1.5 and 2 for a structure with a yield deflection Δ_y = 100 millimetres (3.94 inches). However, only the solution corresponding to a behaviour factor q = 1.25 meets the SLS requirement of 82 millimetres (3.23 inches) of maximum deflection (1% drift). This can be determined by looking at the elastoplastic response in ADRS format and determining where it crosses the serviceability acceleration spectrum. In this case the structure will reach a displacement of approximately 80 millimetres (3.15 inches) under a serviceability-level earthquake.

In Figure C-18 the final column design has an initial period of T = 0.97 seconds, a ductility demand μ = 1.25, a yield displacement d_y (or Δ_y) = 100 millimetres (3.94 inches), a maximum displacement d_m (or Δ_d) = 120 millimetres (4.73 inches), a design base shear V_b = 177 kN (39.8 kip) and an overturning moment M_b = 1,455 KNm (1070 kip × feet).

The result is very similar to the final ULS design found from iterating a traditional force-based design approach. However, the need for iteration was reduced significantly by through the use of this method.

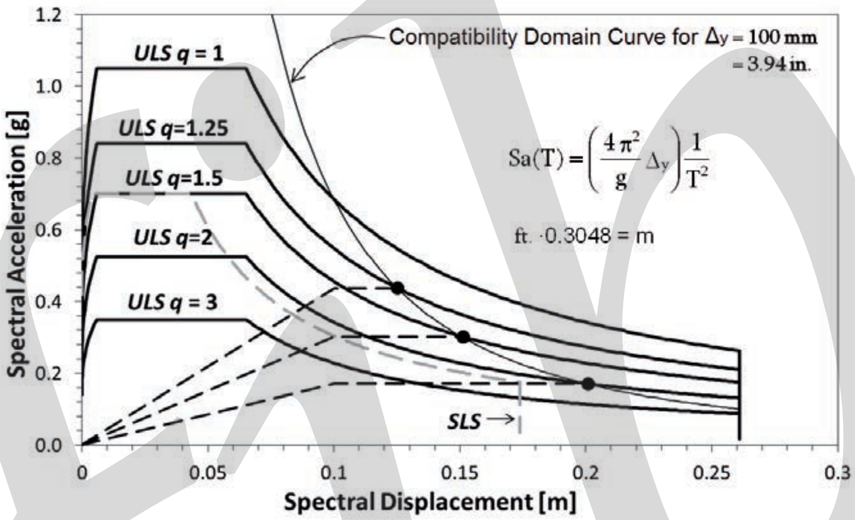


Fig. C-18
Simultaneous design for SLS and ULS using CFBD

C.6 Displacement-based design (DBD) of one-storey industrial building for ULS (collapse prevention)

This example of the DBD approach uses the same building prototype and seismic hazard assumptions. The results are compared with the previously presented FBD approach.

C.6.1 Step 1: Equivalent SDOF system

A DBD first step generally requires the simplification of the multi-degree-of-freedom (MDOF) structure into a simple-degree-of-freedom (SDOF) system, with effective seismic mass, height and stiffness (secant to the target displacement) and the choice of the material properties and section geometry (see Table C-9). In this example the system is already an SDOF, which simplifies the process.

The starting section geometry is assumed to be 600 × 600 millimetres (23.6 × 23.6 inches), as per the initial assumption in the FBD example, for reasons of comparison.

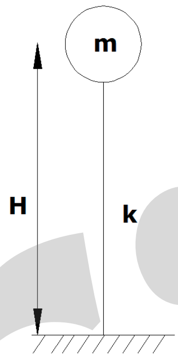


Fig. C-19 SDOF simplification

Table C-9 SLS response iteration

m_e	50.0	t	11.23	kip
P	246.6	kN	55.4	kip
H_e	8.22	m	26.97	ft.
d_{bl}	26	mm	1.02	in.
f_{yk}	450.0	N/mm^2	65.27	kip/in.
f_u	585.0	N/mm^2	84.85	kip/in.
k	0.006			

C.6.2 Step 2: Setting ultimate (target) displacement and calculating yielding displacement, ductility and equivalent viscous damping

A maximum interstorey drift of 2.5% (or maximum displacement d_m [or Δ_d] = 0.2055 metres [8.1 inches]) is set at the beginning of the design. This will eliminate the need for iteration to respect the displacement limits.

The next equations, reproduced from Priestley et al. (2007), describe the steps to finding the above-mentioned values.

$$L_{sp} = 0.022\, f_{yk}\, d_{bl} \tag{C-10}$$

$$L_{sp} = 0.022 \times 450 \text{ MPa} \times 26 \text{ mm} = 257.4 \text{ mm (10.13 in.)}$$

$$\Phi_y = 2.1 \times \frac{\varepsilon_y}{h_c} \tag{C-11}$$

$$\Phi_y = 2.1 \times \frac{0.0023}{600} = 0.000007875$$

$$\Delta_y = \Phi_y \times \frac{(H + L_{sp})^2}{3} \tag{C-12}$$

$$\Delta_y = 0.000007875 \times \frac{(8{,}220 \text{ mm} + 257.4 \text{ mm})^2}{3} = 188.6 \text{ mm (7.43 in.)}$$

$$\Delta_u = \Delta_y + \Delta_p \tag{C-13}$$

$$\Delta_u = 205.5\text{mm (8.09 in.)}$$

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{C-14}$$

$$\mu = \frac{205.5 \text{ mm}}{188.6 \text{ mm}} = 1.09$$

$$\xi_{eq} = 0.05 + 0.444 \times \left(\frac{\mu - 1}{\mu\pi}\right) \tag{C-15}$$

$$\xi_{eq} = 0.05 + 0.444 \times \left(\frac{1.09 - 1}{1.09\pi}\right) = 0.0617$$

C.6.3 Step 3: Entering displacement spectrum and evaluating effective period and stiffness (secant to target displacement)

The base shear and base moment of each column is calculated in this step.

The displacement spectrum is reduced by a displacement reduction factor (EC8, version 1998) that is a function of the equivalent viscous damping expected from the SDOF at the target displacement Δ_d. Damping-ductility relationships, depending on the structural systems and the associated hysteresis rule (Priestley et al., 2007), are used to define the equivalent viscous (area-based) damping.

$$\eta = \sqrt{\frac{7}{2 + \xi_{eq}}} \geq 0.55 \tag{C-16}$$

$$\eta = \sqrt{\frac{7}{2 + 6.17}} \geq 0.92$$

The effective period can be evaluated by entering the reduced displacement spectrum as Δ_d.

$$\Delta_d = 1.72 \text{ s}$$

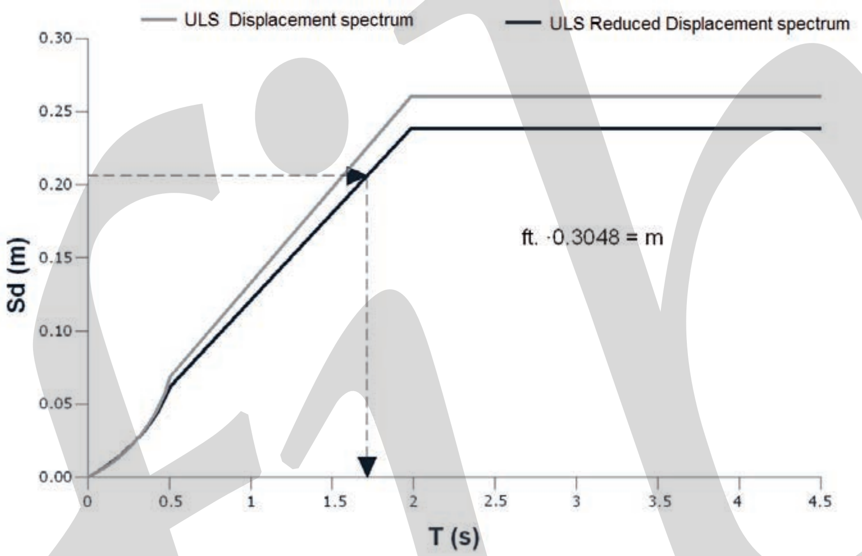


Fig. C-20
Serviceability limit state check for CFBD

The effective stiffness and base shear and moment of the structure can then be calculated as:

$$K_{eff} = \frac{4\pi^2 m}{T_{eff}^2} \tag{C-17}$$

$$K_{eff} = \frac{4\pi^2 50 \text{ tons}}{1.72^2} = 667 \text{ kN/m (45.7 kip/ft)}$$

$$V_b = \Delta_d \times K_{eff} \tag{C-18}$$

$$V_b = 0.2055 \text{ m} \times 667 \text{ kN/m} = 137.11 \text{ kN (30.8 kip)}$$

$$M_b = H_e \times V_b \tag{C-19}$$

$$M_b = 8.22 \text{ m} \times 137.11 \text{ kN} = 1{,}127.08 \text{ kNm (830 kip} \times \text{ft.)}$$

C.7 Displacement-based design (DBD) of one-storey industrial building for SLS (damage limitation)

C.7.1 Step 1: Equivalent SDOF building

The ULS uses the same values as the SDOF that is described above. Only the value for the diameter of the bars was altered and increased to 30 millimetres (1.18 inches).

C.7.2 Step 2: Setting ultimate (target) displacement and calculating yielding displacement, ductility and equivalent viscous damping

The equations described above should be used for the DBD with an SLS target displacement of 82.2 millimetres (3.24 inches) (1% drift) and column sections of 800 × 800 millimetres (31.5 × 31.5 inches).

$$L_{sp} = 0.022 \times 450 \text{ MPa} \times 30 \text{ mm} = 297 \text{ mm (11.7 in.)}$$

$$\Phi_y = 2.1 \times \frac{0.0023}{800} = 0.00000591$$

$$\Delta_y = 0.00000591 \times \frac{(8{,}220 \text{ mm} + 297 \text{ mm})^2}{3} = 142.8 \text{ mm (5.62 in.)}$$

$$\Delta_u = 82.2 \text{ mm (3.24 in.)}$$

$$\mu = \frac{82.2 \text{ mm}}{142.8 \text{ mm}} = 0.576$$

$$\xi_{eq} = 0.05$$

Since the structure remains elastic ($\mu = 0.576 < 1$) the equivalent viscous damping has been taken to be 5%.

C.7.3 Step 3: Entering displacement spectrum and evaluating effective period and stiffness (secant to target displacement)

The elastic displacement spectrum has to be used because $\eta = 1$.

$$\eta = \sqrt{\frac{7}{2+5}} = 1$$

Figure C-21 shows the effective period $T_{eff} = 1.06\text{s}$ for a displacement of 82.2 millimetres (3.24 inches). Thus, the effective (secant to target displacement) stiffness and base shear and moment are calculated as shown below:

$$K_{eff} = \frac{4\pi^2 50 \text{ tons}}{0.96^2} = 2{,}142 \text{ kN/m (146 kip/ft)}$$

$$V_b = 0.0822 \text{ m} \times 2{,}142 \text{ kN/m} = 176.06 \text{ kN (39.5 kip)}$$

$$M_b = 8.22 \text{ m} \times 176.06 \text{ kN} = 1447.2 \text{ kNm (1067 kip} \times \text{ft)}$$

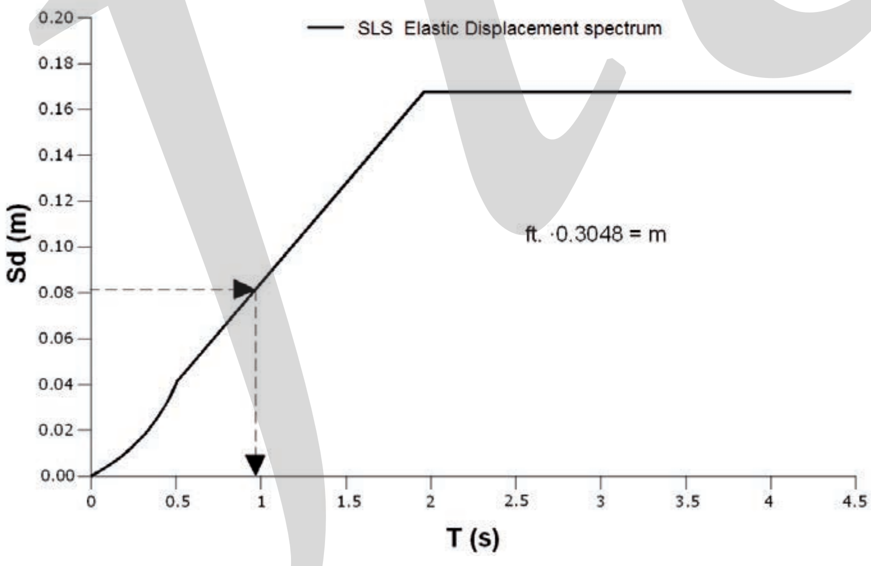


Fig. C-21
Elastic displacement spectrum and effective period for SLS

C.8 Comparison of FBD, closed-form FBD and DBD

Figure C-22 compares the different design base-shear values obtained from the alternative design procedures, namely FBD (initial period assumptions without iteration), iterative FBD or closed-form CFBD, and DBD, which were described in Section C.2 to C.7 for the specific seismic intensity case of a_g = 0.35g.

It is worth noting that when comparing the results for the iterative FBD (or closed-form CFBD) and the DBD the differences of base shear and base moment are less than 3% for all cases.

The more important and non-conservative differences are observed when comparing a traditional non-iterative FBD (reference level set at 100%) against an iterative (closed-form) FBD and a DBD. Design base shear and moment differ markedly from traditional and non-iterative FBD, which leads to very non-conservative design outcomes.

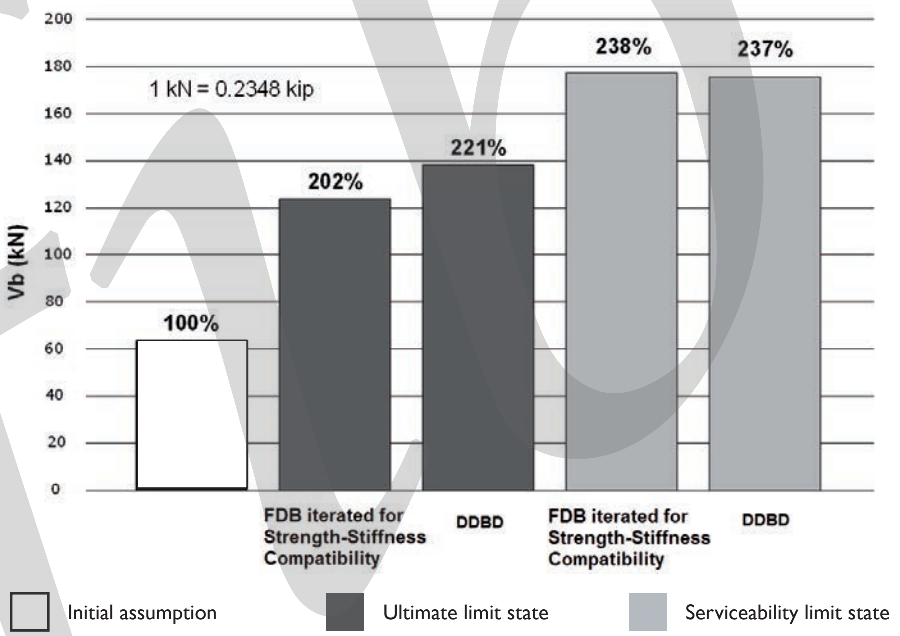


Fig. C-22 Base shear demand per column

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