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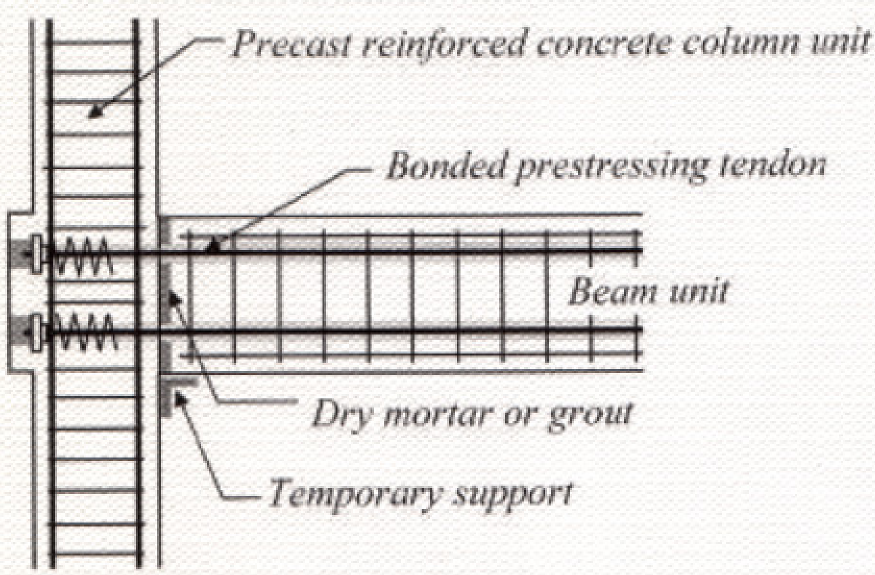

Seismic design of precast concrete building structures

state-of-art report

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State-of-art report prepared by

Task Group 7.3

October 2003

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Cover picture: General reinforcing details of equivalent monolithic post-tensioned system 1 (see Fig. 5-17)

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Foreword

The present document is the outcome of a four year effort made by a large group of experts from the profession and the academia. The original plan, back is early 1999, was to have two separate documents, one dealing with precast construction, the other one with the use of prestressing in earthquake resistant buildings. It soon became clear, however, that this separation was difficult to achieve in a rational way, due to the intrinsic connection existing between the two topics. To give just an example, in Japan most if not all the buildings using prestressing are constructed by assembling precast elements by post-tensioning. If separation proved to be not viable, the organisation of the structure of a combined text was not exactly automatic either. With no other source available for reference, full meetings had to be devoted to discussing the overall content of the document, the order of presentation of the various topics and, most of all, the character the document should have. In the intention of the drafting group, the main destination of the document is the cultured professional with some experience in precast construction wishing to enlarge his view over the international scene and its latest developments. It is not a manual, and less so a catalogue of precast systems. It is not an operative design guide, either, and it could not have been, due to the substantial differences in the design approaches still existing in the different countries, whose coverage would have required the size of an encyclopedia. In brief, it might be described as a State-of-the-Art on the essential aspects distinguishing precast/prestressed from cast-in-situ construction.

The bulk of the information is contained in five of the eleven chapters of the volume: Introduction (Chapter 1), Design Approaches (Chapter 4), Lateral Force Resisting Systems (Chapter 5), Diaphragms (Chapter 6), and Modeling and Analytical Methods (Chapter 9).

From Chapter 1, one learns about the diffusion of precast construction in seven major seismic countries, about the types of structures for which precast is used, and about the design criteria and provisions in force in these countries. References are provided to the relevant design documents. Chapter 4 reviews general seismic design procedures for precast concrete structures. Chapters 5 and 6 contain a comprehensive systematic review of the major distinct ways for assembling precast elements to form moment-resisting frames, walls, dual systems and diaphragms, while giving also concepts and rather detailed explanations on the features of the resulting lateral force resisting mechanisms. Here not only the well established, but also the less diffused innovative solutions are described, notably the so-called hybrid systems where unbonded post-tensioned tendons, in combination with mild steel, are used to assemble the elements in such a way as to minimise the residual drift (the term hybrid is used to signify a response behaviour intermediate between the non-linear elastic and the elasto-plastic, which maintains the re-centering properties of the former and a variable part of the dissipative properties of the latter). A more analytical flavour, closer to the forefront of the research, characterises Chapter 9, whose spirit, however, is that of showing how from more accurate (and complex) models and methods of analysis, affordable, simplified models can be derived, applicable to the wide class of alternative connections made of bonded and unbonded mild and prestressing steels. In each of the mentioned chapters, the interested structural engineer will find hundreds of references to the original sources.

The other more brief, but important, chapters are Lessons from Previous Earthquakes (Chapter 2), Precast Construction Concepts (Chapter 3), Gravity Load Resisting Systems (Chapter 7), Foundations (Chapter 8), Miscellaneous Elements and Structures (Chapter 10), and Conclusions (Chapter 11).

Though everything can always be made more perfect, and this document makes no exception to the rule, it is a fact that nothing similar for width of scope and competence of the authors is available on this subject in the literature of Earthquake Engineering. All those who have contributed need to be thanked; this milestone, however, would not have been reached without the enlightened determination and unfailing patience of the two Convenors, Professors Bob Park and Fumio Watanabe.

Paolo E. Pinto
Chairman of fib Commission 7, *Seismic Design*

Contents

1 Introduction

1.1 General

Precast and prestressed concrete has had significant and successful application in earthquake resisting structures in many parts of the world. Experience of earthquakes and laboratory testing gives confidence that precast and prestressed concrete elements can be used very successfully in structures designed for earthquake resistance providing careful attention is paid to design and construction. Poorly designed and constructed precast and prestressed concrete structures have performed badly in major earthquakes causing the use of precast and prestressed concrete in earthquake resisting structures to be regarded with suspicion in some countries. Structures designed for seismic regions have design requirements in addition to those for ordinary structures designed for gravity and wind loading. For precast concrete structures designed for gravity and wind loading reference can be made to FIP (1994) and fib (2003).

Factors that are important in the design of precast concrete buildings include:

- experience of contractors in the region,
- internal force paths in the different stages during construction,
- crane limits and availability,
- plant and field tolerances,
- transportation distance, weight and size limits
- temporary guying, bracing and shoring

Some of these factors are discussed elsewhere (FIP, 1994).

The aim of this state-of-art report is to discuss the practice of using precast and prestressed concrete in countries in seismic regions and to recommend good practice.

1.2 Scope of the report

This state-of-art report considers precast reinforced concrete and prestressed concrete. It covers lessons learned from previous earthquakes; construction concepts; design approaches; primary lateral load resisting systems (precast and prestressed concrete frame systems and structural walls including dual systems) diaphragms of precast and prestressed concrete floor units; modelling and analytical methods; gravity load resisting systems; foundations; and miscellaneous elements (shells, folded plates, stairs and architectural cladding panels). Design equations will be reported where necessary, but the emphasis will be on principles. Ordinary cast-in-place reinforced concrete is not considered in this report.

1.3 Advantages and disadvantages of incorporating precast and prestressed concrete in construction

The main advantages of incorporating precast and prestressed concrete in construction are:

- The possible increased speed of construction
  — which leads to earlier occupation of buildings and lower interest payments.
- The high quality of precast and prestressed concrete units
  — which is the result of manufacture under factory conditions or of precasting on the building site and the use of high quality materials.
- The improved durability
  — which is the result of the improved quality of construction and/or the closure of cracks after loading of prestressed concrete construction.

- The reduction in site labour
  - which partly offsets a shortage of skilled site workers.
- The reduction in site formwork.
- Post-tensioning is a convenient method for connecting precast concrete elements together
  - precast concrete elements can be made into continuous structures using post-tensioned tendons
- The possible reduction in damage during earthquakes
  - prestressed concrete structures show only small residual displacements and good crack closure after earthquakes.

Other advantages of incorporating precast concrete in construction are discussed elsewhere, for example FIP (1994).

The main disadvantages of incorporating precast and prestressed concrete in construction are:

- Economical and effective methods need to be developed for joining precast concrete units together so as to resist seismic actions and ensure the integrity of the structure.
  - design and construction standards covering all aspects of seismic resisting structures incorporating precast and prestressed concrete are not always available.
- The construction techniques used for joints between precast concrete units may be unfamiliar and construction needs to be conducted with good quality control.
  - poor workmanship at joints between precast units has led to catastrophic behaviour of structures incorporating precast concrete during some past earthquakes.
- Enhanced craneage may be required to lift heavy precast concrete units.
- Relatively small tolerances may need to be worked within.
  - Successful precast concrete construction requires designers, fabricators and contractors to come to terms with the implications of variations in dimensions and hence to have a full understanding of tolerances.
- Prestressing force may be lost in construction using bonded prestressing tendons if inelastic tensile strains develop in the tendons as a result of seismic actions.
  - The use of unbonded post-tensioned tendons overcomes this problem.

1.4 Performance criteria

The required performance criteria for structures incorporating precast and prestressed concrete adopted in design are generally similar to those for cast-in-place construction.

For moment resisting frames and structural walls constructed incorporating precast and prestressed concrete elements the challenge in design and construction is to find economical and practical means of connecting the precast elements together to ensure adequate stiffness, strength, ductility and stability. The design should consider the loadings during the stages of construction and during the life of the structure. The design should ensure that the structure performs satisfactorily in the service load range and at performance levels approaching the ultimate limit state.

According to Priestley (2000), four performance levels can be defined for conventional cast-in-place concrete which are generally the same as for precast and prestressed structures. They are:

- Fully Operational. Facility continues in operation with negligible damage.
- Operational. Facility continues in operation with minor damage and minor disruption in non essential services.
- Life Safety. Life safety is substantially protected, damage is moderate to extensive.

— Near Collapse. Life safety is at risk, damage is severe, structural collapse is prevented.

It is contended by Priestley [2000] that two performance limit states should be considered in design, namely the Fully Operational and a Damage Control performance limit states. The latter would be somewhere between the Operational and Life Safety performance limits and would depend on the importance and function of the structure.

The Fully Operational limit state for concrete structures can be defined by crushing of concrete and unacceptably large residual crack widths. Note that it is the residual crack width, rather than the maximum crack width occurring during seismic response which is of principal interest. Limit compression strains of 0.004 at this limit state would appear to be conservative and reasonable for concrete. Maximum reinforcement tensile strains of 0.01 for beams and 0.015 for columns and walls seem appropriate, since analysis of test data indicates that residual crack widths of reinforced concrete members subjected to peak strains of this level will be in the range of 0.5-1.0 mm [Priestley (2000)].

The definition of a Fully Operational drift limit is less obvious, since the onset of non-structural damage is very dependent on the design details provided to separate non-structural elements from structural elements. However, with good detailing non-structural damage should not be evident at storey drifts of less than 1%.

The Damage Control limit state can also be defined by material strain limits and by design drift limits intended to restrict non-structural damage. For example, a limit compression strain for confined concrete could be taken as greater than 0.004, depending on the confinement by transverse reinforcement. The maximum longitudinal reinforcement tensile strain must be limited to a lesser value than the strain ϵ_{su} at maximum stress to avoid buckling and low cyclic fatigue.

1.5 State of precast construction practice

1.5.1 Canada

Structural precast concrete construction started in Canada in the 1950's with a number of notable buildings. Early examples include a 10,000 m^2 one-storey precast concrete structure with a column and girder framing system and double tee roof members built in Edmonton in 1955 and an eight-storey precast apartment building built in Winnipeg in 1960. In 1962 the publication of the first Canadian Standard for Prestressed Concrete [Canadian Standards Association (1962)] overcame the last objections to using prestressed concrete. Early one-storey, industrial and commercial building structures typically consisted of wall panels, sometimes combined with beams and gravity columns supporting pretensioned single and double tee roof diaphragms. The Canadian Prestressed Concrete Institute published the CPCI Design Handbook in 1964 [Cazaly and Huggins (1964)]. By the 1980's, a number of multi-storey beam and column precast systems were built, with either moment-transfer connections or with added precast or cast-in-place structural walls. The zero-slump hollow core manufacturing system, invented in Winnipeg in 1962, has gained worldwide popularity, with 1,400,000 m^2 of hollow core slabs presently used each year in Canada. Many shear wall residential buildings were built using hollow-core floors and precast concrete interior and exterior walls. The 31-storey Bromley Place apartment building in Calgary, built in 1985, is still the tallest total precast concrete building in Canada. Precast parking garage structures are now being used extensively in most regions of Canada because of the excellent durability of precast pretensioned systems constructed with high-performance concrete. Precast pretensioned wall panel units that are assembled and then post-tenionsed are also used to construct tanks for water treatment facilities.

1.5.2 Chile

Precast concrete construction commenced in Chile in the 1950s in one and two storey housing buildings with large precast panels and slabs, and in one level industrial buildings. Later one level industrial buildings and two level office buildings were constructed using precast flag pole columns with precast beams seated on beam stubs at the columns. In the 1970s four and five storey wall buildings were constructed using precast elements. However, currently precast concrete elements are not being used extensively as primary lateral load resisting systems.

The poor performance of poorly designed and constructed precast concrete structures in overseas earthquakes have caused the use of precast concrete in earthquake resisting structures to be regarded with some suspicion in Chile.

Precast concrete elements are currently being used mainly in Chile in gravity load resisting systems in combination with cast-in-place reinforced concrete systems, mainly walls. The use of precast concrete in flooring systems became commonplace in the 1980s, but most buildings are still constructed using cast-in-place floors.

1.5.3 Italy

Up to the 1940s there was only limited use of prefabrication of structural concrete in Italy, mainly because manpower was cheap and construction was not much industralised. In the 1950s the introduction of prestressing, the growth of manpower costs and the high demand for industrial buildings to contain industrial plant of all kinds turned construction rapidly towards precast concrete solutions. Another innovation that favoured prefabrication was the introduction of lightweight aggregates, better dealt with in plants than on construction sites.

Numerous precasters rose, who, having that experience, laid out their production to other fields, like commercial buildings (storehouses, markets, malls), complex industrial buildings (multi-storey), social buildings (schools, hospitals, gymnasia), car parks, office buildings and, finally, dwellings. This evolution, based on skeleton structures, implied a transition from simple, modular schemes to more flexible ones and the adaption to particular needs, up to tailor-made solutions.

In parallel, also loadbearing wall-panels appeared, this time following foreign systems, for dwelling and social buildings. They had a certain development within large projects but, due to their functional rigidity, they are no longer widely used. These systems have been very popular in Eastern Europe.

Beside the so-called fully prefabricated structures, there have been always partial prefabricated ones too, with only some parts made of precast concrete elements, in the context of cast-in-place concrete or structural steel or masonry buildings.

Floor slabs play an important part in prefabricated concrete structures. Practically most floors are made of precast concrete (planks, beam-and-blocks, hollow-core, etc). In some cases, the whole deck (slabs and beams) is precast, on cast-in-place columns. This is because horizontal elements, if poured on site, require expensive propping for a certain time and, if prestressed, also skilled workmanship which is available better in plants.

Other advantages favouring precast concrete in Italy are that the high performance of concrete is better obtained from dedicated plants rather than on work sites, and precast concrete construction is more environmentally friendly in terms of dust, noise and occupancy time. Hence prefabrication has kept a market share in Italy in spite of some prejudice against non-monolithic structures.

1.5.4 Japan

In late 1950s the use of one-way flooring systems of pre-tensioned precast units started such as single or double T section and hollow core section. In 1970s the reinforced concrete two-way composite flooring system with half precast units and field cast topping concrete was put to use. Since then many of the floors are constructed using above two systems.

Precast reinforced concrete construction of low and medium-rise flats started in 1970s with structural systems of moment resisting frame, structural wall and combination of them. With the wide spread of precast reinforced concrete construction, needs of design guidelines arose. In 1986 the Architectural Institute of Japan (hereinafter abbreviated to AIJ) published book entitled "Design and Construction of Precast Reinforced Concrete buildings". Currently many precast reinforced concrete high-rise buildings are being constructed based on the structural and materials research. However, the Building Standard Law did not cover the precast reinforced concrete until 1999. Therefore the structural design and construction methods needed to be reviewed by the appraisal board of the Building Center of Japan where the structural equivalency to monolithic reinforced concrete construction is checked. In 1999, the performance based design method was adopted to the Building Standard Law. The structural equivalency to monolithic construction is not always required provided that the structural seismic performance of precast system satisfies the safety limit state and serviceability limit state.

Construction of prestressed concrete buildings incorporating precast units started in late 1950s. Design and construction code for prestressed concrete buildings was established by the AIJ in 1961. In 1983 the Ministry of Construction circulated the Notification No. 1320 based on the Building Standard Law. This notification made possible the ultimate strength design. These code and notification covered not only the monolithic construction but also precast prestressed concrete construction. Then precast prestressed concrete construction rapidly spread in Japan. Nowadays almost all prestressed concrete buildings are constructed by assembling precast units by post- tensioning.

1.5.5 Mexico

Precast concrete floor units were introduced in Mexico in the 1950s for the construction of dwellings.

A small number of buildings with moment resisting frames incorporating precast reinforced concrete elements have been constructed since the 1960s. The building codes of Mexico City at the time did not have provisions for precast concrete. The 1976 Mexico City Buildings Code for the first time introduced some regulations for precast construction, basically suggesting the emulation concept. Typical beam-column joints of moment resisting frames had welded connections for the bottom beam reinforcement at the columns. The top beam reinforcement was generally connected by dowels in the columns. The use of welding for connections in precast elements was common practice in Mexico until designers were aware of failures of welded connections during the 1994 Northridge earthquake in California.

Alternative methods have been used more recently for connecting precast elements in frames. One alternative method involves the use of precast concrete columns cast with voids in the beam-column joint regions. The precast beams are placed between the columns and the bottom beam longitudinal reinforcement is hooked in the column voids. The top beam longitudinal reinforcement passes through the voids in the interior columns and is hooked in the voids of the exterior columns. After the beam longitudinal reinforcement has been placed the column voids are filled with cast in place concrete. However, in this method the hooks do not always comply with the code requirements for anchorage length.

Precast reinforced concrete moment resisting frames are generally designed in Mexico as partially ductile structures.

1.5.6 New Zealand

The use of precast concrete in one-way flooring systems very rapidly became commonplace in the 1960s. The precast floor units are generally of pretensioned prestressed concrete. Since the 1960s cast-in-place floor construction has been generally less common and uncompetitive. Now almost all floors are constructed using precast concrete.

In the 1950s and 1960s precast elements post-tensioned together to form continuous moment resisting frames were sometimes used. However, until the late 1970s to early 1980s, the use of precast elements in seismic resisting moment resisting frames and walls was the exception rather than the rule.

The boom years of building in New Zealand of the mid-1980s produced a significant increase in structural applications of precast reinforced concrete for moment resisting frames. Moment resisting frames incorporating precast reinforced concrete are now very common in New Zealand. Normally the joints between the precast elements are designed to emulate monolithic construction so that the whole structure shows equivalent monolithic structural behaviour during an earthquake. Currently the tallest moment resisting frame incorporating precast concrete elements constructed in New Zealand is a little over 40 storeys in height.

Precast concrete structural walls are frequently used for low-rise commercial and industrial buildings. The walls are either designed to emulate monolithic construction or to be jointed systems that do not emulate monolithic construction. Prestressed concrete building structures, other than precast pretensioned floor systems are not common now in New Zealand. However, hybrid frames designed using unbonded post-tensioned tendons and non-prestressed reinforcement are currently being planned.

It is evident that the use of precast concrete in New Zealand is commonplace.

The seismic design provisions of the New Zealand concrete structures standard NZS 3101 used in the 1980s were mainly for cast-in-place reinforced and prestressed concrete structures. However, the most recent edition [Standards New Zealand (1995)] covers many aspects of the seismic design of precast reinforced concrete structures.

As a guide to designers, precasters and contractors, guidelines for the use of structural precast concrete in buildings have been written by a study group of the New Zealand National Society for Earthquake Engineering and the New Zealand Concrete Society, and published by the Centre for Advanced Engineering of the University of Canterbury (1999).

1.5.7 United States of America

Precast prestressed concrete was first used in the construction of Walnut Lane Memorial Bridge in Philadelphia, Pennsylvania, in 1950. Following the introduction of 7-wire strand and high strength concrete, application of precast, prestressed concrete gained momentum in the 1950s. Since that time, the precast industry has grown in the United States by continuously expanding structural applications of precast prestressed concrete.

The notable developments of precast prestressed concrete have been the introduction of standardised products in the 1950s and 1960s, to innovative applications in bridges in the 1970s, to addressing durability issues in the 1980s [PCI, 1999]. With improved durability, pre-topped double tees were introduced for parking structures in that era. A major effort of the precast industry in the 1990s through the present is the development of seismic-force-resisting systems suitable for seismic regions. This effort has now resulted in the construction of the tallest concrete structure in high seismic regions of the United States [Englekirk, 2002]. This precast, prestressed apartment complex building, known as The Paramount, is located in San Francisco, California, with 39 stories and a height of 128 m. In parallel with this effort, the precast industry has also significantly enhanced capabilities to incorporate innovative architectural features into precast products in the 1990s. Precast construction of the

Paramount also took advantage of these developments by utilising architecturally finished structural members in the seismic-force-resisting moment frames.

Precast prestressed concrete structural systems in the United States have both a national character and regional flavor. This is because the precast concrete industry lacks national uniform standards, except for the design guides and manuals recommended for use by the Precast/Prestressed Concrete Institute (PCI). Capabilities and preference of precast producers, whose markets are mainly limited by practical means on hauling distances, have influence at the regional level. Typically the precast manufacturers and a relatively small community of precast specialty consulting engineers work together in the design of precast building systems. Other factors that influence precast design at the regional level include labor and material costs, competition in the market, climate and level of seismic risk.

The building market for structural precast concrete in the United States is largely based around double tees, hollow core components, piles, walls, beams and columns. The floor products and piles are made in large volume in standardised units [PCI 1998, PCI 1999]. The hollow core production uses extrusion equipment. Manufacturing standardised products along with the use of equipment in construction enable the precast industry to produce durable products at competitive prices. Precast walls, beams and columns are typically manufactured on a project basis due to less frequent repetition in the dimensions of these members. However, the industry has a suite of standardised beams and columns which are suitable for gravity framing systems. Architectural cladding is also a main line of product in many precast plants.

The precast industry recognises the significance of adequate connections between precast units. Design connections are performed to satisfy the code requirements, ensuring sufficient strength, ductility, volume changes due to shrinkage, creep and thermal effects, and durability when exposed to environment. Fire protection for the connections is also addressed to ensure that the system performance will not be affected by weakening of the connections. Standard products are also available for connections to improve constructability and reduce construction costs.

1.6 Design approaches

1.6.1 Canada

The National Building Code of Canada (NBCC) (1995) provides earthquake design forces levels and the Canadian Standards Association Standard A23.3 provides the approach to be taken for the seismic design of concrete structures in Canada. The NBCC prescribes force modification factors for determining seismic design base shears. These force-modification factors vary from 1.0, for unreinforced masonry, to 4.0 for ductile moment-resisting frame systems. Because of the variety of different types of precast structures the NBCC does not provide a force modification factor or design procedures for these systems. However, the Design Manual of the Canadian Precast/Prestressed Concrete Institute (1996) provides examples of seismic designs of a one-storey and a four-storey structure, both designed using a force modification factor of 1.5.

1.6.2 Chile

In Chile there are general design provisions for buildings in the current 1996 code "Seismic Design of Buildings" [NCh 433Of.96]. Reinforced and prestressed concrete structures are designed according to the ACI 318 building code. For buildings, the codes contain comprehensive provisions for the seismic design of cast-in-place concrete structures, but contain fewer provisions for the seismic design of precast concrete structures. Nevertheless, significant developments in the use of precast concrete have been made in Chile in spite of the fact that some aspects of the seismic design of precast concrete building structures have not yet been formally codified.

In common with other countries, the seismic design forces recommended for ductile structures in the current 1996 code for "Seismic Design of Buildings" [NCh433Of.96] are smaller than the inertia forces induced if the structure responded in the elastic range to a major earthquake. R factors between 2 and 7 are proposed in this code. The design seismic force is related to an achievable structural ductility not explicitly specified in the Code. A design spectrum and seismic coefficients lead to design horizontal seismic forces at a serviceability limit state (allowable stresses).

The performance criteria for precast concrete structures in Chile are generally the same as for cast-in-place concrete structures.

Connections have been designed to provide sufficient strength so that the chosen means for achieving ductility can be maintained throughout the deformations that may occur. For dual systems of moment resisting frames with cast in place structural walls, the stiffness of walls has been considered to be enough to limit post-elastic deformations in the connections of the frames. Brittle connections between elements of frames have been designed with confidence to remain in the elastic range during a major earthquake, even though capacity design is not used directly in design by most Chilean engineers.

A new Code for Seismic Design of Industrial Facilities [NCh2369], which is currently under discussion in Chile, does include provisions for the seismic design of precast concrete structures. The Code is based on the Chilean experience of past earthquakes. In spite of being a new Code, NCh2369 is still based on forces. Greater design base shears are specified for precast structures using R factor lower than those used for cast-in-place-concrete structures.

In the proposed NCh2369 for the seismic design of precast concrete structures three alternative structural systems are presented: namely, gravity load systems, emulation of monolithic systems, and jointed systems, as summarised below:

(1) Gravity Load Systems

Precast concrete gravity load systems are the most common precast systems used in Chile. The precast structural members are not part of the lateral-force-resisting system and they should be designed mainly to resist gravity loads. Cast in place reinforced concrete walls, reinforced masonry walls or steel frames must be used as the earthquake resisting systems.

Precast concrete elements and their connections must withstand seismic deformations without losing the capacity to support gravity loads. Precast reinforced concrete frames are designed according to section 21.9 of ACI 318-99.

Connections between precast concrete elements and the earthquake resisting system must satisfy a, b, c or d below.

(2) Wet Connections (emulation)

Precast concrete structural systems with wet connections must comply with all requirements applicable to monolithic reinforced concrete construction resisting seismic forces. Wet connections must satisfy ACI 318-99, specially the recommendations for anchorages and splices.

(3) Dry Connections (emulation)

Dry connections must be designed as strong connections. A strong connection must remain in the elastic range while the designated nonlinear regions undergo inelastic response under the design ground motion. If the earthquake resisting system has been designed exclusively with dry connections, a maximum height of 18 m or four levels is allowed.

(4) Jointed Precast Relying on Unique Properties

Connections different from the above mentioned connections can be used only if the designer can prove analytically and experimentally that the system can withstand seismic actions according established principles in the Code. A maximum height of 18 m or four levels is allowed for these structural systems.

1.6.3 Italy

Italy is divided into three seismic zones with design (500 year return period) peak ground accelerations of 0.35g, 0.25g, and 0.15g respectively, plus a zone considered to be non-seismic covering a minor part of the country.

Zoning is established and updated by the seismic code, which is a national law and gives the design rules, containing some prescriptions for prefabricated concrete structures.

Italy also has a design code for the verification of precast concrete structures which imposes a set of horizontal forces equivalent to 0.02 times the maximum gravity loads. Practically this makes all precast concrete structures conceived as seismic resistant, at least qualitatively.

Among "Structural Eurocodes" issued by the European Committee for Standardisation (CEN), Eurocode 8 (various dates) deals with design of seismic structures. It is not yet in force but will become soon a full European Standard (EN 1998-1). However, during its over 10-year elaboration as a pre-standard, it has been acquainted with and has inspired the newest editions of national standards. In future, it will replace them in all European countries.

The selection of the structural scheme may follow two main criteria.

One criterion tends to emulate the monolithic behaviour of cast-in-place construction. Connections are normally wet cast, with through re-bars made continuous by welding, threading, overlapping, or coupling.

The other criterion, more proper to fabrication in individual units, accepts flexural discontinuities due to dry joints, possibly integrated with devices. The precast concrete elements do not show protruding reinforcement, harmful to production, and do not require cast-in-place concrete completion. Structural justification needs ad hoc models and safety measures, with respect to those worked out for ordinary frames.

Buildings with skeleton structures often follow the first criterion (emulation of monolithic), mainly because these are adaptations of previously designed ones (and approved) for cast-in-place concrete. Beams rest on columns' corbels (avoidable for short spans) and have protruding reinforcement at joints, that are grouted on site.

When buildings are conceived for prefabrication, the structure follows mostly the "dual" scheme, where gravity loads are assigned to columns and horizontal actions to shear walls. This scheme is particularly suited for seismic constructions, where it allows for long spans without having heavily stressed beam-column joints and consequent large cross sections in the frames. Instead, the joints can be, at limit, hinged. In seismic zones codes forbid reliance upon friction from gravity. Therefore supports are provided with restraining devices, like anchors or pins, which aim to prevent progressive collapse.

A typical example of the second criterion are the loadbearing panel systems. They are composed of concrete plates, placed in three mutually perpendicular planes, which stabilise each other only connected by linear hinges. Some systems prefabricate assembled sets of plates, vertical and horizontal, thus avoiding even temporary stabilizers during erection. The overall resistance, as well as the prevention of progressive collapse, is obtained by means of steel ties, that can be incorporated or not within the panels. Such structures perform well in earthquakes. However, they are heavy and rigid from the stand point of room distribution. There exist now lighter systems for low-rise buildings which use hollow-core panels for floors and walls.

A further example of the second criterion are buildings for industrial halls, made in general with statically determinate structures. The earlier ones, which started the prefabrication industry, have a structure made of columns cantilevering from the foundation, which support longitudinal beams and tapered transverse beams spanning the whole hall, bearing any kind of short roof slabs, all simply supported. In other schemes, representing now the most typical precast structures in Italy, only longitudinal beams are present, whereas transverse beams and slabs are replaced by a large variety of long roofing systems: these are normally 2.5 m wide, up to more than 20 m long, with thin cross sections, formed from simple T, TT or Y, up to elaborated curved shapes, laid in alternatively with lightings. Such roofs lack in-plane rigidity and cannot develop full diaphragm action. The supports are provided with pins to transmit horizontal forces and to prevent pads from loss of seating, as friction alone is not relied upon under seismic action. Sometimes, these structures are braced longitudinally by walls or diagonal bars.

For these structural schemes, the appropriate behaviour factor to be applied is currently under discussion. In fact, on one hand the columns appear like inverted pendulums, while, on the other hand, their ductility may be good (as they are designed basically for bending and low normal force) and their number offers a large redundancy that assimilates them to frames.

With regard to floors, traditionally precast concrete floors are provided with a topping layer of cast-in-place reinforced concrete deemed to withstand in-plane diaphragm stresses. However, a view in Italy now is that the structural topping is an inconvenience. Its contribution to the resistance in the slab action is proportionally little, as its strength class is lower than the precast part, and the addition of weight is high, as it is a solid layer. The extra weight requires heavier beams, columns and foundations, which altogether mean more mass, thus higher seismic forces, requiring a stronger structure. In some cases, the topping depth, summed up at all storeys may affect the total building height. So, good reasons exist in favour of possibly avoiding structural topping.

Diaphragms made of precast concrete elements, without the addition of a cast-in-place topping (untopped), make it possible to overcome the mentioned inconveniences. They also have the advantage that the resisting mean plane gets closer to the centroid of the structural floor, which represents the largest part of the total mass, whereas a topping is eccentric. However, intrinsic discontinuities of precast assemblies oblige careful design and execution of connections between the precast floor elements. The precast elements profitably used without a structural topping are hollow-core and double-tee slabs, which offer, with their even-finished upper surface, a ready flat plane and may be given a diaphragm function.

1.6.4 Japan

Wall type precast concrete buildings in Japan, which have been used for houses, are designed in accordance with the codes and standards issued by the Architectural Institute of Japan (AIJ) and by the Building Center of Japan (BCJ). The design is the allowable stress design base. However, the structural detailing is provided taking into account the ultimate state of building.

The design of precast concrete frame systems is achieved following the procedures for cast-in-place concrete structures based on the Building Standard Law 1981 (including Building Standard Law Enforcement Order and Notification of Ministry of Construction) and the concrete standards issued by the AIJ, since the design codes and standards for the precast concrete frame systems have not yet been established. The design procedures include the working stress design for moderate earthquakes and the ultimate strength design for severe earthquakes.

Precast concrete frame systems are designed as monolithic structures having equivalent monolithic connections. The design for precast concrete frame systems has often been subjected to the appraisal of the BCJ because of unregulated structure. It was sometimes recommended by building officer to take larger design forces than the forces for a monolithic structure due to possible slippage at precast connections. In the case of a moment resisting frame without walls, for example, the factor of 0.35 or more to reduce the 1.0g level design seismic force was recommended while monolithic structure might take the factor of 0.30. Newly developed precast connections need experimental investigation to verify their seismic behavior. Precast concrete frame systems taller than 60m are required to undergo design appraisal by public institute such as BCJ.

For design of precast members, the common equivalent static seismic load used for ordinary cast-in-place concrete structures may be used, whereas the effects of other loads inherent to precast structural system also need to be taken into consideration. They include the forces caused by (1) change of dimension of precast elements, (2) erection process, and (3) volume change due to temperature, creep and drying shrinkage, etc. Design moment and/or combination of design force are obtained by frame analyses, in which the structures may be modeled as a cast-in-place reinforced concrete construction. Obtained moments and forces are increased taking into account the effects of necessary safety margin. Results of the analysis give design moment, design shear and design requirement for bond. Story drift and eccentricity, the limitations of which are specified in codes, are also calculated based on the analysis above. The geometry of section and amount of reinforcement may be proportioned based on the design moment, shear and their combination. Additional consideration needs to be taken of force transfer at the connection.

The Building Standard Law was revised in 2000 adopting performance based design approach. [Shiohara and Watanabe (2000)]. Target performances (serviceability and safety) are clearly indicated with drift limit for design. Response displacement due to design seismic forces is obtained by the capacity spectrum method. Precast concrete frame systems with equivalent monolithic connections may be designed based on this new approach. In accordance with the revision of the Building Standard Law in 2000, the Architectural Institute of Japan (2000) is proposing design guidelines for precast concrete frame systems with equivalent monolithic reinforced concrete connections which satisfy the following conditions; (1) connections which have enough stiffness such that the design stresses can be predicted with necessary accuracy by structural analyses applied to monolithic cast-in-place concrete structural systems, (2) connections which have equal to or larger strength than it needed to transfer force occurring in member or between members such that equivalent structural seismic resistance would be maintained, (3) members with those connections should have comparable restoring force characteristics by which no significant difference in earthquake response occur, and (4) connections should have equivalent or superior serviceability, durability and fire resistance.

1.6.5 Mexico

Very few precast concrete moment resisting frames were designed in Mexico in the 1960s and 1970s because of the lack of regulations for the seismic design of precast concrete structures. In 1976 the Mexico City Building Code (MCBC) for the first time introduced some regulations for precast concrete construction, basically suggesting the emulation concept. The current MCBC (1993) extended those regulations giving some general provisions for the seismic design of connections for precast concrete structural elements.

The use of welding for connections in precast concrete structural elements in moment resisting frames was common practice in Mexico until designers were aware of failures of welded connections in steel structures during the 1994 Northridge earthquake in California. After this event, designers in Mexico have shown interest in using alternative methods for connecting precast concrete structural elements without the use of welding; for example by using precast concrete columns with voids at the level of the beams. The precast concrete beams are placed between the columns with the beam longitudinal bars passing through. Cast-in-place concrete is placed in the voids.

Precast concrete moment resisting frames in Mexico are designed as partially ductile structures. According to the MCBC (1993) a fully ductile precast concrete frame can be designed only if designers are able to show building officials that the proposed design and construction process for the connections between precast structural elements are adequate. However, the MCBC (1993) does not give specific provisions for arriving at this validation.

The use of precast concrete in flooring systems is not common in Mexico. The two reasons for this situation are firstly that designers feel that the current MCBC (1993) lacks guidelines for the seismic design of precast concrete floors, and secondly that producers of precast concrete floors in Mexico have not been active in their promotion.

1.6.6 New Zealand

The current New Zealand concrete design standard [Standards New Zealand (1995)] contains provisions for the seismic design of precast reinforced concrete moment resisting frames but does not cover all aspects of the seismic design of precast reinforced concrete structural walls.

The seismic design actions at the ultimate limit state specified by the current New Zealand loadings standard [Standards New Zealand (1992)] depend on the structural system as classified below:

The design seismic forces for cast-in-place reinforced concrete structures and for structures incorporating precast reinforced concrete elements of the same available ductility recommended in the New Zealand loadings standard (1992) are identical.

(1) Ductile Structures

Ductile structures are required to dissipate energy by yielding of the reinforcement in specified localities of the structure while undergoing inelastic displacements. For ductile structures the design seismic actions are significantly less than the inertia forces induced if the structure responded in the elastic range to a major earthquake. The design seismic force is related to the achievable structural (displacement) ductility factor $\mu = \Delta max/\Delta y$, where Δmax is defined as the maximum horizontal displacement that can be imposed on the structure during several cycles of seismic loading without significant loss of strength, and Δy is defined as the horizontal displacement at first yield assuming elastic behaviour of the cracked structure up to the design seismic force. For ductile structures $\mu = 5$ or 6 is used to determine the appropriate inelastic spectra of seismic coefficients at the ultimate limit state.

Adequate ductility capacity is considered to be provided if all primary elements of such structures required to resist earthquake induced forces are designed by capacity design and detailed in accordance with the concrete design standard [Standards New Zealand (1995)] for ductile structures.

(2) Structures of Limited Ductility

Structures or structural elements of limited ductility are assumed to have either low inelastic deformation demand or low inelastic deformation capacity and are designed to resist seismic forces derived with the use of lower μ values. $\mu \leq 3$ is used to determine the appropriate spectra of seismic coefficients at the ultimate limit state.

Limited ductility structures or elements are required to be subject to capacity design. Adequate ductility is considered to be provided if all primary elements of such structures required to resist induced force are designed by capacity design and detailed in accordance with the concrete design standard [Standards New Zealand (1995)] for structures of limited ductility.

(3) Elastically Responding Structures

Elastically responding structures are designed using μ = 1.0 or 1.25 to determine the appropriate spectra of seismic coefficients. They are not expected to develop significant inelastic deformations. When energy dissipation is possible only in a form not admitted for ductile structures or structures of limited ductility the mechanism so identified should be detailed for adequate ductility.

To ensure that the most suitable mechanism of post-elastic deformation occurs in a structure during a major earthquake, New Zealand standards require that ductile structures and structures of limited be the subject of *capacity design*. In the capacity design of structures, elements of the primary lateral earthquake load resisting systems are chosen and suitably designed and detailed for adequate strength and ductility for a major earthquake. All other structural elements and other possible failure modes are then provided with sufficient strength so that the chosen means for achieving ductility can be maintained throughout the deformations that may occur [Standards New Zealand (1995), Standards New Zealand (1992)].

For moment resisting frames and structural walls of buildings the best means of achieving ductile post-elastic deformations is by flexural yielding at selected plastic hinge positions, since with proper design the plastic hinges can be made adequately ductile. To ensure that failure in flexure cannot occur in parts of the structure not designed for ductility, or that failure in shear cannot occur anywhere in the structure, the maximum actions likely to be imposed on the structure should be calculated from the probable maximum flexural strengths at the plastic hinges taking into account all the possible factors that may cause an increase in the flexural strength of the plastic hinge regions. These factors include an actual yield strength of the longitudinal reinforcing steel which is higher than the lower characteristic (5 percentile value) yield strength and additional longitudinal steel strength due to strain hardening at large ductility factors (the sum of these two factors is referred to the steel overstrength).

As a result, the shear reinforcement in the plastic hinge regions, and all reinforcement in parts of the structure away from plastic hinge regions, need to be designed for the overstrength flexural actions at the plastic hinge regions, in order to ensure that non-ductile failures do not occur elsewhere. For moment resisting frames column sidesway mechanisms with plastic hinges forming in the top and bottom of one storey are not permitted except for one and two storey frames and in the top storey of taller frames, and in tall frames where the sway displacement is restricted by stiffer structural elements, for example by walls or by some columns remaining in the elastic range. In other cases strong column-weak beam behaviour is required to avoid excessive ductility demand at plastic hinges.

The use of capacity design has given designers confidence that structures can be designed for predictable behaviour during major earthquakes. In particular, brittle elements can be protected and yielding restricted to ductile components as intended by the designer. The capacity design procedure has enabled structures incorporating precast concrete elements to be designed for ductile behaviour, since any brittle connections between elements can be designed with confidence to be strong enough to remain in the elastic range during a major earthquake.

1.6.7 United States of America

In recent decades, design of buildings in the United States has been based on one of the three model building codes: (1) the BOCA National Building Code [BOCA (1999)], (2) the Standard Building Code [SBCCI (2000)], and (3) the Uniform Building code (UBC) [ICBO (1997)]. These model codes are mainly adopted in the northeastern quarter, in the southeastern quarter and in the western half of the country, respectively [Gosh (2002)]. In an effort to replace the three model codes with a single code, an International Building Code

(IBC) was introduced in 2000 [ICC (2000)]. The International Building Code has thus far been implemented only in a few states and localities.

Various design provisions in the model codes adopt standards and resource documents, as it is impossible to address all aspects of design within a code. Concrete design and construction in all codes either adopt or are based on the ACI-318 standard: Building code requirements for concrete structures. Another document that influences seismic design provisions in model codes is the NEHRP Provisions, where NEHRP stands for National Earthquake Hazards Reductions Program [BSSC (2000)].

Seismic design provisions for precast concrete structures were first introduced in the 1994 NEHRP provisions, and then adopted in UBC 1997 [ICBO (1997)]. The design in the UBC is limited to emulative (equivalent monolithic) design of frame structures using only strong connections. More widely applicable seismic design provisions for precast concrete structures were published in the 2000 NEHRP Provisions [BCCS (2000)]. This document permits emulation design of precast structures with both strong and ductile connections, and introduces non-emulative design provisions for precast structures.

Seismic design provisions for precast structures has also been now introduced in ACI 318-02 [ACI 318 (2002)]. Although ACI 318-02 has similar provisions to the 2000 NEHRP Provisions, the scope is somewhat limited with an option of non-emulative design for precast wall structures eliminated. Seismic design provisions in the next version of IBC (i.e. IBC 2003) are expected to adopt the ACI 318-02 standard. There are also current plans for the 2003 NEHRP standards to further improve seismic design of precast systems in seismic regions of the United States could expand significantly in the coming years. Since the ACI 318-02 standard will play a critical role in the application of precast seismic systems for the next several years, more details of this document are presented below.

Table 1-1 identifies the sections of ACI 318-02 that are applicable to the seismic design of precast concrete buildings, along with the equivalent sections that are applicable to cast-in-place structures. In high seismic regions only special precast moment frames and special precast walls are permitted. The provisions for special precast moment frames allow for both ductile and strong connections and are intended to produce behaviour similar to monolithic special moment frames. Precast moment frames that do not emulate monolithic frame behaviour are permitted, provided they satisfy a set of acceptance criteria established by ACI T1.1 (2001). The ACI T1.1 requirement includes validation of the design method by laboratory experiments. For hybrid frames described in Chapter 5, such validation has been successfully performed and this frame has been implemented in high seismic regions of the United States.

Special structural walls constructed using precast concrete are required to meet all provisions for the cast-in-place special walls in ACI 318-02, enforcing emulation behaviour. Jointed wall systems such as that experimentally proven to be effective in the PRESSS program (see Chapter 5) are not all allowed. However, a building official may accept a non-emulative wall system based on the general section 21.5.2.1.5 of ACI 318-02, which allows "a reinforced concrete structural system not satisfying the requirements of [Chapter 21 of ACI 318] if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this chapter".

Special concrete walls are distinguished from ordinary (non-seismic) concrete walls principally in that they have a minimum reinforcement ratio of 0.0025 in the horizontal and vertical directions and require a check for boundary confinement. Unlike the special moment frames, design shear forces in special concrete walls are based on the factored load analyses without considerations to overstrength flexural moments. As a result, structural walls designed to ACI 318-02 have the potential to experience shear failure, which is usually characterised by sudden and substantial deterioration in strength.

In low and moderate seismic zones, ACI 318-02 permits the use of intermediate and special frames and walls; intermediate frames and walls are considered to have less toughness and detailing requirements than special frames and walls. There are no provisions for precast intermediate frames because the precast industry does not envisage an economic incentive to use intermediate moment frames over special moment frames.

Intermediate precast structural walls are permitted in ACI 318-02 and may be designed in moderate seismic zones. It is noted that there is no category of cast-in-place intermediate structural walls. In low seismic regions, structures in ordinary category that does not require special detailing can be adopted.

Precast framing members that are not designated to be part of the seismic force resisting system, i.e. “gravity” framing, is required to meet the following:

(a) Detailing requirements for transverse reinforcement, which include all of the provisions required for cast-in-place gravity frame members,
(b) The structural integrity provisions required of all precast construction (Section 16.5), and
(c) Bearing length of corbel seats at least two inches (50 mm) longer than required by calculation.

There is no explicit requirement for either cast-in-place or precast gravity framing to demonstrate sufficient rotation capacity under seismic deformations.

According to commentary section R21.1, ACI 318-02 provisions include the minimum requirements for seismic design of cast-in-place and precast structures, emphasising the need to ensure inelastic response without significant deterioration in strength. However, not completely protecting failure against shear such as that discussed for special concrete walls may lead insufficient inelastic capacity. To preclude development of unintended nonlinear mechanisms or behaviour modes, the capacity design approach discussed in Chapter 4 of this report must be fully implemented in addition to the requirements provided in ACI 318-02.

Structural System	**Cast-in-place construction**	**Precast construction**
Special moment frames	21.3, 21.4, 21.5	21.6
Special structural walls	21.7	21.8
Intermediate moment frames	21.12	Not used
Intermediate structural walls	Not used	21.13
“Gravity” framing members (i.e., not designed as part of the seismic force-resisting system)	21.11	21.11.4

Table 1-1: Sections of ACI 318-02 (2002) containing provisions for precast construction in moderate and high seismic regions.

1.7 Acceptance procedures

1.7.1 Japan

For ordinary cast in place reinforced concrete construction the Building Standard Law of Japan specifies four member ductility levels of A (fully ductility), B (intermediate ductility), C (limited ductility) and D (non-flexural failure). The limit values of tension reinforcement ratio, magnitude of axial load level, shear span-depth ratio and nominal shear stress are specified for each ductility level. Structural ductility is determined based on the ductility of constitutive members where four ductility ranks are taken as Rank-I (full ductility), Rank II (intermediate ductility), Rank III (limited ductility) and Rank IV (others). Precast reinforced concrete system shall be checked by experiments that the structural ductility is equivalent to one of ductility ranks given for ordinary monolithic construction. In the laboratory tests sub-assemblages are subjected to displacement controlled reversed cycles to the requirement displacement limits. Generally two or three cycles are taken in each displacement cycle. Commonly used evaluation standard is that the measured strengths during cyclic loads do not reduce to less than 80% of the maximum measured strength.

The Notification No. 1320, Ministry of Construction specifies the ductility evaluation method for prestressed concrete structures both for monolithic and precast construction.

When new structural material not covered by the Building Standard Law of Japan is applied to precast construction, experimental verification is required.

1.7.2 New Zealand

The New Zealand loadings standard [Standards New Zealand (1992)] specifies a simple procedure for determining the displacement (structural) ductility factor μ appropriate for a structural form or material not covered by a standard. In a laboratory test the subassemblage is subjected to four displacement cycles of lateral loading to the required displacement limits. It should be checked that during this loading that the measured strengths do not reduce to less than 80% of the maximum measured strength.

1.7.3 United States of America

Innovative Task Group 1 and Collaborators of the American Concrete Institute have developed a document ACI T1.1-01, 2001, which defined the minimum experimental evidence that can be deemed to be adequate to validate the use, in regions of high seismic risk or in structures assigned to satisfy high seismic performance or design categories of weak beam-strong column moment frames not satisfying fully the requirements of ACI 318-99.

The test modules should be representative beam-column assemblies not less than one third full size extending each side of the joint between contraflexure points in the beams and columns. Test modules shall be subjected to a sequence of displacement controlled cycles to predetermined drift ratios. Three fully reversed cycles shall be applied at each drift ratio and testing is continued with gradually increasing drift ratios up to at least 0.035. A record is made of measured column shear force versus drift ratio.

The test module shall be considered to have performed satisfactorily when all of the following criteria are met for both directions of response.

(1) The test module shall have attained a lateral resistance equal to or greater than En before its drift ratio exceeds the value consistent with the allowable story drift limitation of the International Building Code, 2000, where E_n = nominal lateral resistance of the test module determined using the specified material strengths.

(2) The maximum lateral resistance E_{max} recorded in the test shall have not exceeded λE_n where λ is the specified overstrength factor for the test column.

(3) For cycling at the given drift level at which acceptance is sought, but not less than a drift ratio of 0.035, the characteristics of the third complete cycle shall have satisfied the following three requirements:

   (i) Peak force for a given loading direction shall have been not less than $0.75E_{max}$ for the same loading direction;

   (ii) The relative energy dissipation ratio β shall have been not less than 1/8 where β = the area inside the lateral force-drift ratio loop divided by the area of the effective circumscribing parallelograms, and

(iii) The secant stiffness from a drift ratio of –0.0035 to a drift ratio of +0.0035 shall have been not less than 0.05 times the stiffness for the initial drift ratio within the linear-elastic response range for the module.

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2 Lessons from previous earthquakes

2.1 Introduction

This chapter describes the excellent performance of well-designed and well-detailed precast and prestressed concrete structures and illustrates some lessons learned from the performance of poorly designed and detailed precast concrete buildings. The examples were chosen to illustrate some deficiencies in the design and detailing of this common type of construction.

2.2 Performance of well-designed, well-detailed precast and prestressed concrete structures

Although this chapter uses failures of poorly designed and poorly detailed structures to illustrate some important concepts in seismic design, it must be emphasized that well- designed and well-detailed precast and prestressed concrete structures have performed very well in major seismic events.

Fintel (1986) reported that in the 1985 Mexico earthquake only five of the 265 buildings, that either collapsed or were severely damaged, used precast concrete elements. He also reported that there were many precast buildings and multi-storey parking garages in Mexico City that survived the severe ground shaking without damage or distress. Rodríguez (2000) reported that many of the precast concrete structures were constructed with multi-level columns with the beam-column joints interconnected by welding of the top and bottom beam bars in the joint and then casting concrete in the joint region. Camba and Meli (1993) conducted detailed seismic evaluations of five prestressed concrete buildings located in Mexico City. Both linear as well as non-linear analyses were carried out on these buildings that had either cast-in-place post-tensioned beams or precast prestressed concrete beams. These structures were analyzed using input motions, obtained from measured ground motion records that corresponded to the building sites. Camba and Meli concluded that the frame structures investigated showed no signs of distress and that current codes would result in even safer designs, even for the most severe seismic conditions. Recent developments of precast concrete structural systems used in Mexico are described by Rodríguez (2000).

The performance of precast concrete buildings in the 1988 Armenian earthquake clearly demonstrates the importance of proper seismic design [EERI (1989) and Fintel (1995)]. The predominant structures were one to five storey masonry wall structures with hollow core floors. About 90 percent of these structures collapsed due to improper connections of the floor diaphragms to the walls and the brittle nature of the masonry walls. There were many precast concrete frame structures that collapsed due to poor design and poor detailing. However, in contrast to these improperly designed structures, large-panel precast concrete structures behaved excellently. Sixteen of these large-panel structures were located in Leninakan and there was no significant damage to these structures. Furthermore, the five-storey large-panel structure located in Spitak was the only building that did not suffer major damage in this city. These structures had one storey high wall panels that were interconnected by reinforcement and cast-in-place joints between the panels. The cast-in-place joints between the panels had serrated edges to improve shear transfer and had hairpin dowels that overlapped around an interlocking reinforcing bar in the panel joints. The excellent performance of these large-panel structures clearly illustrates the success of proper design and detailing, in spite of the poor quality of the concreting of the joints [EERI (1989)]

Park (1995) reported on trends and developments in New Zealand with the use of precast concrete for floors, moment-resisting frames and walls. The methods used for connecting the precast elements together are typically aimed at emulating the behaviour of well-detailed, cast-in-place construction. Subassemblies incorporating these new methods have been tested under reversed cyclic loading [Restrepo et al (1993)] and various types of these new construction techniques have been used in practice [Park (1995)].

Muguruma et al (1995) described the performance of precast prestressed concrete buildings following the 1995 Kobe earthquake in Japan. A total of 163 precast prestressed concrete buildings were in the region of most severe damage. These structures consisted of 11 precast, prestressed concrete buildings, 49 structures with non-structural precast, prestressed elements, 89 cast-in-place post-tensioned buildings and 14 buildings with non-structural cast-in-place prestressed members. Muguruma et al (1995) concluded that these structures performed "remarkably well". They attributed the excellent performance to the following:

(i) these structures were designed for higher design force levels than other structural types,
(ii) the fact that precast, prestressed concrete buildings typically have regular geometry with symmetrical plan layout of the resisting elements and uniform distribution of mass and structural stiffness over the height,
(iii) the precast prestressed concrete structures typically have high quality construction and higher strength and better quality concrete, and
(iv) these structures are typically newer construction designed according to more recent codes.

There were many examples of the excellent behaviour of precast concrete structures during the 1999 Kocaeli earthquake in Turkey [Saatcioglu et al (2001)]. Although many poorly designed and poorly detailed precast concrete structures collapsed, many precast industrial structures located near the region of severe faulting performed excellently. These structures consisted of single-storey and multi-storey medium and large span frame structures and precast concrete wall structures. It was clear from the inspection of these structures and structures under construction that the main structural elements were well designed and well detailed as well as having proper diaphragm reinforcement and connections.

2.3 Improper design and detailing of ductile elements

For low-rise precast concrete structures the columns or wall panels are the ductile elements, with the floor and/or roof elements designed to be stronger than these ductile energy-dissipating elements in accordance with accepted capacity design principles [Park and Paulay (1975)]. It is of interest that older moment resisting frames and structural walls incorporating precast concrete often have been observed to perform badly during earthquakes. The observed failures have been mainly due to brittle (non-ductile) behaviour of poor connection details between the precast elements, poor detailing of the elements and poor design concepts. Fig. 2-1 and Fig. 2-2 show examples of major damage to precast concrete structures as a result of severe earthquakes. As a result the use of precast concrete was shunned in some countries in seismic zones for many years.

Many precast concrete frame structures collapsed during the 1988 Armenian earthquake [EERI (1989)]. These structures were typically nine storeys in height and contained hollow-core floor slabs. Some of the structures had some walls in one direction but these walls typically contained large openings. The beam-column connections were made by welding the beam bars to steel angles protruding from the precast columns. The floor diaphragms were poorly connected to the frame elements. Column splices were made by welding the vertical column bars. This detail resulted in significant eccentricities of the column bars at the splice location. The columns were poorly detailed with inadequate confinement and had column ties with only 90° bend hooks. These hooks were ineffective once spalling of the concrete cover occurred in the columns.

Fig. 2-1: Collapse of precast buildings in Tangshan, China in 1976

Fig. 2-2: Collapse of precast buildings in Leninakan, Armenia in 1988 [EERI (1989

Fig. 2-3 shows a precast concrete column of the California State University parking structure that failed during the 1994 Northridge earthquake.

This three-storey structure had exterior site-cast frames that were designed and detailed to be ductile [Mitchell et al (1995), Iverson and Hawkins (1994), EERI (1994)]. The main interior girders in the N-S direction are precast pretensioned elements supported by corbels on the exterior cast-in-place columns and on the interior columns. A cast-in-place post-tensioned slab spans between the beams in the E-W direction with the post-tensioning anchored at the exterior frames. The interior columns were designed to be gravity-load columns only with the lateral loads to be taken by the exterior frames. The mix of a very ductile system with the poorly detailed gravity-load columns interconnected by a flexible diaphragm led to brittle failures of several interior columns.

Fig. 2-3: Failure of poorly detailed precast concrete column in the 1994 Northridge earthquake

For frame structures, the column must be capable of developing plastic hinging at their bases without suffering brittle failure modes such as shear, bond failure and loss of confinement of the core concrete. Fig. 2-4 shows the shear failure of a precast column that occurred during the 1999 Kocaeli earthquake in Turkey [Saatcioglu et al (2001)].

The columns of this structure were grouted into foundation sockets to provide fixed bases. The transverse reinforcement is widely spaced and is unable to prevent buckling of the longitudinal reinforcement and is inadequate for confinement. In addition, the 90° bend hooks have lost their anchorage due to spalling of the concrete cover. Fig. 2-5 shows the shear failure at the base of a precast concrete column illustrating the loss of anchorage of the 90° hooks.

Fig. 2-6 shows the ductile flexural yielding that took place in a precast column during the Kocaeli earthquake.

This 450 by 450 mm column contained two 20 mm diameter bars in each corner and was confined with 6 mm diameter plain ties at 100 mm spacing. Although the ties had only 90° bend anchorages with a 120 mm free end extension the column developed significant flexural hinging.

Fig. 2-4: Shear failure of precast concrete column at foundation socket (1999 Kocaeli earthquake

Fig. 2-5: Shear failure at base of precast concrete column (1999 Kocaeli earthquake)

Fig. 2-6: Flexural hinging at base of precast concrete column (1999 Kocaeli earthquake)

It is noted that the flexural cracking indicates that significant reversals of moments did not take place during the earthquake. If more significant reversed cyclic loading had taken place then it is likely that cover spalling and hence loss of anchorage of the ties would have occurred.

Fig. 2-7 shows the severe distress in a 260 mm by 510 mm precast column, some distance from its base while other columns collapsed in this industrial structure in Turkey.

Fig. 2.7: Severe distress in precast concrete column (1999 Kocaeli earthquake)

A close-up of the region of distress shown in Fig. 2-8 indicates that twelve 15 mm diameter bars were continuous over the height of the column while four 20 mm diameter bars were cut-off at the region of distress. The transverse reinforcement consisted of 6 mm diameter plain bars at a spacing of 175 mm, anchored with 135° hooks [Saatcioglu et al (2001)]. Bond and shear failure occurred at this critical section where the vertical bars were curtailed.

Fig. 2-8: Close-up of poor column details (1999 Kocaeli earthquake

2.4 Inadequate diaphragm action

As well as carrying the gravity loads, floors need to transfer the imposed seismic forces to the supporting structure through diaphragm action. Adequate connections to transfer diaphragm forces and adequate support of precast concrete floor units are basic requirements for a safe structure. It is essential that floor systems do not collapse as the result of imposed movements caused by earthquakes which will reduce the seating area at the supports of the precast floor units. Fig. 2-9 shows the collapse of a precast concrete hollow core floor which became unseated during the 1994 Northridge earthquake.

There were many examples of failures of diaphragms in the 1999 Kocaeli earthquake [Saatcioglu et al (2001), EERI (2000)].

The diaphragms that failed were very flexible, made with corrugated fibre concrete or thin metal sheets and the panels of these diaphragms were poorly connected to each and to the main structural components. Fig. 2-10 shows the inadequate clip connection between a corrugated fibre concrete diaphragm and a beam in a precast industrial building that suffered total collapse. Fig. 2-11 shows the failure of a light gauge metal diaphragm in a furniture warehouse.

There were many examples of failures of precast concrete structures due to the extreme flexibility of the diaphragms and in several cases complete failures of unfinished precast concrete industrial buildings [EERI (2000)].

There were many examples of failures and severe distress in older tilt-up wall panels [EERI (1994)]. These types of structures have typically failed due to insufficient connection between the diaphragms and the tilt-up wall panels. A large number of older structures have undergone retrofit to improve these connections.

Fig. 2-9: Collapse of precast concrete hollow core floor (1994 Northridge earthquake)

Fig. 2-10: Flexible connections connecting diaphragm to precast concrete beam (1999 Kocaeli earthquake)

Fig. 2-11: Failure of flexible light-gauge metal diaphragm (1999 Kocaeli earthquake)

2.5 Poor joint and connection details

Beam-column joints of moment resisting frames incorporating precast concrete elements have often failed in earthquakes due to poor details. A significant cause of failure has been the brittle failure of reinforcing bars in the vicinity of welds. Fig. 2-12 shows the failure of a poorly detailed moment-resisting connection between beams and columns as a result of the Tangshan earthquake in China in 1976.

Precast concrete beams seated on columns or on column corbels need to have adequate connection between the beam ends and the bearing areas to avoid the beams sliding off their supports. Fig. 2-13 shows an example of the unseating of precast concrete beams off column corbels due to inadequate connections between the beams and their bearing areas.

Fig. 2-12: Failure of a poorly detailed beam-column connection (1976 Tangshan earthquake, China)

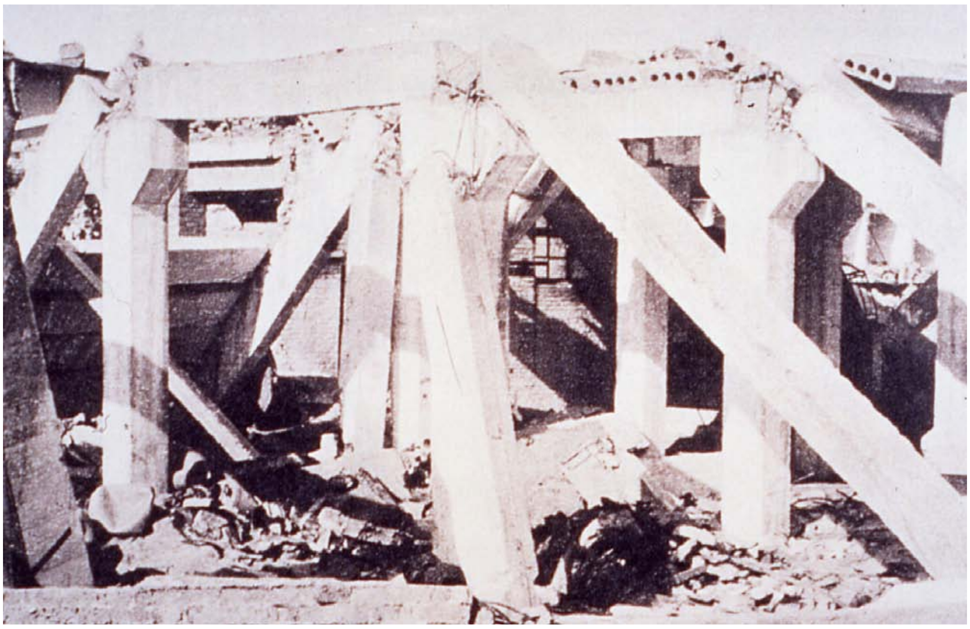

Fig. 2-13: Unseating of precast concrete beams off corbels of columns

Fig. 2-14 shows the failures of the connection between a column and the double cantilever column head used to support the transverse beams of the industrial building [Saatcioglu et al (2001), EERI (2000)]. The building was under construction at the time.

The nuts on the bolts connecting the column head to the column were stripped off their threads as the relatively long flexible bolts underwent significant bending as shown in Fig. 2-15. Such connections should be much stiffer and be strong enough to develop the full strength of the column.

Fig. 2-14: Failure of cantilever head connection on top of columns (1999 Kocaeli earthquake)

Fig. 2-15: Failure of column to cantilever head connection due to failure of bolts (1999 Kocaeli earthquake)

Fig. 2-16 shows the shear distress in a precast column of a one-storey parking structure in Sherman Oaks that occurred during the 1994 Northridge earthquake. Although the interior columns of this structure had been retrofitted following the 1971 San Fernando earthquake, no retrofit was carried out on the exterior columns. The 406 × 610 mm exterior columns were reinforced with #10 (32 mm diameter) vertical bars and had peripheral #3 (9.5 mm diameter) ties at 400 mm spacing.

Fig. 2-16: Beam-to-column connection failure and shear failure of column in precast concrete parking structure (1994 Northridge earthquake)

Not only did many of the inadequately reinforced columns suffer major shear distress but there were also several examples of failures of the connections between the beams and the columns. This connection consisted of connection pins connecting the beams to the columns. In some cases the pins were misplaced such that the pins were outside of the transverse reinforcement of the column, resulting in side shear failure of the cover concrete (see Fig. 2-17).

In other locations the connection pins were placed inside of the column ties. Although the spacing of the ties was reduced to about 85 mm near the top of the column, this reinforcement had no seismic crossties and the anchorage of the transverse reinforcement was made with 90° hooks. As a result the pins pushed out of the core and were not adequately restrained due to the lack of crossties and the loss of anchorage of the tie reinforcement (see Fig. 2-18).

Fig. 2-17: Brittle failure of beam-column pin connection at top of corner column (1994 Northridge earthquake)

Fig 2-18: Failure of beam-column pin connection at top of poorly detailed column (1994 Northridge earthquake)

2.6 Inadequate separation of non-structural elements

Fig. 2-19 shows the collapse of a precast concrete industrial building [Saatcioglu et al (2001)]. The building was under construction at the time. The presence of the stiff masonry infills that interacted with the columns resulted in column failure at the top of the partially collapsed walls where the vertical bars were spliced (see Fig. 2-20).

Fig. 2-19: Failure of columns due to interaction with stiff masonry infill (1999 Kocaeli earthquake)

Fig. 2-20: Failure of column at lap splice connection and at top of infill wall (1999 Kocaeli earthquake)

There were many examples of structures with out-of-plane failures of the masonry infilled walls [EERI (2000)].

2.7 Inadequate separation between structures

Fig. 2-21 shows an overall view of a precast concrete parking structure in Mexico City that partially collapsed in the 1985 Michoacán earthquake [Mitchell et al (1986)]. This frame structure suffered from pounding with an adjacent parking structure due to inadequate separation between the two structures. It also experienced failures at the precast joints between the beams and the columns resulting in brittle failures and poor energy absorption. It is essential that adequate separation between structures be provided to prevent pounding between the structures during an earthquake.

Fig. 2-21: Collapse of precast concrete parking structure due to pounding against adjacent structure in Mexico City (1985 Michoacán earthquake)

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3 Precast construction concepts

3.1 Types of elements in structural systems incorporating precast concrete

3.1.1 Introduction

Precast concrete elements are widely used as structural systems in many earthquake prone countries. Precast floor and roof diaphragms, moment resisting frames, structural wall systems and dual wall-frame systems that emulate cast-in-place concrete construction can be used to construct buildings to resist lateral seismic loads. Precast concrete elements may also be assembled with energy dissipating connections or other seismic damping systems that provide safe earthquake resistant structures without emulating the performance of cast-in place concrete.

3.1.2 Precast concrete floor units

Floor diaphragms may include a composite reinforced concrete structural topping layer to resist seismic shear, or may consist of precast units connected by welded or grouted connections (with a non-structural levelling topping), or may rely on grouted bars to resist diaphragm forces by shear-friction. Some precast floor elements are more suited to use without a structural topping layer than other types (see Table 3-1).

Precast floor units must be detailed to accommodate the localised displacements that are likely to be imposed on them by the actions of the primary lateral load resisting system. The localised actions that can impact on the design of the precast floor elements include:

- Plastic hinge formation in ductile moment resisting frames and the associated beam elongation that accompanies this.
- Gapping joints in hybrid post-tensioned frames.
- Strut and tie node points, where diaphragm forces pass around floor openings, irregularities in the floor plan, or zones of damage at plastic hinge locations.
- Sliding joints in vertically jointed wall systems.
- Transfer diaphragms where, for example, lateral forces from a tower structure are distributed into a lower level podium with different dynamic characteristics.
- Pinned, gravity-load beam connections in structures designed for large lateral displacements (an example is large area parking buildings with a perimeter lateral load resisting system).

The two procedures used to accommodate these actions are:

1) Isolate the precast components from any high displacement demands, with sliding supports and compressible joints. This method is preferred for relatively brittle extruded or slip-formed hollowcore sections, and for some of the more brittle beam and block flooring systems.
2) Reinforce the precast units, and any composite topping concrete to provide adequate ductility to resist the required gravity loads during, and after, the imposed displacements.

Precast floor type	Composite reinforced topping	Welded or grouted connections. No composite topping	No composite topping
Hollowcore units	Recommended for high seismic zones. Hollowcore units require special detailing in areas associated with high localised displacement.	Limited cyclic test data available.	Adequate for moderate seismic zones with shear friction bars between the precast slabs or with undulating shear keys and perimeter tie beams.
Double tees and single tees	Recommended for high seismic zones.	Used in high seismic zones	Not required with this system. Bars can be cast into the top flange.
Channel slabs	Recommended for high seismic zones	Used in high seismic zones	Adequate for moderate seismic zones. Bars can be cast into the top flange for high seismic zones.
Flat slabs	Recommended for high seismic zones	Used in high seismic zones	Adequate for moderate seismic zones. Bars can be cast into the top flange for high seismic zones.
Beam and block systems	Recommended for high seismic zones. Use without shear reinforcement requires special detailing in areas associated with high localised displacement.	Not practical with this system.	Not practical with this system.

Table 3-1: Recommended precast concrete diaphragm construction

3.1.3 Precast beams

Precast concrete beams can be designed either with connections to emulate the seismic performance of cast-in-place construction, or can be pinned at their supports, or can form part of a hybrid system – with end supports incorporating unbonded post-tensioning and damping devices. In each case, the designer may choose the depth of the precast portion of the beam to suit the required construction sequence. The options are:

1) The precast beam has its top surface at the underside of the precast flooring system (e.g. Fig. 3-1, Type 1). This method minimises the weight of the precast beam for transport and erection, and can simplify the end connections, but the half-beam usually requires extensive propping to support construction loads.
2) The precast beam has its top surface at the top of the precast flooring system (e.g. Fig. 3-1, Type 2). This method can eliminate mid-span propping, which reduces the dead-load continuity moments at the supports, and can therefore reduce the ductility demand on the columns.
3) The beam is precast full depth, to the finished floor level (e.g. Fig. 3-1, Type 3). This method simplifies site construction, but requires adequate seating for the precast floor units and careful consideration of construction tolerances and displacement compatibility.
4) A precast, pretensioned beam shell is cast with the top surface at the underside of the precast flooring (e.g. Fig. 3-2). The beam shell is often designed to support the full self-weight of the composite beam and the precast flooring system, spanning simply

supported between the columns. This system is extensively used in New Zealand in ductile frame structures up to six floors in height as it simplifies the assembly of the joint reinforcement.

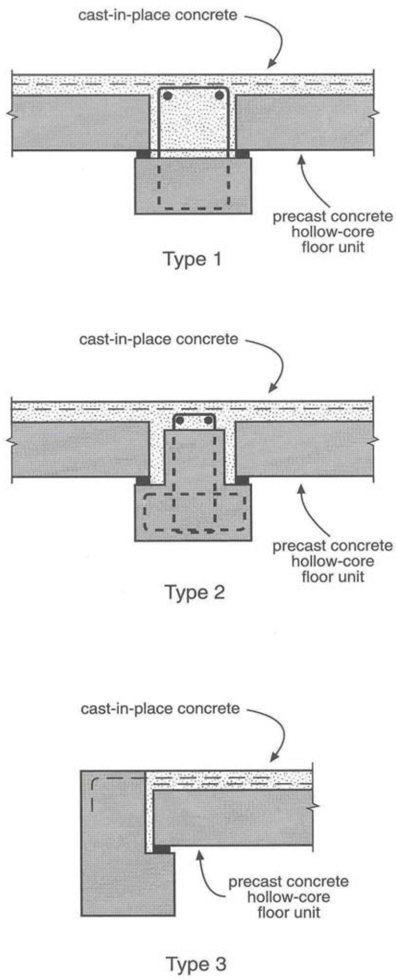

Fig. 3.1: Precast beam types [Centre for Advanced Engineering, 1999]

3.1.4 Precast columns

Precast concrete columns may be cast full height, for low-rise structures, or may be jointed at each floor. A variety of connection methods are used, but the common aim is to provide secure temporary support for the beams to allow fast erection and simple beam to column connections.

Pre-tensioned columns have been used for industrial buildings (e.g. Fig. 3-3, Caxton Pulp Mill). These columns can be detailed for ductility using the recommendations for ductile prestressed concrete piles [Park et. al. 1984].

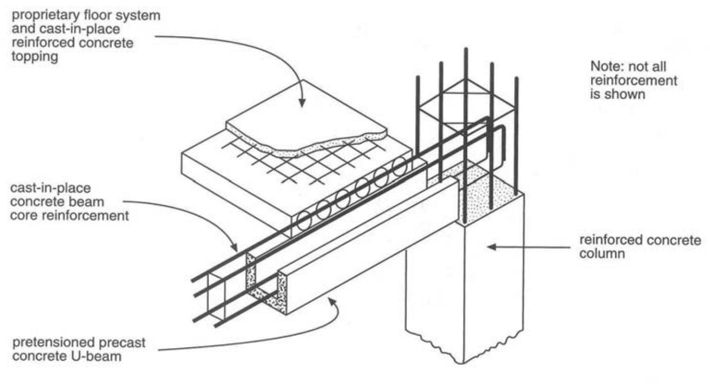

Fig. 3-2: Precast, pre-tensioned beam shell [Centre for Advanced Engineering, 1999]

Fig 3-3: Precast, pre-tensioned columns and rafter (Caxton Paper Mill, Kawerau)

3.1.5 Combined beam-column systems

Precast beam-column units, "**T**" shaped or cross-shaped, and spanning one to two floors are used to construct moment resisting frame structures (e.g. Fig. 3-4). They are fast to erect and have the advantage that the complex beam-column joint reinforcement is assembled in the precasting factory, rather than at the job site. Joints are located at the mid-span of the beams, and at mid height of the columns, away from the points of high ductility demand.

Fig. 3-4: Two storey precast beam-column units forming a perimeter ductile moment resisting frame

3.1.6 Precast structural walls

Precast wall construction may emulate cast-in-place construction, with lapped reinforcing bars in concreted or grouted joints (e.g. Fig. 3-5). Alternatively they may be designed with discrete joints that are capable of dissipatingenergy through ductile connections or damping devices (e.g. Fig. 3-6).

3.2 Types of connections between precast concrete elements in moment resisting frames and structural walls

3.2.1 General

The construction of moment resisting frames and structural walls incorporating precast concrete elements usually fall into two broad categories; either “equivalent monolithic” systems, or “jointed” systems. The distinction between these types of construction is based on the design of the connections between the precast concrete elements.

Fig. 3-5: Lapped reinforcing bars in wall joints, emulating cast-in-place construction.

Fig. 3-6: Wall panels with discrete joints and ductile connectors

3.2.2 Equivalent monolithic systems

The connections between precast concrete elements of equivalent monolithic systems (cast-in-place emulation) can be subdivided into two categories:

(1) **Strong connections of limited ductility**

Strong connections of limited ductility for equivalent monolithic systems are designed to be sufficiently strong that the connections remain in the elastic range when the building is satisfying the ductility demand imposed by the earthquake. That is, the yielding occurs elsewhere in the structure. Precast reinforced concrete elements have protruding longitudinal bars that are connected either by lap splices in a cast-in-place concrete joint, or by non-contact lap splices involving grouted steel corrugated ducts, or by splice sleeves, or by welding, or by mechanical connectors. These connections may have limited ductility if subjected to cyclic yielding. In moment resisting frames and structural walls these connections are protected by a capacity design approach that ensures that flexural yielding occurs away from the connection region.

An example of a strong connection of limited ductility of an equivalent monolithic system is the column-to-beam connection of System 2 of Section 5.2.4.1, which is designed to be strong enough not to enter the post-elastic range in a severe earthquake.

(2) **Ductile connections**

Ductile connections of equivalent monolithic systems are designed for the required strength and with longitudinal reinforcing bars or grouted post-tensioned tendons in the connection region that are expected to enter the post-elastic range in a severe earthquake.

In moment resisting frames, the plastic hinge region may extend a distance along the end of the member as in cast-in-place construction.

Experimental testing under simulated seismic loading has shown that provided the connection is well designed and constructed this system performs very well. Care is needed to prepare the ends of the beam and/or the column face at the connection with adequate mechanical keys or roughness to transfer the vertical shear force.

An example of a ductile connection of an equivalent monolithic system is the beam-to-column connection of System 1 of Section 5.2.4.1, where plastic hinging is expected to occur in the beam at the column face in a severe earthquake and yield penetration will occur into the connection region.

3.2.3 Jointed systems

In jointed systems the connections are weaker than the adjacent precast concrete elements. Jointed systems do not emulate the performance of cast-in-place concrete construction. The connections between precast concrete elements of jointed systems can be subdivided into two categories.

(1) **Connections of limited ductility**

Connections of limited ductility, for jointed systems, are usually dry connections formed by welding or bolting reinforced bars or plates or steel embedments, and dry packing and grouting. These connections do not behave as if part of monolithic construction and generally have limited ductility. An example of a jointed system involving structural walls is tilt-up construction. Generally such structures are designed for limited ductility or elastic behaviour.

(2) **Ductile connections**

Ductile connections of jointed systems are generally dry connections in which unbonded post-tensioned tendons are used to connect the precast concrete elements together. Such connections can be designed to perform in a ductile manner. The post-elastic deformations of the member are concentrated at the interface of the precast concrete elements where a crack opens and closes. The unbonded post-tensioned tendons remain in the elastic range.

Hybrid systems have dry connections, which combine both unbonded post-tensioned tendons and longitudinal steel reinforcing bars in moment resisting frames, or unbonded post-tensioned tendons and energy dissipating devices (e.g. flexing steel plates or friction devices). The post-elastic deformations of the member during an earthquake are concentrated at the interface of the precast concrete elements where a crack opens and closes.

Experimental testing under simulated seismic loading has demonstrated that hybrid systems, if well designed and constructed, perform at least as satisfactorily as ductile connections of equivalent monolithic systems, and have the added advantage of reduced damage and of being self-centering (i.e. there is practically no residual deformation) after an earthquake.

3.3 Tolerances, fabrication and erection issues

3.3.1 Introduction

The design and construction of precast concrete structures to resist seismic actions requires an appreciation of product and construction tolerances; knowledge of the capabilities and limitations of precast element production processes, and an understanding of how the structure will be safely assembled. In many respects seismic resistant precast concrete construction is similar to non-seismic precast, but with tighter tolerances on some connection details. The more complex connections that are used for seismic construction will often also require access for specialised equipment. The designer and erector must also have an appreciation of displacement compatibility (as it relates to seismic separation and the protection of brittle elements) to ensure that the structure will behave in the intended manner.

3.3.2 Tolerances

There are three types of tolerances that apply to precast concrete construction:

1) **Product tolerances**

Industry standard product tolerances [Prestressed Concrete Institute (1985a)] should be used for design of precast components and joints where possible (see Table 3-2). Many seismic joint details however will require tighter tolerances (for the location of connection hardware) than the precast construction industry would normally work to. This requires consultation between the designer and representatives of the precast industry, to ensure that those critical tolerances are practical. Examples of connections with limited tolerances are:

- Proprietary grouted sleeve connectors.
- Hooked or bent reinforcing bars that are lapped in congested joint regions.
- Weld plates where eccentricity must be minimised.
- The location and alignment of post-tensioning ducts.

With high quality formwork, and the degree of repetition normally encountered on precast concrete structures, these tighter than normal tolerances do not significantly affect the cost of construction.

Product	Length	Width	Depth or thickness	Location of embedded items
Flooring units	+/- 25mm	+/- 6mm	+/- 6mm	+/-12mm
Beams	+/- 20mm	+/- 6mm	+/- 6mm	+/-12mm
Columns	+/- 12mm	+/- 6mm	+/- 6mm	+/-12mm
Wall panels	+/- 12mm	+/- 6mm	+/- 6mm	+/-12mm

Table 3-2: Examples of Typical Product Tolerances [Prestressed Concrete Institute (1985a)]

2) **Erection tolerances**

Industry-standard erection tolerances [Prestressed Concrete Institute (1985b)] should be used for the design of precast components and joints (see Table 3-3). Where tolerances tighter than these are required, they should be very clearly noted on the contract drawings. Examples of locations where accumulated erection tolerances may affect the seismic performance of the structure are:

- The width of seismic gaps.
- The rotation clearances of pinned gravity-load joints.
- The connection of stairs and ramps.
- The alignment of weld plates, dowels or ducts.
- Minimum seating for precast floor slabs.

Product	Elevation of bearing surfaces	From reference grids	Plumb variation	Joint width
Columns	+12mm, -6mm	+/- 12mm	6mm in 3m	n/a
Beams	+12mm, -6mm	+/- 25mm	3mm /300mm	12mm (exposed) 20mm(hidden)
Floor units	+/- 20mm (topped) +/-6mm (untopped)	+/- 25mm	n/a	+/-12mm
Wall panels	+/- 12mm	+/- 12mm	+/-6mm/3m	+/-10mm

Table 3-3: Examples of Typical Erection Tolerances [Prestressed Concrete Institute (1985b)]

3) **Interfacing tolerances**

Interfacing fit affects the way in which other materials, installed at some other time, or by other trades, interact with the seismic resistant precast structure. Sub-contractors installing their components before or after the precast has been erected must understand the importance of critical clearances. Examples of interfacing considerations are:

- The separation of non-structural masonry partition walls from the primary lateral load system to limit damage and to ensure that the non-structural components do not contribute adversely to the torsional stiffness of the structure.
- Glazing and curtain wall separations.
- Mechanical services openings and attachments. These should preferably be located away from beam hinging zones or strut and tie node points.
- The choice of fire and acoustic stopping materials.

3.3.3 Fabrication

The economical production of precast concrete requires repetition in the use of formwork and an understanding, by the designer, of how a precast element will be cast. Standard hollowcore floor slabs, double tees and other mass produced components can be easily adapted for seismic structures, but the range of options for inclusion of fully ductile connections or projecting reinforcing bars may be limited by the production method.

A precast manufacturer will also be aware of transportation and handling limitations, which may affect the maximum size of precast elements and hence the location of joints.

Many seismic connections require more precise tolerances than precast concrete connections for non-seismic construction. Adequate training, repetition of standard details and a suitable quality assurance system are the keys to successful precast production.

3.3.4 Erection of seismic resistant precast structures

The erection of seismic resistant precast concrete construction differs from non-seismic precast by requiring tighter tolerances in limited number of key areas, such as:

- minimum floor slab seating that must be maintained;
- seismic separations must not be reduced by accumulated tolerances,
- rotation compatibility for pinned gravity connections will require careful attention to the location and thickness of bearing pads and clearances between beam ends and supporting columns.

Seismic connections may also require access for specialised equipment (grouting, welding, or post-tensioning) and access for more than one worker. Detailing a temporary support structure that provides propping, bracing and safe access ensures that a high quality joint can be easily assembled and checked (e.g. Fig. 3-7).

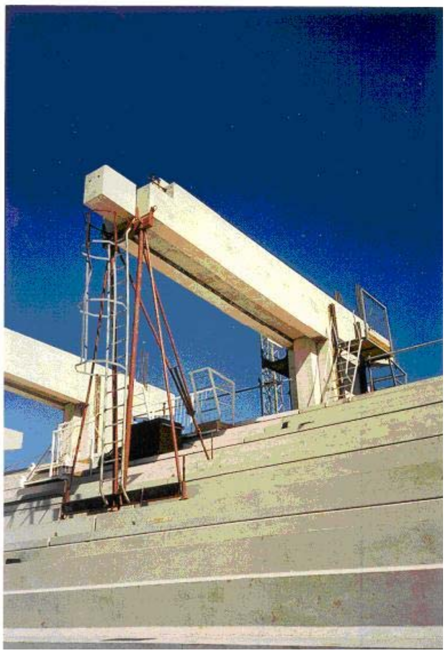

Fig. 3-7: Temporary support with access ladders and work platforms for grouted post-tensioned joints

3.4 Materials and quality assurance

3.4.1 Introduction

The use of materials of appropriate quality, correctly installed, is essential if the precast structure is to have the required seismic performance. Because many of the components are difficult to inspect after installation, a comprehensive quality assurance program is required, both for the precast component fabrication and for the site installation.

Alignment affects the performance of all precast connections. Quality assurance records must include details of any joint misalignment and the means of resolving any lack of fit.

3.4.2 Grouted connections

Precast concrete components incorporating these connections are fast and simple to install. Precast factory production checks must ensure that the dowels and sleeves are correctly located and that the required embedment length can be achieved in the assembled connection. Grout tubes must be clear of obstructions and dowel bars must be the correct grade of steel.

On site, the connections must be correctly shimmed to level and sealed to contain the grout. Proprietary pre-packaged grouts with the correct strength and flow characteristics are strongly recommended. The grouting sequence and grout inlet points must be selected to ensure complete filling of the joint [Thurston (1986)]. Temporary props and bracing must remain in place long enough to ensure that the joint components do not move while the grout is curing.

A pressure pump is recommended, to ensure a smooth and continuous flow of grout across joints and into vertical ducts. Quality assurance records should record the start and finish time for each joint, measures of the grout flow characteristics and its strength. Pumped grout volumes must be checked against the theoretical joint volume and any discrepancy must be accounted for. Grout strength must be tested at 28 days, and at the time the braces or props are removed, or when the next level of precast is installed.

Grouted joints must be protected from rapid drying, or from freezing.

3.4.3 Welded connections

All welding, in the precast factory and on the site, must be carried out by competent welders, and in accordance with best industry practice. Quality assurance records should show evidence of the welder’s competence. Ensuring that all weld plates in a precast component are electrically connected to the reinforcing steel during manufacture allows faster assembly on the site.

Welding procedures and joint preparation must be selected to ensure ductile welds of adequate strength to resist the imposed seismic forces and displacements. Site welding can cause minor concrete spalling around weld plates unless the plates are edged with compressible tape to allow for thermal expansion.

In some countries, some grades of reinforcing steel are not weldable, while other bar grades require specialised welding procedures. Reinforcing bar welding procedures should be confirmed by the bar manufacturer, or by testing.

Welded connections will often require fire or corrosion protection.

3.4.4 Bolted connections

The key considerations in bolted connections are the length, strength grade and tension, of the bolts. For seismic connections, some bolts will require preset tensions to control service load deflections while other bolts may require to be loose to allow members to slide. Quality records must show evidence of bolt tension control.

Nuts must be capable of developing the strength of the bolt, and must be locked with jamb-nuts where required.

Bolt, or threaded insert embedment depths must be sufficient to ensure ductile behaviour where this is required to resist seismic loads. Local reinforcement may be used to enhance ductility with reduced embedment. Ductile yielding of the local reinforcement should occur before a pullout failure from the concrete.

Bolts will often require corrosion protection.

3.4.5 Post-tensioned connections

Post-tensioned connections require the use of tested proprietary hardware, with tendons correctly stressed and grouted, or ungrouted, as required by the design. Manufacture, installation and grouting quality assurance procedures should follow industry best practice [FIP (1998)]. Unbonded tendons must be protected from corrosion [***fib*** (2001)].

Connections in some hybrid post-tensioned structures may require the positioning of dampers or energy dissipaters in the joints. These structures may also require fibre, or mesh-reinforced grout, to prevent the grout dropping out as the joints open under seismic actions.

3.4.6 Cast-in-place concrete connections

Concrete joints should use concrete of similar strength to the concrete in the adjacent precast components. Clearances must be adequate to allow consolidation of the joint concrete if self-compacting concrete is not used.

Joints should be flushed with water to pre-wet them before the concrete is placed, but must not contain free water.

Joints must be adequately cured, and must be protected from freezing.

3.5 Serviceability considerations

3.5.1 Introduction

The seismic performance of a precast concrete structure can be adversely affected if the designer fails to consider the long-term effects of service load conditions. Reduced or damaged seating areas can then fail prematurely when subjected to the vertical acceleration and displacement of even moderate earthquakes. Corrosion can also severely reduce the long-term strength of seismic connections.

3.5.2 Temperature effects

Thermal movements can reduce the minimum seating length of precast flooring units in large area carpark structures. The use of movement-limiting connections is recommended to ensure that temperature shortening in cold weather occurs uniformly at all control joints, rather than at the joint that offers the least resistance to movement.

The thermal bowing that occurs due to differential temperature gradients through a precast floor or roof system [Williams et. al, Fintel et. al, (1988), and Libby (1987)] can also adversely affect the seismic performance of a precast structure. Unless the seating details of the affected precast elements are designed to accommodate this regular thermally induced sliding, low cycle fatigue will lead to spalling at the beam edge, and/or at the end of the precast floor unit (e.g. Fig. 3-8). If it is not detected and repaired, this spalling can lead to collapse of the upper floor, and at worst, a progressive collapse of the whole structure during relatively minor earthquakes. Low friction plastic bearing pads that are textured to grip on the beam or wall support, while allowing the floor or roof unit to slide, are a simple method of eliminating this potential problem.

3.5.3 Creep and shrinkage

The long-term effects of restrained creep and shrinkage can lead to floor or roof diaphragm connections being overloaded before they are subjected to any seismic loads. Failures of long span pre-tensioned roof units during the 1964 Anchorage, Alaska earthquake have been attributed to this effect [Sutherland, 1965].

Connection details must be designed with sufficient ductility to accommodate the combined effects of long-term creep and shrinkage, in the floor or roof units, without causing unmanageable restraint forces at the lateral load resisting vertical elements of the structure.

Base isolated structures are able to accommodate creep and shrinkage movements without generating high restraint forces, but isolating devices may require re-centring after the most of the creep and shrinkage has occurred. Clearances provided to accommodate ground movements must allow for long-term effects.

Fig. 3-8: Double tee, stepped end detail, that has been damaged by thermal bowing in a parking building

3.5.4 Durability

Precast structures designed for earthquake resistance must also be detailed for adequate durability. For exposed parking structures, in marine coastal areas, or in those regions that apply salt to roads for winter safety, corrosion protection and regular maintenance is essential. Connection details and maintenance programs should be designed to ensure that the full design strength and ductility of the connections is available for the life of the building.

Structures that incorporate unbonded post-tensioning cables to restore them to their original pre-earthquake alignment, are not expected to develop permanently open cracks, but unbonded cables must be protected against corrosion [***fib*** (2001)].

3.5.5 Fire and acoustic performance

Seismic gaps, installed to ensure that the separate parts of the structure will deform without adversely influencing the performance of adjacent parts, must be maintained in spite of the need to comply with fire resistance rating and acoustic requirements. Similar separation requirements occur around non-structural partitions (or elements such as stairs or ramps). Compressible fibre packing has proven to be adequate for this purpose.

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4 Design approaches

4.1 Performance objectives and limit states

4.1.1 General

Performance objectives define the level of functionality that a structure is to have under a specific level of earthquake ground motion. Table 4-1 defines terms related to performance objectives. A performance objective is a combination of a limit state (or performance level) with an earthquake hazard level. An example of a performance objective is no collapse under an earthquake with 5% probability of occurrence in 50 years.

Limit states define the state of the structure, member or cross-section. Examples of each type are given in Table 4-2.

In the past, performance objectives have not been stated explicitly or in great detail. For example, the Structural Engineers Association of California (SEAOC) stated in the 1988 "Blue Book" [SEAOC (1988)] that the objective of the recommendations is to limit damage to moderate levels in a moderate earthquake and for the structure not to collapse in a severe earthquake. This has been the implicit objective for most building codes. This assessment of the seismic performance to be expected from code-designed structures was based on the judgment of the SEAOC seismology committee rather than on a technical evaluation of seismic performance.

Performance-Based Engineering has attracted significant attention since about 1995, following publication by SEAOC of *Vision 2000: Performance based seismic engineering of buildings* [SEAOC (1995)]. No consensus yet exists over the exact definition of performance-based engineering, but it implies that designers should be able to design a structure to perform at a selected level in response to an earthquake of a specific hazard level. In the U.S., performance-based engineering is the subject of several research projects, and is identified as a central focus of the Pacific Earthquake Engineering Research (PEER) Center [www.peer.berkeley.edu].

4.1.2 Structural performance levels

Traditionally, three limit states or performance levels have been used to describe overall structural performance [Paulay and Priestley (1992)]:

(1) Immediate occupancy (or serviceability)

A structure that can be immediately occupied after an earthquake typically would have no sign of structural distress, and limited non-structural damage. Perception may play a significant role in judging the state of a building, since tagging of buildings may be done by individuals who are not structural engineers, and who have been brought in at short notice directly after an earthquake with scant training. To be "green-tagged", (i.e. occupation is permitted) a building must not only be undamaged, but it must also appear to be undamaged.

In reality, some structural damage can be accepted in a green-tagged building, if that damage does impair the structure's ability to be safe in an aftershock.

Earthquake Hazard Level	The measure of severity of earthquake effects at a given site, usually expressed in probabilistic terms. For example a 5% chance of exceedence in 50 years. See Section 4.2 of this report.
Limit State	A limit state defines the specific condition of a structure, member, or cross-section. Examples of limit states include *performance levels* and member conditions such as listed in Table 4-2.
Performance Level	A performance level is a specified degree of functionality for a structure after it experiences an earthquake. A performance level is the same as a *limit state,* although the term performance level is typically applied to the performance of an entire structure rather than to a member or cross-section. Examples of performance levels include immediate occupancy, damage control, and collapse prevention. Performance levels can correspond to the performance of structural members such as beams, columns, or walls, or to that of non-structural elements or systems such as cladding or utilities.
Performance Objective	A performance objective or goal consists of a *performance level* associated with an *earthquake hazard level.* An example would be a performance level of “immediate occupancy” for an earthquake hazard level with a 50% chance of exceedence in 50 years. Structures are typically designed for more than one performance objective.
Performance-Based Engineering	Refers to engineering in which performance objectives are explicitly chosen or defined. Performance-based engineering or design is sometimes taken to mean the use of involved analyses and research results in design, but others interpret that simplified or prescriptive design rules qualify as performance-based engineering if they are accurately derived from research.

Table 4-1: Terms related to seismic performance objectives

(2) Damage control

At this performance level, the structure should not be occupied after an earthquake, but repairing structural damage is feasible and economical. Repairable damage covers a wide range. An owner’s decision to repair may depend on how many elements have been damaged as much as on the seriousness of the damage in each element.

(3) Collapse prevention (or survival)

At this performance level, damage is extensive, but the structure does not collapse, and thus the potential for casualties is reduced.

Global structural performance levels	Immediate occupancy Damage control Collapse prevention
Global structural engineering limit states	P-Δ instability Limitation on peak story drift Limitation on residual story drift
Member limit states	Flexural cracking Shear cracking Limitation on residual crack width Flexural yielding Concrete spalling Shear failure
Cross-section limit states	Unbonded post-tensioning remains elastic Concrete strain of 0.005 Steel strain of 10 times yield strain

Table 4-2: Examples of limit states applicable to precast reinforced or prestressed concrete buildings

An interesting aspect of the above three performance levels is that they each relate to a different structural characteristic. Serviceability depends largely on structural stiffness. damage control depends on strength, and collapse prevention depends principally on ductility capacity [Paulay and Priestley (1992)].

In the FEMA 356 seismic retrofit guidelines [FEMA (2000)], a performance level of “life safety” is defined in addition to those listed above.

In structural engineering practice, performance objectives are often most useful if they have been specifically defined by the owner and engineer to correspond to the specific use of the structure. For example, the seismic retrofit of the Thorndon Overbridge in Wellington, New Zealand, considered the following three limit states: full service to traffic, limited service (the bridge is acceptably safe for emergency vehicles, while other traffic is prohibited), and collapse prevention. For hospital buildings, unique performance levels of interest include [ATC (2002)]:

- The ability to allow continued occupancy of the most acute patients and attending staff, even if damage has occurred. (This is because of the risk involved in evacuating acute patients.)
- The ability to access a damaged building to retrieve medical equipment, supplies, medicine, or records.

For buildings such as electrical substations that are part of utility lifelines systems, the ability to access a damaged building to operate switches can be an important performance level.

4.1.3 Non-structural performance levels

Non-structural limit states or performance levels relate to the performance of building utilities, systems, or components. Building systems include electric power, water, fire protection sprinklers, elevators, natural gas, and telephone. In essential facilities like fire stations, police stations, or hospitals there can be additional systems and an increased importance of non-structural performance. The performance of building components such as partitions and cladding, and the protection of potentially hazardous contents, are also non-structural performance considerations.

Nonstructural performance relies on the design of the building systems or components themselves to accommodate seismic motions, and on the structural response. Building equipment and systems can be affected by the level of acceleration in the structural response, and also by relative displacements induced, for example between stories.

4.1.4 Parameters and response measures associated with performance

Table 4-2 shows that limit states can be general performance levels, such as damage control, or specific engineering measures, such as limitations on story drift or concrete strain. To design a structure that achieves certain performance goals, the engineer must relate general performance levels to more specific parameters or response measures, which can depend on the type of structure. Examples of specific response measures — peak drift, residual drift, floor accelerations, and reinforcement strain — are discussed below.

The peak drift reached during earthquake shaking is a principal parameter affecting structural instability and damage to nonstructural systems that span between floors (such as partitions or piping).

Consideration should be given to the relationship between drift and damage for different classes of structure. For example, parking structures, which constitute a large part of the market for precast concrete in the US, seldom have partitions or non-structural walls. Such structures could accept drift levels related solely to the structural drift capacity (and if necessary to elevator performance).

The residual drift after an earthquake can affect the post-quake functionality and reparability of the structure. If the residual drift is too large, doors and windows may not open and elevators may not run properly. Large residual drifts could make the repair of a structure uneconomical.

High acceleration can also lead to damage. High accelerations are likely to occur in stiff systems with short periods. Acceleration can affect building equipment or its mountings, including computer equipment.

There are some key differences between different types of precast concrete systems in how well they can achieve particular performance levels. Jointed precast systems with un-bonded post-tensioning are well suited to reducing or eliminating residual deformation. Jointed systems also suffer less concrete cracking than monolithic (or equivalent-monolithic) concrete systems because earthquake deformations occur in joint openings rather than in cracks in the concrete. By reducing residual drift and cracking, jointed systems can more easily achieve immediate occupancy or damage-control limit states.

The maximum strain in steel reinforcement in monolithic concrete systems is a function of rotational demands at plastic hinges and the effective plastic hinge length. In jointed concrete systems, reinforcement strain is often controlled by de-bonding a specified length of the reinforcement at the location of the joint that accommodates seismic rotations.

4.2 Representation of earthquake ground motion

4.2.1 General

The seismic design of buildings — precast concrete or otherwise — requires a characterization of the earthquake ground motion. Typically the seismic hazard is related to the probability of the specified ground motion. Probabilistic seismic hazard can be expressed either as a return period (e.g. 475 years) or as a probability over a specified time period (e.g. 10% chance of exceedence in 50 years). The two types of presentations can be related according to equation 4-1:

$$T_r = - I / \ln(1 - P) \qquad (4\text{-}1)$$

where T_r = return period, years; I = time interval of interest, years; P = probability of exceedence in that time interval.

The probabilistic seismic hazard includes contributions from all identified earthquake faults that can significantly affect the site. Occasionally, ground motion information is given for a deterministic rather than a probabilistic seismic hazard. In the deterministic case a single earthquake fault and scenario is considered.

Ground motion input for structural analysis can be represented by response spectra or by time-history records. Design response spectra are used as the input to linear static and linear dynamic analyses. They are also used to characterize demand versus capacity for nonlinear static analyses. Time-history records are used as the input to nonlinear dynamic analyses.

4.2.2 Earthquake response spectra

4.2.2.1 General

The ground motion of a site for a given return period is typically represented by a design response spectrum. The response spectrum shows ground motion amplitude across a range of period of vibration. The response spectrum can be represented as acceleration versus period, deformation versus period, acceleration versus deformation, or on a tri-partite log-log plot.

A response spectrum shows the peak deformations, velocities, or accelerations of elastic single-degree-of-freedom systems of different periods to the specific ground motion. Response spectra can have different curves that give the responses of systems with different amounts of viscous damping.

When developed for a specific ground motion, the response spectrum is jagged. The local peaks are functions of the specific ground motion and are likely to be different for another earthquake at the same site. If several individual spectra are averaged, a smoother curve is obtained. In designing structures for future earthquakes, a design response spectrum is created. The design response spectrum is smooth because it does not have local peaks caused by idiosyncratic effects of an individual ground motion.

4.2.2.2 Development of design response spectra

The engineer determines the design response spectrum for a site based on either a site-specific study done by a ground-motion expert, or on seismic zone maps of spectral ordinates combined with soil-profile factors. Somewhat different approaches can be behind the creation of design response spectra, but typically the information needed to create a site-specific spectrum or a seismic zone map includes:

- what earthquake faults are present;
- characteristic magnitude of each fault;
- earthquake probability for each fault;
- distance from each fault to the site;
- attenuation relationships that relate ground motion parameters to magnitude and distance.

The information used in estimating ground motions at a site has a significant amount of uncertainty and variability.

4.2.2.3 Presentation of response spectra

Earthquake response spectra can be presented in various ways. Seismologists sometimes present response spectra as a function of frequency on a logarithmic scale. For structural engineering, it is best to present response spectra on a non-logarithmic scale, and using period of vibration, T, rather than frequency, ω. ($T = 2\pi/\omega$) The ordinate of the response spectra is typically spectral displacement S_d or spectral pseudo-acceleration S_a. Spectral displacement is the peak relative displacement between the structure mass and the ground for a single-degree-of-freedom structure of given period. Spectral pseudo-acceleration is nearly identical to true spectral acceleration and is more convenient to use [Chopra (2001)] because of its relationship to spectral displacement given by:

$$S_d = (T^2/4\pi^2)\, S_a\, g \qquad (4\text{-}2)$$

where S_d = spectral displacement, cm; T = period of vibration, seconds; S_a = spectral pseudo-acceleration, as a fraction of g; and g = gravitational constant = 981 cm/s^2.

This relationship is used to relate three useful ways of presenting earthquake response spectra, shown in Figure 4-1. The figure shows the site-specific design response spectrum for the design of a building near San Francisco, California. The spectrum corresponds to 5% viscous damping and a 10% probability of exceedence in 50 years (475-year return period.) Figure 4-1(a) shows acceleration plotted versus period. Figure 4-1(b) shows spectral displacement plotted versus period. Figure 4-1(c) shows acceleration plotted versus spectral displacement. In this presentation, lines of constant period are radial lines.

Each of the three presentations contains exactly the same information. The different ways of presenting the information are more suitable for different design approaches. As discussed in Section 4.6, an acceleration-versus-period spectrum is typically used for force-based design. Acceleration-versus-displacement or displacement-versus-period presentations are convenient for displacement-based design. The acceleration-versus-displacement presentation is particularly interesting because the structure’s force displacement capacity curve can be plotted on the same coordinates. [ATC-40, (1997)]. This way of representing response was first suggested by [Freeman et al, (1975)].

4.2.2.4 Characteristics of response spectra

Systems with very short periods, less than about 0.03 seconds, are so stiff that they move essentially as rigid bodies with the ground. Thus the spectral acceleration as T approaches zero is equal to the peak ground acceleration. Systems with very long periods, perhaps greater than about 15 seconds, are so flexible that the mass essentially remains still while the ground moves beneath it. Thus the spectral displacement as T approaches infinity is equal to the peak ground displacement.

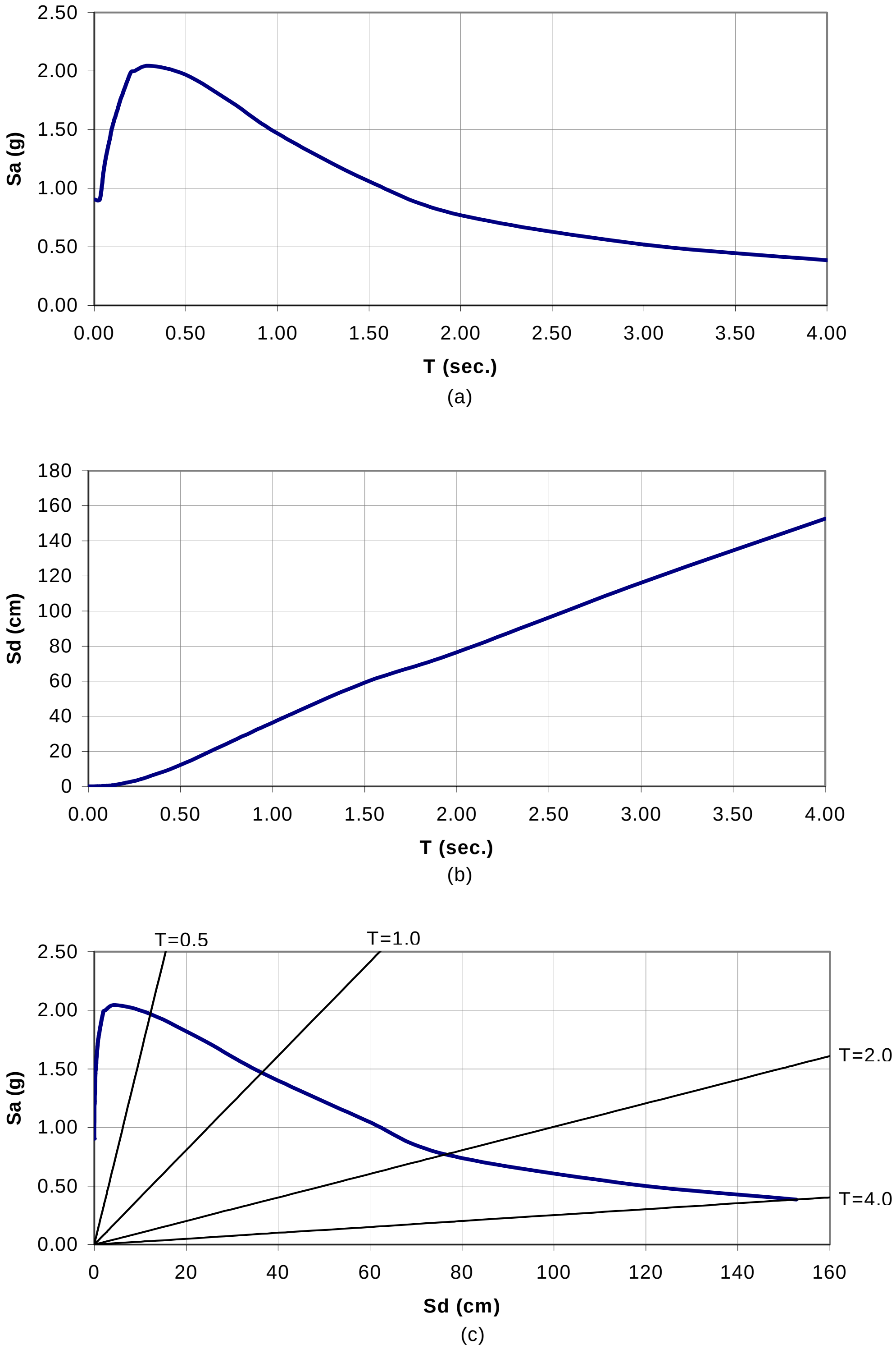


Fig. 4-1: Site-specific design response spectrum for a building near San Francisco, California, 975 year return period, 5% damping.
(a) Acceleration versus period, (b) displacement versus period, (c) acceleration versus displacement

Between these two extremes, [Blume, et al (1961)] identified three zones: an acceleration-controlled region, a velocity-controlled region, and a displacement-controlled region. Chopra [2001] gives details of how these regions can be defined and how spectral values can be assumed to vary across the regions of the response spectrum.

4.2.3 Earthquake time-history records

Time history records are taken from recorded ground motions in past earthquakes. A time history provides the full details of a single ground motion recording. Time history records are needed if nonlinear dynamic analysis is to be conducted.

A time history record consists of numerical values of ground acceleration taken at a short time step over the duration of earthquake shaking. An example of a ground acceleration record, given the textbook by [Chopra (2001)], contains 1559 data points at equal time spacings of 0.02 seconds. The data points represent the north-south component of the ground motion recorded in El Centro, California during the earthquake of 18 May 1940.

Hundreds of ground motion records, for various faulting types, magnitudes, and distances, are available from research databases. In structural engineering practice, a ground-motion expert usually selects and scales the earthquake records to be used in the analysis. The records are typically scaled in amplitude to be representative of the design response spectrum developed for the site. The records are selected from the database of available records to have faulting mechanisms, magnitudes, and fault distances compatible with the situation at the site. Occasionally, synthetic records are used rather than recorded accelerations.

Time-history analyses always demonstrate significant record-to-record variability. For this reason, a number of records must by used. The IBC and UBC design codes [ICC (2000), ICBO (1997)] require taking the average result of a minimum of seven appropriate records, or the maximum result of a minimum of three appropriate records.

The practice of creating or scaling records to closely match a design response spectrum in all period ranges must be used cautiously. Depending on the method of scaling used, this practice can produce earthquake records with greater energy input than actual records [Naeim (1995)].

4.3 Capacity design

4.3.1 General

Before about the mid 1970s it was customary in the seismic design of structures to use linear elastic structural analysis to determine the bending moments, axial forces and shear forces due to the design gravity loading and seismic forces and to design the members to be at least strong enough to resist those actions. The engineer was not required to check the nonlinear mechanism of response or the governing behavior mode of members. Nor did the engineer choose or identify the regions of the structure, such as plastic-hinge regions, where nonlinear behavior was to take place.

As a result, when the structure as designed and constructed was subjected to a severe earthquake, the manner of post-elastic behaviour was a matter of chance. Flexural yielding of structural members could occur at any of the regions of maximum bending moment, and shear failures could also occur, depending on where the flexural and shear strengths of members and joints were first reached. Hence the behaviour of such structures in the post-elastic range was somewhat unpredictable.

In New Zealand in the 1970s it was realised, [Park and Paulay (1975)] that the possible effects of overstrength of some regions of the structure need to be carefully considered in seismic design to ensure that undesirable failure modes do not occur during severe earthquakes. For this reason the design approach termed capacity design was developed in New Zealand. The basic idea of capacity design was first proposed by [Hollings (1969)] and developed by discussion groups of the New Zealand National Society for Earthquake Engineering in the 1970s and by [Park and Paulay (1975)] and more recently by [Paulay and Priestley (1992)]. New Zealand design standards have required since 1976 that ductile structures and structures of limited ductility be the subject of capacity design.

The design recommendations of this report require capacity design for precast structures that respond in the nonlinear range.

4.3.2 Definition of capacity design

4.3.2.1 General

Capacity design is a seismic design approach in which distinct parts of the structure, such as member plastic hinges or rotating joints, are chosen and detailed for nonlinear behavior according to an identified mechanism of nonlinear response. Capacity design includes three essential steps:

1. A mechanism of nonlinear lateral displacement is chosen for the entire (global) structure, which identifies the distinct locations of the structure that will experience nonlinear behavior. In precast concrete structures, the nonlinear locations will typically be flexural plastic hinges in members or rotating joints in or between members.
2. The nonlinear locations of the structure are given adequate design strength and ductility capacity for the seismic demands.
3. All other regions of the structure and other possible behavior modes are then provided with sufficient strength to ensure that nonlinear behavior occurs in the intended locations according to the chosen mechanism, and that undesirable failure modes do not occur.

These steps are each discussed in the sections below.

4.3.2.2 Global mechanism of nonlinear response

The mechanism of nonlinear lateral displacement for a structure is determined by the relative strengths for each member and action affecting the structure. A plastic or nonlinear analysis can be used to determine the governing mechanism.

Some mechanisms of nonlinear response include story mechanisms such as shown in Figure 4-2(b). The story mechanism is undesirable in structures over about two stories because all of the nonlinear displacement demand for the building is concentrated in the single story with column hinging. This causes large rotational demands on the column plastic hinges of that story, and if the demands exceed the ductility capacity of the column ends, catastrophic collapse will result.

By contrast, the mechanisms shown in Figure 4-3 are such that none of the plastic hinges suffer excessive rotation.

In concrete wall structures with high ductility demand, shear failure in diagonal tension is an unacceptable mechanism and mode of behavior for two reasons. First, shear failure is an undesirable behavior mode at the member level, because it involves rapid strength degradation upon failure. Secondly, in a taller wall structure, a shear failure acts like a story mechanism because nonlinear deformation tends to concentrate at a single story or over a limited height, as shown in Figure 4-4 and Figure 4-5.

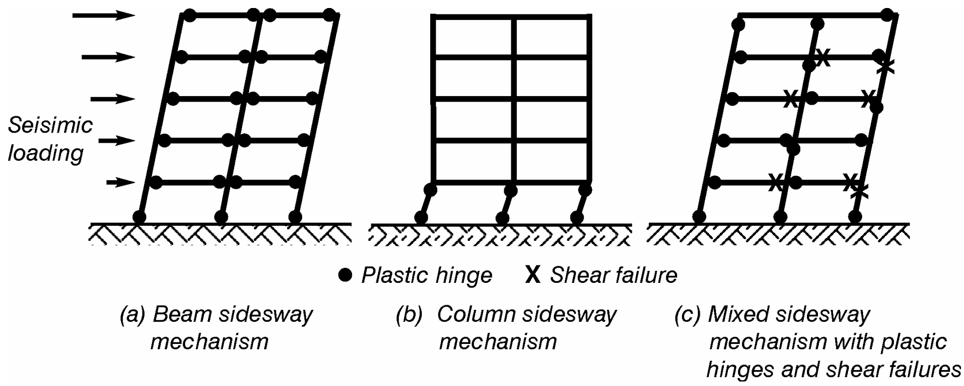


Fig. 4-2: *Some mechanisms of post-elastic deformation for tall seismically loaded moment resisting frames in the post-elastic range [Park et al (1975, 1997)].*

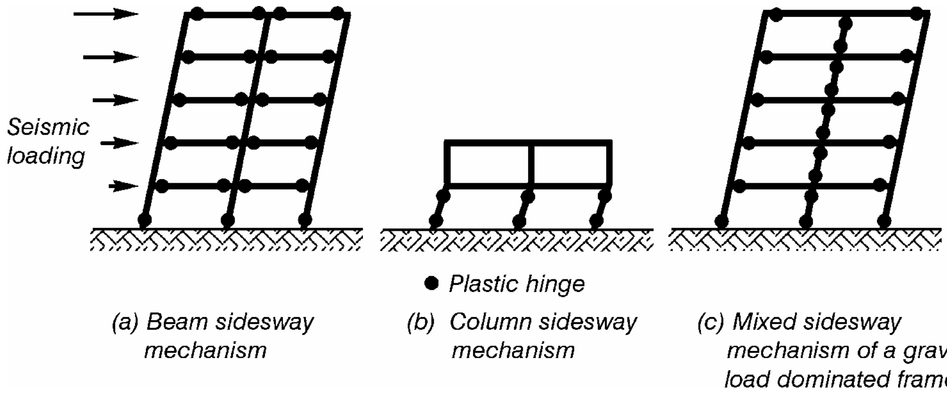


Fig. 4-3: *Desirable mechanisms of post-elastic deformation of moment resisting frames during severe seismic loading according to standards New Zealand (1995)*

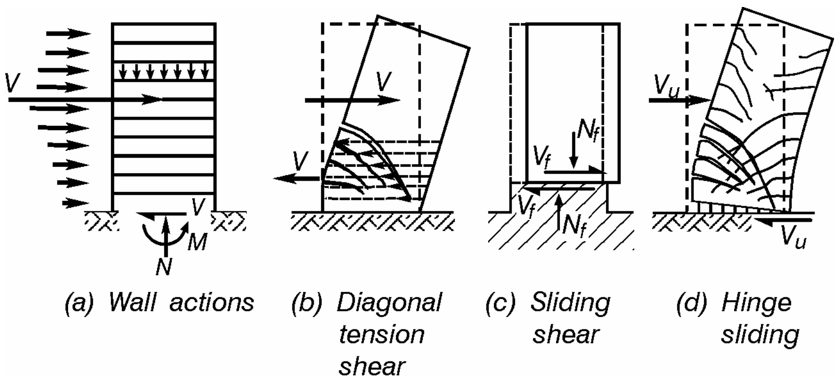


Fig 4-4: *Undesirable modes of behaviour for seismically loaded cantilever structural walls in the post-elastic range [Park et al (1975, 1997)]*

In cast-in-place concrete structures and equivalent-monolithic precast structures the acceptable nonlinear mechanisms typically consist of flexural plastic hinges acting as the nonlinear locations. In jointed precast structures the nonlinear mechanism typically consists of rotation at selected joints. Rotation at a precast joint is similar to a flexural plastic hinge, except for the differences discussed in Section 4.5.

In addition to plastic hinging and joint rotation, rocking between a structure's foundation and the supporting soil can be an acceptable nonlinear location. In the special case of seismically isolated structures, the nonlinear mechanism typically consists of all nonlinear lateral displacement occurring in the isolators.

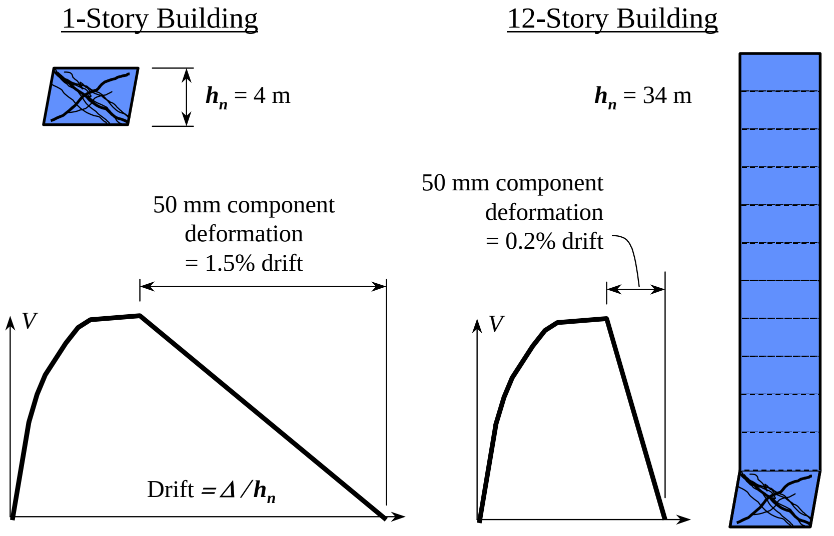


Fig. 4-5: Effect of wall shear failure on global force (V) versus roof drift response, illustrating how shear failures in taller walls act like a story mechanism.

4.3.2.3 Strength and ductility capacity of nonlinear locations

The design of the chosen nonlinear locations determines the global strength and deformation capacity of the structure. The type of nonlinear locations used, such as equivalent monolithic plastic hinges or rotating joints, determines the response characteristics of the structure, as discussed in Section 4.5. The relationship of required strength and deformation capacity to the level of earthquake shaking depends on a seismic demand versus capacity assessment, as discussed in Section 4.6. Such an assessment can be made using either a force-based or a displacement-based design procedure. The available methods are based either on effective initial stiffness and ductility capacity or on effective secant stiffness and equivalent viscous damping, as discussed in Section 4.6.

In New Zealand practice, design strength has been used for the design of nonlinear locations, according to the strength levels discussed in Section 4.3.4. The nonlinear locations are detailed for ductility as discussed in Section 4.5.

4.3.2.4 Design of linear regions and actions

All regions of the structure outside of the selected nonlinear locations are designed to remain linear and elastic for the maximum forces that can be delivered by the mechanism. Determining these forces requires evaluating the upper-bound strength or overstrength of the nonlinear actions, as discussed in Section 4.3.4.

As an example, consider a precast wall structure where the nonlinear mechanism is the individual walls rotating at their joint with the foundation. For this example the foundation is a linear and elastic region and is designed for forces corresponding to the moment overstrength of the wall-foundation connection. The shear in the wall is a linear action and is designed to exceed the shear force corresponding to the moment overstrength of the wall-foundation connection.

In New Zealand practice [Standards New Zealand (1995)], the nominal strength of linear-elastic regions and actions is compared to forces corresponding to the overstrength of nonlinear locations.

4.3.3 Implementation and advantages of capacity design

4.3.3.1 General

Capacity design offers the advantage of improved seismic performance, greater reliability of seismic performance, and a more rational and understandable design process. The approach has been implemented, to varying degrees, in seismic design codes throughout the world. Table 4-3 compares specific capacity-design measures that have been implemented in different design codes for concrete structures.

For equivalent seismic performance, capacity design procedures should result in more economical construction. In capacity design, special requirements for seismic forces and displacements are applied only to those identified areas of the structure where they are needed.

In some design situations, where there is a large uncertainty in strengths or in the distribution of earthquake demands, the capacity design approach should allow more than one potential mechanism of response. For example a structure could be designed to allow either wall rotation at the wall's joint with the foundation, or to allow rocking between the wall's foundation and the soil. Such a design might be necessitated by a large uncertainty in foundation resistance and the expense of designing a foundation with overturning resistance greater than the overstrength of the wall system.

Another example would be to allow some yielding of floor or roof diaphragms while still detailing for the intended mechanism in the vertical seismic system. Diaphragm yielding might occur during peak periods of the earthquake response when the amplification factors on the diaphragms become large. This approach is discussed in Chapter 6.

4.3.3.2 Codes not based on capacity design

Some countries, notably the U.S., have lagged behind in a comprehensive implementation of capacity design for concrete structures. Designs that do not follow the capacity design approach may experience an unexpected nonlinear mechanism or behavior mode. This compromises the engineer's effort to achieve adequate and predictable seismic performance and could potentially lead to collapse for some systems.

Building codes that are not based on capacity design hinder the structural engineering community's ability to:

- achieve predictable seismic behavior in structures, given the large randomness and uncertainty in the design earthquake input;
- provide a consistent and understood level of seismic protection for all structural systems;
- develop code provisions for new systems with appropriate design factors and with seismic performance comparable to established systems or to performance intentions.

Country:[1]	Canada	Europe	Japan	New Zealand	USA
Global behavior					
Explicit identification of nonlinear mechanism of response.	Yes	Yes	Yes[2]	Yes	No
Story mechanisms prevented.	No[3]	Yes[3]	Yes[2]	Yes	No[3]
Collectors designed for mechanism overstrength.	No[5]	Yes	Yes[2]	Yes	No[4]
Diaphragm shears and moment strength taken from mechanism overstrength.	No[5]	Yes	Yes[2]	Yes	No
Foundations designed for mechanism strength, or foundation-rocking mechanism explicitly identified.	Yes[6]	Yes	Yes[2]	Yes	No
Ductile moment frame design					
Beam shear strength exceeds beam flexural overstrength.	Yes	Yes	Yes	Yes	Yes
Column shear strength exceeds column flexural overstrength.	Yes	Yes	Yes	Yes	Yes
Inelastic dynamic amplification of column moments and shears, including bi-directional seismic effects.	No	No	Yes[2]	Yes	No
Shear force in beam-column joints taken from overstrength of beam plastic hinging.	Yes	Yes	Yes[2]	Yes	Yes
Bond strength of bars passing through beam-column joints exceeds flexural overstrength.	No[7]	Yes	Yes[8]	Yes	No
Lap splices permitted at column ends only if column end does not yield.	No	No[15]	No	Yes	No
Tension-shift effect considered for length of confined region of columns.	Yes[9]	Yes	Yes[2]	Yes	Yes[9]
Ductile wall design					
Wall shear strength exceeds flexural overstrength.	Yes	Yes	Yes[10]	Yes	No
Inelastic dynamic amplification effects for wall shear.	Yes[11]	Yes	Yes[2]	Yes	No
Tension-shift and higher-mode effects considered for curtailment of wall longitudinal reinforcement.	No[5,12]	Yes	No	Yes	No
Sliding shear failure prevented.	No[13]	Yes	No[14]	Yes	No[13]

Table 4-3 Capacity design practices currently required in design codes for concrete structures[1]

(For notes see next page)

Notes for Table 4-3

[1] Design codes considered are:

- Canada: CSA A23.3-94
- Japan: 1999 AIJ Allowable Stress Standard, 1988 AIJ Ultimate Strength Guidelines, and 1997 AIJ Inelastic Displacement Concept Guidelines. The standard used depends on the scale and height of the building.
- New Zealand: 1992 SNZ loading code and 1995 SNZ concrete code
- USA: 2000 NEHRP seismic provisions and 2002 ACI concrete code
- Europe: Eurocode 8, Draft No. 6, January 2003 (Ref. No: prEN 1998-1:200X)

[2] The answer is *yes* for the AIJ Ultimate Strength and Inelastic Displacement Concept Guidelines, but *no* for the AIJ Allowable Stress Standard.

[3] Strong-column-weak-beam provisions apply on a joint-by-joint basis but do not necessarily prevent story mechanisms [SEAOC 2000].

[4] Collectors are designed for an amplification factor of 2.5 to 3 but not checked against actual overstrength.

[5] Based on proposals preliminarily accepted, will change to *yes* in upcoming code.

[6] Rocking footing mechanism not allowed.

[7] No capacity design but more conservative than ACI, uses 24 bar diameters instead of 20.

[8] The answer is *yes* for the AIJ Inelastic Displacement Concept Guidelines, but *no* for the AIJ Ultimate Strength Guidelines and Allowable Stress Standard.

[9] Uses 1/6 of column clear height, which is less than New Zealand requirements in cases of high axial load or inflection point not near column mid-height.

[10] Walls governed by shear are permitted with design for increased seismic forces.

[11] Explanatory notes refer to requirements similar to those for New Zealand.

[12] Explanatory notes provide a procedure for this.

[13] Shear friction requirements and limit of shear stress (0.67 $\sqrt{f_c'}$ to 0.83 $\sqrt{f_c'}$ MPa) provide some protection against sliding shear.

[14] In the AIJ Ultimate Strength and Inelastic Displacement Concept Guidelines, a limitation on story-drift angle provides some protection against sliding shear.

[15] Lap splices are permitted even in critical regions.

4.3.3.3 Capacity design for precast concrete structures

Capacity design offers advantages in the seismic design of any structural system, but the approach can be crucial in designing precast concrete structures. The consequences of developing an unexpected nonlinear mechanism or behavior mode can be more hazardous for a precast concrete structure compared to a monolithic cast-in-place structure, because of the nature of the construction.

The design recommendations of this report require capacity design for precast structures that experience nonlinear response in an earthquake.

4.3.3.4 Other considerations

Capacity design procedures often require a more thorough structural engineering design process. Compared to pre-1970s practice, construction of some structural elements can be more expensive because of capacity design. For example, in moment frame structures, columns may need to be larger to prevent story mechanisms.

4.3.4 Levels of member strength

4.3.4.1 General

To ensure that undesirable mechanisms or behavior modes do not govern the earthquake response, the capacity design approach considers the potential variability in member strengths. Typically lower bound, mean, and upper bound strengths should be considered.

The New Zealand concrete design standard [Standards New Zealand (1995)] requires consideration of three levels of member strength: design strength ϕS_n, nominal strength S_n, and overstrength S_o (see Figure 4-6).

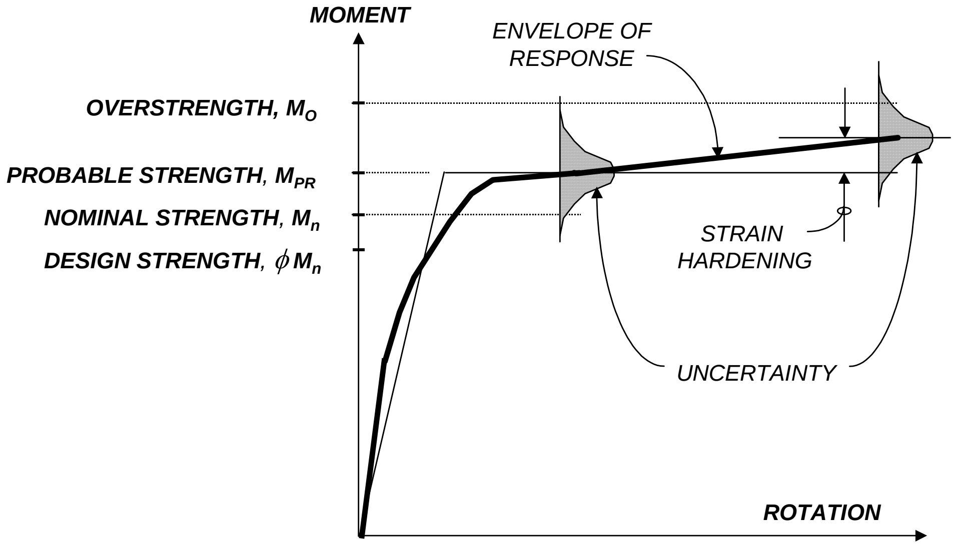


Fig. 4-6: Illustration of overstrength, probable strength, nominal strength and design strength on a graph of moment versus rotation

Design strength is the nominal strength S_n multiplied by the appropriate strength reduction factor, $\phi \leq 1.0$, where ϕ is to allow for smaller material strengths than assumed in design and variations in workmanship, dimensions of members and reinforcement positions. The factor ϕ is used in New Zealand, U.S., Japanese, and Canadian codes. In European standards material factors γ_s and γ_c are used to reduce the characteristic steel and concrete strengths, respectively, instead of ϕ factors.

Nominal strength S_n is the theoretical strength calculated using the lower characteristic strengths (5 percentile values) of the steel reinforcement and concrete and the member cross sections as designed.

Overstrength S_o is the maximum likely theoretical strength calculated using the maximum likely overstrength of the steel reinforcement and of the concrete. For concrete structures, the engineer typically needs to assess the *flexural overstrength,* which is the maximum likely flexural strength at the plastic hinges. Yield strength of reinforcing steel is typically the most important factor assessing this overstrength.

4.3.4.2 Overstrength due to longitudinal steel reinforcement

In practice the actual yield strength of the steel will normally exceed the lower characteristic yield strength (5-percentile value) used in design. Also, longitudinal reinforcement in the plastic hinge regions of a ductile concrete structure may reach strains that are several times the strain at first yield, and a further increase in steel stress due to strain hardening may occur.

Statistical analysis of the results of tests on samples of several years of steel production, and moment-curvature analyses, in New Zealand [Andriano and Park (1986)] determined that the flexural overstrength at plastic hinges in concrete beams reinforced using New Zealand manufactured reinforcing steel should be taken as $1.25M_n$, where M_n is the nominal flexural strength of the section calculated using the lower characteristic yield strength of the steel (5 percentile value). This 25% increase in M_n takes into account the probability of the actual steel yield strength being greater than the lower characteristic value (approximately a 17% allowance is assumed, as given by the ratio of the upper (as percentile) to lower (5 percentile) characteristic yield strengths of 1.17) and the steel strength increase above the yield strength due to strain hardening at high strains (approximately an 8% allowance).

To adequately assess overstrength, it is important for statistical information on the stress-strain properties of reinforcing steel used in seismic regions to be available. Knowledge of the likely variations of steel properties, to obtain overstrength factors, is important in capacity design. Without this information, the engineer must assume conservative values of overstrength (i.e. assume a high ratio of overstrength to nominal strength). This will make the design less economical.

4.4 Precast structural systems and expected response

4.4.1 General

Using the principles of capacity design, precast structural systems are conveniently classified according to the characteristics of their seismic response. Some structural systems, with high inherent lateral strength, respond linearly to earthquake demands, while the majority of precast structures will respond non-linearly to severe earthquakes. Nonlinear systems can be classified as either equivalent monolithic, jointed hybrid, or jointed rotating. The different systems and the key features of their seismic response are described in the sections below.

4.4.2 Linear response

Some types of precast concrete structures can be effectively designed to respond linearly and elastically to earthquake shaking. In such a case the strength of the structure must be larger than the elastic forces imposed by the maximum earthquake. In linearly responding structures there is not a need for capacity design or ductile detailing.

The most appropriate structures to design for linear response are those with high inherent lateral strength compared to the seismicity. Low-rise wall buildings are an example of structures that can have high lateral strength without special design measures. If the number and strength of walls is sufficient, then the best design approach can be to check that during the elastic response of the structure to the earthquake the forces in the walls, foundations, and other parts of the structure do not exceed the design strength. Precast buildings with relatively solid walls around the entire perimeter are a common building type that can be designed for linear response. Buildings located in regions of low seismicity will also be easier to design for linear response.

Designing for linear response means that the building should suffer no structural damage as a result of the design earthquake. This makes linear-response design suitable for buildings that need to meet an immediate occupancy performance level. (As discussed in the Section

4.4.6, designs for nonlinear response might also meet the immediate occupancy objective, if the nonlinear response has re-centering characteristics.)

The typical force-displacement response of linearly responding structures is shown in the first row of Table 4-4. In comparison to nonlinear structures, the response has higher strength, and typically higher stiffness.

In the design of non-ductile, linearly responding structures it is important to carefully choose the maximum level of ground motion to be considered. The aim is to provide appropriate overall reliability of the seismic performance, given the variability and uncertainty of the predicted ground motion and the consequences of failure. In ductile structures there tends to be a margin of deformation capacity in case seismic demands are larger than expected. In non-ductile structures this margin may not be present.

4.4.3 Nonlinear response and nonlinear locations

Most new structures, precast or otherwise, are designed to respond nonlinearly under severe earthquake shaking. This means that in the critical locations, the structural members need to be able to deform or rotate beyond the linear limits of the member or connection. As discussed in Section 4.3, the nonlinear locations in the structure must be explicitly chosen and designed by the engineer. In precast concrete structures, the nonlinear locations will typically be flexural plastic hinges in members or rotating joints in or between members.

The global nonlinear seismic response of a structure depends on the response characteristics of the nonlinear locations. Thus, for seismic design purposes, precast structural systems can be categorized according to the behavior of the nonlinear locations. As shown in Table 4-4, four types of nonlinear structural systems can be identified:

- equivalent monolithic;
- jointed hybrid;
- jointed rotating;
- foundation rocking.

These are shown in the table in addition to the linear response category discussed in Section 4.4.2.

The principal difference between jointed systems and equivalent monolithic systems is that the nonlinear locations of jointed systems allow rotation by joint opening, while the nonlinear locations of equivalent monolithic systems allow rotation by plastic hinging.

4.4.4 Equivalent monolithic systems

Figure 4-7 shows examples of equivalent monolithic structural systems. The key characteristic of these systems is that the nonlinear locations are plastic hinges with distributed flexural cracking. The plastic-hinge behavior is comparable to that in a monolithic, cast-in-place structure. Earthquake energy is dissipated in plastic hinges by the yielding, in tension and compression, of the mild steel longitudinal reinforcement.

Figure 4-7(a) shows an example of an equivalent-monolithic moment frame. In this frame, column-beam cruciform sections are precast and connected to each other at the mid-span of the beams and columns. The plastic hinging is designed to occur at the beam ends. In this case the connections at mid-span of the beams are designed as strong connections located away from the plastic-hinge region.

Figure 4-7(b) shows an example of an equivalent-monolithic concrete wall. For equivalent-monolithic behavior, the wall must be provided with enough connection strength and foundation resistance to force a plastic hinge to occur at the base region of the wall.

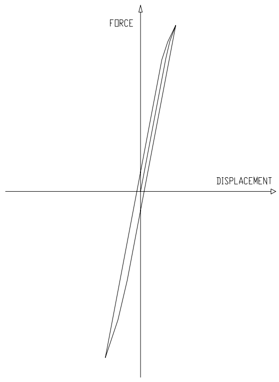
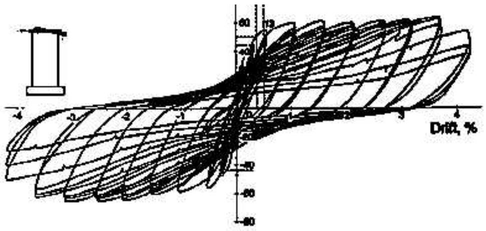
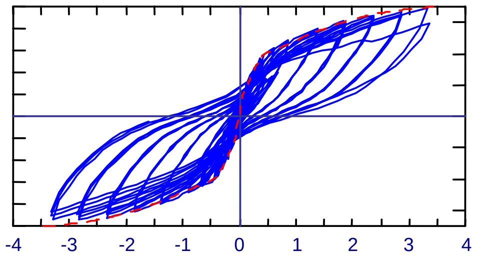
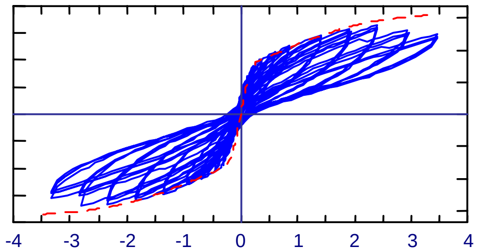
Precast structural system type	***Nonlinear locations*, and behavior**	**Typical force-displacement hysteretic response** [Corley et al 1981; Cheok et al 1996]
Linear response	*There are no nonlinear locations*. All parts of the structure are designed to remain linear and elastic.	
Equivalent monolithic	*Flexural plastic hinges*. Flexural cracking and yielding over the plastic-hinge region allows nonlinear rotation. Yielding of mild steel longitudinal reinforcement in tension and compression provides ductility and dissipates energy.	
Jointed hybrid	*Rotating joints plus energy dissipation.* Joint opening allows nonlinear rotation. Unbonded prestressing and/or dead load provides re-centering forces. Yielding of mild steel longitudinal reinforcement or ductile connectors between walls dissipates energy.	
Jointed rotating	*Rotating joints.* Joint opening allows nonlinear rotation. Unbonded prestressing and/or dead load provides re-centering forces.	
Foundation rocking	*Foundation-soil interface.* Overturning of foundation allows nonlinear rotation. Dead load provides re-centering forces. Radiation damping or nonlinear soil response provides energy dissipation.	Hysteretic response similar to that for jointed-rotating systems

Table 4-4: Precast structural system types, nonlinear locations, and response

4.4.5 Jointed hybrid, jointed rotating, and jointed rocking systems

If the nonlinear locations of a precast structure rotate by a mechanism of joint opening rather than by plastic hinging, the structure is categorized as a jointed system. In jointed

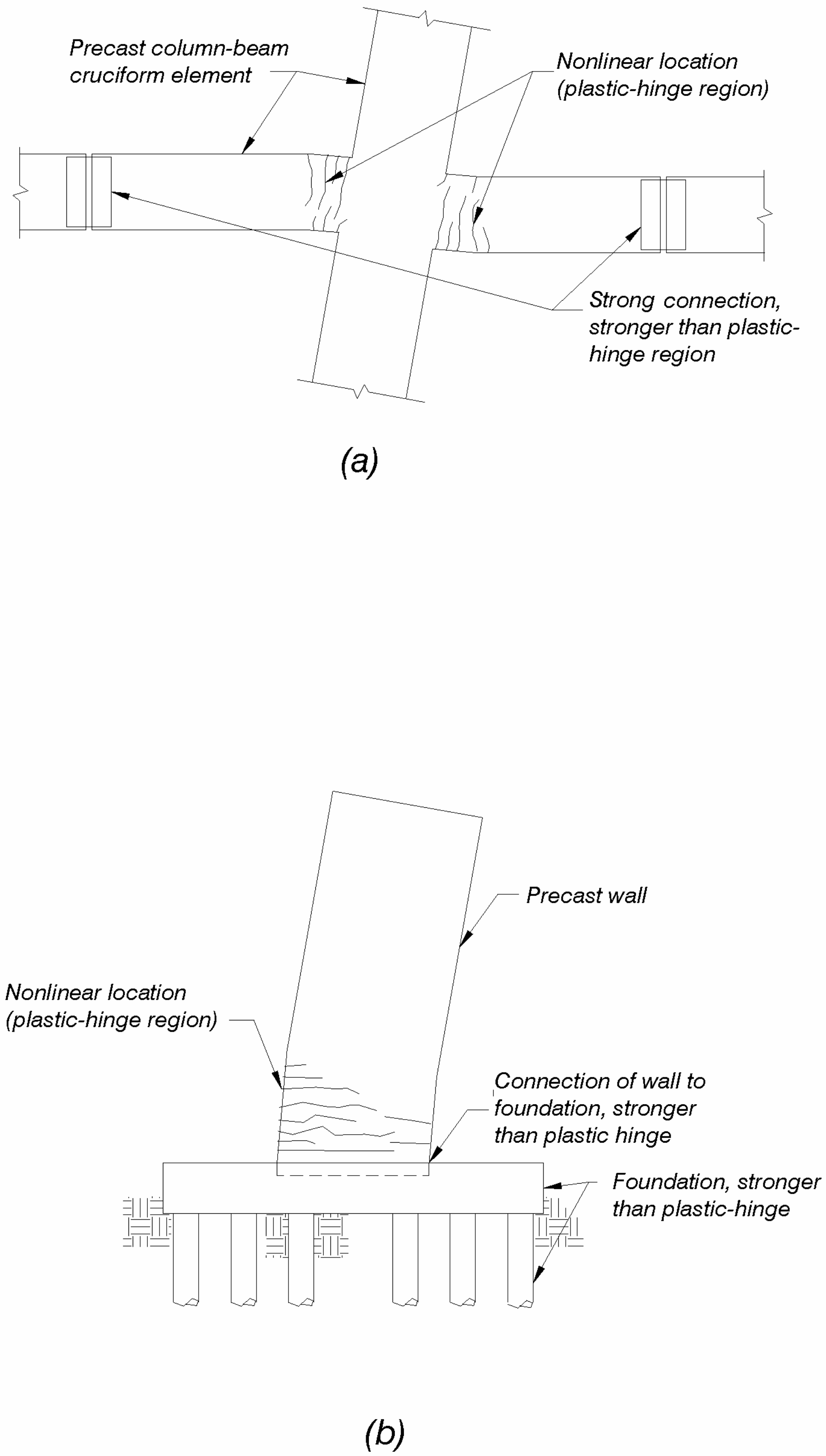


Fig. 4-7: Examples of equivalent monolithic precast systems: (a) moment resisting frame, (b) structural wall

systems nonlinear rotation is accommodated at a single crack or joint opening rather than over several cracks in a plastic hinge region. Jointed systems make use of either unbonded prestressing or dead load to provide a restoring force that counteracts seismic deformations. Three jointed systems are described below.

4.4.5.1 Jointed hybrid systems

Jointed hybrid systems include special measures to provide energy dissipation similar to that occurring in equivalent-monolithic systems. The systems are called hybrids because they use the restoring-force characteristics of jointed-rotating systems combined with energy-dissipation characteristics of equivalent monolithic systems. Figure 4-8 shows examples of jointed hybrid structural systems.

Figure 4-8(a) shows an example of a jointed hybrid moment resisting frame. The nonlinear locations are the joints that open up at the ends of the beams. Unbonded post-tensioned tendons along the beam centerline, passing through the column, provide the restoring force. Mild steel reinforcement at the top and bottom of the beam section provides energy dissipation by yielding in tension and compression. The mild steel reinforcement is de-bonded for a specified length to control the reinforcement strain. This type of moment resisting frame system was shown to provide excellent performance in testing of a 5-story structure under the PRESSS program [Priestley et al (1999)] and has been used in California for buildings up to 39 stories [Englekirk (2002)].

Figure 4-8(b) shows an example of a jointed hybrid wall. The nonlinear location for the wall is joint at the base of the wall that opens up under seismic displacement demands. In this case, the dead load of the wall is used to provide the restoring force. Mild steel dowels between the foundation and the wall are designed to yield in tension and compression to provide energy dissipation.

Figure 4-8(c) shows another example of a jointed hybrid wall. The wall is composed of two precast wall units connected by ductile energy dissipation devices. The nonlinear locations for the wall are the joints at the base of each wall unit that open up under seismic displacement demands. Unbonded post-tensioning tendons down the centerline of each wall unit and anchored into the foundation provide the restoring force. The connectors between the wall units provide energy dissipation from the relative vertical movement between the two units. This type of wall system was shown to give excellent performance in testing of a 5-story structure under the PRESSS program [Priestley et al (1979)]

4.4.5.2 Jointed rotating systems

Jointed precast systems can also be constructed without added measures for energy dissipation. Figure 4-9 gives examples of such systems. Figure 4-9(a) shows an example of a jointed rotating precast moment frame. This is similar to the jointed hybrid moment resisting frame of Figure 4-8(a), except that the yielding mild steel is not used. The nonlinear locations are the joints that open up at the ends of the beams. Unbonded post-tensioning tendons along the beam centerline, passing through the column, provide the restoring force. Figure 4-9(b) shows an example of a jointed rotating wall designed by the same concept as the jointed rotating moment resisting frame shown in Figure 4-9(a). The nonlinear location is the joint opening at the base of the wall, and unbonded prestressing provides the restoring force.

Figure 4-9(c) shows an example of a jointed-rotating precast wall system that uses dead load rather than prestressing to provide the restoring force. The nonlinear locations are the joints opening at the base of the each wall as the wall panels accommodate seismic displacement by rocking on their foundation.

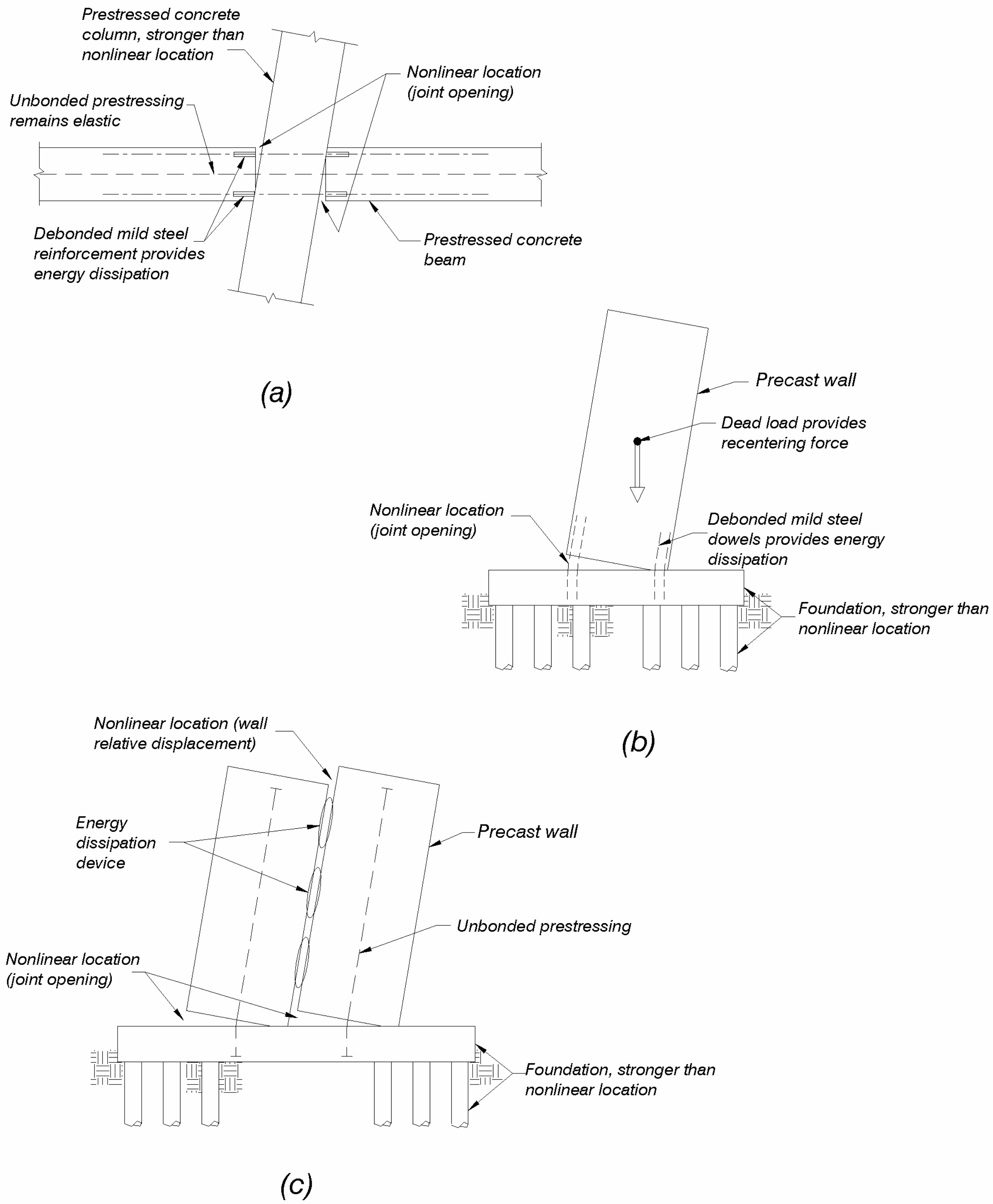


Fig. 4-8: Examples of jointed hybrid precast systems: (a) moment resisting frame, (b) wall with dowels, (c) wall with unbonded prestressing and energy dissipation devices

4.4.5.3 Rocking systems

Foundation rocking systems can be considered as a type of jointed system, except that the “joint” where nonlinear rotation occurs is the interface between the foundation and the supporting soil. Figure 4-9(d) shows an example of a precast wall designed to rock on its foundation. In such a system, the dead load on the wall provides the restoring force that counteracts earthquake deformations. Energy dissipation can be provided to the system by radiation damping in the soil acting under impact from the foundation.

Foundation rocking response can also occur in foundations using piles or drilled piers. In this case, the foundation overturning resistance is often governed by the yielding of tension reinforcement connecting the pile cap to the piles.

4.4.6 Force-displacement hysteretic response

When structural components are cyclically tested, a continuous record is made of the applied force versus displacement. When plotted, the force-versus-displacement record appears as hysteresis loops corresponding to the cycles of imposed displacement. The area inside a hysteresis loop equals the amount of energy dissipated in that cycle of deformation. The hysteretic characteristics of the nonlinear locations of a structure determine the force-displacement characteristics of the structure as a whole as it responds to earthquake motion.

The different types of precast systems exhibit different characteristics in their force-displacement hysteretic response. The hysteretic characteristics reflect the actions such as energy dissipation and restoring forces at the nonlinear locations of the structure.

Table 4-4 shows the hysteretic response for different types of precast systems. The hysteresis loops for linear response show the relatively higher stiffness and strength of such systems, and linear behavior with practically no energy dissipation. The hysteresis loops for equivalent-monolithic systems show nonlinear response and substantial energy dissipation. The hysteresis loops for jointed-rotating systems show nonlinear response with less energy dissipation and a strong tendency towards re-centering. (The re-centering action is indicated when the structure is unloaded to zero force and the hysteresis loop comes back close to zero displacement.) The jointed-hybrid system has a hysteresis loop shape with characteristics in between those of the equivalent-monolithic and the jointed rotating systems.

4.5 Detailing for ductility

4.5.1 General

As presented in Section 4.3, detailing for ductility includes:

(1) determining that nonlinear locations of the structure have the ability to deform as required without any fracture or other strength degradation, and

(2) determining that all other actions and elements in the structure have sufficient strength to ensure that nonlinear behavior occurs in the intended nonlinear locations.

The second item includes provisions for overall structural behavior and provisions for members, which are discussed in Sections 4.5.2 and 4.5.3 below. These provisions are equally applicable to precast as well as cast-in place structures and members. The first item, detailing at nonlinear locations, is discussed in Section 4.5.4 and is generally different for jointed precast members than it is for cast-in-place or equivalent-monolithic members.

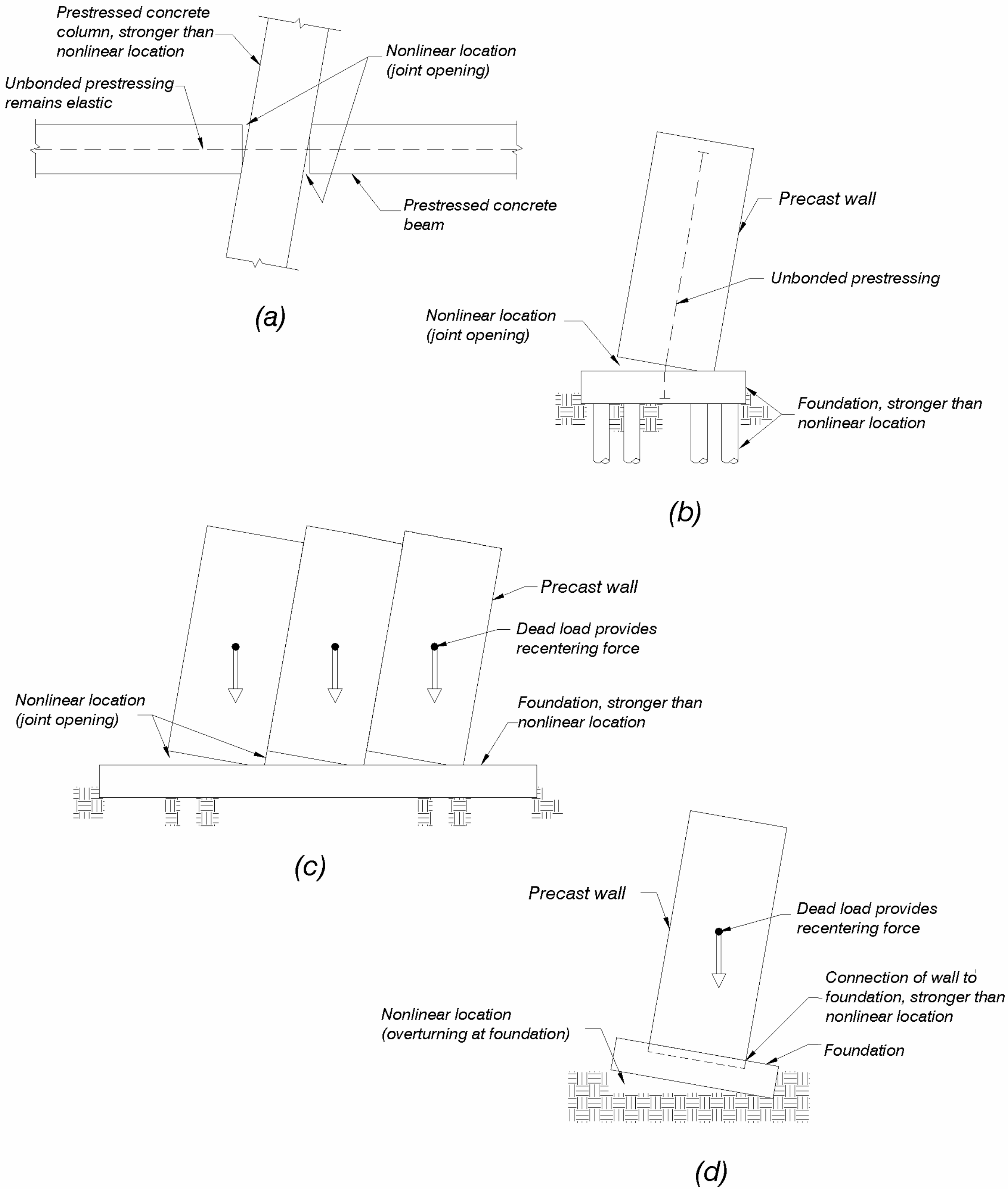


Fig. 4-9: Examples of jointed rotating precast systems: (a) moment resisting frame, (b) wall with unbonded prestressing, (c) walls rocking at base of wall, and (d) wall rocking at base of foundation

Requirements for the ductile detailing of cast-in-place concrete structures are well established [Park & Paulay (1975), Paulay & Priestley (1992), Standards New Zealand (1995)]. The requirements that are applicable to precast structures are summarized in the following sections, but the above references should be consulted for further explanation.

4.5.2 Ductility provisions for overall structural behavior

Ductility provisions for overall structural behavior include protecting against story mechanisms, as discussed in Section 4.3.2.1. In moment resisting frame structures, this is achieved by providing adequate column strength in relation to beam strength. Typically, code provisions have required columns to be stronger than beams at every beam-column joint. A more appropriate criteria is to have the sum of column strengths, considering all columns below a story level be greater than the sum of beam strengths at that story level [SEAOC Commentary (1999)].

In wall structures, preventing shear failures or sliding shear failures prevents a concentration of deformation occurring over a single story or limited height. Walls with flexural plastic hinges, joint opening, or foundation rocking at the base provide for an even distribution of nonlinear story drift demands, which is desirable.

In dual systems (consisting of walls working in combination with frames) the deformations of the frames can be controlled and limited by the stiffness and strength of the walls. Figure 4-10 shows two possible mechanisms for dual systems. In Figure 4-10(b) the hinging of the columns is acceptable because the strength of the wall prevents a story mechanism from developing. In such a case, a plastic or nonlinear analysis should be used to verify the mechanism and wall shear demands.

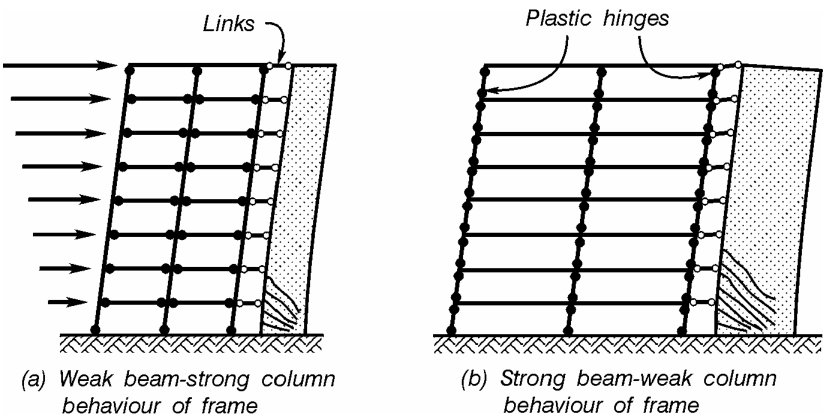


Fig. 4-10: Mechanisms of nonlinear response for dual systems

Another aspect of assessing overall structural behavior for ductility capacity is to ensure that floor and roof diaphragms, collectors, and foundations have adequate strength. The strength of these elements should be sufficient to force the nonlinear behavior into the intended locations of the seismic-force resisting system. As discussed in Chapter 7 on diaphragms, for most structures this is easily achieveable. For structures that have long diaphragm spans (say > 40 metres) between parallel walls or moment frames, diaphragm response may not always be linear and elastic, unless special measures are taken to strengthen diaphragms. Alternatively, some yielding of diaphragms can be permitted, using the philosophy described in Section 4.3.3.4 of allowing more than one potential mechanism of response. In this case it should be shown that either potential mechanism provides adequate global ductility capacity.

4.5.3 Ductility provisions for members

One of the key ductility provisions for members is to ensure that shear strength exceeds the shear corresponding to the overstrength of the governing nonlinear mechanism. For beams of moment resisting frames, the shear demand is typically that corresponding to the development of flexural overstrength at each end of the beam.

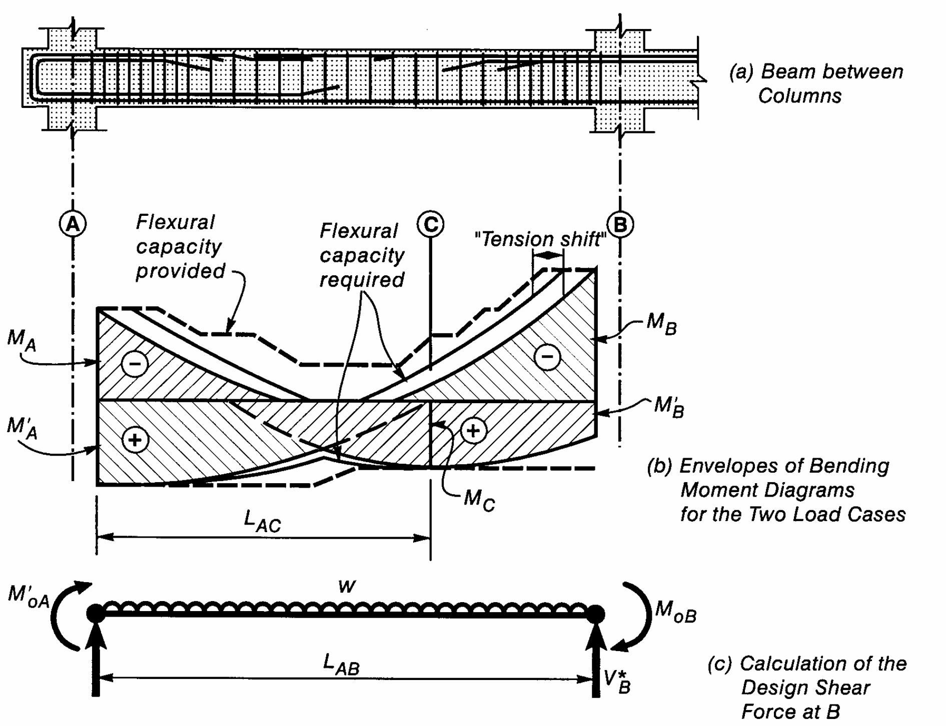


Fig. 4-11: Envelopes of bending moment and the calculation of the design shear force of a beam in a moment resisting frame

For beams with substantial gravity load or curtailed reinforcement, the possibility of yielding occurring within the span of a beam must be checked. Figure 4-11 illustrates for a beam how gravity moments are considered in combination with seismic moments, including the effect of tension-shift, and the possibility of a plastic hinge occurring under positive moment within a span.

For columns and walls, dynamic amplification factors are applied to shear demands to account for inelastic higher mode effects. The amplification factors increase for taller structures where higher modes can be more significant. Figure 4-12 illustrates how column moments are magnified by a system-overstrength factor ϕ_0 and by a dynamic amplification factor, ω. The maximum moment gradient gives the maximum column shear.

In members where longitudinal reinforcement is curtailed, the effect of tension shift must be considered. Reinforcement must extend beyond the point where it is no longer required to resist flexure by a distance not less than the flexural depth of the member.

In addition to protecting against shear failure in diagonal tension, shear stresses should be small enough that sliding shear or diagonal compression shear failures do not occur.

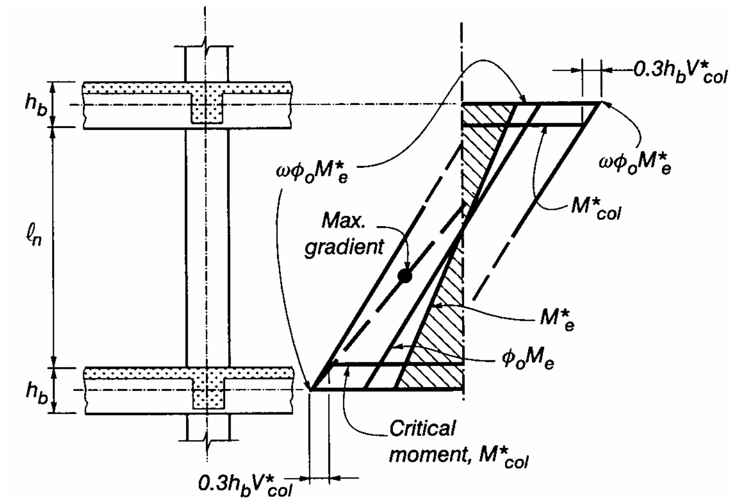


Fig. 4-12: Amplification of column moments

In moment resisting frames, beam-column joints are to be designed for the maximum forces that can be delivered by the overstrength of the yielding beams. Reinforcement passing through the beam column joint must be adequately developed.

4.5.4 Ductility provisions for nonlinear locations

Ductility provisions for nonlinear locations depend on the type of nonlinear location. Equivalent monolithic precast systems have flexural plastic hinges as their nonlinear locations, and the detailing requirements for the plastic hinges are the same as those established for cast-in-place concrete, as indicated in Section 4.5.4.1.

Jointed precast systems have unique requirements, as indicated in Section 4.5.4.2

4.5.4.1 Equivalent-monolithic systems

In plastic-hinge regions, longitudinal reinforcement should be continuous and lap splices should be avoided. If not enclosed by tie reinforcement, the splices tend to progressively unzip with cycles of displacement. Even well confined lap splices that do not slip are undesirable in plastic-hinge regions, because they prevent an even distribution of yielding along the length of the flexural reinforcement. When splices are present, steel yielding and strains tend to concentrate over a short length of reinforcement at one or both ends of the lap-splice length, which can reduce the rotation capacity of the plastic-hinge region.

In addition to providing adequate shear strength, transverse reinforcement is necessary in plastic-hinge regions to prevent premature buckling of longitudinal bars and to provide confinement to concrete. Figure 4-13 shows arrangements of transverse reinforcement and how the reinforcement acts to provide confinement. Figure 4-14 shows the construction of an equivalent monolithic moment frame with precast beams and cast-in-place columns, where the column transverse reinforcement can be seen.

4.5.4.2 Jointed systems

In jointed systems, the provisions required for ductility capacity depend on the type of nonlinear locations incorporated. For the types of systems presented in Section 4.4.5, some of the required checks are summarized below.

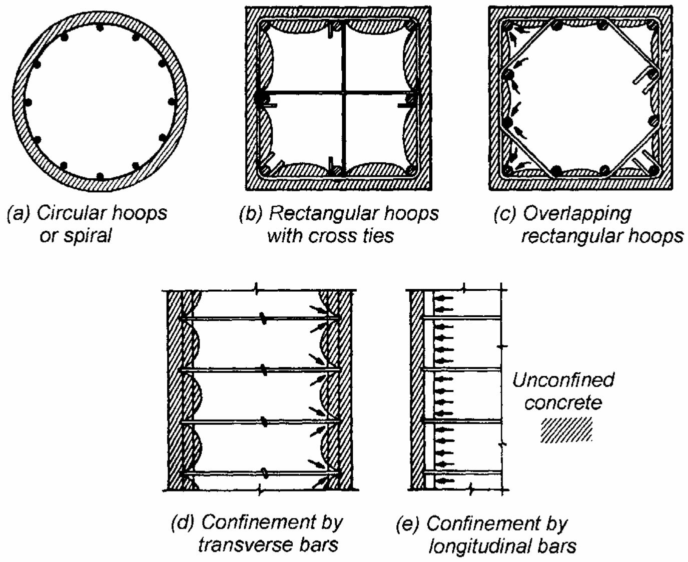


Fig. 4-13: Arrangements of reinforcement that confine the concrete and prevent premature buckling of longitudinal reinforcement of columns

Fig. 4-14 Construction in New Zealand of an equivalent-monolithic moment frame using precast beams and cast-in-place columns, showing column tie reinforcement

When unbonded prestressing is used to provide a restoring force at the nonlinear location, the strain in the unbonded prestressing must be checked to ensure that the prestressing remains elastic under the maximum earthquake displacements. The strain tends to be more critical for members with a smaller ratio of length to flexural depth.

When yielding mild reinforcement is used at joint-opening locations to provide energy dissipation, the reinforcement generally must be de-bonded over a specified length to prevent excessive strains. The length of de-bonding is calculated based on the rotation demands at the joint and the acceptable strain in the mild steel reinforcement.

When wall-coupling devices are used, the devices must be designed to accommodate the imposed deformations without fracture or strength degradation. Typically, an appropriate design would be validated by testing of the device.

To provide for re-centering in a precast system, the restoring force provided by dead load and/or unbonded prestressing must exceed the resisting force of yielding mild reinforcement or wall coupling devices.

4.5.5 Levels of ductility

In designing a structure for nonlinear seismic response, it is possible to design for high levels of ductility capacity or restricted levels of ductility capacity. A structure designed for restricted ductility capacity needs to have greater lateral strength than if it were designed for high ductility capacity.

Typically the additional expense in detailing for high ductility capacity compared to restricted ductility is minor. However, for structures with high inherent lateral strength compared to the seismic demands, it can be more economical to design for restricted ductility capacity.

Designing for restricted ductility is covered in Chapter 8 of the textbook by Paulay & Priestley (1992). The chapter is written for cast-in-place concrete but most of the principles are applicable to precast structures.

4.6 Demand versus capacity assessment

4.6.1 General

Along with capacity-design and detailing provisions, one the essential steps of seismic design is determining if the structure has an adequate combination of strength and ductility capacity compared to the earthquake demands. (This is specified as step 2 of the capacity design process in Section 4.3.2.1.) Either of two approaches can be used for the demand-versus-capacity assessment: force-based design or displacement-based design.

The sections below give a description and comparison between the two design methods, and key aspects of force-based design are discussed. Displacement-based design is discussed in detail in Appendix A.

4.6.2 Comparison of force-based and displacement-based design

Structures have traditionally been designed using force-based procedures. In this method, the designer first calculates the inertia forces that would be induced in the structure if it were to remain elastic. The forces are then reduced by a code-specified factor (R or R_μ) related to the ductility capacity of the structural system. The resulting forces depend heavily on the initial period of the structure and the force reduction factor. The initial period in turn depends on the assumed elastic initial stiffness. After designing the members for strength, the designer must also provide enough stiffness that the displacement of the structure meets the maximum drift limits. Displacements are calculated by applying the design loads to an elastic model of the structure, and multiplying the displacements by a code-specified empirical factor that is intended to reflect the effects of inelasticity.

Displacement-based design provides an alternative approach to the assessment of seismic demand versus capacity, and can be better suited to the characteristics of precast reinforced and prestressed concrete structures, Priestley (2000), Priestley and Kowalski (2000) and Priestley (2003) have developed and described the displacement based design method in detail. In displacement-based design, the design drift is the starting point. The structure is characterized by (secant) stiffness and equivalent viscous damping at maximum displacement response (see Fig. 4-15), rather than the initial stiffness characterization used for force-based design. From the selected displacement, the secant stiffness and damping are determined, and design forces are directly found. The assumed level of damping is checked, and if necessary, the design forces are adjusted, though the adjustments are generally small.

The sequence of design operations for force-based and displacement-based design is outlined in Table 4-5. Although the sequence of design differs greatly between the two procedures, the resulting difference is only in how the required strength of the non-linear regions of the structure is calculated.

The displacement-based design procedure can be further simplified by using the acceleration-versus-displacement spectrum (see Figure 4-1(c)), rather than the displacement-versus-period spectrum (see Figure 4-1(b)). In this case steps 3 and 4 are replaced with a single step: “Enter the acceleration-versus-displacement spectrum with the design displacements and determine the design force levels”.

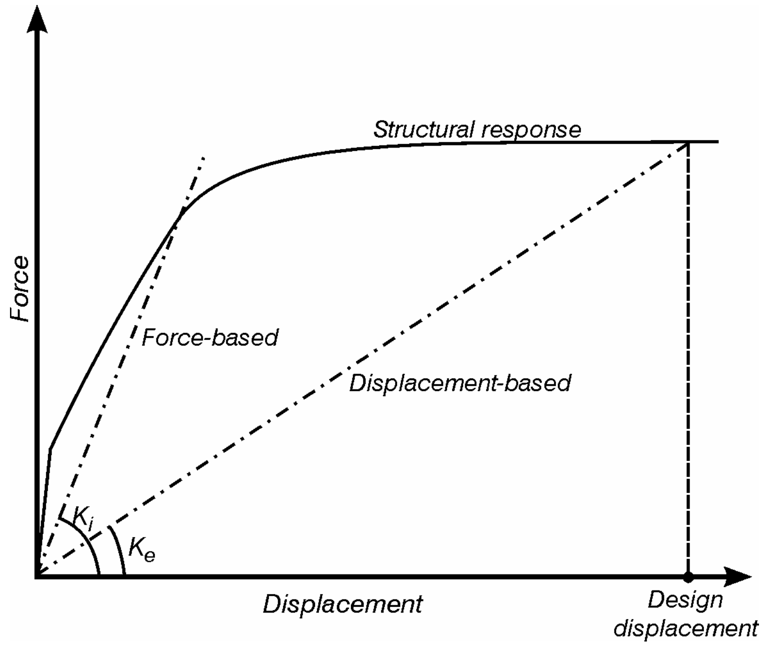


Fig. 4-15: Effective stiffness for design

4.6.3 Key differences between force-based and displacement-based design

Some of the key differences between force-based design and displacement-based design are summarized below, and the issues are discussed in further detail is Sections 4.6.4 and 4.6.5.

One essential difference is that force-based design is based on initial stiffness, and displacement-based design is based principally on effective secant stiffness.

In force-based design, initial stiffness is used to determine seismic displacements and period (although for period an empirical calculation, independent of initial stiffness, often governs). In displacement-based design, secant stiffness is used to relate displacement values to required design forces. In displacement-based design, initial stiffness may also be needed to determine ductility demands, which affect equivalent viscous damping values.

The second key difference is in how the elastic spectrum is reduced. In force-based design, a factor R or R_μ based on ductility capacity is used to reduce the 5% elastic spectrum and give the required design forces. In displacement-based design, the concept of equivalent viscous damping is used instead. The ductility capacity is related to an equivalent viscous damping value that is used to reduce the 5% elastic spectrum.

Force-based design	Displacement-based design
1. Estimate effective initial period.	1. Calculate or select design displacements.
2. Obtain elastic force from 5% acceleration spectrum.	2. Estimate equivalent viscous damping.
3. Determine force reduction factor or ductility factor.	3. Calculate effective secant period from displacement spectra.
4. Calculate design force levels.	4. Calculate effective secant stiffness and design force levels.
5. Calculate member forces at plastic hinges.	5. Calculate member forces at nonlinear locations (plastic hinges).
6. Check displacements versus drift limits. If necessary, redesign for increased stiffness and return to Step 1.	6. Check equivalent viscous damping, and return to step 3 if necessary.
7. Provide capacity protection of linear sections and actions.	7. Provide capacity protection of linear sections and actions.

Table 4-5: Comparison of force-based versus displacement-based design procedures

4.6.4 Applicability of force-based and displacement-based design

Both force-based and displacement-based procedures can be used, with appropriate adjustments, for any type of structure. Depending on the assumed relationships between ductility and equivalent viscous damping in displacement-based design, and on the assumed initial stiffness and period values in force-based design, they may or may not give similar solutions.

For application to jointed-precast systems, displacement-based design accounts more directly for the special force-deformation hysteretic characteristics that these systems exhibit. If force-based design is used for jointed precast systems, the R_μ value used should be adjusted to account for the effects of hysteretic characteristics.

Additional advantages of displacement-based design include the following:

- A direct and transparent selection of the maximum displacement of the structure as the first step.
- A single procedure that achieves drift control and determines required design forces.
- Typically less sensitivity to initial stiffness, which can be difficult to determine for some structure types.

A disadvantage of displacement-based design is that there has been disagreement as to the appropriate ways of selecting equivalent viscous damping values.

4.6.5 Aspects of force-based design

Force-based design has been the traditional approach of seismic design. Aspects of force-base design are as follows:

4.6.5.1 Design force

Equation 1 gives a general form of the equation defining the required base shear strength for force-based design.

$$V_B = \frac{C_T I(gM_e)}{R_\mu} \tag{1}$$

where C_T is the basic seismic coefficient that depends on seismic intensity and period T, I is an importance factor that reflects the level of acceptable risk for the class of building, M_e is the effective mass, g is the acceleration of gravity, and R_μ is the force-reduction factor, which is related to the ductility capacity (μ) of the structural form and material.

The New Zealand Loading Code [Standards New Zealand (1992)] presents one of the most straightforward versions of force-based design. The ductility capacity μ is used directly to determine the seismic coefficient according to a graph such as shown in Figure 4-16.

Equation 1 may be used as the single operative equation in an equivalent lateral force analysis, or applied to each of the significant elastic modes of vibration in a multi-modal analysis. In the latter case, the different structural responses (forces, moments, shears, displacements) are combined a modal combination rule such as the Square Root Sum of the Squares (SRSS) or Complete Quadratic Combination (CQC). Details are given in standard texts on the subject (Chopra, 2001, Clough and Penzien, 1975).

The variables in Eqn. 1 are treated very differently by the design codes of different countries, with the result that the design loads also differ. The major differences are outlined in the following discussion.

4.6.5.2 Period

The elastic period(s) of vibration depend on the structural configuration and member stiffness. In some provisions the initial uncracked (gross-section) member stiffness is used to calculate period, while in others an estimate of the effective stiffness at first yield is used, on the basis that this better represents the stiffness as the structure enters the inelastic range. For reinforced concrete members, the ratio of cracked-section to gross-section stiffness can vary between 0.2 to 0.7, depending on the reinforcement ratio and axial load ratio on the section [Priestley, (2000)]. Thus, significant differences in calculated period can result from the different assumptions.

In some countries, notably the United States [ICC, 2000)], a fundamental period is estimated based on the structural form, material and building height, rather than on geometry and member stiffness. The period equation is of the form

$$T = C_1(h_n)^{0.75} \tag{2}$$

Where h_n = building height. Although modal analysis based on realistic member stiffness is permitted, the base shear so calculated must not be less than that calculated from the height-dependent period equation, using an equivalent lateral force analysis, by more than 20%.

Equation 2 generally results in shorter periods than would result from structural analysis. For example, the ICC (2000) code specifies C_1=0.0731 for concrete frames, when h_n is expressed in metres. For a typical 10 story frame building, Equation 2 results in a period of about 1.0 sec, while structural analyses, based on gross-section and cracked sections, result in periods of about 1.6 sec and 2.4 sec respectively. Since the typical form of the period dependent coefficient C_T in Equation 1 is inversely proportional to period for moderate to long periods, this implies very large differences in elastic force levels.

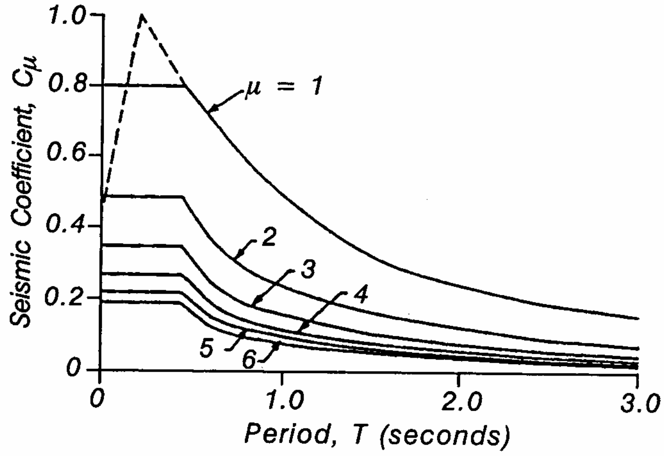


Fig. 4-16: Basic seismic hazard coefficient, for intermediate soil sites, for different displacement ductility capacities, μ, used in New Zealand [Standards New Zealand 1992]

4.6.5.3 Force-Reduction Factor R_μ

Codes specify maximum force-reduction factors for different structural types. It can been shown, however, that this approach does not adequately allow for the effect of geometry on ductility capacity or the influence of foundation compliance. For example, slender walls will deform inelastically by formation of plastic hinge at the base, whereas stocky walls are more likely to slide along the construction joint. Despite these differences in expected behavior, the same force-reduction factor is specified for both [(ICC (2000)]. Design codes of different countries also specify quite different force-reduction factors for the same class of structure.

Comparison between force-reduction factors is complicated by the presence or absence of other factors that influence the design forces. Relevant issues include questions of whether strength-reduction factors are applied to seismic loads calculated from Equation 1 or to calculated member capacities, and whether moment-redistribution may be applied to combined gravity and seismic member forces to improve structural efficiency. However, the effective range of permitted R_μ factors appears to be about 2 to 8 with South American (Chile) and Japanese values at the lower end, and United States values (ICC 2000) at the top end. Values in Europe (EC8) and New Zealand are typically about 5 or 6 for moment resisting frame structures. The variations may be loosely interpreted as a trade-off between provision of strength and ductility capacity, at the extremes of which Japan favors strength and the United States favors ductility.

A further complication is the incorporation in some codes of a "seismic performance" factor, S_p, applied to the result of Equation 1. In New Zealand and Canada, values of $S_p = 0.67$ are applied, thereby reducing the design base shear. By comparison with other codes which do not include this factor, it could be argued that the design force reduction factor for these codes is effectively 1.5 times higher than the listed value.

It should be noted that Equation 2 generally leads to low estimates of building periods and consequently high design loads. The equation was developed by fitting an equation to a group of measured building periods. The equations used in design codes are a lower bound to the measured periods. This is conservative for estimating elastic forces, but can be unconservative for estimating displacements. All the buildings measured had a 10 ft (3 metre) story height, so the period estimates for buildings with taller story heights may be underestimated.

4.6.5.4 Seismic displacement

Many codes calculate the maximum expected displacements based on the equal displacement approximation, which holds that the peak displacements of an elastic and an inelastic system with the same initial stiffness will be the same, when they are subjected to the same ground motion. However, some countries typically Japan use an equal energy approximation, which relates the elastic and predicted inelastic displacement by the equation

$$\Delta_{\mu} = \Delta_{el} \frac{R_{\mu}^{2} + 1}{2R_{\mu}} \qquad (3)$$

Thus, a structure with $R_\mu = 4$ would have predicted inelastic displacements that are 2.13 times those for the corresponding elastic structure, whereas the equal displacement approximation would predict the same displacement. In the New Zealand standard [Standards New Zealand (1992)] the force reduction factor is period dependent, and is incorporated directly in inelastic spectra for different design ductility levels, which are specified in preference to force reduction factors. In this case, the design displacements are found by multiplying the elastic displacements corresponding to the reduced design force levels by the ductility factor.

Nonlinear time-history analyses show that the equal displacement assumption is on average valid at periods longer than the site period. At shorter period, nonlinear displacements tend to exceed elastic linear displacements.

More recently, however, evidence has been presented [Stewart (2003)] that earthquake demands on structures with shorter periods may be less than indicated by recorded ground motions. The effective ground motions transferred to a foundation system of typical building dimensions can be significantly less at shorter periods than free-field ground motions recorded by an instrument at a single point. This base-slab averaging effect counteracts (to some extent) the increased ratio of nonlinear to linear demands predicted for short period structures.

In code provisions, the design displacement is taken as the calculated elastic displacement corresponding to some level of design forces. In the UBC (1997) design displacement is taken as 0.7 times the displacement that occurs under 5% damped elastic forces. In the ICC 2000, design displacement is taken as C_d/R times the displacement that occurs under 5% damped elastic forces, with the ratio C_d/R ranging from 0.5 to 0.9. These code provisions for displacement do not agree well with the research, which would suggest a factor of 1.0 for medium to long periods, and a factor potentially greater than 1 for shorter periods.

4.6.5.5 Drift limits

Code provisions give an upper limit on the drift ratio (lateral deformation over height) that any story of the structure can reach. The intent of the drift limit is to prevent damage to non-structural elements such as cladding that are connected from floor to floor. Generally the upper limit on drift ratio is about 0.02 to 0.025. Structures without elements that can be damaged by relative story displacement (e.g., some parking structures) could be allowed to reach higher drift levels, but this is not explicitly recognized in most code provisions.

In some codes (such as in Chile) the drift limits refer to elastic response to the reduced design load, but more commonly the maximum drift corresponding to the full design seismic intensity is specified. In some countries, significantly lower drift limits are specified. For example inferred maximum drifts in Chile are in the range 0.004 to 0.010.

In force-based design, structures are governed either by the force-reduction factor (R or R_μ) or by the drift limit. For concrete moment resisting frames, the drift limit rather than the R or R_μ factor tends to govern the seismic design. For concrete wall structures, the R factor rather than the drift limit tends to govern the design. In displacement-based design, meeting required strength and achieving displacement control are both accomplished in the same process.

4.6.5.6 Estimating yield drift

The yield drift can be directly estimated based on the dimensions of structural members. For moment resisting frames governed by beam plastic hinging, drifts at first yield may be estimated (Priestley, 2000) as

$$\theta_y \approx 0.5\varepsilon_y \frac{l_b}{h_b} \tag{4}$$

where ε_y is the yield strain of the beam reinforcement, l_b is the bay length (distance between adjacent column centerlines) and h_b is the beam depth. Typical l_b/h_b values lie in the range 5 to 10, for which Equation 4 gives yield drifts of 0.005 to 0.010.

For a peak drift equal to the common limit of 0.02, the corresponding ductility demand is between 2.0 and 4.0. The force reduction factor then lies between 2.0 and 4.0 if the equal displacement approximation is accepted, or between 3.7 and 7.9 if the equal energy approximation of Equation 3 is chosen. These values all fall below the ICC value of $R_\mu = 8$ and demonstrate an inconsistency in that code. The inconsistency is greatest for buildings with periods in the approximate range $0.6 < T < 4.0$ sec., where the equal displacements approximation might be expected to govern.

4.6.6 Recommendations for equivalent-monolithic precast systems

Since equivalent-monolithic precast systems by definition behave like cast-in-place reinforced concrete systems, force-based design requirements for cast-in-place concrete may be adopted directly. However, it should be apparent from the foregoing discussion that considerable differences in required strength will result from compliance with different seismic codes. The following recommendations indicate current trends in design philosophy that are likely to be incorporated into the next generation of design codes.

4.6.6.1 Period

Period calculations should be based on standard conditions that are used for all concrete structures. Realistic estimates of member stiffness at first yield of the flexural reinforcement offer a suitable basis. Member effective stiffness depends on axial force and reinforcement ratio. The reinforcement ratio or member strength will not be known at the start of design, and hence some iteration may be required. Neglecting the effect of member strength and reinforcement ratio, initial estimates of period for precast reinforced concrete members may be made assuming the following values for member stiffness:

Beams:	$0.35\ I_{gross}$
Columns:	$0.4\ I_{gross} - 0.7\ I_{gross}$ depending on axial load ratio
Walls:	$0.3\ I_{gross}$

4.6.6.2 Force-Reduction Factors

Limits on R factors depend on structural form, and amount and distribution of transverse reinforcement in potential plastic hinges. Assuming validity of the equal displacement approximation, the limit for R is equal to the displacement ductility capacity. Based on this, reasonable limits to R are:

Frames:	R <5
Walls:	R <4

The value for walls is set lower than that for frames mainly in recognition of reduced energy dissipation capacity, and of greater sensitivity to foundation compliance effects. The value for wall structures warrants further investigation and refinement in the light of undesirable diaphragm behavior, which is more often apparent in wall structures than in frame structures because of the greater force concentrations in the former.

The *R* values given above are consistent with the recommended values for effective initial stiffness given here. They are not consistent with more traditional stiffness assumptions, which tend to overestimate initial stiffness. If larger initial stiffness are assumed, the predicted yield displacements will be smaller and the calculated ductility capacity to achieve an ultimate deformation level will be greater. This would indicate higher *R* values.

The appropriate ductility level, and hence the design *R* value, will be limited by maximum permitted drift limits, as discussed above.

4.6.6.3 Yield curvature

It has been shown that curvatures at first yield of reinforced concrete members are essentially independent of reinforcement content and axial load ratio (Priestley, 2000), and may be approximated with adequate accuracy for design by the following relationships

Beams (rectangular or flanged):	$h_b\phi_y = 1.7\varepsilon_y$	(+/-) 10%	(5a)
Circular columns:	$D\phi_y = 2.35\varepsilon_y$	(+/-) 15%	(5b)
Rectangular columns:	$h_c\phi_y = 2.12\varepsilon_y$	(+/-) 10%	(5c)
Rectangular walls:	$l_w\phi_y = 2.0\varepsilon_y$	(+/-) 10%	(5d)

For non-rectangular walls, ϕ_y can be estimated as $1.5\varepsilon_y / (l_w - c)$, where c is the neutral axis depth at the nominal moment strength M_n.

The advantage of using Equations 4 and 5 is that the equations are based only on the steel yield strain (a known quantity), and on the geometry of the structural members. This enables yield drifts to be estimated for different structural forms before the required strength is known.

Thus, from the code specified drift limits it is possible, at the start of the design, to establish the maximum displacement ductility factor, and hence the maximum force reduction factor, that could apply to the design. For moment resisting frame structures, this limit will almost always be less than the structural limit corresponding to material damage. Taller wall structures can similarly be controlled by drift limits, when realistic estimates of elastic displacements are used.

In force-based design, the equations given above for yield curvature can be used to calculate effective moment of inertia, I_{eff}, based on member strength and geometry. Based on the relationship that $\phi_y = 1.5\varepsilon_y / (h - c) = M_n / E_cI_{eff}$, rearranging gives: $E_cI_{eff} = M_n(h - c) / 1.5\varepsilon_y$.

Substituting $\varepsilon_y = f_y / E_s$ gives $I_{eff} = (E_s/E_c)M_n(h - c) / 1.5f_y$.

Once the member strength is known, calculations from this equation can be used to refine stiffness estimates that were made based on the recommendations of 4.6.6.1.

4.6.6.4 Drift limits

Current drift limits of 2% to 2.5% are likely to remain for control of non-structural damage. In some codes a distinction is made depending on the method used to determine the expected drift. If this results from inelastic time-history analysis using records complying with the design spectral shape, a higher limit is permitted, with lower drift limits being specified if simplified analyses are used. In certain classes of structure, such as parking structures, the opportunities for and consequences of non-structural damage are reduced. A case exists for permitting higher drifts, with limits based on restricting non-structural damage. However, in such cases the drift due to all elements, including diaphragms, should be included. The issue of the effects of diaphragms on deformation is discussed in greater detail in Chapter 6.

The re-centering action of jointed and jointed-hybrid systems means that these systems should have minimal residual displacements after a strong earthquake. The lack of residual displacements reduces the likely need for post-earthquake repair, making such systems suitable for damage-control or immediate-occupancy performance levels.

Chapter 9 on analytical methods provides further the appropriate modeling of hysteretic characteristics of precast structures.

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Appendix A: Direct Displacement-Based Design

A.1 Overview

The formulation known as direct Displacement-Based Design (DBD) is described in the following. Several alternative procedures have also been developed. The design approach is intended to lead to structures that achieve, rather than are bounded by, a specific performance state under a given seismic intensity. The result is uniform-risk structures, which is philosophically compatible with the uniform-risk seismic spectra incorporated in most codes. The strength of the structure is selected by rational means to ensure that the displacement goal is met. The displacement is the key parameter, and the loads are derived from it. This approach is essentially the reverse of the traditional Force Based Design (FBD), in which the loads are derived first and an estimate of the drift is then obtained from the computed loads.

Priestley (2000) has described the Displacement Based Design method in detail, so only a summary is given here. The method has its origins in the work of Gulkan and Sozen (1971) and Shibata and Sozen (1976), but has undergone considerable development since then [(Priestley and Kowalsky, 2000), Priestley (2003)], and is now incorporated into the SEAOC Recommended Lateral Force Requirements (1999) as an approach to determining seismic loads.

Two important assumptions underlie direct displacement-based design:

- First, the response of the structure is dominated by a deformed shape that resembles the fundamental inelastic mode shape. This is essentially the same assumption as is made in the Equivalent Lateral Force embodiment of existing force-based design. It generally leads to a reasonable description of the building and story drifts but, because it excludes higher mode effects, it cannot be used alone to calculate individual story forces and the corresponding floor-to-frame connection forces. The inelastic mode shape is defined in the same way as an elastic mode shape, namely the shape that leads to identical distributions of load and response. However the inelastic shape is not a mode shape in the sense of an eigenvector to a linear system, because it lacks some of the properties, such as orthogonality, of a true mode shape. The shape also varies with the intensity of the applied load. In DBD, a shape is chosen that approximates the displaced shape of the inelastic system at the design load.
- Second, the peak displacement of an inelastic single-degree-of-freedom (SDOF) system is the same as that of a viscously damped elastic system if the two have, at peak drift, the same secant stiffness and energy dissipation per cycle.

A.2 Notation

A_{loop} = area of hysteresis loop
A_{rect} = area of rectangle circumscribing hysteresis loop
$\{e\}$ = vector with elements = 1.0 in DOFs parallel to ground motion and 0.0 elsewhere
C_d = ratio of inelastic to elastic drift
f_{pu} = specified strength of pre-stressing tendon material
f_{py} = specified yield strength of pre-stressing tendon material
f_{sy} = specified yield strength of deformed reinforcement
K = stiffness of SDOF system
K_{eq} = secant stiffness of true hysteretic system at maximum displacement
L = earthquake participating mass
M = mass of SDOF system
$[M]$ = mass matrix
M^* = generalized mass in first mode
$M_{cap,tot}$ = total moment capacity
$M_{cap,p}$ = moment capacity provided by pre-stressed reinforcement
$M_{cap,s}$ = moment capacity provided by yielding reinforcement
$M_{cap,tot}$ = total moment capacity
M_{eff} = effective mass in first mode
$M_{s',des}$ = resisting moment provided by compression deformed reinforcement at design limit state
R = seismic response modification factor
S_a = spectral acceleration
S_d = spectral displacement
T = period of linear elastic SDOF system
T_{eq} = period of equivalent viscously-damped linear SDOF system
V_{des} = design base shear
V_{eq} = design base shear of equivalent viscously-damped linear system
V_{max} = peak shear experienced during pushover analysis

Γ	= earthquake participation factor
Δ_M	= inelastic drift of structure under reduced earthquake load in 1997 UBC
Δ_S	= elastic drift of structure under reduced earthquake load in 1997 UBC
$\Delta_{tar,MDOF}$	= target displacement for multi-degree-of-freedom (MDOF) system
$\Delta_{tar,SDOF}$	= target displacement for SDOF system
θ_{des}	= design interface rotation, consistent with design moment, M_{des}
λ	= over-strength factor
λ_p	= over-strength factor for pre-stressed reinforcement in tension
λ_s	= over-strength factor for deformed reinforcement in tension
$\lambda_{s'}$	= over-strength factor for deformed reinforcement in compression
ξ	= viscous damping
ξ_{eq}	= viscous damping in equivalent linear system
$\xi_{eq,calc}$	= calculated viscous damping in equivalent linear system
$\xi_{eq,est}$	= estimated viscous damping in equivalent linear system
$\{\phi_{eq}\}$	= equivalent mode shape, or assumed deformed shape
ω	= natural frequency of SDOF system
ω_{eq}	= natural frequency of equivalent linear SDOF system

A.3 Procedure

The core of the DBD method may be explained most easily by considering the simplest case of a viscously damped linear Single Degree-of Freedom (SDOF) system in which the system mass and damping are assumed to be known. The objective is to design the system so that it will reach a specified displacement (the design, or target, displacement, $\Delta_{tar,SDOF}$) when subjected to a specified ground motion. The procedure is illustrated in Figure A-1. It requires the elastic Displacement Response Spectrum (DRS) for the ground motion in question. This can easily be obtained from the traditionally used Acceleration Response Spectrum (ARS) for the motion by dividing each ordinate of the ARS by ω^2, since

$$S_d = \frac{S_a}{\omega^2} \tag{A-1}$$

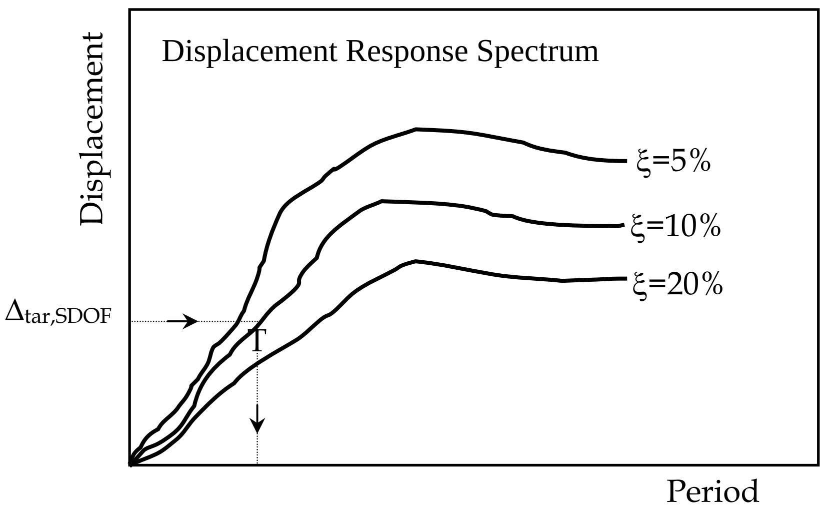


Fig. A-1: Use of DRS to find period corresponding to a target displacement

Figure A-1 shows curves typical of a DRS for a specific ground motion. For design, the DRS is likely to be idealized by a series of smooth curves or lines, such as shown in Figure A-2. The conversion from ARS to DRS is illustrated for a typical design spectrum in Figure A-3.

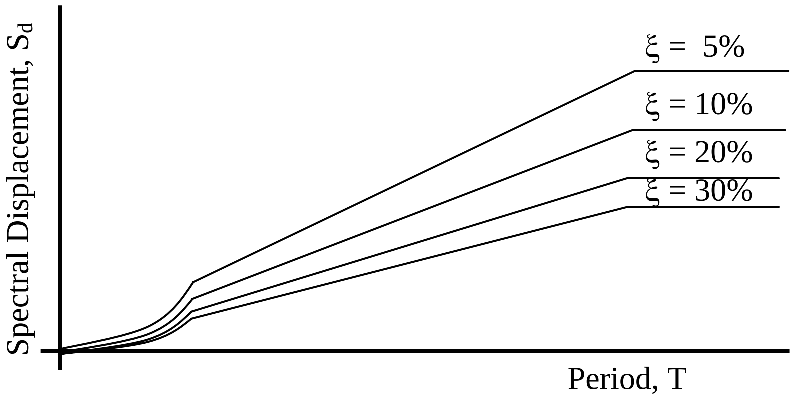


Fig. A-2 Typical design Displacement Response Spectra (DRS)

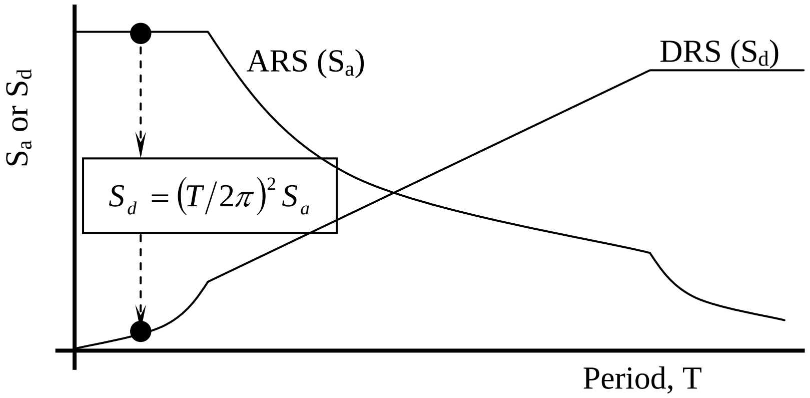


Fig. A-3: Conversion from ARS to DRS

The design calculations consist of entering the DRS with the design displacement, $\Delta_{tar,SDOF}$, and finding the period, T, that corresponds to it. This is illustrated in Figure A-1. The curve corresponding to the correct damping for the system relates the displacement and the period.

The required stiffness can then be obtained from the period, T, and the known mass, M, because

$$K = M\omega^2 = M\left(\frac{2\pi}{T}\right)^2 \qquad \text{(A-2)}$$

Once the required stiffness is known, the member sizes can be selected and the design is complete.

If the true system is hysteretic, rather than elastic, the procedure must be augmented by a second step that relates inelastic and elastic behavior. The (SDOF) hysteretic system is approximated by an equivalent viscously damped one. The two are equivalent in that the stiffness of the elastic system is the same as the secant stiffness of the inelastic system at the design drift. The viscous damping is selected to give the same energy dissipation per cycle as exists in the hysteretic system at the design drift. The hysteretic energy dissipation per cycle, and therefore the equivalent damping, is assumed here to be known at the start of the design. (A modification to address the case in which it is not known is described below). The DRS is entered, as before, with $\Delta_{tar,SDOF}$ to find a period. The resulting period is T_{eq}, the period of the equivalent elastic system that, when combined with the known damping, will result in the desired $\Delta_{tar,SDOF}$. The equivalent stiffness, K_{eq}, corresponding to T_{eq}, is then computed from

$$K_{eq} = M\left(\frac{2\pi}{T_{eq}}\right)^2 \quad \text{(A-3)}$$

An elastic system with this stiffness will result in the desired displacement, $\Delta_{tar,SDOF}$. The required strength, V_{eq}, of the real hysteretic system can be obtained from the equivalent elastic one by reference to Figure A-4 and Equation A-4.

$$V_{eq} = K_{eq}\Delta_{tar,SDOF} \quad \text{(A-4)}$$

Since $\Delta_{tar,SDOF}$ was selected by the designer, and is therefore known, V_{eq} can be computed directly from Equation A-4.

Once the required strength, V_{eq}, has been computed, the member sizes can be selected and the complete load vs. displacement curve can be constructed. The area inside the hysteresis loop is equal to the energy dissipated per cycle, from which the equivalent damping can be computed. For systems with the same properties in each direction, the equivalent damping [Chopra, (1999)] is given by Equation A-5.

$$\xi_{eq} = \frac{2}{\pi}\frac{A_{loop}}{A_{rect}} \quad \text{(A-5)}$$

where A_{loop} = area enclosed by the hysteresis loop
A_{rect} = area of the rectangle circumscribing the hysteresis loop

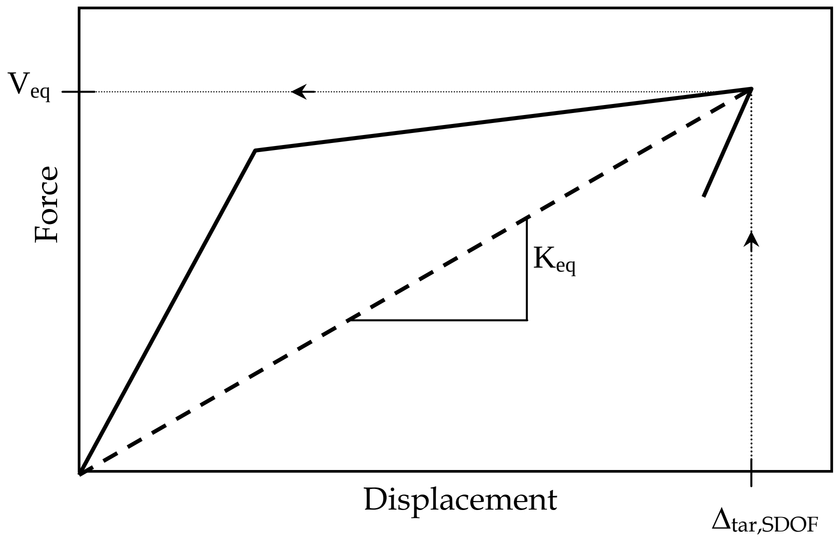


Fig. A-4: Relationship among $\Delta_{tar,SDOF}$, V_{eq} and K_{eq}.

If ξ_{eq} differs from the value assumed at the start of the analysis, the computations must be repeated with the new ξ_{eq} until convergence is achieved.

If, in addition, the system has more than one Degree of Freedom (DOF), a procedure is needed for reducing the Multi-Degree-of-Freedom (MDOF) system to an equivalent SDOF one, so that the DRS can be used. This is done using classical modal analysis procedures, except that a deformed shape, $\{\phi_{eq}\}$, is assumed for the MDOF hysteretic system and is used in place of the true first elastic mode shape. Approximate shapes are suggested by Priestley and Kowalsky [2000]. Use of this equivalent mode shape leads to

$$M^* = \{\phi_{eq}\}^T [M] \{\phi_{eq}\} \quad \text{(A-6)}$$

$$L = \{\phi_{eq}\}^T [M] \{e\} \quad \text{(A-7)}$$

$$\Gamma = \frac{L}{M^*} \quad \text{(A-8)}$$

$$M_{eff} = \frac{L^2}{M^*} \quad \text{(A-9)}$$

$$\Delta_{tar,SDOF} = \Delta_{tar,MDOF} / \Gamma \quad \text{(A-10)}$$

where $\{\phi_{eq}\}$ = equivalent mode shape (i.e. shape chosen by the engineer to represent the deflected shape of the structure at the design drift)
[M] = mass matrix
{e} = vector with elements = 1.0 in DOFs parallel to the ground motion and 0.0 elsewhere
Γ = earthquake participation factor
L = earthquake participating mass
M* = generalized mass in first mode
M_{eff} = effective mass in first mode

The equivalent mode shape, $\{\phi_{eq}\}$, should be normalized so that it has the value 1.0 at the location where the target displacement, $\Delta_{tar,MDOF}$ is measured in the MDOF system.

After computing the design base shear, V_{des}, on the basis of an assumed level of damping, and designing the corresponding member sizes and strengths, the true damping supplied by the MDOF hysteretic system must be computed. This may be done by conducting a single-cycle "pushover" analysis on the MDOF system, using imposed lateral displacements distributed in the assumed displaced shape (the equivalent mode shape). The structure should be pushed to $\Delta_{tar,MDOF}$, reversed to $-\Delta_{tar,MDOF}$, then taken back to zero displacement. Equation 2.5 can then be used to determine the equivalent damping, for symmetric systems. In Equation 2.5, A_{loop} is given by the energy dissipated by the MDOF system and A_{rect} is given by ($4\Delta_{tar,SDOF}*V_{max}$), where V_{max} is the peak base shear experienced during the pushover analysis.

For hysteretic MDOF systems, the procedure may be broken into nine steps, shown in Figure A-5.

Step 1. ***Select the target displacement, $\Delta_{tar,MDOF}$.*** A possible basis for the choice is the amount of drift-induced damage to the building that is deemed to be tolerable under the intensity of ground motion being considered.

Step 2. ***Estimate the equivalent viscous damping of the structure, $\xi_{eq,est}$.*** The exact value is not important at this stage because it will be corrected in subsequent iterations. The value depends on the ductility demand. In the absence of better information, the following starting values may be used as a guide: 5-8% for undamped unbonded pre-stressed systems, 8-15% for unbonded pre-stressed frames and walls with damping, and 15-25% for yielding or yielding gap frames.

Step 3. ***Select a deformed shape, {ϕ_{eq}}.*** The shape should resemble the expected deformed shape of the structure at the design drift. This shape is used as the equivalent mode shape in the analysis that follows.

Step 4. ***Compute the Earthquake Participation Factor**, Γ,* that converts the roof displacement of the MDOF system to the displacement of the associated SDOF system, using conventional modal techniques and treating the deformed shape of Step 3 as an elastic mode shape. If the equivalent mode shape is normalized so that the element corresponding to the roof displacement is 1.0, and if the MDOF target displacement is measured at the roof, the Earthquake Participation Factor, Γ, is defined by

$$L = \{\phi_{eq}\}^T [M]\{e\} \tag{A-11}$$

$$M^* = \{\phi_{eq}\}^T [M]\{\phi_{eq}\} \tag{A-12}$$

$$\Gamma = \frac{L}{M^*} \tag{A-13}$$

The target drift of the SDOF system is then

$$\Delta_{tar,SDOF} = \Delta_{tar,MDOF} \; / \Gamma \tag{A-14}$$

Step 5. ***Compute the Equivalent Period, T_{eq},*** necessary to achieve $\Delta_{tar,SDOF}$ during the design earthquake, given the estimated damping, $\xi_{eq,est}$, and the DRS.

Step 6. ***Compute the Equivalent Stiffness, K_{eq},*** for the SDOF system from the equivalent period and the known mass of the structure, by

$$K_{eq} = \left(\frac{2\pi}{T_{eq}}\right)^2 M^* \tag{A-15}$$

Step 7. ***Compute the Design Base Shear*** from

$$V_{des} = K_{eq}\Delta_{tar,SDOF} \tag{A-16}$$

Compute the story loads using the distribution defined by the equivalent mode shape.

Step 8. ***Analyze the structure*** under the loads of Step 7, obtain the member forces and design the members to resist them.

Step 9. ***Determine the Energy Dissipated per Cycle*** (EDC) by the structure from a cyclic push-pull analysis or otherwise, and re-evaluate the equivalent viscous damping. It is given by

$$\xi_{eq,calc} = \frac{2}{\pi}\frac{A_{loop}}{A_{rect}} \tag{A-17}$$

If the value of $\xi_{eq,calc}$ differs significantly from the previous estimate, $\xi_{eq,est}$, repeat Steps 5 to 9 with the new estimate of damping.

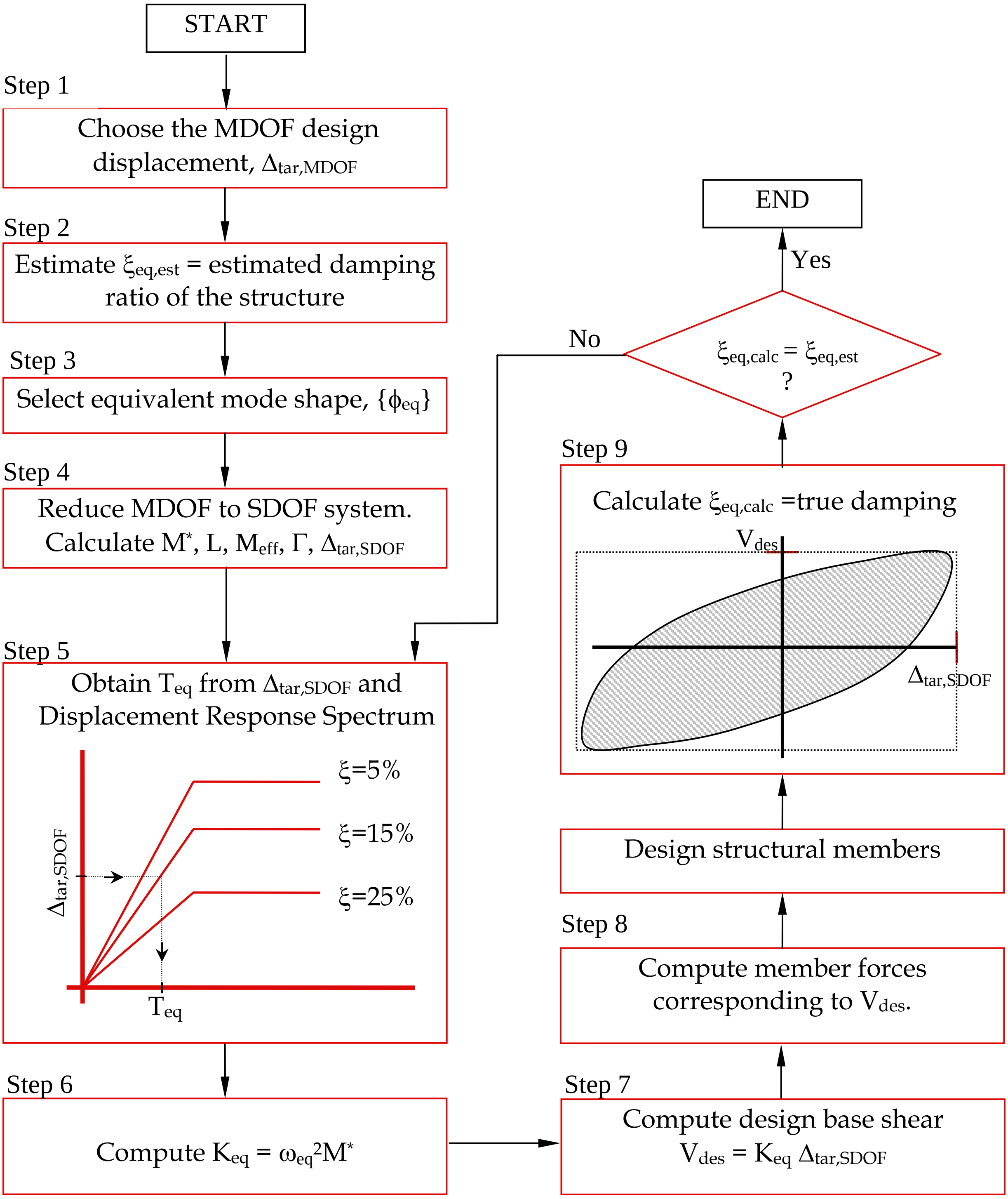


Fig. A-5: Flowchart for Displacement-Based Design

A.4 Discussion of Displacement-Based Design

Displacement-Based Design addresses primarily the response in the fundamental mode of vibration. In almost all two-dimensional systems, this represents the vast majority of the displacement. However, local forces, such as floor-to-wall forces in a wall structure, may be much larger than those predicted from the basic DBD procedure because of the local effect of higher modes. The same is true for the FBD procedure. Consideration of such forces is important at least in the design of connections. One approach to estimating the effects of higher modes is given by Eberhard and Sozen (1993). It is applicable to both FBD and DBD.

One of the consequences of using DBD is that the required strength is related to the available damping. Thus, if two systems have identical stiffnesses but different damping characteristics, the one with the smaller damping will require a larger strength if both are to reach the same prescribed drift. This feature of the DBD procedure reflects the physics of the problem, but is not taken into account by FBD. As an example, if a pre-stressed frame with damping and a pre-stressed frame without damping are under consideration for a particular building, if DBD is used, and if the same drift limit is imposed for both systems, the design loads will be higher in the system without damping.

A.5 Recommendations for precast jointed and other systems

A.5.1 Equivalent mode shape

The equivalent mode shape can be estimated from characteristic displacement profiles at maximum response based on elastic time history analysis. The following equations, though approximate, have been shown to be adequate for design purposes in buildings with equal storey masses. [Loeding et al, (1998)].

For frames with n storeys:

for n < 4: $$\Delta_i = \theta_d h_i \quad \text{(A-18a)}$$

for 4 < n < 20 $$\Delta_i = \theta_d h_i \left[1 - \frac{0.5 h_i (n-4)}{16 h_n} \right] \quad \text{(A-18b)}$$

for n > 20 $$\Delta_i = \theta_d h_i \left(1 - 0.5 h_i / h_n \right) \quad \text{(A-18c)}$$

where h_i and h_n are the heights to the ith storey and the roof respectively.

For cantilever wall buildings, the maximum drift occurs at the top of the building. Assuming a linear distribution of curvature with height, the yield drift at roof level is

$$\theta_y = \varepsilon_y h_n / l_w \quad \text{(A-19)}$$

Hence the design drift becomes:

$$\theta_d = \varepsilon_y h_n / l_w + (\phi_m - \phi_y) l_P \le \theta_c \quad \text{(A-20)}$$

where l_P is the plastic hinge length.

The design displacement profile for the wall can then be estimated (Priestley, 2000) as:

$$\Delta_i = 0.67 \varepsilon_y \frac{h_i^2}{l_w} \left[1.5 - \frac{h_i}{2h_n} \right] + \left[\theta_d - \frac{\varepsilon_y h_n}{l_w} \right] \left[h_i - \frac{l_P}{2} \right] \quad \text{(A-21)}$$

Effective mass: From consideration of the mass participating in the first inelastic mode, the effective system mass for the equivalent SDOF system is

$$m_e = \sum_{i=1}^{n} \left(m_i \Delta_i\right) / \Delta_d \tag{A-22}$$

where Δ_d is the design displacement

Typically, for building structures:

$$m_e \approx 0.8 \sum m_i \tag{A-23}$$

Effective damping: The effective damping depends on the structural system and the displacement ductility factor $\mu = \Delta_d / \Delta_y$, where the design and yield displacement may be calculated as above.

The following approximate relationships may be used to relate damping (ξ, expressed as a percentage of critical damping) to ductility factor for different structural systems:

Reinforced concrete frame:	$\xi = 5 + 30(1 - \mu^{-0.5})$ %	(A-24a)
Reinforced wall structures:	$\xi = 5 + 23(1 - \mu^{-0.5})$ %	(A-24b)
Frames or walls with unbonded prestressing:	$\xi = 5$ %	(A-24c)

For frames or walls where the strength and energy dissipation is provided by a combination of unbonded prestressing steel and mild steel reinforcement, the effective damping should be interpolated between Equations 16c and 16a in proportion to the fraction of flexural strength provided by the mild steel reinforcement.

A.5.2 Design displacement spectra

A major difference from force-based design is that design utilizes a set of displacement-period spectra (eg, Figure A-2) for different levels of equivalent viscous damping, rather than the acceleration-period spectra for 5% damping adopted by most force-based codes. It is appropriate to place a cap on peak response displacements, since at long periods, structural displacements tend to reduce, eventually equalling the peak ground displacement. The European seismic code (EC8, 2003) adopts a cap at T = 3 sec. above which displacements are considered to be independent of period. Geotechnical considerations indicate that the cap period should depend on the foundation condition, with lower periods applying for rock than for soft soil.

Design Displacement Spectra were discussed in Section 4.3.3. A site-specific spectrum is the best option, but in its absence a DRS may be generated from the appropriate 5% damped design ARS as described in Section 4.3.3. In most cases, the effective period of the structure will be longer than the period that separates the acceleration and velocity controlled regions of the spectrum (usually between 0.5 and 0.75 seconds), so the acceleration controlled region may reasonably be ignored. Response curves for damping levels other than 5% may be approximated by

$$S_d(T,\xi) = S_d(T,5)\left[\frac{7}{2+\xi}\right]^{1/2} \tag{A-25}$$

where $S_d(T,\xi)$ is the 5% spectral displacement for ξ% damping.

Equation 18 is used in EC8. SEAOC presents an alternative relationship, based on work by Newmark and Hall (1980).

A.5.3 Distribution of base shear force

The base shear calculated in accordance with the above procedure should be vertically distributed based on the vertical mass and displacement profiles. Thus:

$$F_i = V_B(m_i \Delta_i) / \sum_{i=1}^{n}(m_i \Delta_i) \tag{A-26}$$

The similarity with force-based design is apparent. The difference is that the forces are distributed in accordance with the design displacement profile, rather than a height-proportional profile.

A.5.4 Analysis for member design actions

Because direct Displacement-Based Design is conducted at maximum displacement response, inelastic action will have occurred at critical sections. Thus, any distribution of internal forces that satisfies equilibrium is acceptable, provided that the corresponding inelastic deformations do not exceed the member ductility capacities. If the member strengths are chosen, the corresponding ductility demands may be established directly by inelastic analysis. Alternatively, the desired ductilities may be selected and the required strengths may then be obtained approximately by analyzing an elastic model in which the member stiffnesses are reduced to reflect the ductility demand. For example, in a frame designed for beam hinging, the beams will respond inelastically, and the appropriate stiffness is

$$I_b = I_{cr}/\Upsilon_b \tag{A-27}$$

where Υ_b is the expected beam displacement ductility demand. Analyses have shown that member forces are relatively insensitive to the level of stiffness assumed, and thus is it acceptable to assume $\Upsilon_b = \Upsilon_s$, the frame design ductility. More complete details are provided in (Priestley, 2000)

5 Lateral force resisting systems

5.1 Introduction

The implementation of innovative ideas for connecting precast elements together, and subsequent verification through experimental procedures, has resulted in significant advances for the precast concrete industry in seismic regions of the world in the past two decades. For example, in New Zealand precast concrete has been used in moment resisting frames since the 1980s [Park, (1990)]. In common to other structural systems, the main challenge when using precast concrete elements as part of the lateral force resisting system is to find economical and practical means of connecting the elements together to ensure adequate stiffness, strength, lateral deformation capacity and stability. The design should ensure that the structural system performs satisfactorily, that is, with minimum residual cracking and damage when subjected to moderately strong and commonly occurring earthquakes and that the system has sufficient lateral deformation reserve to survive a rare large magnitude earthquake without collapse. In addition, and equally important, the design should have competitive advantages over other construction systems.

The types of connections between precast concrete elements in moment resisting frames and structural walls are discussed in Section 3.2.

The effective design, fabrication and erection of precast concrete elements composing the lateral force resisting system depend on several factors including:

1) design approach
2) horizontal and/or vertical jointing
3) connection reliability
4) internal force paths and three-dimensional kinematics of the system
5) foundation sub-structure

In general, lateral force resisting systems built incorporating precast or precast/prestressed concrete elements are either moment resisting frames or structural walls. Dual systems, in which these two systems are combined to provide lateral force resistance in the same direction, are also possible, but such systems have largely been unexplored. It is common practice, particularly in regions of low and moderate seismicity, to combine moment resisting frames or structural walls in parallel with precast gravity loading frames. Gravity loading frames are discussed in detail in Section 8.

Moment resisting frames are often chosen in design because of their inherent space flexibility. Frames have advantages over wall systems that energy dissipation takes place in many regions and that they are highly redundant. The main challenge when using moment resisting frames as the lateral force resisting system is to size the beam and column elements to provide an effective control to earthquake induced inter-storey lateral displacements. This control is particularly important when the building incorporates parts not specifically designed to undergo large inter-storey lateral displacements.

Structural walls have long been recognised as a very efficient lateral force resisting system in low and high-rise buildings [Fintel (1995), Park and Paulay (1975), Paulay and Priestley (1992)]. The large lateral stiffness of structural walls and their deformed shape when subject to lateral forces make them highly desirable for controlling earthquake induced inter-storey lateral displacement demands, and hence, minimizing non-structural damage, in building structures.

Structural walls are often used for construction of low-rise commercial and industrial buildings to provide, in addition to lateral force resistance, an aesthetically pleasant façade, a fire barrier and a support for climatic insulation. Structural walls have common use in parking structures where functional and gravity support requirements provide opportunities to integrate walls into the structural system. Structural walls are the preferred choice in some parts of the world for the construction of the building envelope in low-rise commercial and industrial buildings where security is a major design consideration.

The cost associated with the loss of business operation, damage to equipment and structural damage following moderately strong earthquakes has become main performance criterion since the cost of the consequences of damage and lack of operation to business in recent earthquakes worldwide has been quantified to be excessive. Such cost is often comparable, if not greater, than the cost of the structure. Emerging structural systems are being developed to provide not only life-safety but also damage-protection to building structures. For example, an alternative structural emerged in 1993 when Priestley and Tao (1993) proposed the use of lateral force resisting systems built using jointed precast concrete elements connected through unbonded prestressing tendons. This proposal was supported by a series of non-linear dynamic time-history analyses that showed the viability of such system. Priestley and Tao pointed out that a main advantage of the system investigated was the lack of residual drift following a strong earthquake. Since then, several systems have been proposed and tested, mainly as part of the PRESSS research program in United States. This program recently ended with the testing of a 60 percent scale five-storey building [Nakaki et al. (1999), Priestley et al. (1999)]. Such systems are becoming accepted in some countries and being used in practice.

5.2 Moment resisting frames

5.2.1 Member partitioning

Fig. 5-1 shows a number of precast concrete arrangements and location of connections that can be used for the construction of moment resisting frames in seismic regions. Beam, column and beam-column cruciform units can easily be precast and connected in site by several means. In the majority of cases, beam units are connected through cast-in-place joints either at the beam-column joint region or at the beam midspan. Beam and column shell units are employed when the crane capacity limits the use of other precast concrete alternatives. Precast concrete columns can be connected at the column ends or at mid-height through grouted steel sleeves or non-contact bar splices into grouted corrugated ducts.

5.2.2 Classification

Moment-resisting frames are arranged to provide lateral force resistance in one- or two-ways. Some possible arrangements are shown in Fig. 5-2. One-way frames are built with or without transverse pin-ended beams whose main purpose is to carry gravity loading. When one-way frames are built at the perimeter of the building, the transverse beams should also be designed to provide the necessary restraint to ensure frame in-plane stability. Two-way frames are often preferred over one-way frames in regions of high seismicity since every column in the structural system, as well as the foundations, participate in the lateral force resistance in two-directions. Furthermore, an advantage of two-way frames is that inertia forces arising from diaphragms generally require a short path before they are collected and transferred into the columns.

Fig. 5-3 classifies moment-resisting frames in terms of the response characteristics. Moment-resisting frames can be designed with mild steel and/or prestressing to provide lateral force resistance by responding elastically, thus, requiring only reinforcing details for nominal ductility. Note that truly linear-elastic behavior does not occur in most frames due to the development of cracking in beams, columns and joints in reinforced concrete frames and, principally, in the columns in frames with prestressed beams. Cracking of the concrete results in stiffness degradation. In the design of such systems, allowance should be made for the stiffness degradation that is expected to result from cracking of the concrete.

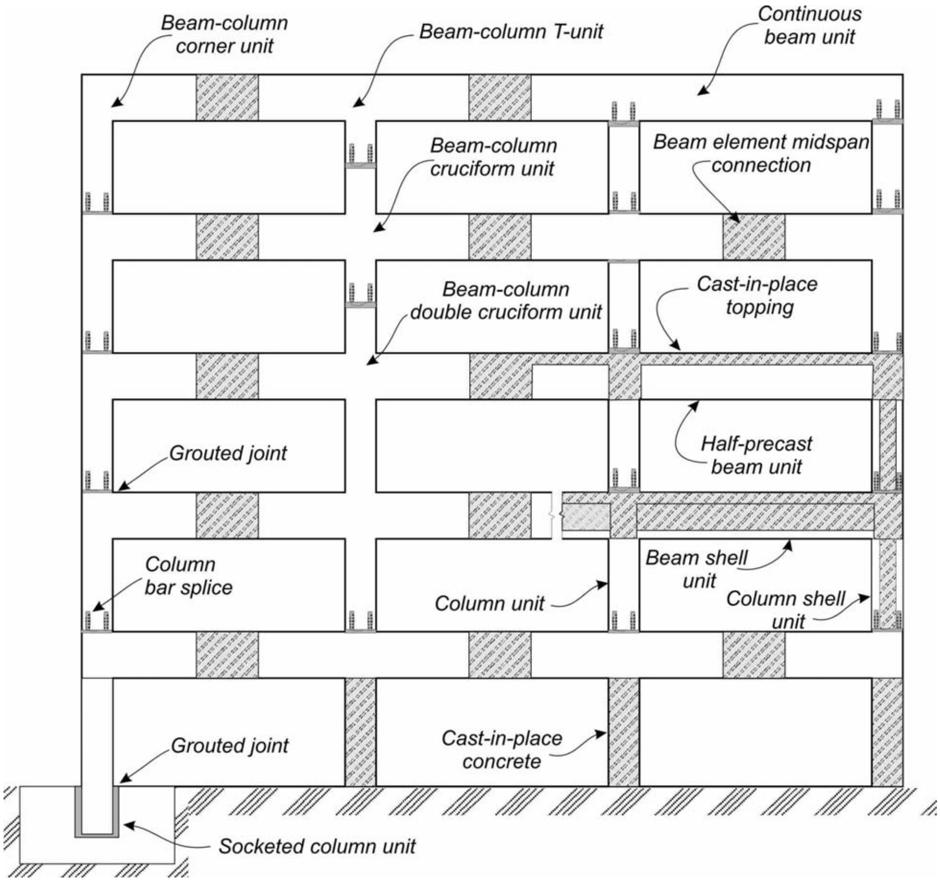


Fig. 5-1: Common arrangement of elements, components and connections in precast concrete moment-resisting frame construction

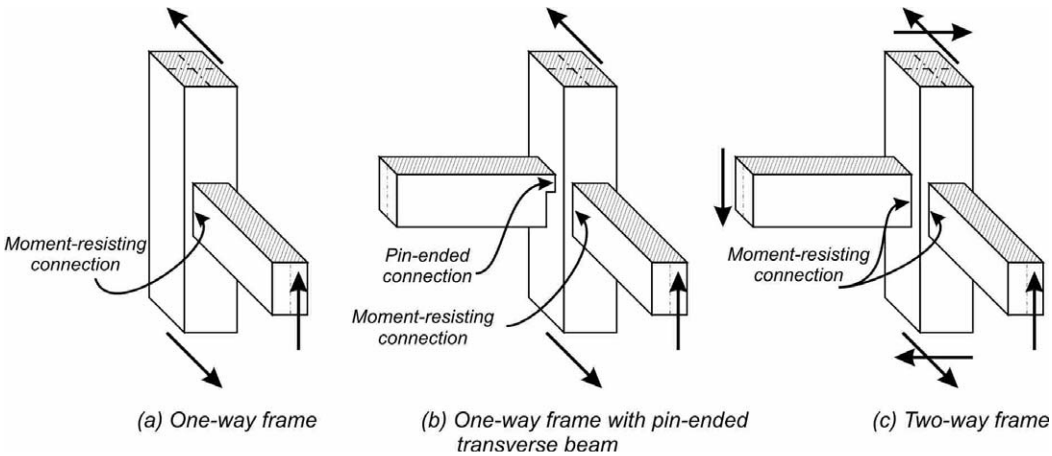


Fig. 5-2: Classification of precast concrete moment-resisting systems according to the in-plane lateral force resisting mechanisms

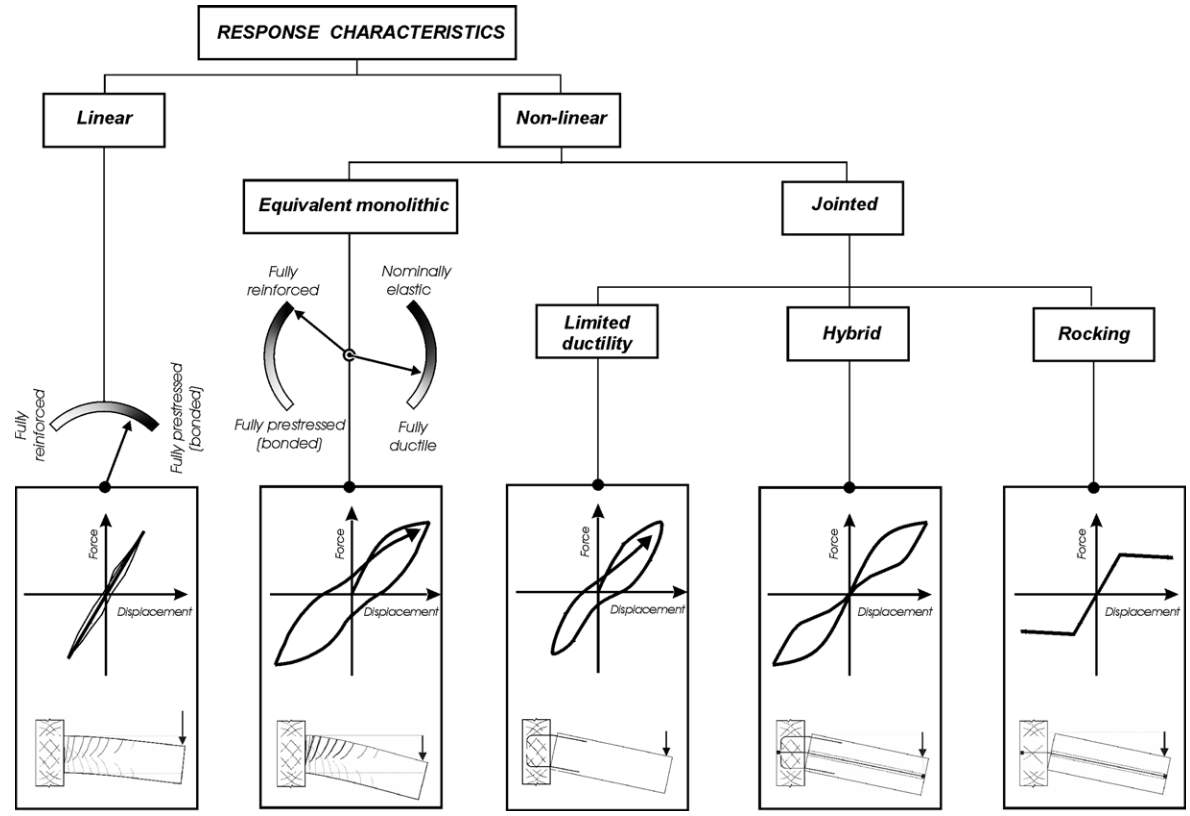


Fig. 5-3: Classification of precast concrete moment-resisting systems according to the lateral force – lateral displacement response characteristics

Moment-resisting frames are, however, often designed to provide lateral force resistance by responding non-linearly. Systems displaying non-linear response characteristics can be broadly grouped in two main categories:

1) Equivalent monolithic systems

These systems are designed to closely emulate the response of conventional cast-in-place reinforced or prestressed construction in terms of stiffness, strength, ductility capacity and energy dissipation characteristics. The equivalence between a precast concrete system and the cast-in-place counterpart should be determined through experimental work. For example, in Japan a precast concrete system is considered to be equivalent to a monolithic system if the drift of the precast system is within 80 to 120 percent of the cast-in-place counterpart and if the energy dissipation in the second loading cycle is no less than 80 percent of that obtained from the response of the cast-in-place counterpart.

Equivalent monolithic systems are designed to ensure that the overall lateral force-lateral displacement non-linear response arises from the development of flexural plastic hinges in pre-selected regions of the frames. Such regions are detailed for ductility, depending upon the expected local ductility demand expected. Other regions and elements within the structure are made overstrong to ensure they will always remain in the elastic range thereby avoiding the development of an unexpected and undesirable mode of response.

A major advantage of equivalent monolithic systems is that their design can generally follow the recommendations given in standards for seismic design of cast-in-place concrete systems.

In equivalent monolithic systems, the system ductility capacity depends on the mechanism of inelastic deformation and the ductility capacity of the critical sections in the plastic hinges. The system ductility capacity ranges from incipient or nominally elastic to fully ductile. The hysteretic response in equivalent monolithic systems depends primarily on the degree of prestressing [Park and Thompson (1977), Thompson and Park (1980a and 1980b), Nishiyama (1990)]. The hysteretic response of frames composed of non-prestressed members is reasonably large and comparable to monolithic reinforced concrete frames. Equivalent viscous damping ratios of up to 25 percent are expected for this type of construction. On the other hand, fully prestressed frames have a relatively narrow hysteretic response, with equivalent viscous damping ratios of the maximum 8 percent.

2) Jointed systems

These systems are designed such that the non-linear response takes place at pre-selected connection interfaces following a pre-determined mechanism of non-linear deformation. In jointed systems, all members are deliberately made overstrong to ensure that non-linear deformations arise only at the connections.

Hybrid and rocking systems are two jointed systems with distinct lateral force–lateral displacement response characteristics, see Fig. 5-3. In hybrid frame systems, mild-steel reinforcement and unbonded prestressing tendons are combined at the critical connections to obtain a centered-oriented hysteretic response that results in little or no residual lateral displacements. These systems display equivalent viscous damping ratios of up to 18 percent.

Rocking frame systems display a non-linear elastic response due to the elastic restoring force provided by the presence of the prestressing tendons. Rocking frame systems differ from the hybrid systems in that prestressing solely provides moment resistance at the connection. In practice, some energy dissipation does take place in rocking systems. The source of energy dissipation is crushing of the compressed concrete at the extreme ends at the connections. Equivalent viscous damping ratios for rocking frame systems are typically no more than 5 percent.

Some reinforced concrete systems that have use relatively weak connections between elements, achieved mainly by welding bolting and dry packing, can be considered as a branch of jointed systems. They are in the limited ductility category shown in Fig. 5-3.

5.2.3 General seismic design approaches for buildings incorporating moment resisting frame systems

5.2.3.1 Conceptual design

The various approaches available for the seismic design with precast concrete construction were examined in detail in Section 1.6. It is usually agreed that the design level of lateral forces should be related to the overall system lateral deformation capacity and energy dissipation characteristics. Relatively large lateral forces, of the same order of those required to ensure linear-elastic response, are chosen for moment resisting frames whose overall non-linear lateral deformation capacity is low. In contrast, lateral forces that are much smaller than those required for linear-elastic response can be used for the design of frames that possess large non-linear lateral deformation capacity through a kinematically admissible mechanism.

5.2.3.2 Mechanisms of non-linear lateral deformation

The recognition of a mechanism of non-linear deformation in design enables the explicit detailing of critical regions of moment-resisting frames where yielding occurs and also enables the design of connections between elements in regions that are deliberately protected from yielding.

A concentration in rotation in critical regions of the frames must occur in order to ensure the development of non-linear deformations required to satisfy the earthquake-induced displacement demand in frame systems. In equivalent monolithic frames the concentration of rotation occurs in the plastic hinge regions whereas in jointed systems the rotation concentrates at the critical connection interfaces.

Three possible statically admissible mechanisms on non-linear lateral deformation for equivalent monolithic moment-resisting frames are illustrated in Fig. 5-4. Similar mechanisms can develop in jointed moment-resisting frames.

The beam sideways mechanism, see Fig. 5-4 (a), distributes the non-linear deformations over the entire structure, involving mainly the development of beam plastic hinges. Columns are required to develop plastic hinges at their bases. It is also desirable from the detailing perspective that plastic hinges also develop in the columns in the upper level. The beam-sideways mechanism is preferred over other mechanism for multi-storey buildings.

The mixed sideways mechanism, see Fig. 5-4 (b), is also suitable for multi-storey buildings. This mechanism involves the development well distributed plastic hinges at the column and beam-ends. It is important in this mechanism that some columns (typically exterior columns) remain in the elastic range so as to prevent a column sideways mechanism from forming.

The column-sideways mechanism shown in Fig. 5-4 (c) concentrates the entire non-linear lateral deformation in a single level of the building requiring the development of plastic hinges at the column ends. Because of the concentration of rotation in a single level, this mechanism should only be used for low-rise buildings of up to two or three storeys high.

It should be noted that all three mechanisms depicted in Fig. 5-4 require the development of plastic hinges at the base of the columns. For this reason, the use of bar splices at the column bases should be restricted to only frames designed for elastic response. For systems designed for non-linear response, the first level columns should either be cast-in- place or socketed in the foundation and grouted as indicated in Fig. 5-1. Such connection detail is described in Section 8.2.

5.2.3.3 Kinematical considerations

The mechanisms of non-linear deformation described in previous section showed that non-linear lateral displacements take place as a result of concentration of rotation in the critical regions of the frames. In reinforced and prestressed precast concrete members this rotation is coupled to an axial member lengthening due to the offset of the neutral axis depth from the member’s mid-depth towards the extreme compressive fibre.

Fig. 5-5 shows the kinematical effects in a deep beam of a frame component. The frame component has been displaced monotonically well beyond the limit of proportionality so that the elastic deformations are negligible. Evidently, an increase in the beam length will occur due to the migration of the neutral axis depth at the two critical regions in the beam. The elongation of an unrestrained monolithic frame component is about twice of that computed from the kinematics of the monotonically deformed component due the accumulation of axial strains during reversed cyclic loading [Restrepo et al. (1993)]. The elongation per bay at 2 percent drift is of the order of 5 percent of the distance between the beam top and bottom reinforcement. In prestressed frames the restoring force due to prestressing is expected to somewhat reduce the accumulation of the axial strains caused by reversed cyclic loading at the critical regions. In jointed systems the accumulation of axial strains is negligible because the elastic restoring force of the unbonded tendons required the closure of the joints.

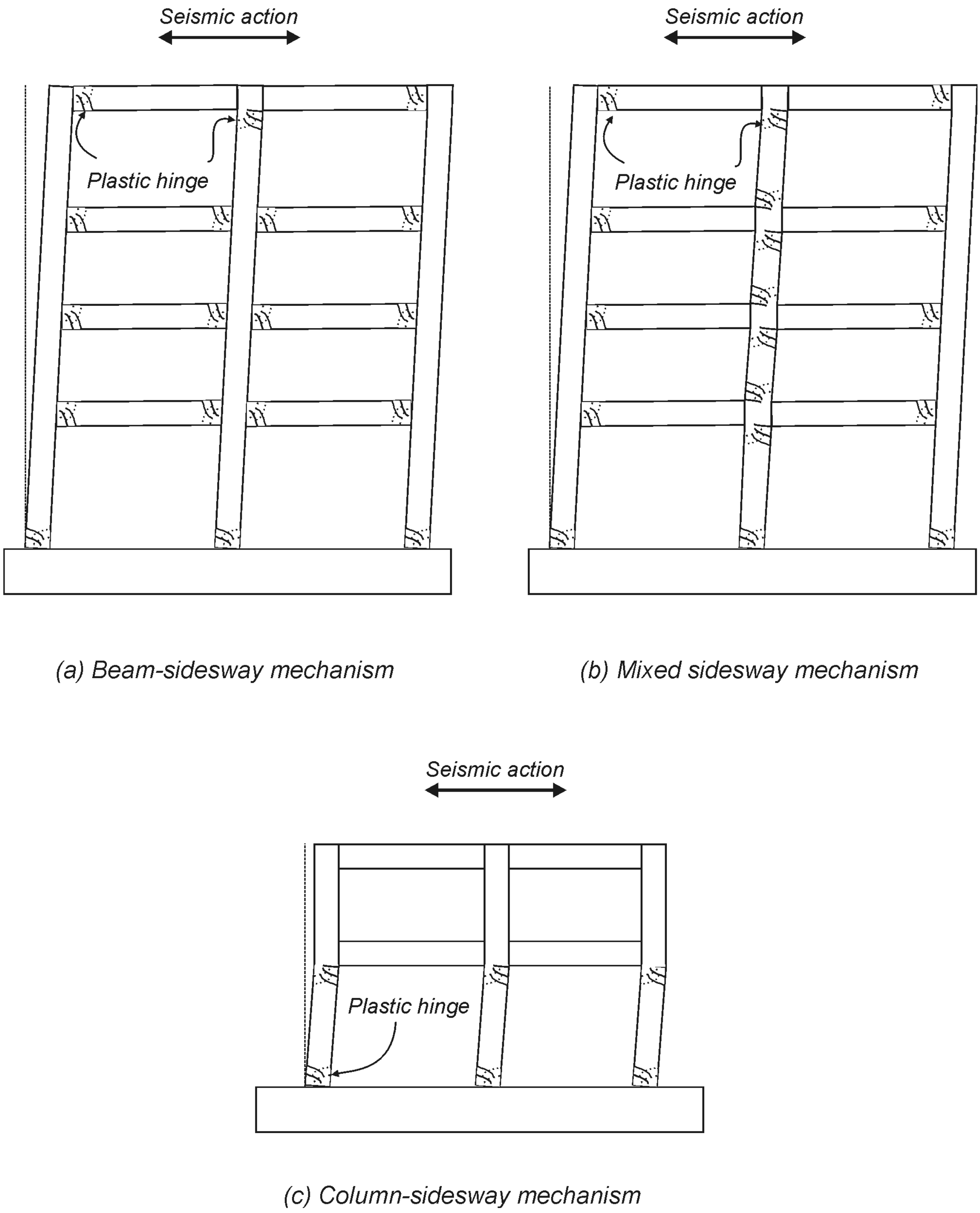


Fig. 5-4: Mechanisms of non-linear deformation in moment resisting frames

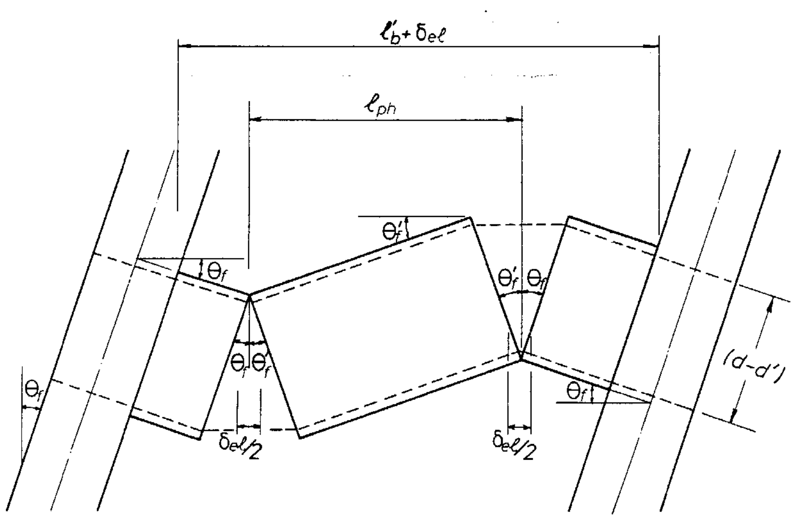


Fig. 5-5: Beam axial elongation resulting from the shift of the neutral axis depth

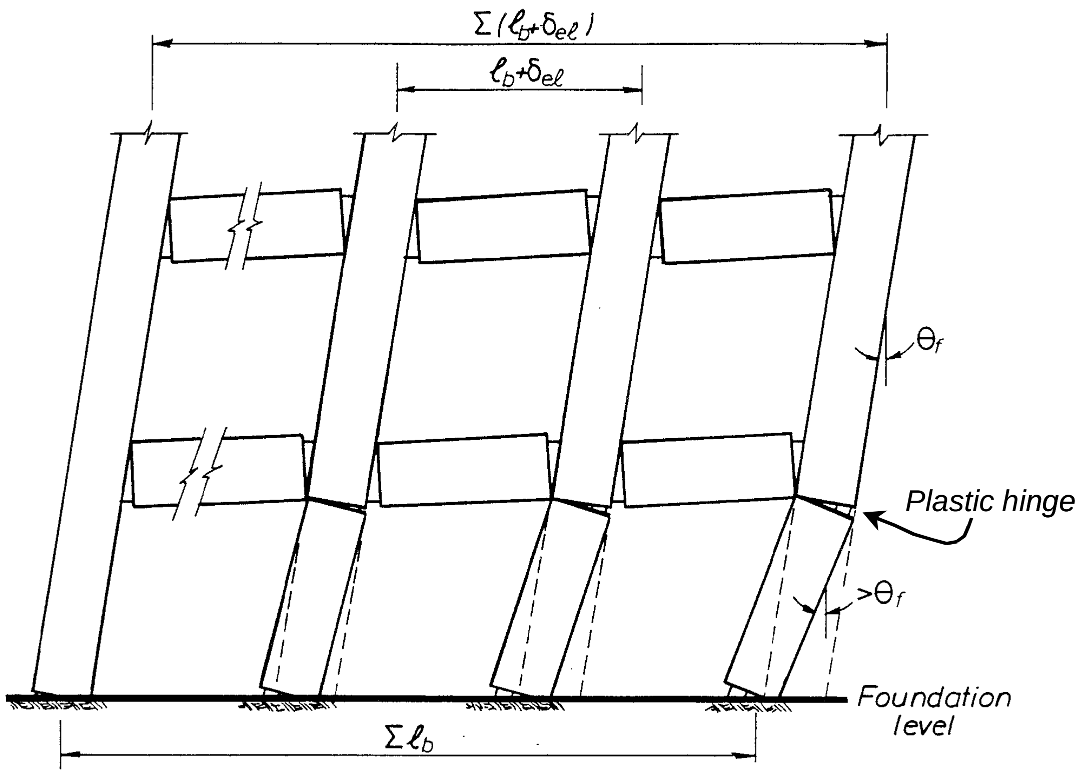


Fig. 5-6: Effects of beam axial elongation in a moment resisting frame

The effects that beam elongation has in a multi-bay multi-storey frame system are complex. For example, the elongation at the first floor is somewhat restrained by the columns below because of the degree of fixity provided by the foundation. The restraining action will induce an axial compression force in the first level beams, which will enhance their flexural capacity and reduce the ratio between the columns to beam capacities. In the worst case the exterior columns, and also the interior columns located to the exterior columns in multi-bay frames, could develop plastic hinges at the beam faces as Fig. 5-6 shows. Such plastic hinge development is often ignored in the mechanisms of non-linear lateral deformation as these mechanisms are based on line element models to represent beams and columns. At those levels above the first level, the magnitude of the beam elongation is likely to be of the same order as that measured in tests on beam-column components.

The kinematics of a moment resisting frame suggests that allowances should be made in design to appropriately proportion the seating details for precast concrete floor units and for the design of connections for cladding systems.

5.2.4 Design practice

A great deal of precast concrete has been incorporated in moment resisting frames in Japan [Kanoh, Y. (1986)] and in New Zealand [Park (1995)]. Best practice design guidelines have been published in these two countries [Architectural Institute of Japan (2000), Centre for Advanced Engineering (1999)]. The following sections give details of the systems commonly used in Japan and New Zealand.

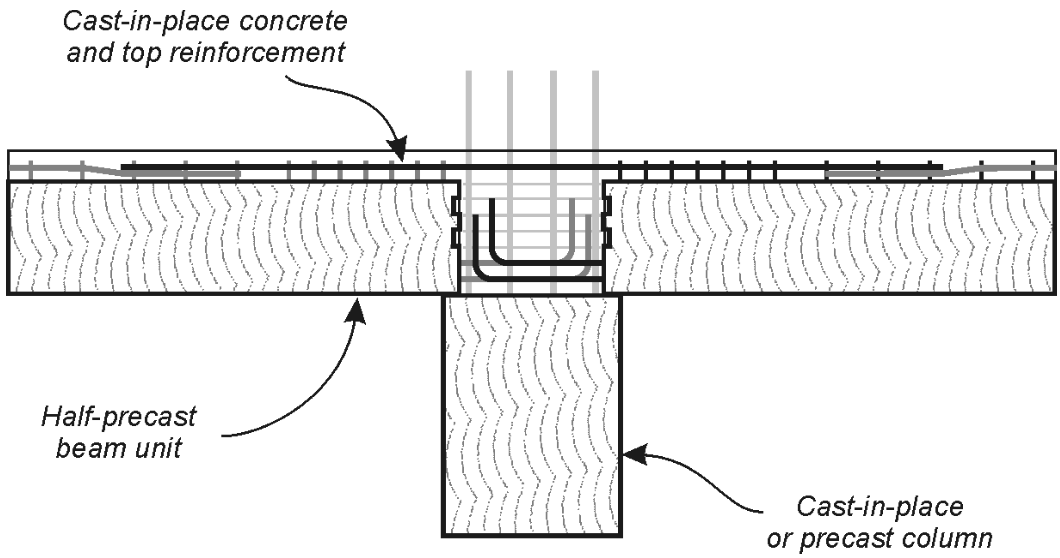
5.2.4.1 Systems for equivalent monolithic construction incorporating non-prestressed connections

Four equivalent monolithic reinforced concrete systems reported in Japan and in New Zealand are shown in Figs. 5-7 to 5-16 and are described in the following sections. It is considered that these systems are suitable for the construction of elastically responding right through to fully ductile moment resisting frames.

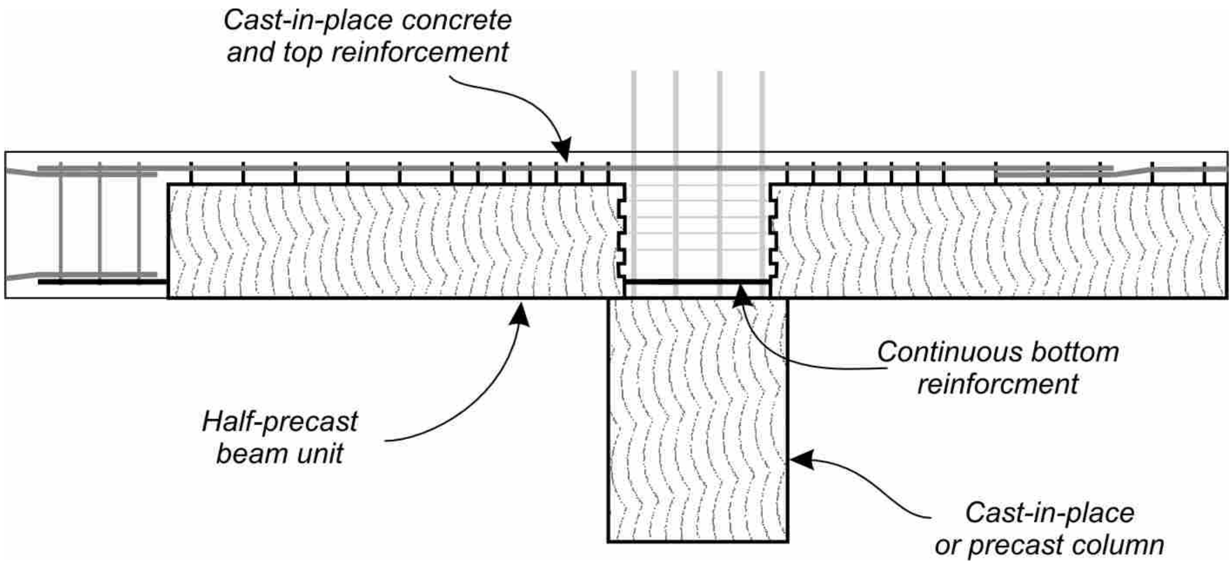
1) System 1- Precast beam units between columns

An arrangement involving the use of precast reinforced or precast partially prestressed members to form the lower part of the beams is shown in Fig. 5-7. Fig. 5-8 shows examples of construction with this system. The precast beam elements are placed between columns and seated on the cover concrete of the previously cast-in-place or precast reinforced concrete column below and/or propped adjacent to the columns. A precast concrete floor system is placed, seated on the top of the precast beam elements and spanning between them. Reinforcement is then placed in the top of the beams, over the precast floor and in the beam-column joint cores. The topping slab over the floor system and the beam-column joint cores is cast. This system can be combined with cast-in place or precast concrete columns. The precast columns can be solid or voided. In the latter case concrete is placed into the inner void prior to the placement of the concrete in the beam-column joint region. The longitudinal reinforcement in the precast columns is spliced at the column bases through grouted steel sleeves.

One approach with this system, commonly used in New Zealand, is to splice the beam bottom bars using hooked anchorages in interior beam-column joints in the joint core, see Fig. 5-7 (a). This approach can lead to congestion in the joint core. Hence the column dimensions need to be reasonably large to accommodate the required development length and to reduce the congestion caused by the hooked anchorages. The anchorage of beam bars within the joint core in System 1 can then be designed using the same code rules as for anchorage in an exterior column. In New Zealand, the anchorage of these bars should not be less than three-quarters of the column depth or $L_{dh} + 8d_b$ from the beam face at which the bar enters the column, whichever is less, where L_{dh} is the development length of the hooked bar in tension and d_b is the bar diameter [Centre for Advanced Engineering (1999)].



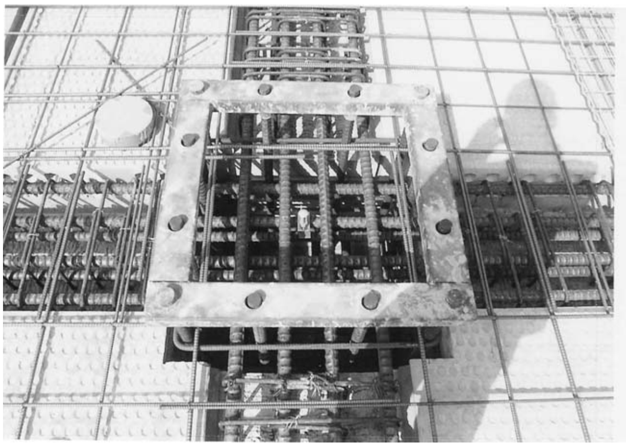
(a) Approach using hooked bar anchorages



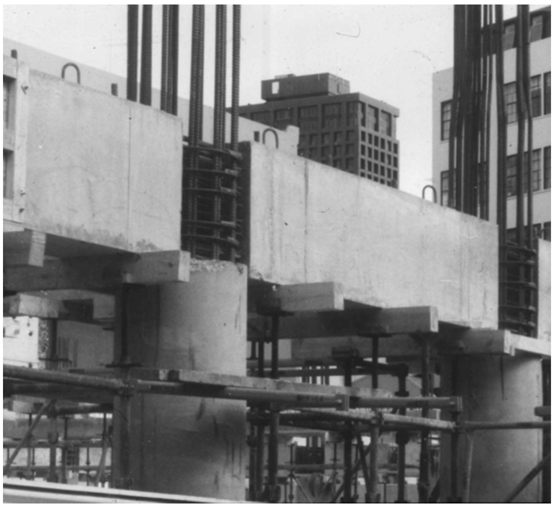
(b) Approach using straight bottom beam bars

Fig. 5-7: Equivalent monolithic system 1

(a) Example from Japan: View of beams before the placement of the floor slab (Courtesy of F. Watanabe)



(b) Example from Japan: View of top of beams after the placement of the precast floors and topping reinforcement (Courtesy of F. Watanabe)



(c) Example from New Zealand (Courtesy of R. Park)

Fig. 5-8: Examples of use of system 1

An alternative to hooking the bars in the joint region commonly used in Japan, is shown in Fig. 5-7 (b). In this alternative, the lower longitudinal beam reinforcement is continuous along the joint. The longitudinal and transverse beam units are seated on the column below. Then the beam-column joint reinforcement is tied to the column longitudinal reinforcement and the top beam longitudinal reinforcement is tied to the stirrups. Cast-in-place concrete is placed in the beam-column joint region and in the upper portion of the beam units. It should be noted that this alternative is of wide use in Japan where it is used to construct two-way frames.

A potential problem with this system is that the critical section of the potential plastic hinge region in the beam occurs at the vertical joint at the column face between the cast-in-place concrete of the joint core and the precast beam. Full-scale laboratory tests in New Zealand, however, demonstrated that no vertical movements occurred between the end of the precast concrete beam and the column if the beam was seated on 30 mm of concrete cover of the column below even if the vertical joint is rather smooth [Beattie, G.J. (1989), Restrepo et al. (1995a)]. This is only if the shear stresses in the plastic hinge do not exceed $0.2\sqrt{f'_c}$ MPa, where f'_c is the concrete cylinder compressive strength in MPa. In New Zealand it is recommended that the surface of the joint should be clean, free of laitance and intentionally roughened to a full amplitude of not less than 5 mm. Alternatively, it is recommended that mechanical keys be built into the beam to transfer shear [Centre for Advanced Engineering, 1999]. In Japan this system is recommended only if the vertical joints incorporate mechanical keys [Architectural Institute of Japan (2000)].

From the practical standpoint this system is sensitive to construction tolerances. It should be borne in mind that beam units that slightly longer than anticipated could restrict the placement of joint hoops as usually very small tolerances are left for this purpose.

2) System 2 – Precast beam units through columns

An arrangement that makes more extensive use of precast concrete, and avoids the placing of cast-in- place concrete in the congested beam-column joint core regions, is shown in Fig. 5-9. The success of this system depends on tighter than normal tolerances. The reinforced concrete columns can be either precast or cast-in-place to occupy the clear height between beams. The precast portions of the beams extend from near midspan to midspan, and hence include within the precast element over the columns the complex arrangement of joint core hoop reinforcement, which is fabricated in the precast factory. The precast portions of the beams are seated on the concrete column below and propped for construction stability.

In this system, the protruding longitudinal bars from the reinforced concrete column below pass through preformed vertical holes preformed in the precast beam element and extend above the top surface of the precast element. The holes in the precast beam elements are preformed using corrugated steel ducting and are grouted with the horizontal interface gap after the column bars have been passed through. The columns bars are spliced above the joint using grouted steel sleeves or by grouting them into corrugated steel ducts embedded in the column above (see Figs. 5-33 and 5.34).

The grouting operation is performed once the gap between the column below and the precast beam unit is sealed around the perimeter. A shrinkage-compensating high-strength and low-viscosity grout is pumped from an inlet port through this gap so it displaces the air progressively. This typically requires vents at the other three corners. The vents are blocked once the air is expelled and the grout begins to flow-out. At this point the grout should begin to flow into the corrugated ducts. Because of loss in the hydraulic head, pumping of grout may be required for those ducts further away from the inlet port. This can be done using a tremie tube or any other method that avoids air-locks.

Protruding bottom bars of the precast beam elements are lapped in a cast-in-place joint at midspan, or are connected by welding or by mechanical couplers (see Figs. 5-24 to 5-32) in the cast-in-place joint. A precast floor system is seated on top of the precast beam elements and spanning between them. Reinforcement is then placed in the top of the beam and topping slab, and the topping concrete is finally placed.

One variation for this system may be to cast the top steel of the beam within the precast beam section. This is particularly suitable for perimeter beams as no edge formwork for the topping is then required. Columns of the next storey are then positioned above the beams using grouted steel sleeves to connect the vertical bars if columns are precast, or using normal reinforced concrete details if columns are cast-in-place.

Fig. 5-10 shows a construction of a 22-storey building in New Zealand using this system. The structure consists of moment-resisting perimeter frames with interior frames carrying mainly gravity loading. Some construction details are shown in Figs. 5-11 and 5-12.

Laboratory testing of this system has been reported by Beattie (1989), Restrepo et al. (1995a) and Lin et al. (2000). The system can be used as an equivalent monolithic for the design of fully ductile moment resisting frames.

3) System 3 – Precast T or cruciform units

An arrangement incorporating T–shaped, cruciform precast concrete elements or even multi-storey cruciform units is shown in Fig. 5-13. In this arrangement the vertical column bars in the precast units are connected using grouted steel sleeves. Cast-in-place connections of the beams for this system are identical to those employed for System 2.

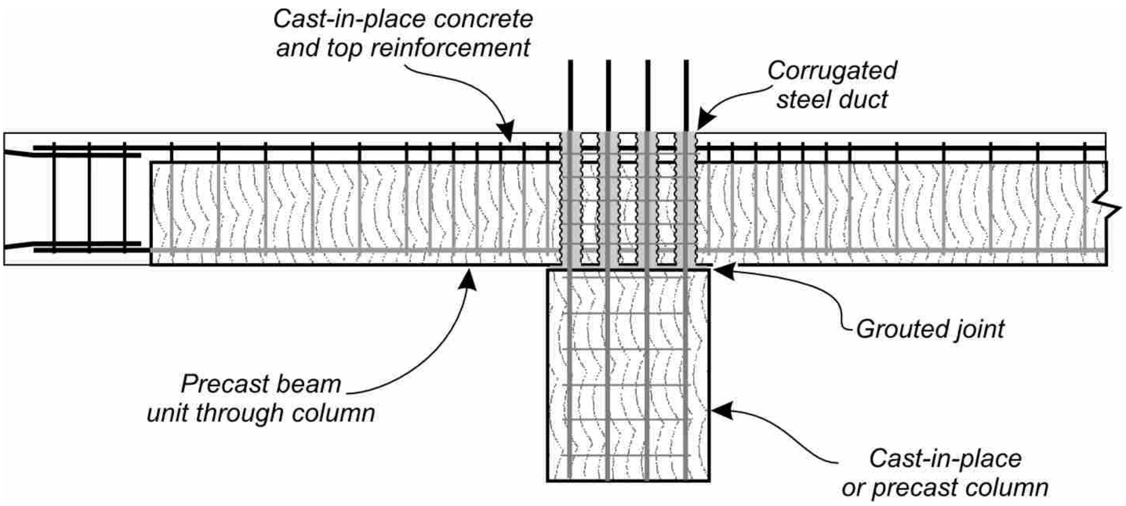


Fig. 5-9: Section through a beam-column assemblage of system 2

Fig. 5-10: Example of building constructed in New Zealand with system 2 (Courtesy of Wilkins & Davis)

Fig. 5-11: Beam corner unit being lowered into place using temporary plastic tubes as guides (Courtesy of J. Restrepo)

Fig. 5-12: Column bars after being grouted in the joint core (Courtesy of J. Restrepo)

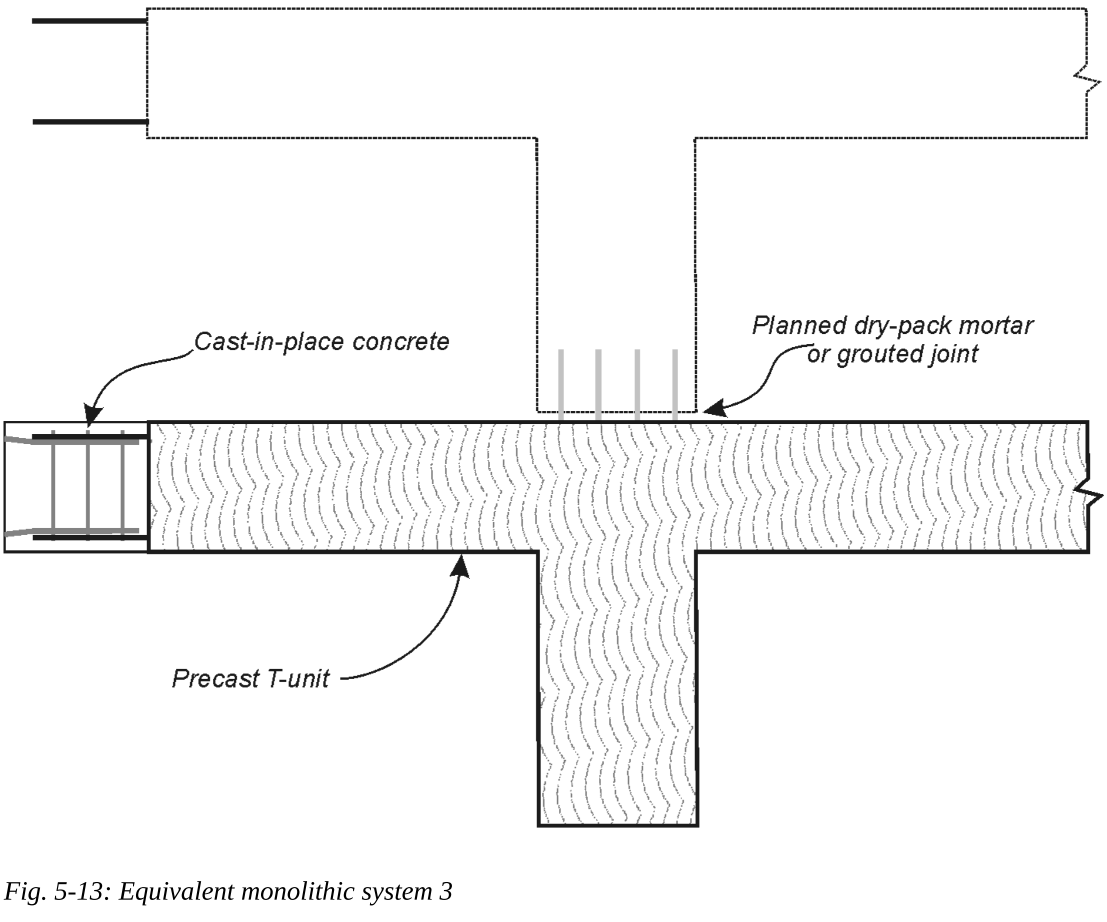


Fig. 5-13: Equivalent monolithic system 3

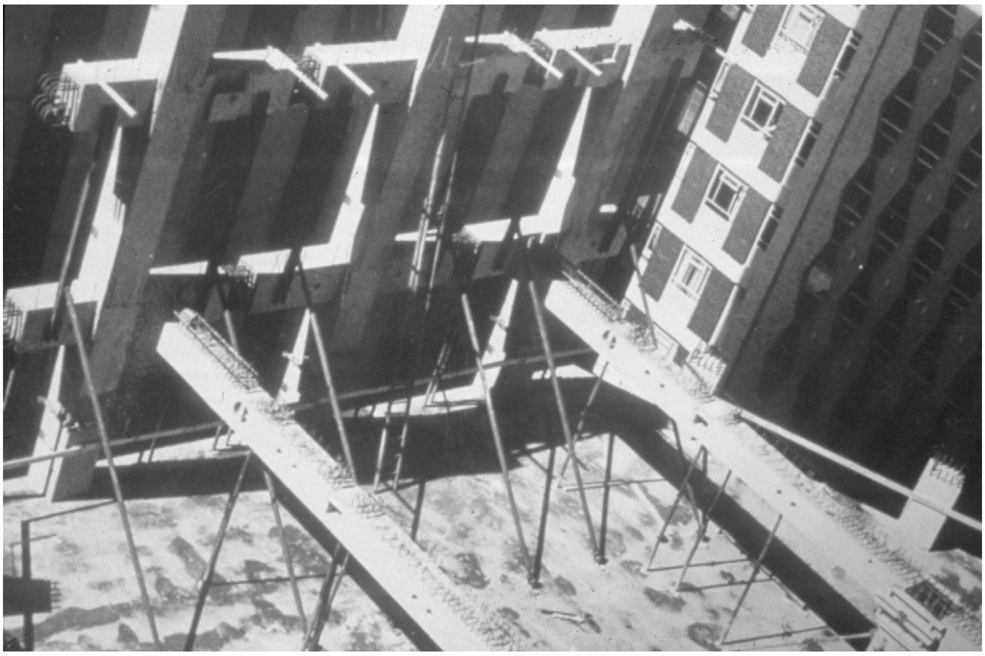


Fig. 5-14: Example of building constructed in New Zealand with system 3 (Courtesy of A O'Leary)

An advantage of System 3 is the extensive use of precast concrete possible and the elimination of the fabrication of complex reinforcing details on the construction site. A possible constraint is that the precast elements are heavy and crane capacity may be an important consideration.

Fig. 5-14 shows a perimeter frame of a building constructed using precast concrete cruciform-shaped units two storeys in height, with grouted steel sleeves connecting column bars at mid-storey height and hooked splices connecting beam bars in cast-in-place concrete joints at midspan.

4) System 4 – Precast shell units

A further system, which is primarily used in New Zealand, uses pretensioned precast concrete beam shell units as permanent formwork for beams. The typical structural organisation of a building floor and moment-resisting frame system incorporating precast pretensioned U-beam units is shown in Fig. 5-15. The precast beam shells are typically pretensioned U-beams and are left permanently in position after the cast-in-place reinforced concrete core has been cast.

The precast U-beams support the self-weight and construction loads and act compositely with the reinforced concrete core when subjected to other loading in the completed structure. Precast U-beams are generally not connected by reinforcement to the cast-in-place concrete of the beam or column. Reliance is normally placed on the bond between the roughened inner surface of the precast U-beam and the cast-in-place concrete core to achieve composite action. Occasionally, protruding stirrups or ties from the U-beams have been used to improve the interface shear strength.This form of composite beam construction has been used in multi-storey moment-resisting framed structures. In this application, the composite beams will be required to develop plastic hinges during major earthquakes. Fig. 5-16 shows a ductile frame under construction using precast U-beams.

Doubts have been expressed by some designers and checking authorities concerning the ability of this form of composite construction to be able to perform as ductile moment-resisting frames. It had been felt that cracking may concentrate in the beam at the column face at the discontinuity caused by the end of the precast U-beam. However, tests have demonstrated [Park and Bull (1986)] that, during severe seismic loading, there is a tendency for the plastic hinging to spread along the cast-in-place reinforced concrete core within the precast U-beam due to breakdown of bond. Hence plastic hinge rotation does not concentrate in the beam at the column face and no undesirable concentration of curvature results. Seismic design recommendations for such construction are available [Park and Bull (1986)].

It is very important to ensure during construction that the inside surface of the shell beams is clean when the cast-in-place concrete is placed, otherwise sufficient bond between the shell and core cannot develop. A site failure of a beam due to lack of bond because of a dirty interface has been observed.

5.2.4.2 Systems for equivalent monolithic construction with connections incorporating bonded post-tensioned tendons

Although energy dissipation in frames incorporating post-tensioned grouted tendons at the critical regions is relatively low, the seismic performance is not necessarily inferior. For example, during the Kobe earthquake in 1995 more than 150 prestressed concrete buildings in the region of strong shaking survived unscathed and only three buildings suffered significant structural damage [Muguruma et al. (1995)]. The response of these buildings clearly showed the main advantages of post-tensioning: deflection recovery and immediate occupancy.

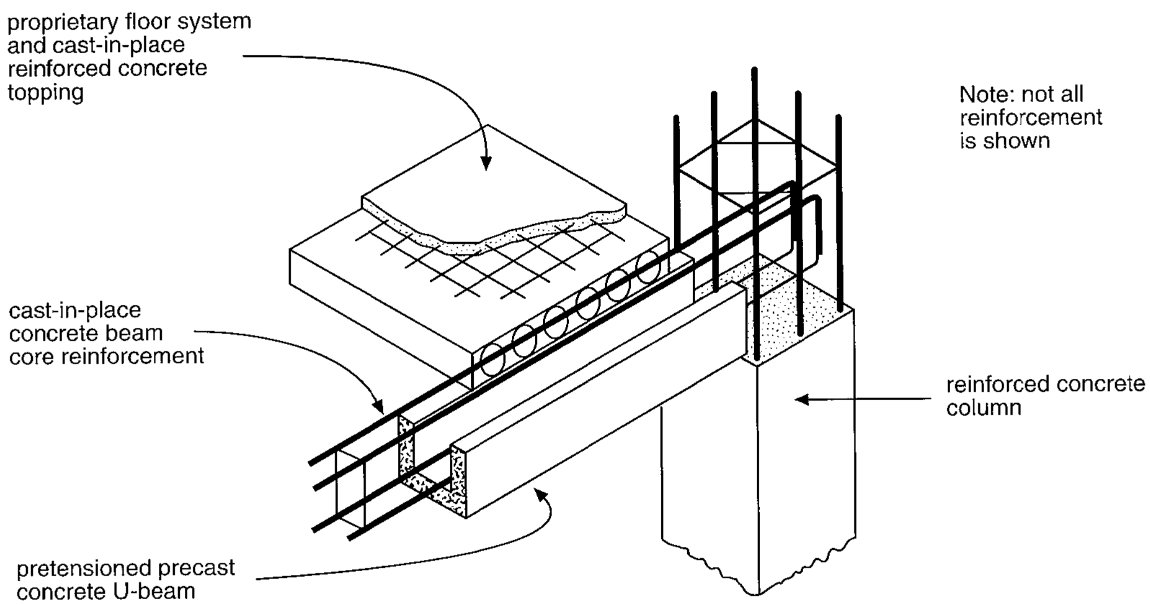


Fig. 5-15: Equivalent monolithic system 4 (Courtesy of D. Bull)

Fig. 5-16: Example of building constructed in New Zealand with system 4 (Courtesy of J. Restrepo)

Equivalent monolithic construction in which the critical connections incorporate bonded post-tensioned tendons is widely used in Japan and have been used in New Zealand in the 1960s and 1970s. There are two main systems in use. These systems are described below.

1) System 1 – Precast beam units post-tensioned between columns

The arrangement shown in Fig. 5-17 makes use of precast concrete beams and columns. Multi-storey high columns can be used with this system. The beams, which are precast with a length slightly less than the clear span between columns, are temporary seated on steel brackets or propped. Then, the gap between the beam-ends and the column faces is grouted followed by the post-tensioning operation and grouting of the tendons. In this system, plastic hinges develop at the beam-column connections and energy dissipation takes place mainly as a result of yielding of the tendons. The hinges spread into the joint and into the beam towards midspan as a result of the loss of bond between the tendon and the surrounding grout. It should be noted that the presence of non-prestressed reinforcement in the precast beam units could slightly constrict the plastic hinges from spreading as much as in monolithic construction. Tests, however, have demonstrated that the behaviour of this system is comparable to equivalent monolithic prestressed concrete construction and that the system can be utilised for the design of fully ductile moment resisting frames [Blakeley and Park (1971)]. An application of this system in Japan and New Zealand are illustrated in Figs. 5-18 and 5-19. In New Zealand it is recommended that the post-tensioned tendons in the beam be anchored in an end block outside the column (see Fig. 5-19) so as to remove the bursting stresses from the anchorage away from the beam-column joint, which is expected to be severely stressed as a result of joint shear.

2) System 2 – Supported precast beam units post-tensioned between columns

In this system the beams are seated on corbels that transfer gravity loads to the adjacent columns. The beams can be built prismatic or with dapped-ends, depending on the position of the corbel, see Fig. 5-20 and then post-tensioned. The columns in this system in Japan are normally one-storey in height. Columns are connected through a mechanical coupler or grouted steel sleeve.

This system is suitable for buildings designed for nominally elastic response. This is because the kinematics of the mechanism in this system is complex due to the presence of the corbel. In particular, large shear forces are expected to develop in the beams when loaded beyond the elastic limit in the downward direction.

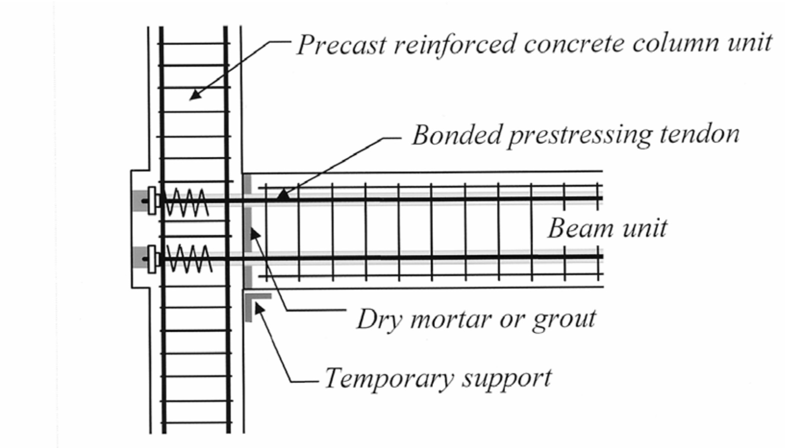


Fig. 5-17: General reinforcing details of equivalent monolithic post-tensioned system 1

Fig. 5-18: Example of building constructed in Japan with post-tensioned system 1 (Courtesy of F. Watanabe)

5.2.4.3 Jointed systems

The advantages of jointed systems utilizing unbonded post-tensioned tendons for lateral force resistance were discussed in Section 5.1. Experimental work on jointed frame systems has been reported by Cheok et al. (1998), El-Sheikh et al. (1999), Macrae and Priestley (1994), Palmieri et al. (1996), Priestley et al. (1999), Sritharhan (2002), Stanton et al. (1997) and Stone et al. (1995).

Of the jointed systems developed, two have received special attention because of their excellent seismic performance during the test on a jointed precast five-storey building at 60 percent scale in United States [Nakaki et al. (1999), Priestley et al. (1999), Sritharhan (2002)]:

1) Hybrid system – Beam units post-tensioned between columns

The hybrid system was developed in United States at the National Institute of Standards and Technology [Stone et al. (1995), Stanton et al. (1997)]. In this system, the beams are connected to multistorey columns by unbonded post-tensioning tendons that run through a PVC duct in the center of the beam and through the columns, see Fig. 5-21. Before jacking the tendons, the gaps between the beam units and the column faces are grouted with fibre-reinforced grout.

In this system the prestressing tendons are initially stressed to a force level of typically less than $0.5f_{pu}$, where f_{pu} is the ultimate tensile strength of the prestressing steel. This lower level of stressing is typically determined to ensure the tendon will not reach the limit of proportionality before the connection rotation attains 0.04 radians.

Mild steel reinforcing bars which are placed into corrugated ducts at the top and bottom of the beam, through the column, and then grouted with a flowable shrinkage-compensating grout. The purpose of these bars is to provide energy dissipation by yielding in tension and compression over debonded regions next to the beam-to-column connection. The amount of mild steel reinforcement and post-tensioning steel are balanced so that the frame self-centers after a major seismic event. The concrete at the beams ends in this system need to be protected from premature crushing due to the large rotations at the beam ends. This is usually achieved by armouring the beam ends (see Fig. 5-21) or by adequately confining the concrete with spiral reinforcement. The mild steel bars are placed in the precast beam units prior to erection. Typically, the entire length of the bars is placed inside over-long corrugated steel ducts. Upon erection, the bars are slid pass through the joint and into the opposite beam prior to grouting, see Fig. 5-21.

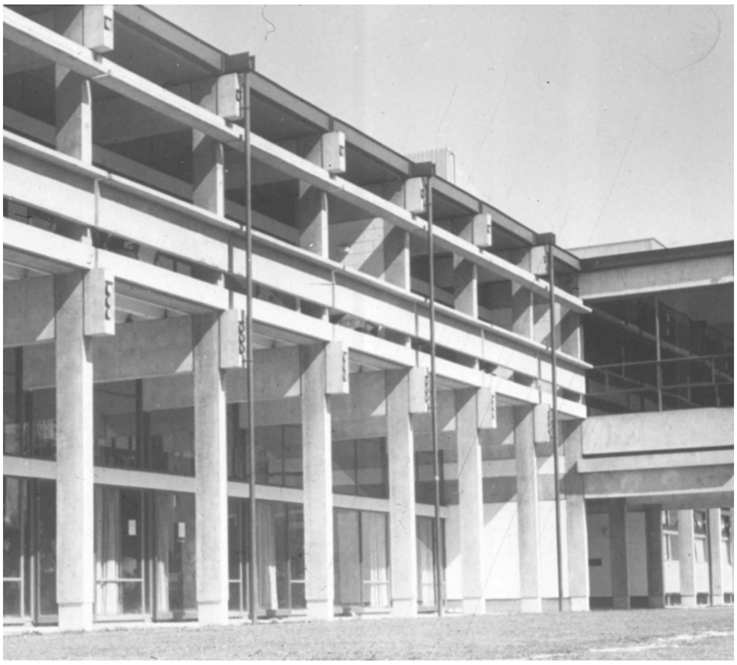


Fig. 5-19: Example of building constructed in New Zealand with post-tensioned system 1 (Courtesy of R. Park)

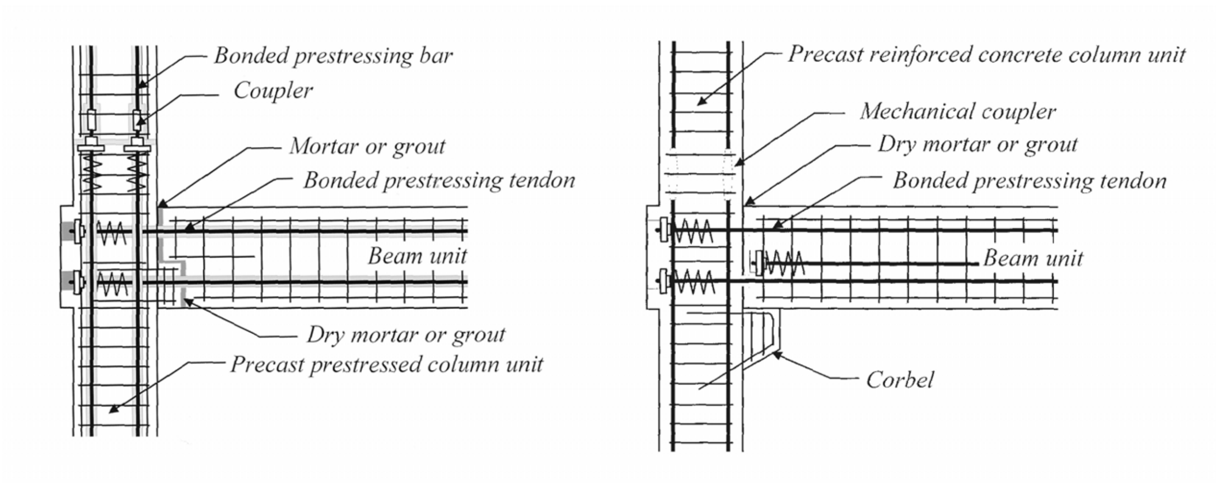


Fig. 5-20: General reinforcing details of equivalent monolithic post-tensioned system 2

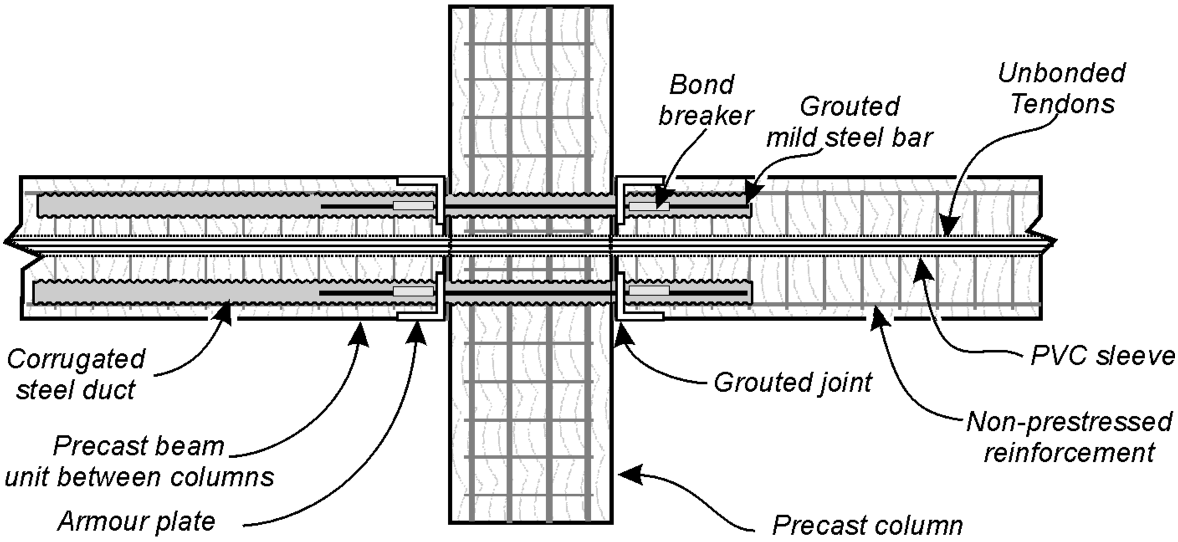


Fig. 5-21: Section through a beam-column assemblage of a hybrid frame system

Fig. 5-22: Example of a 39 storey building constructed in United States using a hybrid frame system (courtesy of Kwan Henmi, Architecture and Planning)

Few buildings have now being built with hybrid frames. Fig. 5-22 shows an example of a high-rise multistory building constructed in United States with this system [Englekirk (2002)].

2) Rocking system – Beam units through columns

This rocking system was initially proposed by Priestley and Tao (1993) and tested by MacRae and Priestley (1994) using post-tensioned beams. The system shown in Fig. 5-23 consists of multi-span beams that are cast in a conventional pretensioned casting beds, with specified lengths of the pretensioning tendons debonded. The beam-to-column connection in this system is identical to that described for System 2 in Section 5.2.4.1. The initial level of prestressing applied to this system is similar to the hybrid frame system.

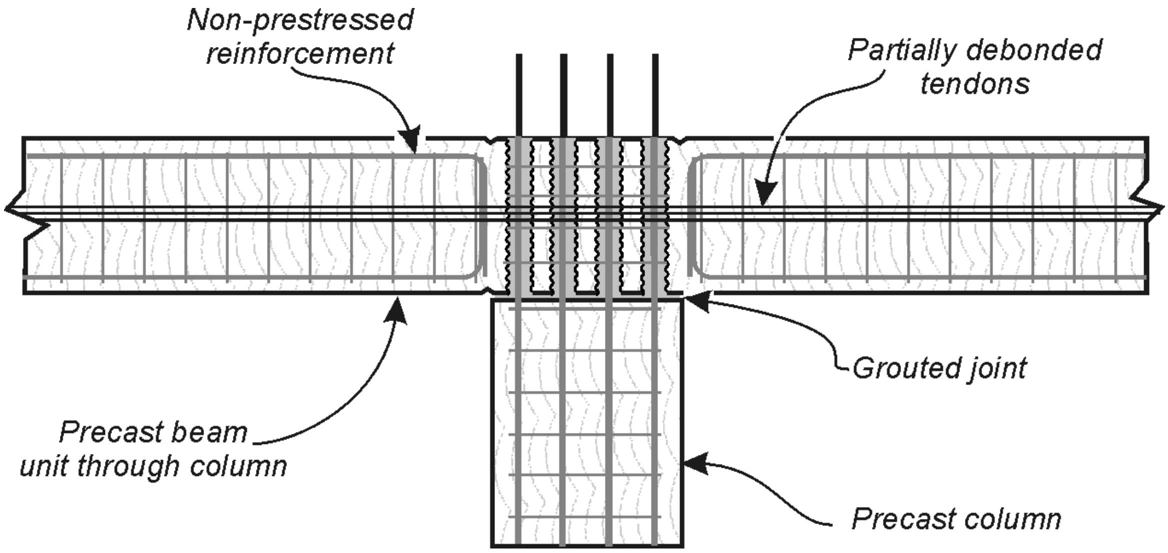


Fig. 5-23: General reinforcing details of a rocking frame system

5.2.4.4 Strong connections for moment resisting frames

1) Connections between beams

In Systems 2 and 3 of Section 5.2.4.1 the protruding bars from the precast element are lapped in a connection near or at midspan which is then cast with concrete. The connection needs to be located away from the potential plastic hinge regions. In New Zealand such beam-to-beam connections can be take place at a distance d from the column faces [Standards New Zealand (1995)], where d is the effective beam depth. Similarly, in System 1 of Section 5.2.4.1 the beam-to-beam connection is performed at or near midspan when using the alternative of straight beam bars through the column as in Japan.

In these connections it is recommended that the surface of the precast beam ends be roughened, clean and be free of laitance. Alternatively, the units can be cast with mechanical keys to improve shear transfer. Restrepo et al. (1995b) provides design information on a variety of beam-to-beam connections.

Figs. 5-24 to 5-32 depict a several approaches for connecting beams together. Non-contact straight bar lap splices are used to connect beams with short aspect ratios, see Fig. 5-24. The longitudinal reinforcement at the connection in the mating beams is offset to minimize a potential clash that might occur during erection.

When the span between columns does not constrain the beam connections to within a small length, a double-straight bar lap splice can provide a simple way of connecting the precast beam units together. This connection detail is illustrated in Figs. 5-25 and 5-26.

An alternative approach to using non-contact straight bar lap splices is to use “drop-in” double hooked bars as shown in Figs. 5-27 and 5-28. This approach greatly simplifies the erection process. This connection approach is suitable for connecting beams of perimeter frames at near midspan near where the point of inflection is expected to be during peak earthquake loading. To prevent splitting of the concrete due to the large radial forces that develop around the hooks, transverse rods of the same diameter than the hooked bars anchored are placed in contact with the hooked bar as Fig. 5-27 shows.

A beam-to-beam connection can also be achieved by welding the longitudinal beam bars protruding from the mating beam units. Welding of the bars can be achieved only under strict construction tolerances and site welding quality assurance procedures. Figs. 5-29 and 5-30 show general details of this approach and an application of a welded connection, respectively.

A further connection approach uses mechanical couplers to connect the longitudinal threaded beam bars, see Fig. 5-31. Strict construction tolerances should be required here also. An example of application of this approach is depicted in Fig. 5-32. An alternative approach, for use with standard deformed bars is to splice the bars using proprietary grouted steel sleeves. It should be noted that disturbance of the grouting material during the setting process is detrimental.

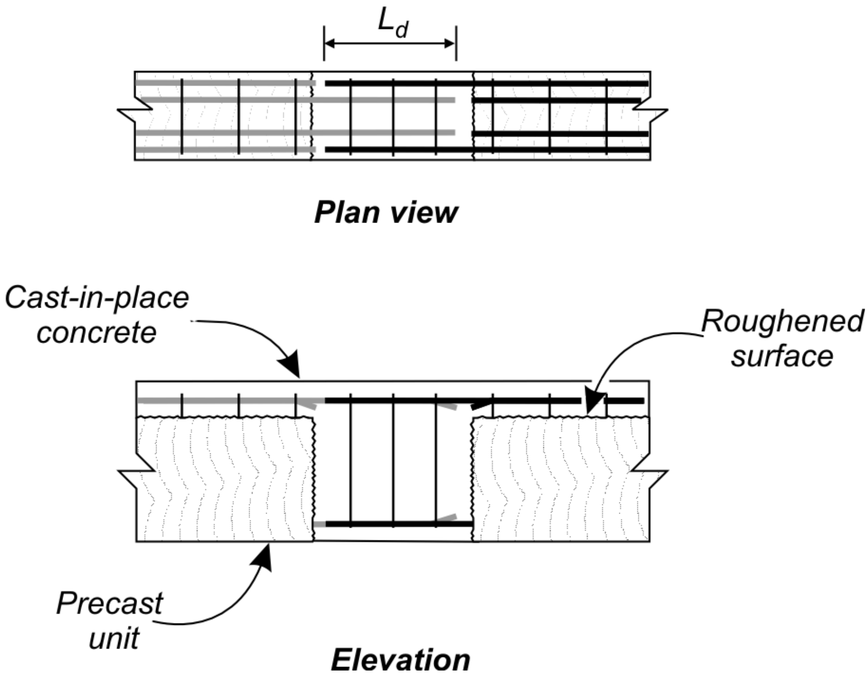


Fig. 5-24: Beam-to-beam connection using non-contact straight bar laps

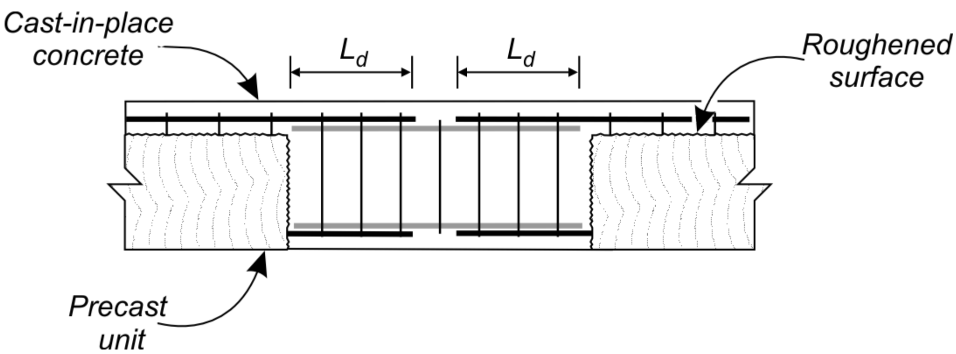


Fig. 5-25: Beam-to-beam connection using double-straight bar laps

Fig. 5-26: Example of a connection with double-straight bar laps in a building in New Zealand (Courtesy of J. Restrepo)

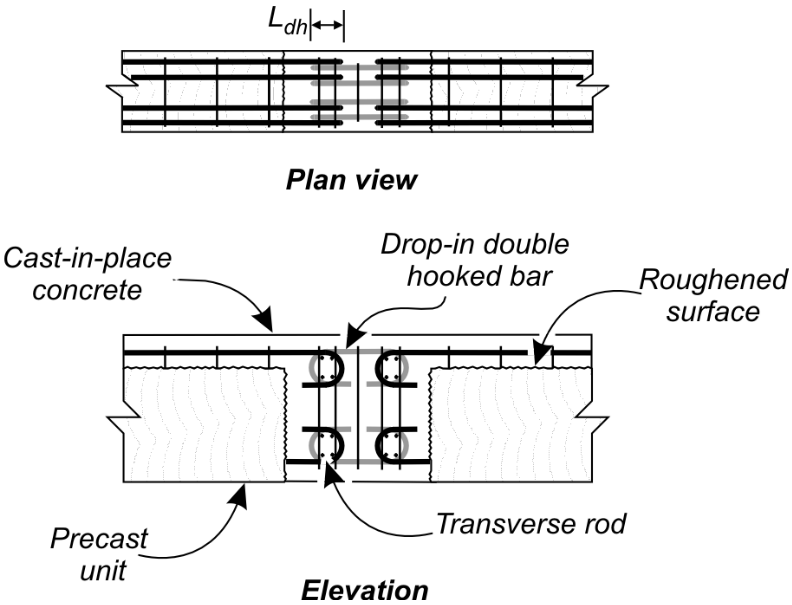


Fig. 5-27: Beam-to-beam connection using drop-in double hooked bars

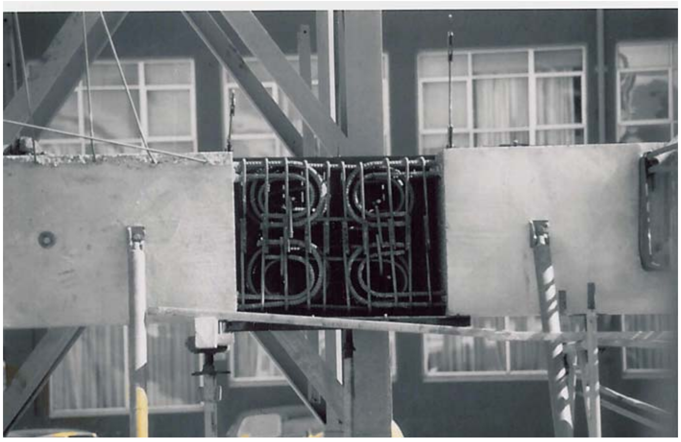


Fig. 5-28: Example of a connection with drop-in-double hooked bar in a building in New Zealand (Courtesy of J. Restrepo)

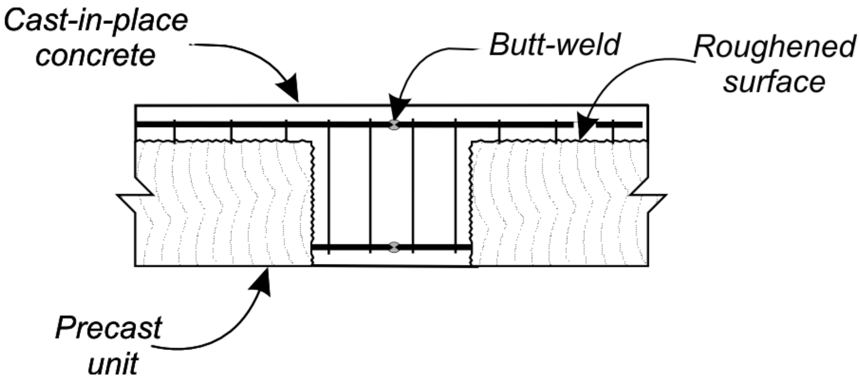


Fig. 5-29: Beam-to-beam connection using welded bars

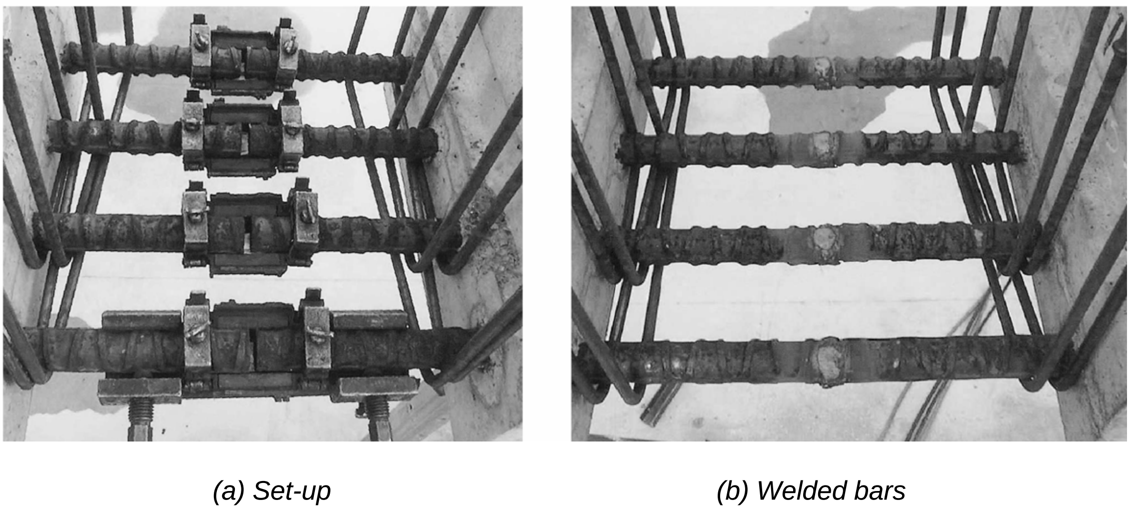


Fig. 5-30: Example of a welded connection in a building in Japan (Courtesy of F. Watanabe)

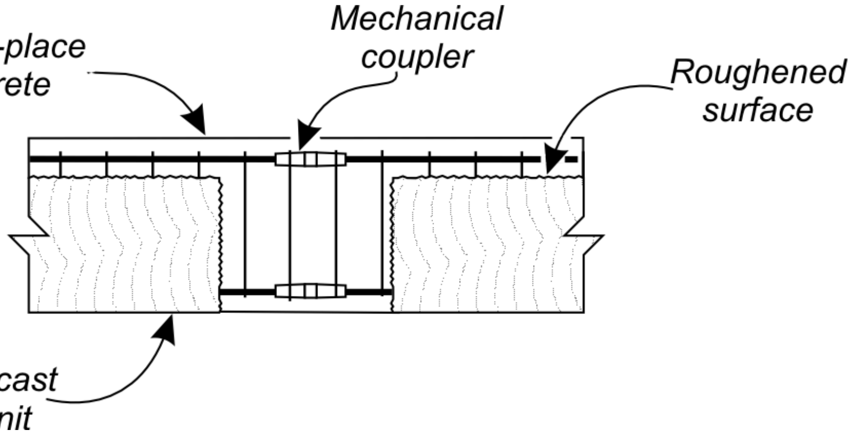


Fig. 5-31: Beam-to-beam connection through mechanical couplers

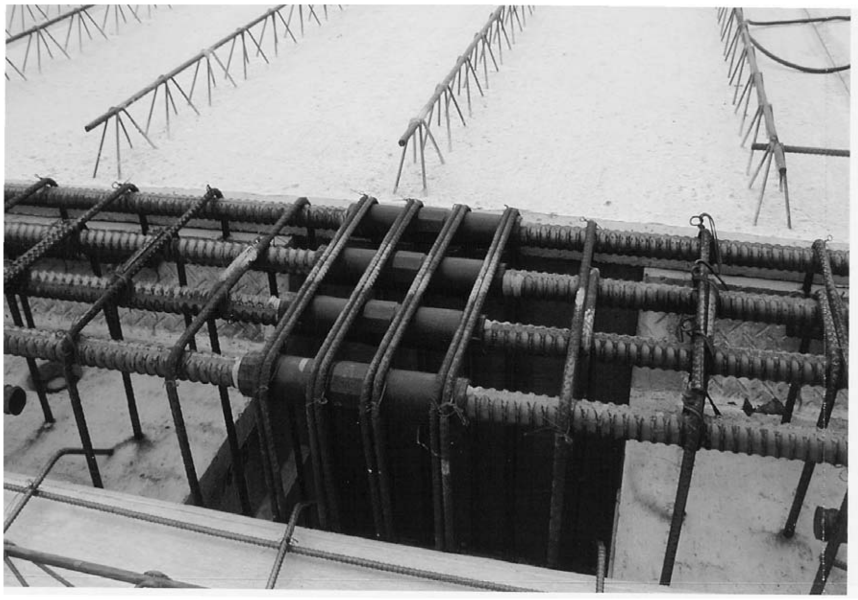


Fig. 5-32: Example of a connection through mechanical couplers in a building in Japan (Courtesy of F. Watanabe)

2) Connections between columns

Precast concrete columns are typically spliced either at the bottom end above a beam or at mid-height between floors. Splices at the column ends should only be performed when the development of a plastic hinge there is explicitly precluded in design.

There are two main approaches for connecting precast columns. In both approaches a gap is left between the jointing surfaces. The surfaces must be roughened, clean and free from laitance.

The first approach makes use of proprietary grouted steel sleeves. In this approach dry-pack bedding mortar is placed in plastic state so that the column being positioned squeezes some mortar out before seating on plastic or steel shims. At this point the splicing bars are aligned inside the steel sleeve and ready for grouting. Care must be taken before the grouting operation to ensure the sleeve is free from debris. Note that the use of steel sleeves requires the oversizing of the transverse reinforcement in the column end region. This implies that, to maintain the minimum concrete cover requirements specified by the design codes, column bars should be placed more towards the center of the column than when using cast-in-place construction. Figs. 5-33 and 5-34 show general details of this method.

Another approach consists in connecting the column longitudinal bars through non-contact bar splices [Lin et al. (2000)]. This approach is depicted in Fig. 5-35. The longitudinal bars protruding from the precast column, Bars "G", are grouted into corrugated steel ducts in the mating column. Adjacent to the ducts there are two small diameter bars, identified as Bars "L" each having at least one-half of the cross section area of the grouted bar. Bar "G" are lap-spliced a distance not less than the development length of the Bars "L". In this approach the gap between the jointing surfaces is grouted at the same time as the ducts. The grouting operation follows the same procedure described for System 2 in Section 5.2.4.1.

5.2.4.5 Pinned joints

Pinned joints in moment resisting frames may appear to be irrelevant but there are situations these joints are required to connect in beams to beams or to columns to either control the response, or for construction reasons. Response control using pinned joints or moment limiting joints is useful where stiffness needs to be reduced in particular bays or storeys of a frame. Excessive stiffness resulting from short or stiff beams, or short columns are often not desirable as they attract too much load, or contribute to soft storey effects above or below a stiff storey. Also, secondary beams can be connected to main beams by pinned joints. Examples of reinforcing detailing of pinned joints between secondary beams and main beams are shown in Figs. 5-36 and 5-37.

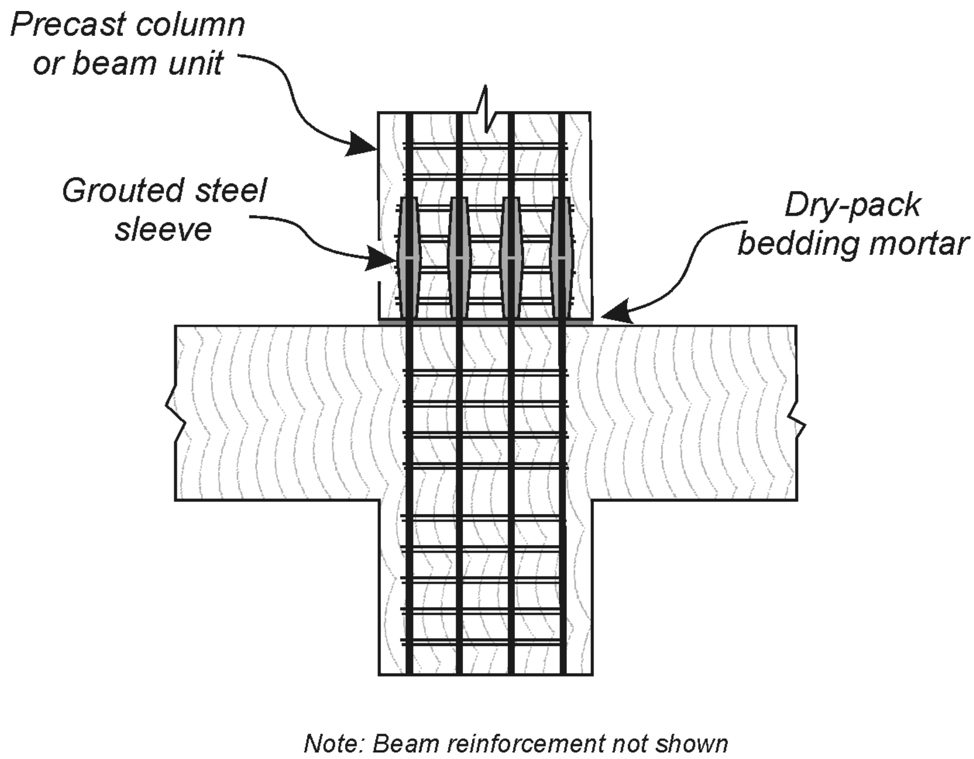


Fig. 5-33: General details of column-to-column connection through grouted steel sleeves

Fig. 5-34: Example of column-to-column connection through grouted steel sleeves in a building in New Zealand (Courtesy of A. O'Leary)

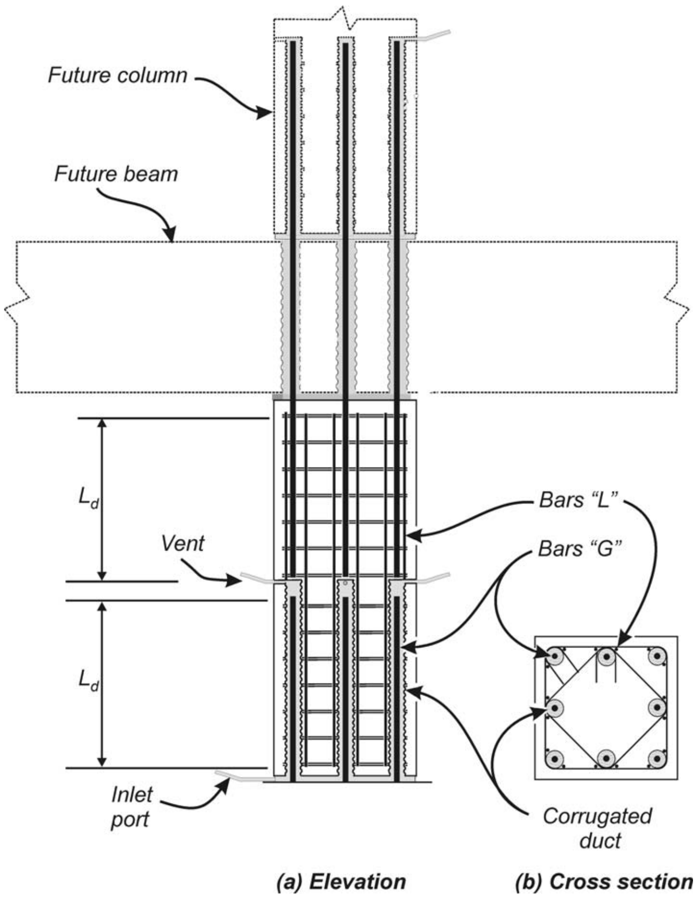
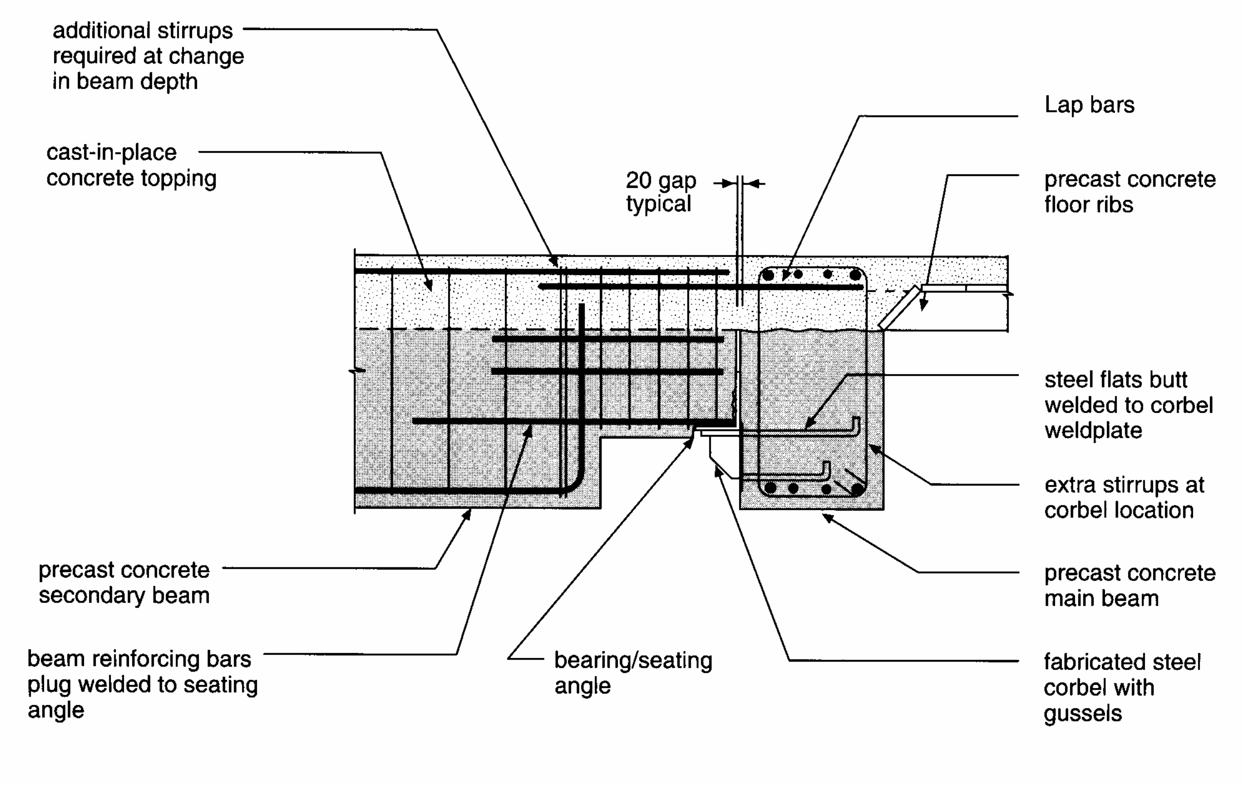


Fig. 5-35: Column-to-column connection through non-contact lap splices

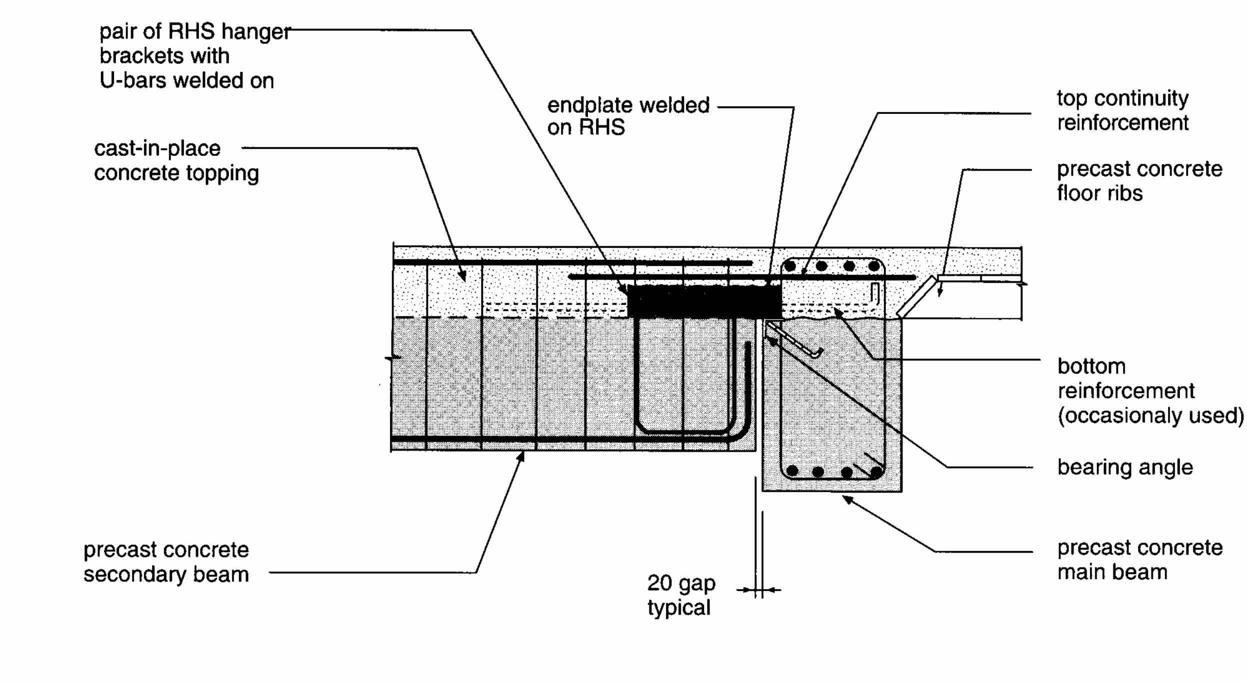
Moment limiting joints can consist of areas of reduced effective depth achieved by artificial discontinuity of the compression zone. A type of pinned joint in columns is the "concrete hinge", formed of diagonally crossed longitudinal bars which result in reduced flexural strength in the region where the bars cross. This type of pin will be an integral part of a precast element and is unlikely to be part of a construction connection.

Three main issues need to be addressed in the design of elements incorporating pinned or moment limiting joints:

1) Shear transfer: diagonal or crossed reinforcing details or details incorporating structural steel elements embedded in the concrete beam can be detailed to transfer shear force.
2) Kinematics: the geometry of the joints need to be properly establish to ensure that the shear force in pinned joints and the moment and shear force in moment limiting joints does not increase due to kinematical incompatibility resulting from side-swaying of the primary lateral force resisting system. For example, corbel type joints perform satisfactorily under gravity loading and when subjected to small rotations but the corbel and the beam can be severely damaged when subjected to large rotations.
3) Fatigue: moment limited beam joints can be designed for precast elements and are sometimes used where a reduced moment capacity is required to "protect" some other part of the system. In detailing such joints particular attention needs to be paid to fatigue because such joints often yield at the serviceability limit state.



(a) Using Steel Corbel Angle Bracket



(b) Using RHS Hanger Brackets

Fig. 5-36: Examples of pinned joints at secondary beam to main beam connections [Centre for Advanced Engineering (1999)]

Fig. 5-37: Example of the end reinforcement of a precast prestressed secondary beam with a pinned joint in a construction site in New Zealand (Courtesy of J. Restrepo)

5.2.4.6 Industrial buildings

Totally precast concrete frames have proved to be suitable for the construction of single storey industrial buildings with large degree of repetition, for enclosing processes giving off corrosive vapour (for example, pulp and paper processing).

Typically, the frame consists of precast pretensioned concrete members. The transverse roof beams are simply supported at their ends on cantilever columns. Short span roof slabs can span between the transverse of beams. The cantilever columns are socketed in the foundation to provide moment resistance. In these types of buildings, the roof should be properly braced to enable the transfer of inertia forces to the columns and to minimize relative lateral displacements between the cantilever columns.

More elaborate precast concrete industrial buildings are constructed in some countries such as Italy where the transverse beams and slabs are replaced by a large variety of long roofing elements, for example 2.5 m wide and 20 m long with thin cross sections from simple T, TT or Y sections up to elaborate curved shapes. Such roofs lack in-place rigidity and cannot develop full diaphragm action. Sometimes these structures are braced longitudinally by walls or diagonal bars.

5.3 Structural wall and dual systems

5.3.1 Member partitioning

Limitations in crane capacity often require structural walls to be partitioned. Walls are usually partitioned horizontally, vertically or in both directions. The general aim when partitioning walls is to ensure that the horizontal connection between the wall segments does not influence the overall wall response by adding flexibility and/or by reducing the capacity. That is, horizontal connections are deliberately made strong with good details to ensure that relative movement between the wall panels is minimized. Vertical connections, on the other hand, can be designed to provide flexibility and to dissipate energy. In some cases vertical connections are deliberately made overstrong to ensure monolithic behaviour. Two examples of wall partitioning are shown in Fig. 5-38.

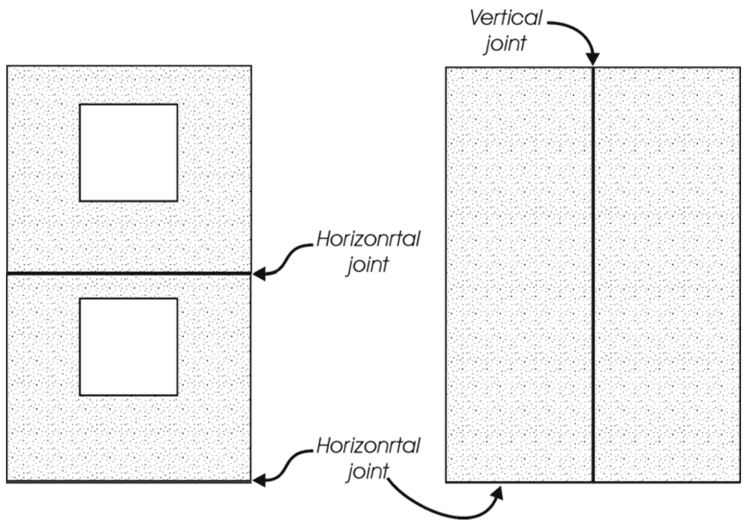


Fig. 5-38: Examples of wall partitioning

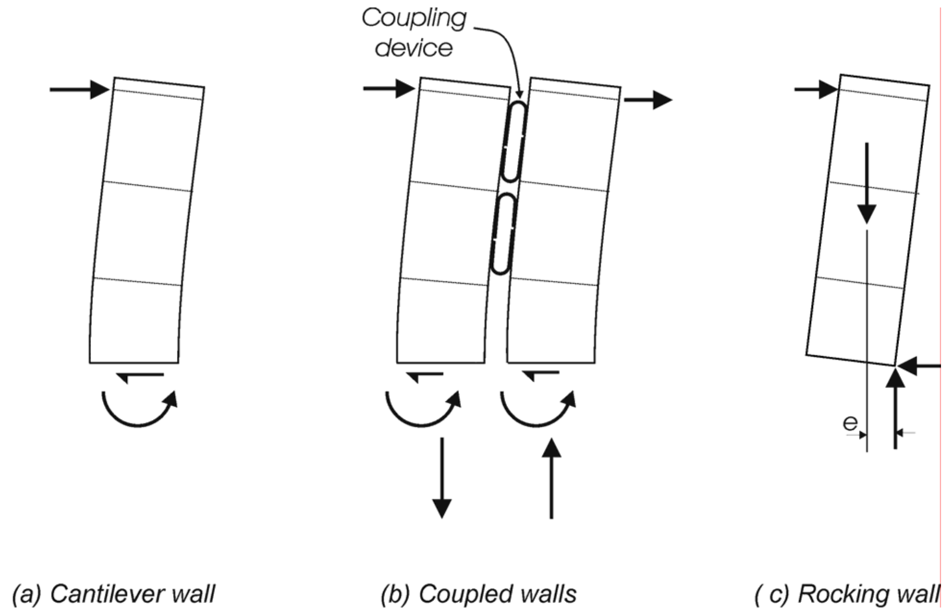


Fig. 5-39: Classification of precast concrete wall systems according to the in-plane lateral force resisting mechanisms

5.3.2 Classification

Precast concrete structural walls are generally arranged to provide in-plane lateral force resistance primarily through three ways. They are by cantilevering from the foundation structure, through coupling with beams or other special devices, and by rocking about their foundation. Such possible ways are illustrated in Fig. 5-39. Cantilever walls resist the overturning moment resulting from the lateral forces by bending. Coupled walls resist the overturning moment not only by bending of the individual walls but also through an axial force couple. Rocking walls resist overturning moment at the base of the walls through the couple arising from the eccentricity between the acting gravity load and the reaction at the wall-foundation interface. In some cases, rocking walls may be prestressed with unbonded tendons to increase the overturning moment capacity.

Fig. 5-40 shows a classification of precast concrete wall systems according to their response and source of non-linear behavior. Structural wall systems can be classified as linearly elastic when the stresses in the reinforcing steel remain below the elastic limit and the compressive strains in the concrete remain below 0.002 in each wall in the system. Note that truly linear-elastic behavior does not occur in most precast reinforced concrete wall systems due to the development of cracking in the wall panels. Cracking of the concrete results in some stiffness degradation. Nevertheless, individual walls in a system can be idealized as having linear-elastic response characteristics by accounting for the stiffness degradation caused by cracking.

Structural wall systems showing strong non-linear response characteristics can be grouped into either equivalent monolithic or jointed systems:

1) Equivalent monolithic systems

These systems are designed to closely emulate the response of conventional cast-in-place construction in terms of stiffness, strength, ductility and energy dissipation characteristics. Walls in this category are generally designed so that the non-linear response results from the development of flexural plastic hinges in selected regions, usually at or near the wall bases. Such regions are detailed for ductility for the expected ductility demand. The system ductility ranges from incipient or nominally elastic to fully ductile. The hysteretic response of equivalent monolithic wall systems is characterized by large hysteresis loops, with equivalent viscous damping ratios of up to 28 percent [Holden et al. (2003)]. A major advantage of these systems is that the design can generally follow the recommendations given in standards for the design of cast-in-place concrete wall systems.

2) Jointed systems

These systems are designed so that the non-linear response takes place at one or several pre-selected connection interfaces following a pre-determined mechanism of non-linear deformation. Three jointed systems are described below.

The first jointed system comprises walls designed for limited ductility response. In one limited ductility jointed wall design the connection between the precast reinforced concrete wall panels is such that planes of significantly reduced stiffness and strength exist at the interface between adjacent precast concrete wall panels. Such construction has been extensively used in New Zealand in tilt-up construction generally of one to three storey apartment, office and industrial buildings. Generally tilt-up wall panels are secured to the adjacent elements using jointed connections comprising various combinations of concrete inserts, bolted or welded steel plates or angle brackets, and lapped reinforcement splices within cast-in-place joining strips. Tilt-up construction is generally designed for elastic or limited ductile response. In another limited ductility jointed wall design the reinforcing bars which pass through the wall-to-foundation connection are designed to yield and to provide energy dissipation. A major design consideration in this system is the effect that sliding shear can have on the response. Sliding shear leads to pinching and can result in large permanent lateral deformations. Sliding shear is minimized in this system when the gravity load exceeds the ultimate tensile force of the reinforcing bars passing through the connection. In this particular case a self-centering response, similar to the response of hybrid wall systems described below, is obtained.

The second jointed system comprises hybrid walls. In hybrid walls an energy dissipation device (e.g. mild-steel reinforcement, u-plate coupling devices, etc.) and unbonded prestressing tendons are combined to obtain a self-centering mechanism that eliminates residual displacements following an earthquake. This results in a centered-oriented hysteretic response, with equivalent viscous damping ratios of up to 20 percent [Restrepo (2002)].

The third jointed system comprises rocking walls. In rocking walls the non-linear response results from the opening of a gap at the wall-foundation interface. The response of rocking walls is essentially non-linear elastic. These systems also have the self-centering characteristics of hybrid systems but lack the energy dissipation capacity.

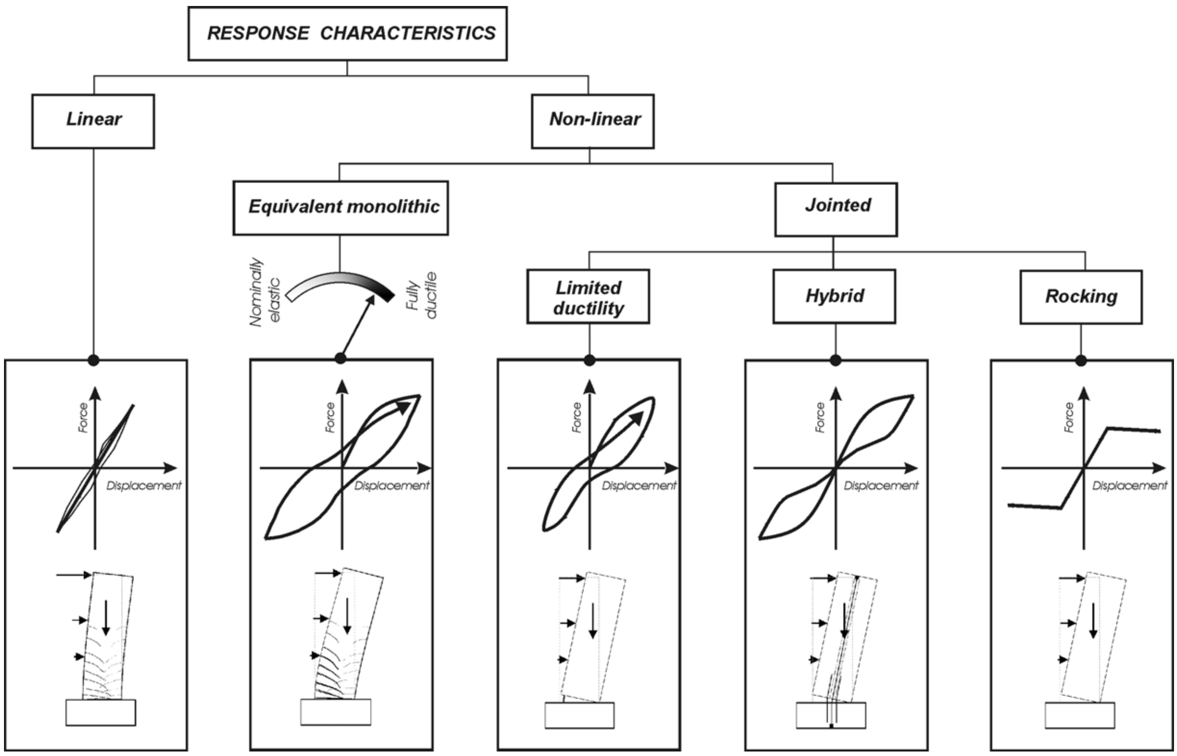


Fig. 5-40: Classification of precast concrete wall systems according to the lateral force – lateral displacement response characteristics

5.3.3 General seismic design approaches for buildings incorporating structural wall systems

5.3.3.1 Conceptual design

Chapter 4 examined different approaches for the seismic design of buildings incorporating precast concrete elements. In common with other structural lateral force resisting systems, structural wall systems can be designed for high or relatively low seismic forces depending on the response characteristics of the system chosen in design or the prescriptive requirements of the applicable building code.

Relatively high seismic design forces are required to ensure linear-elastic response. Walls in a system designed for linear-elastic response do not require special reinforcement detailing. Linear-elastic response is usually selected if one of the following situations arises during the conceptual design stage:

1) The system's dependable base shear capacity exceeds that required by the building code to ensure elastic response.
2) Highly irregular building configurations where a mechanism of non-linear deformation involving flexural plastic hinges or concentrated joint rotations cannot be postulated. This is commonly found in buildings incorporating a number of pierced walls and in buildings founded in highly sloping terrain. The response beyond the elastic range of such buildings is highly uncertain and, as a result, a design for elastic response could ensure a more predictable structural performance.
3) The design overturning moment due to wind forces is greater than that from the seismic forces required for elastic response. This is often the case of tall buildings located in regions of medium and low seismicity and high wind exposure.
4) Use of precast concrete systems possessing very limited ductility capacity. Some precast concrete systems show inadequate response when loaded beyond the elastic range. The use of such systems should be limited to designs for elastic response.

In regions of moderate seismic risk, design forces that have been reduced to reflect some degree of inelastic behavior may be used even when special detailing is not prescribed. This is the practice in the United States where the building code provisions and the quality control requirements ensure well-constructed buildings with structural integrity features in all regions. These design forces are higher than those prescribed for systems with special detailing. For most practical designs of structural walls in low and moderate seismic regions in United States, a primary consideration is resistance to overturning. Wall configurations and connections are chosen to take advantage of the buildings dead load in resisting overturning moment. The dead load may not be directly imposed on the wall, but its resisting capacity can be mobilized by the appropriate choice, design and construction sequence of connections.

The non-linear response of structural systems can be advantageous in many design situations. Economy can result from systems that respond in a non-linear manner as lower seismic forces and overturning moments are required in design. These systems are more appropriate when several of following situations are found during the conceptual design stage:

1) The building has a relatively low density of structural walls. The reduction in the design seismic forces and overturning moment can result in economical design, particularly, through cost reduction in the foundation structure.
2) The design overturning moment determined from the reduced design seismic forces is greater than that due to wind forces.
3) The precast concrete system emulates the behaviour of cast-in place concrete construction and has adequate ductility capacity.
4) The precast concrete system chosen is jointed and has adequate non-linear deformation capacity.

5.3.3.2 Mechanisms of non-linear lateral deformation

The design of buildings incorporating systems with non-linear deformation response characteristics generally requires that a kinematically admissible spatial mechanism of non-linear deformation be established during the conceptual design stage. Fig. 5-41 shows several mechanisms of non-linear deformation for precast structural wall systems. In cantilever wall systems designed to emulate the behavior of conventional cast-in-place concrete, the preferred mechanism involves the development of plastic hinges. In buildings with a regular height distribution, the best location for the development of these plastic hinges is at the base of the walls, see Fig. 5-41 (a).

In buildings with transfer diaphragms, the best location for the plastic hinges is in those walls carrying lateral forces above the transfer diaphragm, see Fig. 5-41 (b). Plastic hinges that develop at the level of the transfer diaphragm tend to spread up and downwards. When the analysis shows that potentially high shear stresses could develop in the walls below the transfer diaphragm, the spread of the plastic hinge to this region should be precluded to avoid a shear dominated non-linear response. A way to preclude the migration of the plastic hinge downwards is through the deliberate increase of the flexural strength in the wall below the transfer diaphragm.

Figs. 5-41 (c) and (d) show suitable mechanisms of non-linear deformation for hybrid cantilever and coupled wall systems, respectively. The best source of non-linear deformations in hybrid cantilever walls is at the base of the walls, where specifically designed energy dissipation devices are cast with the foundation and properly anchored through a grouted connection in the wall panels. The sources of non-linear deformations in coupled hybrid walls are at the base of the walls and at the coupling links. In coupled hybrid walls energy dissipation takes place primarily at the coupling links.

Fig. 5-41 (e) shows the preferred mechanism of non-linear deformation in pierced walls. Piercing of walls that form part of the lateral force resisting system is often required for architectural reasons. The potential for a brittle shear failure in the piers next to the piercings should be considered in design and deliberately precluded. Specific recognition should be given in the design of pierced walls to the increase in shear force demand caused by overstrength and by the higher modes of dynamic response.

5.3.3.3 Kinematical considerations

The presence of a slab can result in unintended coupling of walls, particularly when these walls are built next to each other and only a small gap separates them, see Figs. 5-42 (a) and 5-42 (b). Such coupling can result in the development of forces greater than those anticipated during design and can result in unintended damage to the slab, see Fig. 5-42 (c) [Restrepo et al. (2002)].

The detrimental effects caused by unintended coupling can be avoided with suitable detailing of the floor-to-wall connection or by vertically connecting the walls to avoid the relative vertically displacement from occurring. For example, in the test of a five storey precast concrete building in United States the slab was effectively decoupled from the rotation and uplift of the walls by using a special wall-connection detail [Nakaki et al. (1999)].

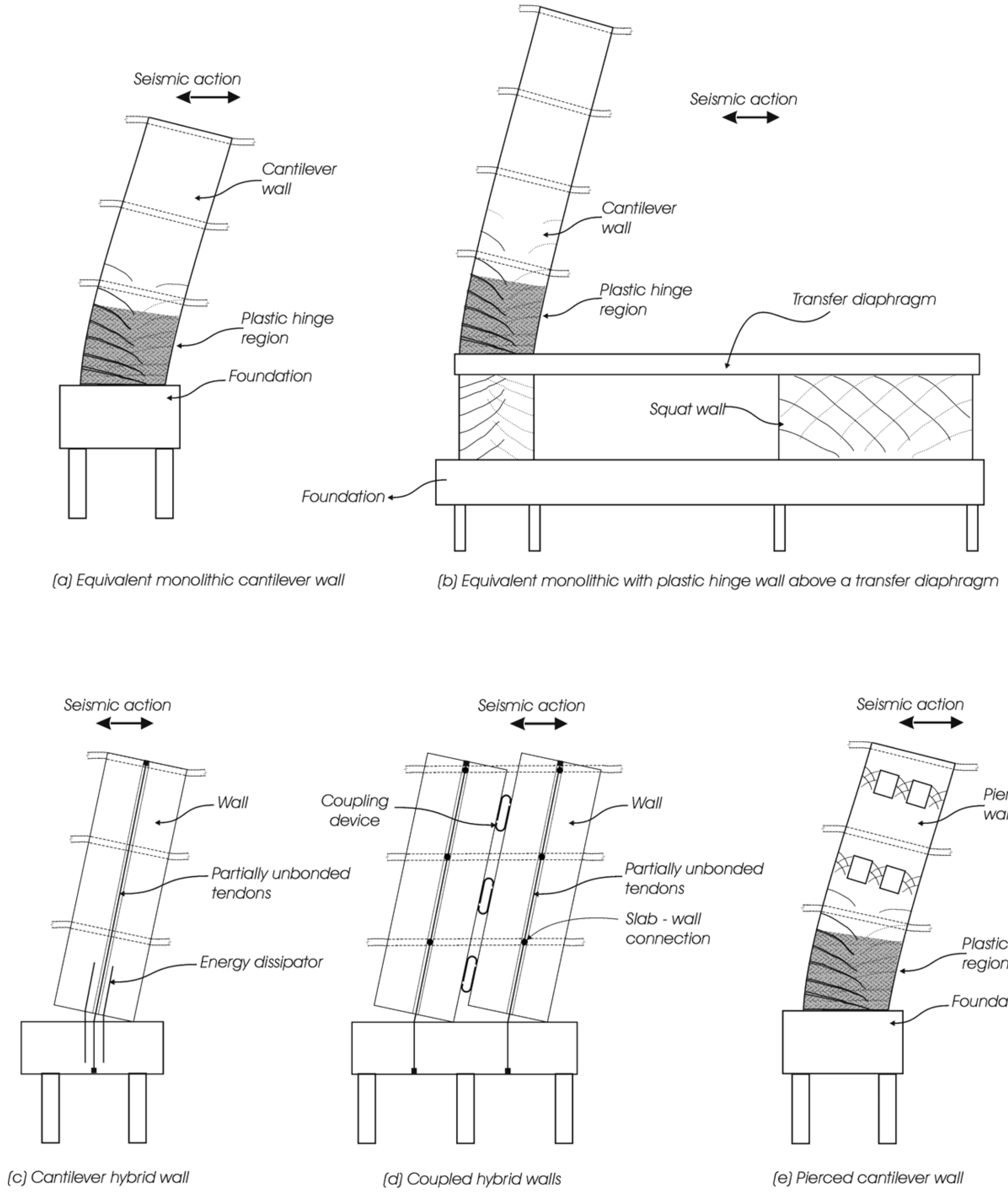
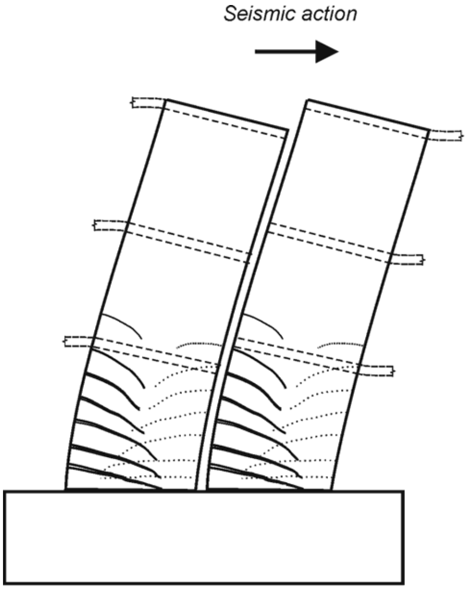


Fig. 5-41: Mechanisms of non-linear deformation in precast concrete wall systems

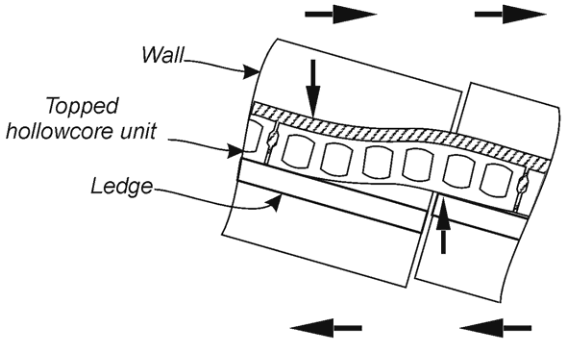
5.3.4 Design practice

5.3.4.1 United States approach

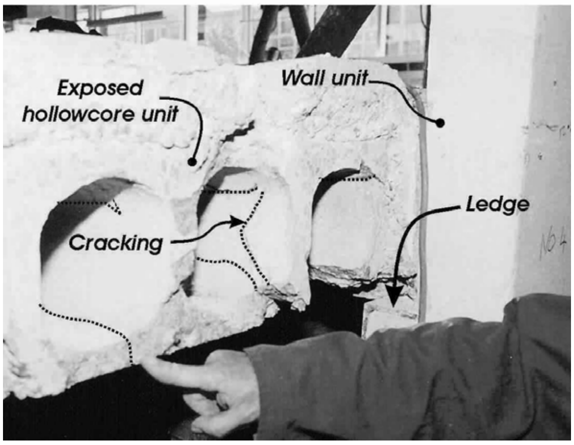
In the United States, structural walls are by far the most common choice for the lateral force resisting system in precast concrete buildings. Building types that incorporate precast concrete in the domestic market often provide ample opportunity to integrate walls required by other functions as lateral load resisting elements.



(a) Relative vertical displacement in adjacent walls



(b) Coupling effect at floor level



(c) Damage to a hollowcore floor unit as a result of unintended coupling.

Fig. 5-42: Unintended wall-slab coupling (Adapted from Restrepo et al. (2000))

The requirements for ductility incorporated into the detailing are determined by the level of seismic risk assigned to a building site. This risk is assessed based on the seismic event with a chance of exceedance of 2% in 50 years (2500 year event) and the soil characteristics at the site. From this assessment, seismic performance categories are assigned as A or B for low seismic hazard, C for moderate hazard and D, E or F for high seismic hazard.

1) Code derivation of equivalent lateral forces

Code provisions for seismic design in United States are advanced through a government-sponsored program called the National Earthquake Hazard Reduction Program (NEHRP). Since 1985, recommended provisions have been published every three years as a resource to the model and material code organizations for improvements to seismic design. In the most recently published recommendations [NEHRP (2000)], new provisions have been included that address strong and ductile connections. A strong connection is designed so that non-linear post-yield behavior occurs away from the connection. A ductile connection is designed to permit non-linear flexural or tension action at the connection, but to ensure that the shear capacity that corresponds to this action the greater than twice the corresponding load.

Equivalent lateral forces are calculated based on an acceleration spectra derived from mapped short period and one-second period acceleration coefficients and tabulated soil factors. The spectral accelerations are modified by R factors (response modification factors) that are defined for the wall systems. These values vary depending on the prescriptive ductility detailing used.

Under recently updated US code provisions, structural walls are assigned response modification factors based on application and detailing. Walls used in regions of low seismic hazard need only conform to the requirements of Chapters 1 to 18 of ACI 318-2002 [ACI (2002)], which are the provisions for concrete design without special seismic detailing. These provisions include requirements for structural integrity, minimum base anchorage, and minimum reinforcing but not for ductile detailing. Walls used in load-bearing wall systems have an R of 4. Walls used as the bracing for essentially complete gravity frames have an R of 5. In regions of moderate risk, there are no additional requirements for cast-in-place structural walls, but new provisions in ACI 318-2002 [ACI (2002)] do apply to precast, called “intermediate precast walls.” These provisions simply add the requirement that connections to and between precast concrete walls have their capacity limited by the yielding of reinforcing and not by other, less ductile failure mechanisms. No reduction in the R factors is assigned for in this new classification of precast walls.

Precast walls required in regions of high seismic hazard are now defined as “Special precast walls”. These walls must conform to ductile detailing requirements included in Chapter 21 of ACI 318-2002 for cast-in-place concrete walls in seismic regions as well as to the requirements for connections for intermediate precast walls. In the strictest sense, these precast walls emulate monolithic cast-in-place construction by conformance to all the prescriptive detailing requirements for cast-in-place construction. These requirements include bar laps and splices that are difficult to meet with jointed precast construction. In the past, there has been very little consideration of equivalent connections that are considered to meet emulation requirements. Prior to the changes introduced in the latest codes, connections details that have been used successfully in other countries to produce strong and ductile connections between walls were not recognized. With the general introduction of new connection provisions for special precast walls, it can be interpreted that such connections will be permitted, although there are no additional detailed prescriptive requirements that describe their application.

Recent changes to the ACI building code have reduced the requirements for boundary elements in walls in high seismic regions when the axial loads on these walls are moderate. However, when boundary elements are required, they also must emulate monolithic construction so it is likely that end columns would be made monolithic with the walls. Special walls used in load-bearing wall systems have an R of 5. Special walls used as the bracing for essentially complete gravity frames have an R of 6.

2) Connection considerations

Precast structural walls are generally designed to cantilever from the base and connected using the “strong” connection concept. The desirable mode for limiting capacity is flexural yielding at the base. In low and moderate seismic hazard regions, the axial load component from superimposed or mobilized dead loads may contribute significantly to the base moment resistance.

Since the expected primary behavior is flexural yielding at the base, the joint at the base requires careful detailing consideration. If the design includes yielding of reinforcing that crosses the joint, then that reinforcing must be anchored on either side of the joint to develop the full probable strength of the bars. ACI 318-2002 requires a Type 2 splice for bars in the yielding region. A Type 2 splice is intended to develop the tensile capacity of the bars, estimated as 150% of the minimum yield strength of the bar. If the wall is not to yield at this joint, then it must be designed and constructed to have sufficient strength to ensure that the ductile behavior will occur away from the joint.

3) Rocking of foundations

Most shear wall structures are designed under the assumption that the bases of the walls are rigid supports for the walls. This often may not be the case. In some US codes prior to 2000, in fact, the design overturning on a wall base was taken as only 75 percent of the overturning moment used for wall design. In general, walls designed for ductility through flexural yielding at the base should be supported by foundations designed with sufficient size and strength to mobilize the flexural yielding.

In some cases it may be acceptable to design for soil-foundation interaction that recognizes rocking. One condition is where walls are short and gravity loads are low. Here, the rocking of the wall with its foundation may be an acceptable means of non-linear response. If this approach is used, careful consideration should be given to maintaining the elastic behavior of the walls and connections as discussed in the rigid wall system section below.

The US codes also recognize that soil-structure interaction can be considered as a behavior that will reduce the equivalent lateral design forces [IBC (2000)]. The referenced procedure makes an evaluation of the added displacements that can be induced by seismic accelerations as the foundation moves. These displacements tend to lengthen the building period. For structures with longer periods, this increase tends to result in lower design forces. The increase in deformation, however, must remain within prescribed drift limits and added P-delta effects must be considered.

4) Rigid wall systems

In US practice, there is no wall system recognized by the codes that is based on design by elastic response. There are currently no provisions that permit the relaxation of detailing requirements in high seismic hazard regions for the use of lower response modification factors. There is, however as discussed above, an allowance to reduce the base shear for structures where the effect of soil-foundation interaction is evaluated and found to result in the effective lengthening of the overall system’s period.

5) Regions of high seismic hazard

In regions of high seismic hazard, the current US codes do not recognize the use of jointed wall systems. The principal guidelines in detailing the walls for ductility are the special seismic provisions of ACI 318, Chapter 21 [ACI (2002)]. These provisions dictate requirements for spacing of reinforcing as well as criteria for the design of boundary elements when needed. These provisions are severely lacking, however, in addressing the detailing needs for ductile connections in precast. Walls that do not conform to the detailed requirements of Chapter 21 are only permitted if it is demonstrated by experimental evidence and analysis to provide equivalent strength and toughness.

In the current development of NEHRP 2000, new recommendations expand the provisions that qualify walls as emulating monolithic construction. These provisions include alternatives for strong and ductile connections. They recognize that in a broad sense, emulation can mean design of walls with comparable behavior, including their non-linear response characteristics and deformations. Since a monolithic wall experiences deformation as a cantilever from a fixed base, a comparable precast system would be designed to exhibit the same deformation characteristics. This comparison would permit opening of horizontal joints with ductile connections since this produces deformations comparable to flexural cracking on the tension edge of the wall. If such a wall were constructed with vertical joints, the connections across those joints would require design as strong connections. These strong connections are designed to ensure that non-linear behavior occurs away from the connection. In this way, the deformation across the joint would be limited so the wall would deform as if it were monolithic.

Under the new provisions, walls connected with ductile connections across vertical joints might also be designed for emulation, but the comparable structure is different. The deformation of such a wall would not be that of a single monolithic wall, but as walls connected by coupling beams. In this case, the ductile connections provide resistance to overturning, energy dissipation and some resistance to drift, but the walls deform nearly independently. Actually, these walls might be considered as independent walls in a stiffness analysis. The ductile connections, then, would be considered only as an externally applied force to be considered for stability and resistance to overturning.

Several configurations of precast joints have been tested to determine their non-linear response characteristics. In horizontal joints, connections utilizing mechanical splices meeting the ACI-318 2002 Type 2 capacity requirements have been well demonstrated to provide significant ductility. Tests have also been performed on a number of vertical joint connections. With vertical joints, a significant challenge for ductility is the ability of the connection to sustain a significant level of its capacity as it experiences the large deformations along the vertical joint. Tests that use a ductile mechanical link between embedded parts show success in developing yield at the link, but have also demonstrated limited strain capacity. One of the PRESSS connections used a ductile plate bent into a “U” shape and fastened to the embedded plates. This connection showed excellent non-linear deformation capacity during the test of the PRESSS programme of the five storey precast concrete building [Priestley et al. (1999)].

5.3.4.2 New Zealand approach

A large percentage of buildings in New Zealand incorporate precast concrete structural walls as the primary lateral force resisting system. Precast concrete walls are often found to be most economical than other construction alternatives in low-rise commercial and industrial buildings.

Very often the seismic design of buildings incorporating precast concrete walls is carried out using the provisions for the design of cast-in-place concrete walls given by the New Zealand Concrete Structures Standard [Standards New Zealand (1995)]. Therefore, the majority of systems used in New Zealand follow the emulation approach. Specific best-practice guidelines are available for the design of the different systems used [Centre for Advanced Engineering (1999)].

The design of buildings incorporating precast concrete walls is independent of the method of precasting. Depending on economical advantages and site space availability, walls panels are either precast in-situ or in a commercial precast concrete yard.

Wall systems can be designed for nominally elastic, limited ductility or full ductility response regardless of the seismicity of the region. Nevertheless, designs for nominally elastic and limited ductility response are more commonly adopted in regions of low and moderate seismicity.

The magnitude of the design lateral forces decreases, whilst the complexity in the detailing of the reinforcement in the critical regions increases, as the design approach goes from nominally elastic to full ductile response. The New Zealand standard requires that a mechanism of non-linear deformation be established, regardless of the design approach chosen. When designing for limited ductility and full ductility responses, the standard requires the use of capacity design to ensure that the mechanism chosen can develop and be maintained in strong earthquakes.

For nominally elastically responding wall systems the New Zealand Concrete Structures Standard does not require the use of special reinforcement detailing. In general, systems designed for elastic response are subjected to low levels of axial compression, have a rectangular cross sections with a thickness of 125 to 150 mm, have unsupported height to thickness ratios of less than 50 and use a single layer of longitudinal and transverse reinforcement.

According to the New Zealand Concrete Structures Standard, wall systems designed for limited ductility response may incorporate a single layer of longitudinal and transverse reinforcement in the potential plastic hinge regions, depending on the ratio between the neutral axis depth and the length of the wall panel. When the threshold limit is exceeded, the region of compressed concrete at the wall ends over the potential plastic hinge region must be adequately confined with transverse reinforcement. The longitudinal reinforcing bars in this region of the wall panel must be appropriately restrained to avoid premature bar buckling. Moreover, the standard recommends that splices of longitudinal reinforcement in the potential plastic hinge regions be staggered. The standard also gives recommendations to ensure the wall panel thickness is adequate to prevent instability of the panel over the plastic hinge region. The standard requires the use of capacity design for wall systems designed for limited ductility response.

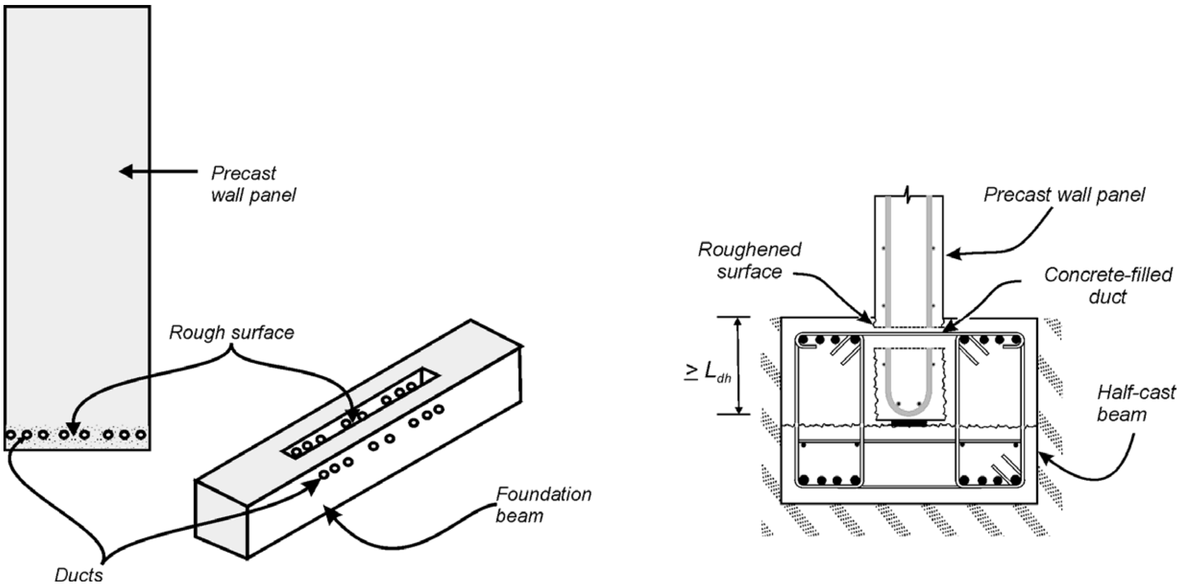
The requirements of the New Zealand Concrete Structures Standard for wall systems designed for fully ductile response are similar but more stringent provisions than those for systems designed for limited ductility response. A major difference in the design recommendations between the two levels of ductility is that the standard does not allow the use of a single layer of longitudinal and transverse reinforcement over the potential plastic hinge region in wall panels designed for full ductile response.

In principle the New Zealand Concrete Structures Standards allows the use of jointed systems, but the standard gives no prescriptive design requirements for such systems.

5.3.4.3 Systems for equivalent monolithic construction

A system that is used for constructing walls to emulate monolithic response is illustrated in Fig. 5-43. In this system the wall panels are seated inside a recess left on the foundation beam and then grouted, Fig. 5-43 (a), or on a partly cast foundation beam that is subsequently completed in a second placement to embed a portion of the wall panel, Fig. 5-43 (b). Transverse reinforcement is passed through horizontal holes left in the wall panels. The wall's longitudinal reinforcement is anchored in the wall a distance at least equal to the development length, L_{dh}.

The amount of transverse reinforcement passing through the ducts in the wall panels can be determined from a strut-and-tie analysis as shown in Fig. 5-44. This system can be used for designing walls for all elastic through fully ductile response [Holden et al. (2003)]. Fig. 5-45 shows an example of application of the use of this system in New Zealand.



(a) Wall panel embedded in a recess in the foundation beam *(b) Wall panel seated on a half-cast foundation beam*

Fig. 5-43: Connection detail for walls embedded in the foundation beam

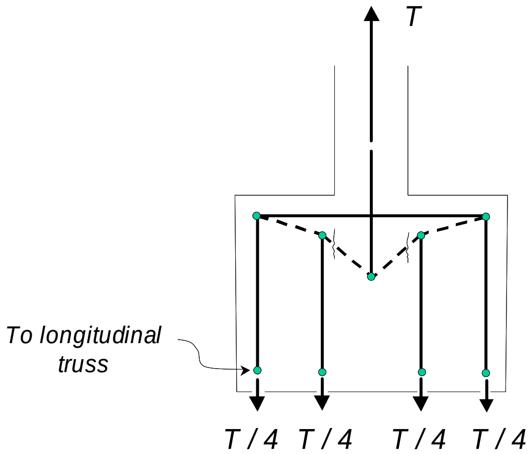


Fig. 5-44: Strut-and-tie model showing the internal force flow of a wall embedded in the foundation beam

5.3.4.4 Jointed systems

1) System 1 – Large panel system

Large panel system construction is a system that was widely used in seismic regions of Europe and Latin America in the 1960s and 1970s. This system is composed of precast concrete floor and walls, which are jointed in three directions to provide overall stability. The precast element are connected at the edges by a variety of ways, see for example Fig. 5-46.

Large panel systems group a variety of patented methods. Structures built using this systems have generally performed very well in earthquakes. In low-rise building construction the inherent capacity of the system results in most cases in elastic response. In high-rise buildings in regions of high seismicity, buildings are generally designed for limited ductility response [Fischinger et al. (1987)]. In such buildings the preferred method of energy dissipation is through shearing at the vertical joints. These joints can be "tuned" to provide a desired capacity. However, these joints generally present a pinched response resulting from shear deformations. Experimental studies carried out in Slovenia have been used to characterized the non-linear dynamic response of these systems [Fischinger et al. (1987), Zeck and Becker (1976)]. Some examples of application of large panel construction are illustrated in Figs. 5-47 to 5-49.

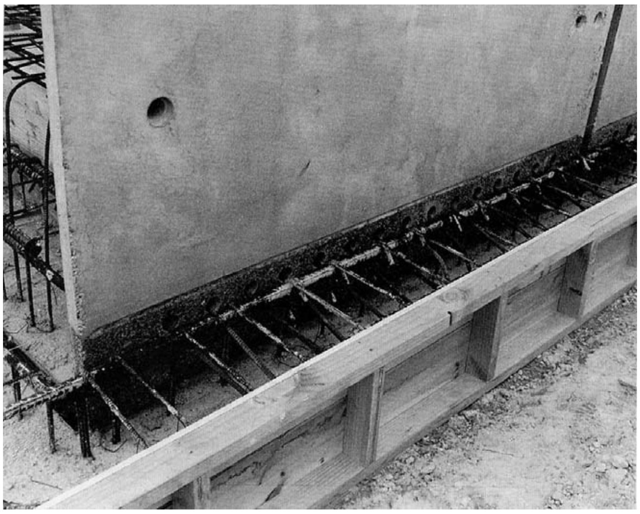
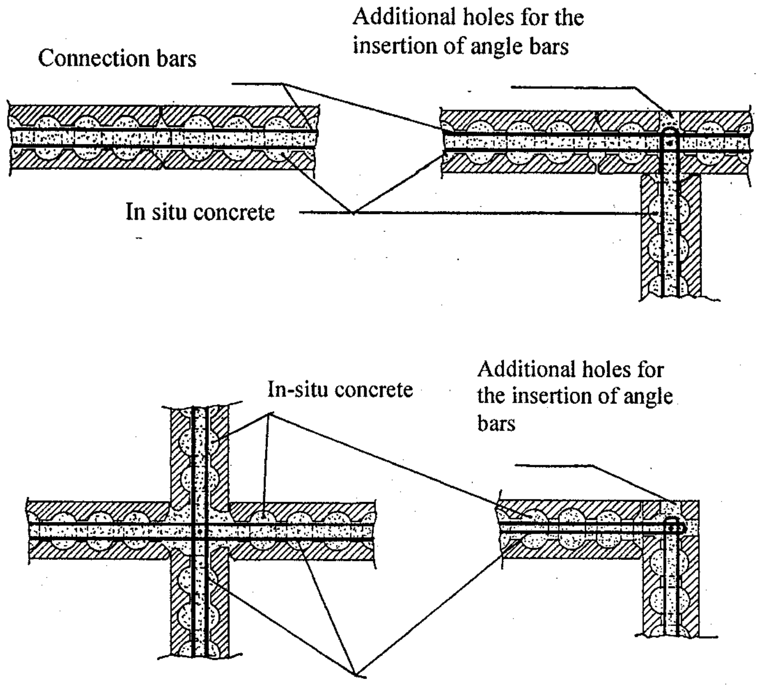


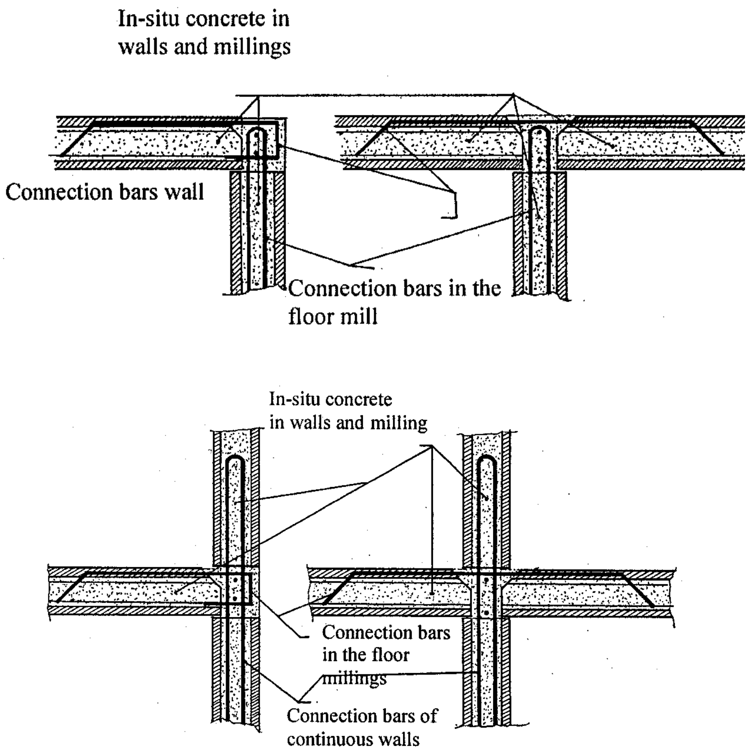
Fig. 5-45: Example of a wall embedded in the foundation beam

2) System 2 – Cantilever walls connected through grouted ducts

Another system uses the foundation-wall connection approach shown in Fig. 5-50. Wall panels are lowered into position ensuring that the vertical reinforcing bars protruding from the foundation beam are projected at least a distance equal to their development length into galvanized corrugated steel ducts present in the precast wall panels. The ducts and the horizontal interface gap are grouted in a single operation. Shrinkage-compensating cement-based grouts, which are either pumped or poured through the interface gap, are normally used for the grouting operation. The grout is pumped in to ensure it flows in one direction to avoid the entrapment of air. Air is expelled through vents placed at several locations on the gap between the foundation as well as at the upper end of each duct. A minimum distance of at least 75 mm between the end of the vertical bar and the end of the duct is recommended. This distance is in recognition that most grouts bleed and the quality of the grout at the top end of the duct is relatively low. Prior to grouting, the jointing wall panel and foundation beam surfaces are roughened and cleaned with an oil-free air pressure gun to improve shear transfer. The connection in this system relies on the force transfer between the vertical reinforcing bars protruding from the foundation and longitudinal lapping bars in the wall panels. The force transfer is achieved through non-contact bar splices from bars that are grouted into corrugated steel ducts and bars that are embedded in the wall panel close to the grouted bars. Such system concentrates the non-linear response at the wall-to-foundation interface. As a result, large strains are expected to develop in the reinforcing bars passing through the interface. An issue that needs to be addressed in design with this system is the potential for sliding shear that may occur at the wall-foundation interface after yielding of the reinforcing bars has taken place. Testing and analysis have demonstrated that this system is suitable for elastic through to limited ductility design [Crisafulli et al. (2002)]. Fig. 5-51 shows an application of this system.



(a) Vertical joints between walls



(b) Wall-to-floor joints

Fig. 5-46: Typical connection details in large panel construction (Courtesy of M. Menegotto)

Fig. 5-47: Overall view of a construction site in Slovenia using the large panel system (Courtesy of M. Fischinger)

Fig. 5-48: Wall panel connection detail in large panel construction (Courtesy of M. Fischinger)

Fig. 5-49: Example of a large residential complex in Italy built using large panel construction (Courtesy of M. Menegotto)

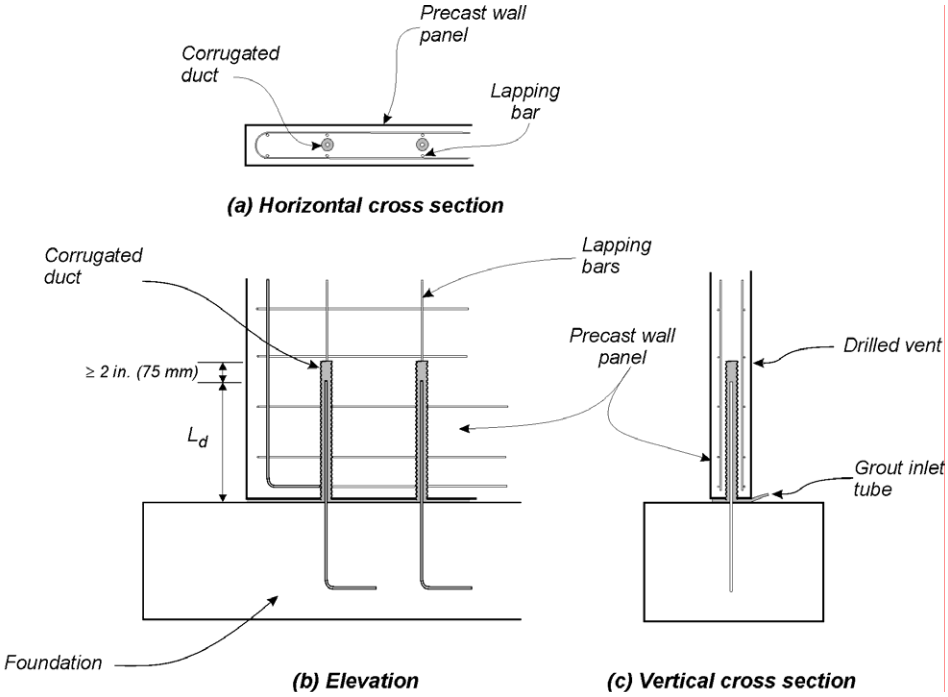


Fig. 5-50: Wall-foundation beam connection through grouted ducting

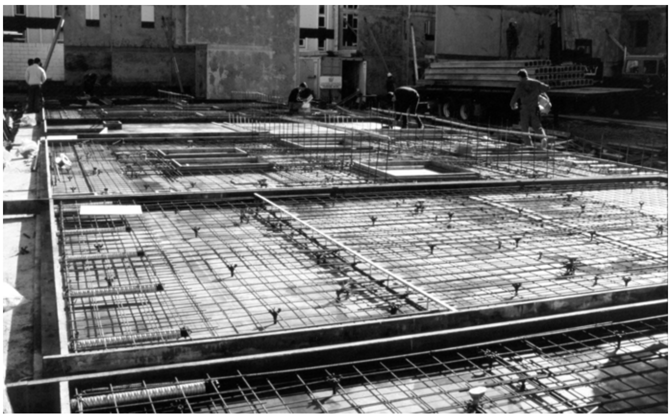


Fig. 5-51: Wall panels incorporating a connection details through grouted ducts under construction in New Zealand (Courtesy of J. Restrepo)

3) System 3 – Hybrid walls

A number of hybrid and rocking wall systems have recently been tested to observe their overall seismic response and predictability [Priestley et al. (1999), Rahman and Restrepo (2000), Holden et al. (2003)]. The hysteretic response of these two systems is shown in Fig. 5-40. The restoring force in these systems results from prestressing and gravity loading combined. The tendons are generally prestressed to stress levels lower than those used in conventional prestressed systems and are left partially or totally unbonded. The initial stress level and the unbonded length of the tendons are calculated to ensure that the limit of proportionality in the prestressing steel is not reached as a result of the elongation caused by the opening of the gap at the wall base during the largest expected lateral displacement demand.

Energy dissipation capacity can be incorporated into the system by several means. For example, in the case of the hybrid cantilever precast wall shown in Fig. 5-41 (c), energy dissipation can take place if mild steel bars, generally having a milled segment in the form of a “dog-bone”, are cast in the foundation and then grouted into the wall. Energy dissipation results from extensive yielding in tension and compression in the tensile strain domain within the milled portion of the bar only. Note that buckling cannot occur as the milled portion of the bar is surrounded by concrete in the elastic foundation beam. Alternatively, adjacent wall panels can be coupled with rolling plates as Fig. 5-41 (d) shows. The energy dissipation devices are proportioned to ensure that closing of the gap at the horizontal connection is ensured upon unloading. A consequence of this action is that the hysteretic response of the overall system is characterised by loops showing energy dissipation and no residual lateral displacements, see Fig. 5-40. These emerging systems are ideal for use in conjunction with a displacement-based seismic design approach [Priestley (2000)]. This is because their hysteretic response can easily be linearised and associated with an equivalent viscous damping ratio.

Figure 5-52 shows the a five-storey precast concrete building tested at the University of California, San Diego, under the PRESSS programme [Priestley et al. (1999)]. Precast concrete walls prestressed with unbonded tendons and coupled with stainless steel rolling plated provided the lateral force resistance in one direction of loading. Figure 5-53 shows the construction of a building in Chile. In this building, the lateral force resistance in one direction is provided by cantilever walls post-tensioned with unbonded tendons.

5.3.4.5 Strong connections for precast walls

1) Connections between walls

Numerous details are currently used to connect the wall panels at vertical joints. These connections can be divided in four groups: welded connections, bolted connections, monolithic joints and connections to adjacent columns. The selection of adequate connections in each particular case should be done not only on the basis of the structural requirements but also considering the construction techniques and the erection tolerances.

From the structural point of view, connection details are classified in two groups. The first group comprises those connections capable of transmitting the force resulting from the interaction of the adjacent wall panels, as occurs in welded or cast-in-place concrete connections. The second group comprises connection details that allow unrestrained movement such as bolted connections with oversized holes. The use of oversized holes may result from the intention of the designer to avoid interaction between the precast wall panels or from practical considerations conceived to simplify the erection of the panels. Only those connection arrangements employing a vertical cast-in-place concrete strip are described in the following paragraphs.

Figs. 5-54 to 5-56 show three possible vertical wall-to-wall connection details that make use of cast-in-place concrete. Typically, these connections are made overstrong to ensure behaviour as if “monolithic”. The wall panel interfaces are typically roughened and in some case incorporate shear keys, see Fig. 5.47. The shear friction concept is generally used to determine the amount of reinforcement that protrudes from the wall panels. In the case of the large panel system described in Section 5.3.4.4, the connections are sometimes designed as a weak link and used to dissipate energy [Fischinger et al. (1987)].

The vertical joint depicted in Fig. 5-54 has enough space to accommodate the lap-splice length of the transverse bars that protrude from the precast wall panels. Another possible connection detail is shown in Fig. 5-55. The transverse reinforcement protruding from the wall panels is bent in the form of a rectangular hoop. The hoops of adjacent walls do not overlap to ease the erection process. The connection is completed by adding longitudinal bars and “drop-in” stirrups.

Fig. 5-52: Overall view of the five-storey building tested under the PRESSS programme [Priestley, (1999)]

Fig. 5-53: Erection of a cantilever wall in a building in Chile built with hybrid frames and walls (Courtesy of P. Bonelli)

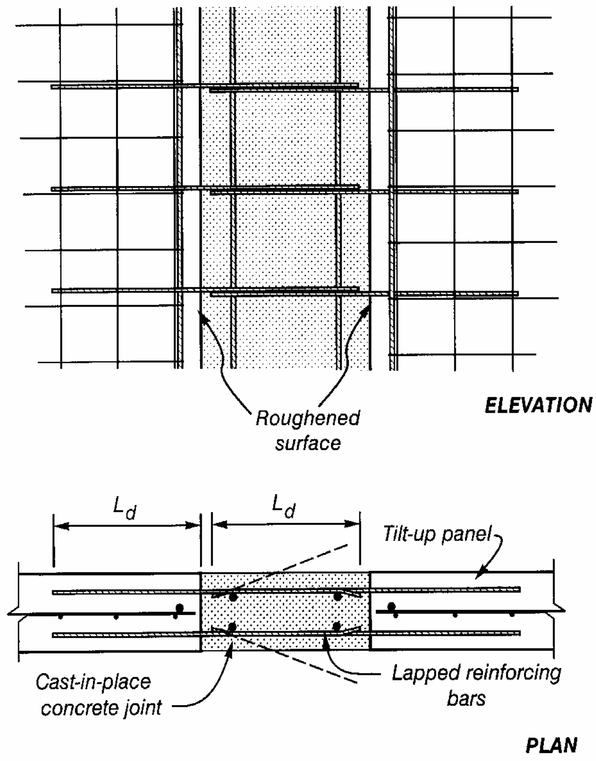


Fig. 5-54: Vertical connection detail between walls: Connection with straight bars

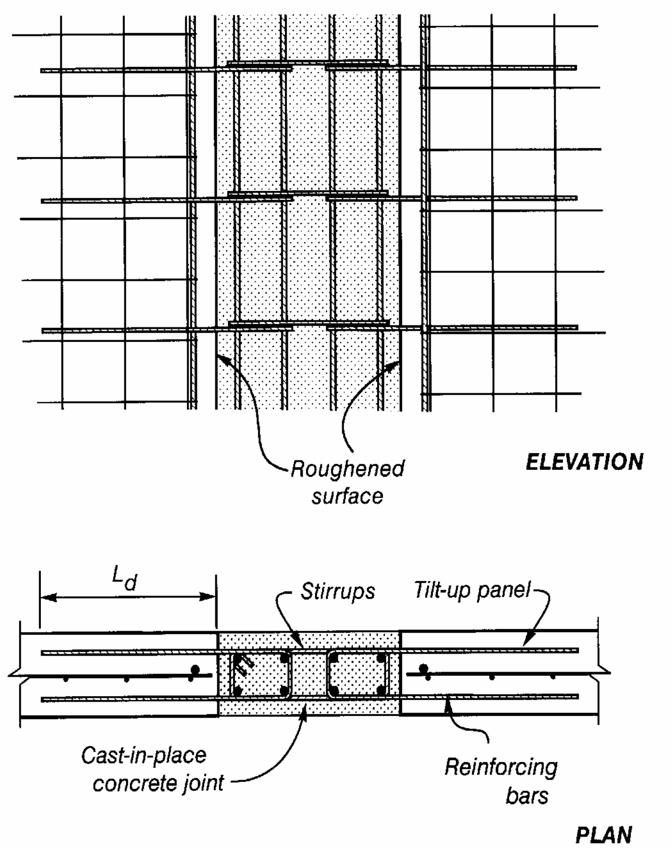


Fig. 5-55: Vertical connection detail between walls: Connection with drop-in hoops

Fig. 5.56 shows another possible connection detail. The transverse hoops protruding from the wall panels overlap, making it possible to have a narrow vertical joint. This arrangement is most suitable when the wall panels are connected through a cast-in-place side strip or are embedded in the foundation beam. Fig. 5.57 illustrates the use of vertical wall panel connections in the construction of three low-rise theatre buildings. The vertical connections between wall panels in the buildings in the foreground and background have already been cast and some temporary inclined props have already been removed as the partially completed roof structure provides stability. The walls in the middle building have not been cast yet and the gaps between the walls are clearly visible in the photo.

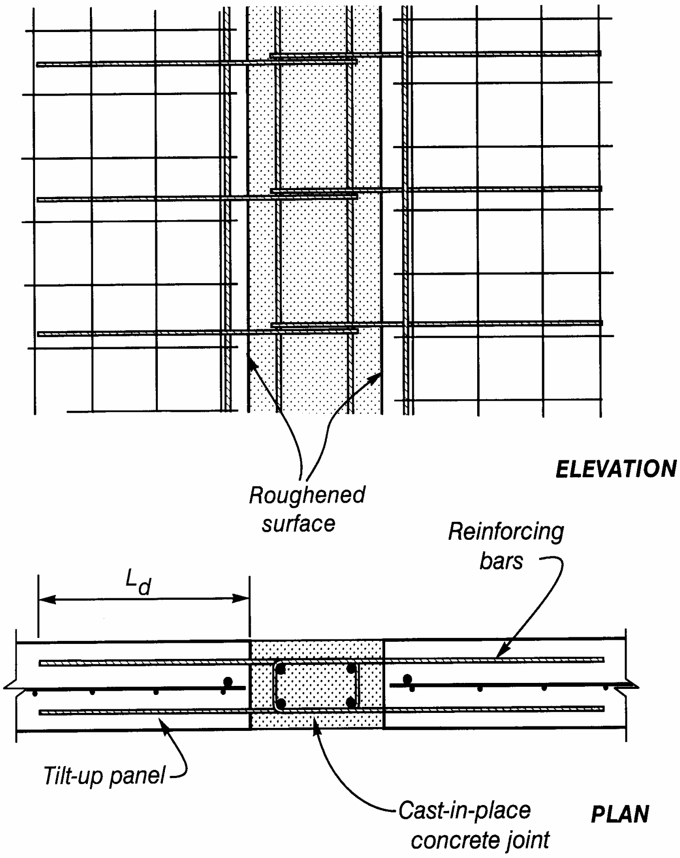


Fig. 5-56: Vertical connection detail between walls: Connection with overlapping hoops

2) Wall-to-slab connections

A variety of connection details are employed to connect precast concrete floors and the precast wall panels. These connections are generally designed to transfer floor inertia forces to the wall panels and often detailed to avoid unseating or concrete pull-out failure during earthquakes.

Connections between precast floor units and interior wall panels are achieved through mainly two methods and they are illustrated in Figs. 5-58 and 5-59. In the first method, see Fig. 5-58, the wall panel is cast continuous over the slab-to-wall intersection. A recess is left in the wall to provide the seating for the precast concrete floor units and tie reinforcement passes through orifices left in the wall panels. When using hollowcore floor units, cast-in-place concrete fills the cores at either side of the supports.

Fig. 5-57: Example of theatre buildings built with precast concrete walls incorporating cast-in-place vertical joints (Courtesy of J. Restrepo).

The second method, shown in Fig. 5-59, is employed when using partitioned walls. The floor units seat on the wall below leaving a gap to let the wall longitudinal reinforcement passes through. Then cast-in-place concrete is placed on top of the slab and on the gap between the adjacent floor units. The wall panels are connected through grouted ducts at the slab-wall intersection. This connection detail is similar to the wall-foundation detail through grouted ducts depicted in Fig. 5-50 and described earlier. Fig. 5-60 shows an example of this connection approach in a multi-storey building. Before erection and after casting to concrete topping, the jointing surfaces are carefully roughened and cleaned and the connection above the cast-in-place concrete is sealed, see Fig. 5-60 (a). Then, the wall is lifted into position, see Fig. 5.60 (b), braced and the gap between the wall and the slab and the ducts are subsequently grouted.

Connections between precast floor units and exterior wall panels are achieved through three main methods. The methods depicted in Figs. 5-61 and 5-62 are employed when the wall panels are continuous over the floor. In these two methods, the floor units are either supported on a steel ledge or on an armoured concrete corbel. The third method incorporates a recess on the wall panel, which is used to support the floor units. This method is used when the walls are partitioned horizontally, see Fig. 5-63.

In all the connection methods shown in 5-61 to 5-63 the reinforcing bars protruding from the panels are designed to transfer the inertia forces at each floor level through a shear friction mechanism. It should be noted that, given the sensitivity of the anchorage length to construction tolerances and the consequences of a pull-out failure of the concrete, the New Zealand Concrete Structures Standard [Standards New Zealand (1995)] requires that these bars be anchored in the walls to ensure they can yield extensively.

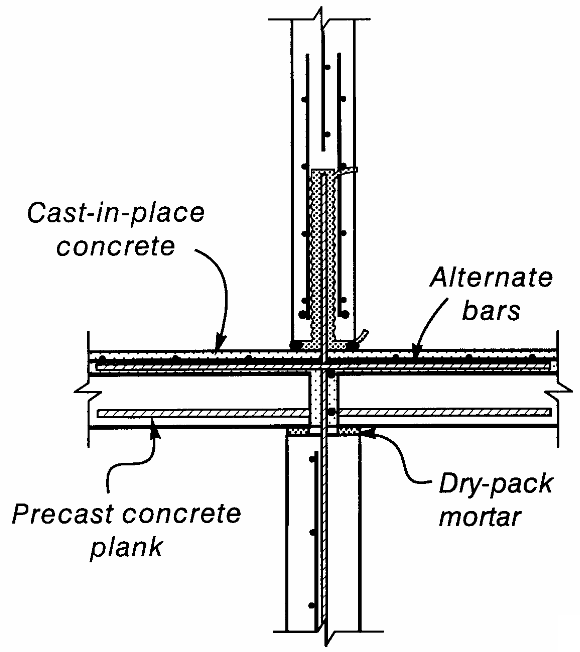


Fig. 5-58: Interior wall-floor slab connection detail: Continuous wall

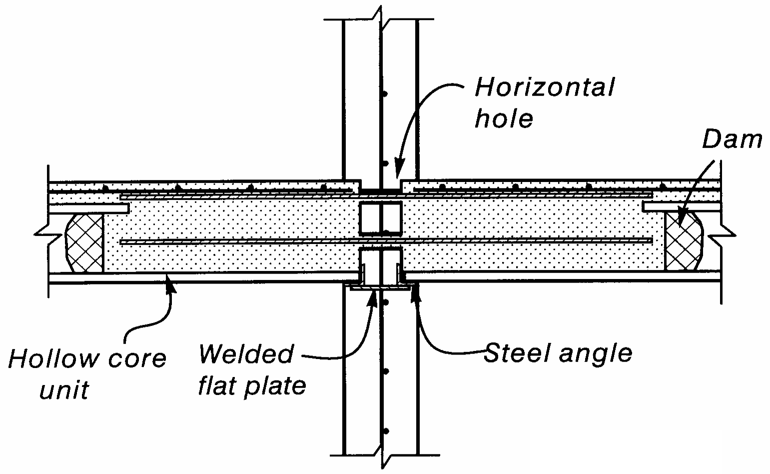
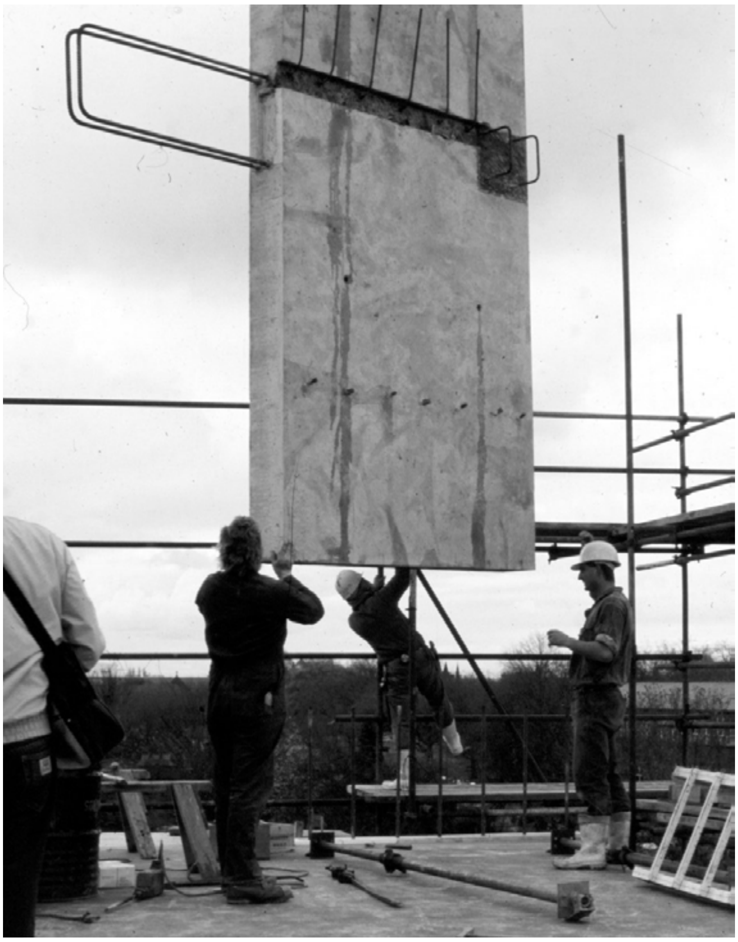


Fig. 5-59: Interior wall-floor slab connection detail: Partitioned wall with horizontal joint

(c) Close up of horizontal connection detail



(b) Erection

Fig. 5-60: Example of a grouted wall-wall connection in New Zealand (Courtesy of J. Restrepo)

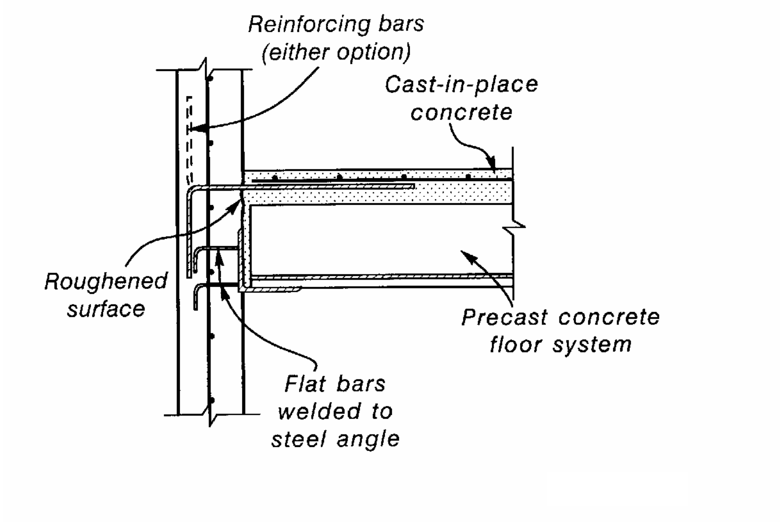


Fig. 5-61: Exterior wall-floor slab connection detail: Continuous wall with steel ledge

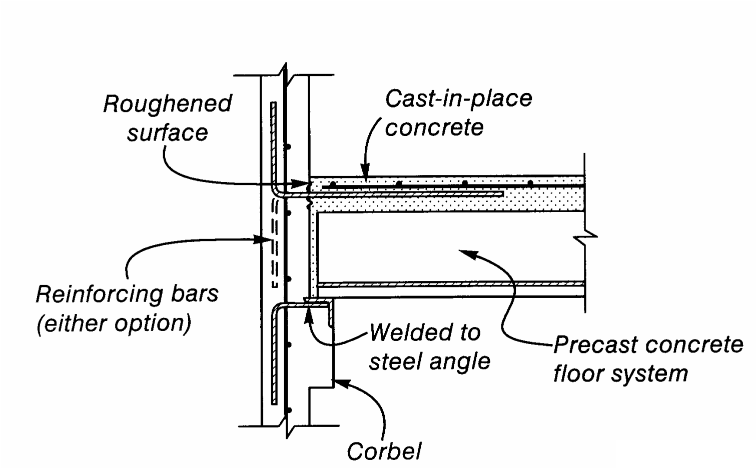


Fig. 5-62: Exterior wall-floor slab connection detail: Continuous wall with armoured corbel

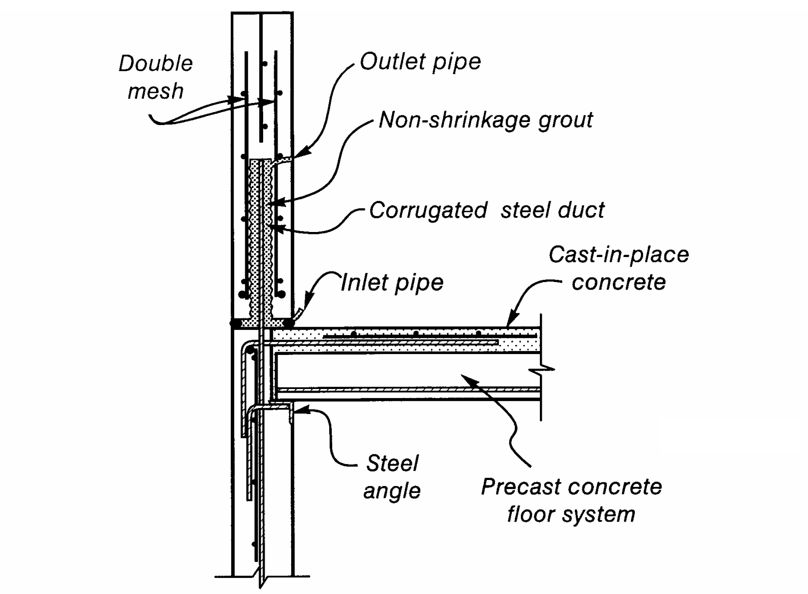


Fig. 5-63: Exterior wall-floor slab connection detail: Partitioned wall with horizontal joint

5.3.5 Dual systems

5.3.5.1 General

The combination of cantilever walls and frames acting in parallel and designed to resist lateral forces in buildings is referred to as dual systems. The contrasting lateral displacement shapes of individual cantilever wall and frame systems, when combined in a dual system, forces them to deform in a unique way that brings several structural advantages such as excellent drift control and excellent seismic energy dissipation, particularly for high-rise multi-storey buildings. In dual systems, the large lateral stiffness of the walls leads to a good lateral displacement control during earthquakes, eliminating the possibility of column sideway mechanism (soft story) in the frame, particularly in the lower storeys of a building. Frames, on the other hand, help to control inter-storey drifts in the upper storeys of a building. The degree in which walls and frames interact depends on the way the individual systems deform laterally. For example, the interaction is greatest when slender cantilever walls, which deform principally in bending, are combined with frames that have relatively large beam depth to column depth ratios, and that deform primarily in shear. This interaction, however, results in the development of internal forces whose magnitude is not negligible.

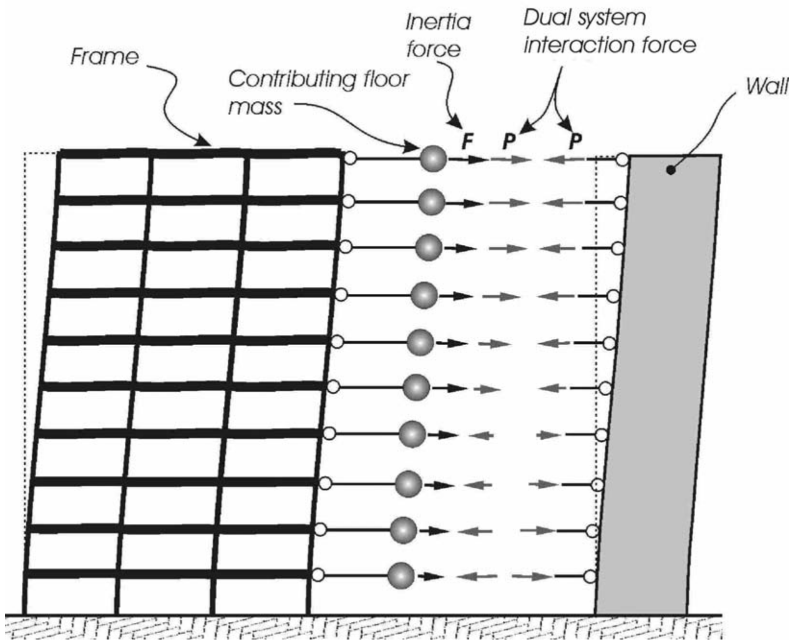
When walls and frames are not co-planar, as is often the case, the interaction forces in both cast-in-place and precast systems must be transferred through the diaphragm. In some cases the forces can be similar or greater in magnitude than the floor inertia forces arising from the acceleration acting on the floor's distributed mass. Fig. 5-64 (a) illustrates the forces that result from the interaction between a frame and a wall. In the upper levels, interaction forces develop in the same direction as the inertia forces, thus increasing the internal forces in the diaphragm. In contrast, in the lower storeys the interaction and inertia forces oppose each other. Fig. 5-64 (b) shows a plan view of the top floor of the dual system depicted in Fig. 5-64 (a). It is clear in this figure that the forces diaphragm cannot be designed for the inertia forces alone.

In precast reinforced concrete structures incorporating dual systems in which the depth of the walls is much greater than the depth of the beams most of the inelastic behavior of the system is expected to be provided by the walls, with a significant reduction of inelastic demands in the frames as compared to the demands in a system based only on frames. Although this is a general feature of dual systems, it is particularly convenient in precast construction in seismic areas since less levels of detailing would be required in frames in a dual system as compared to those necessary for a structural system based only on frames.

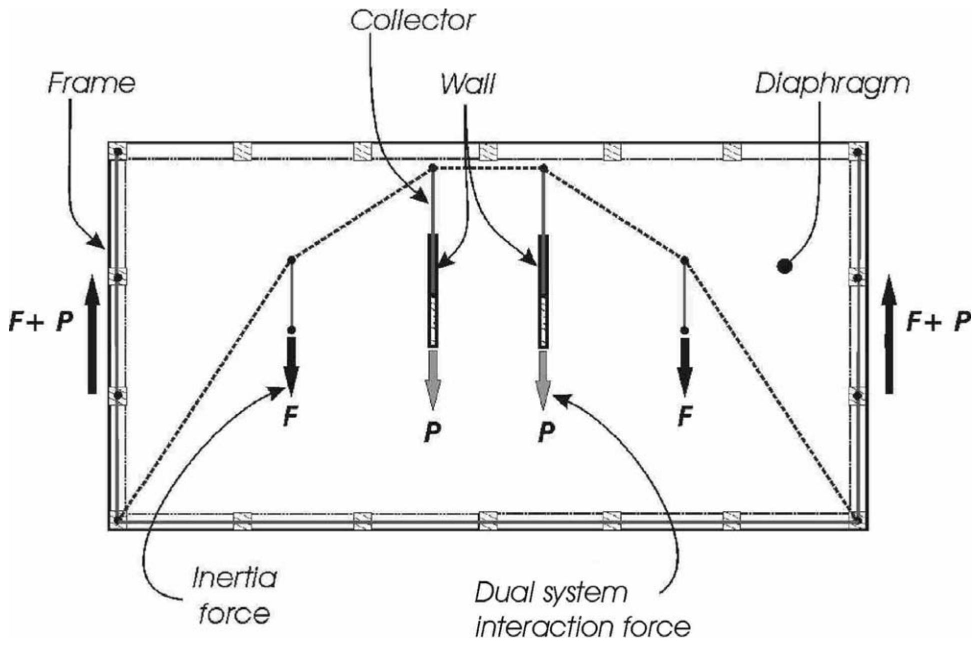
5.3.5.2 Displacement and ductility compatibility in buildings with dual systems

Displacement and ductility compatibility is a relevant issue in the design of buildings with dual systems, particularly in what is regarded as the detaining of the frame and wall members [Paulay and Restrepo (1999)]. Fig. 5-65 shows the idealized lateral force, lateral displacement response of the walls and frames in a dual system. The overall response of the dual system is obtained as the superposition of the response of the walls and frames. This simple model shows that the seismic response of the dual system is controlled by the displacement ductility capacity of the walls. According to the overall response, the frames remain elastic when the walls reach their ultimate displacement. This finding suggests that elements in the frames do not need to be designed for full ductility, in spite of the dual system being fully ductile. This simple model explains that the design ductility for the walls and frames in a building with a dual system is not independent and should be chosen on the basis of a rational analysis where deformation compatibility is considered.

Such concept has been examined experimentally [Rodriguez et al. (2002)]. These researchers conducted a seismic load test on a two-storey precast building incorporating a dual system for lateral load resistance. A plan and lateral elevation of the specimen is shown in Fig. 5-66. Fig. 5-67 plots the base shear, V, normalized in terms of the ideal base shear strength, V_R, versus the roof drift ratio, D_r. Lateral forces were applied in the direction of the structural walls. An evaluation of the specimen's damage at the end of testing revealed that most of the inelastic response of the system was concentrated at the walls' bases with only little inelastic response of precast frame elements and their connections. Fig. 5-67 shows results of a push-over analysis against the experimental response envelope. Both plots in this



(a) Elevation



(b) Plan view of top floor

Fig. 5-64: Development of inertial forces and frame-wall interaction forces in diaphragms of buildings built with dual systems figure show that there is a reasonable correlation between measured and predicted results. These predicted results are also shown in Fig. 5-68 in terms of the contribution of the walls and frames. These results are expressed in terms of global drift ratio, D_r, and global displacement ductility ratio, μ. It can be seen that results in Fig. 5-68 are conceptually similar to those shown in Fig 5-65, that is the lateral displacements of the dual system is controlled by the lateral deformation capacity of the walls. Results shown in Fig. 5-68 also indicate that the precast elements of the frame sub-system do not need to be designed for full ductility. This is a convenient feature in dual systems and can be used in precast construction with dual systems for reducing the levels of detailing in frame elements and their connections. A building designed with a dual system is shown in Fig 5-69. The central cast-in-place reinforced concrete structural walls forming the service core of the building were designed to resist the seismic forces. The perimeter frame of precast concrete truss beams and precast concrete circular hollow columns infilled with cast-in-place concrete was designed mainly for gravity loading.

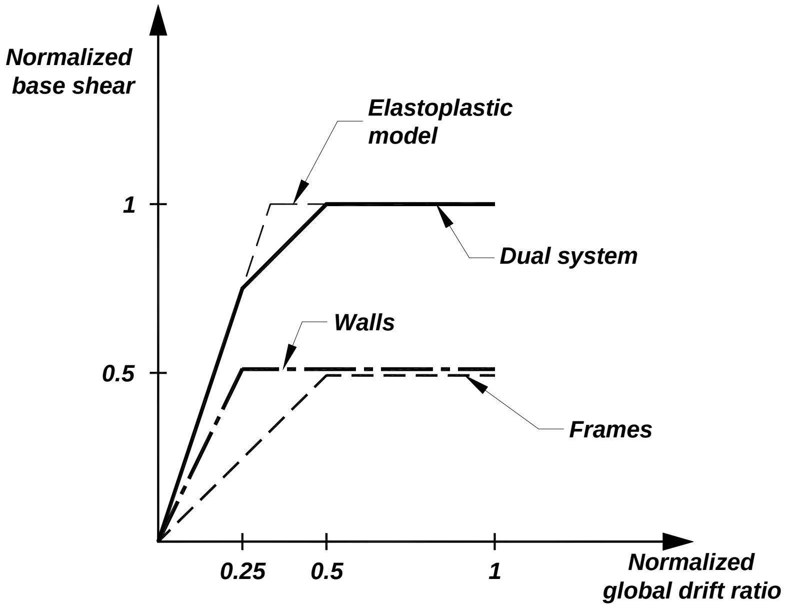
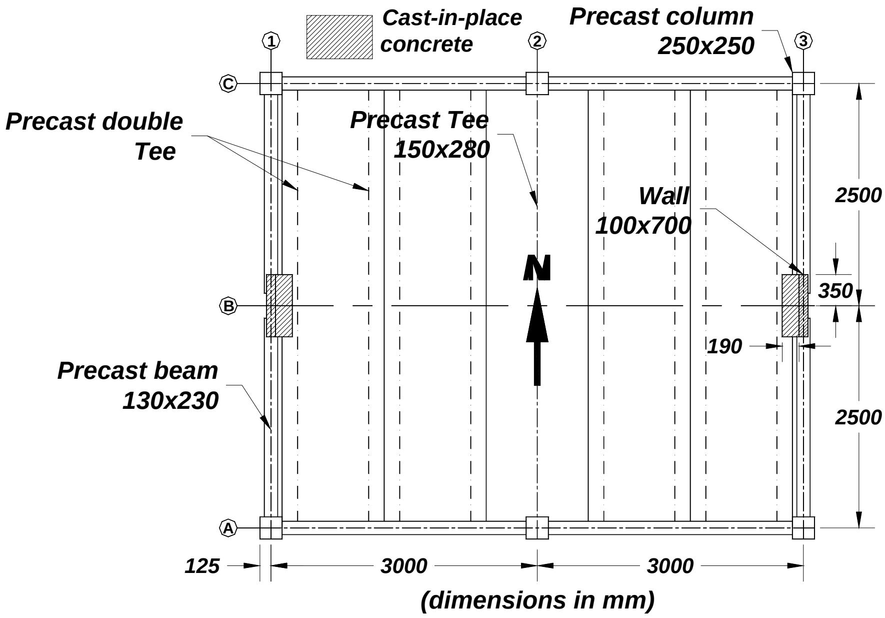
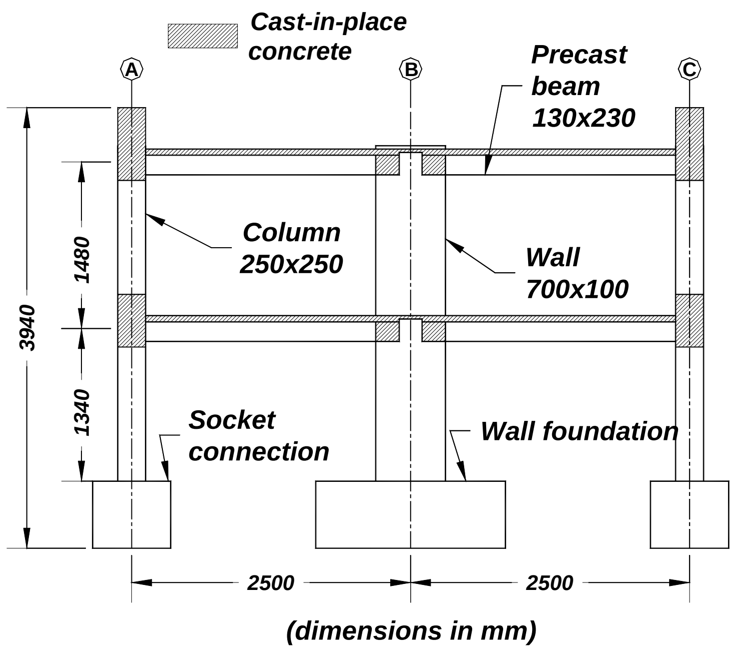


Fig. 5-65: Normalized lateral force-lateral displacement response in a dual system and in the individual sub-systems



a) Plan



b) Axis 1 elevation

Fig. 5-66: Plan and elevation of a test specimen with a dual system [Rodriguez et al. (2002)]

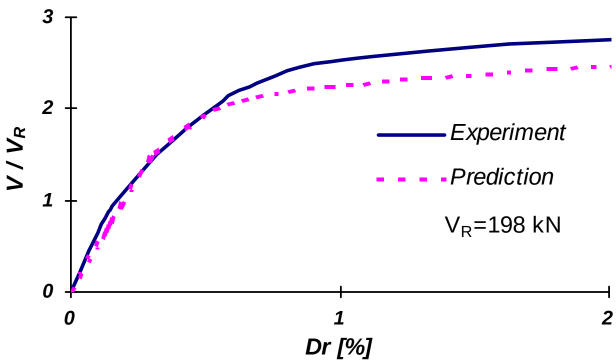


Fig. 5-67: Measured and predicted normalized lateral force-lateral displacement response envelope of a test specimen with a dual system [Rodriguez et al. (2002)]

5.3.5.3 Issues related to the seismic design and behaviour of dual systems

Building codes specify values for the response modification factor, R, for structures with different structural systems, including dual systems. As is known, this factor is also related to the displacement ductility capacity of the system. However, designers should be aware that, due to deformation compatibility, the sub-systems are subjected to different displacement ductility demands. This implies that the displacement ductility demand for the dual system would be less than the individual values of walls and larger than the individual values corresponding to frames. As a consequence, in the design of a dual system in precast construction the detailing of walls control the displacement ductility capacity of the dual system, implying that the detailing for ductility in the critical regions of the frames should not be as stringent as in the critical regions of the walls.

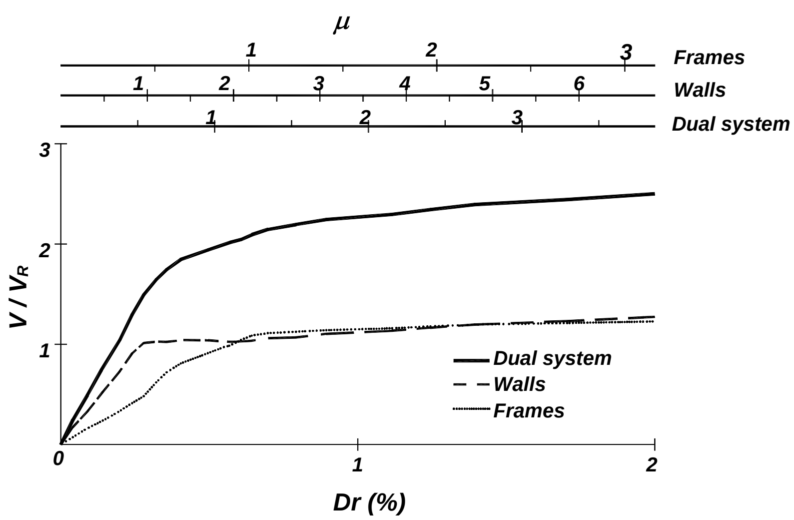


Fig. 5-68: Predicted lateral force-lateral displacement response obtained for a test specimen with a dual system [Rodriguez et al. (2002)]

Fig 5-69 Precast concrete perimeter frame in a building in New Zealand with seismic forces resisted mainly by an interior core of cast-in-place concrete structural walls (Courtesy of R. Park)

The evaluation of diaphragm forces is a relevant issue that needs to be considered in the seismic design of dual systems, and has been particularly raised after the observed behavior of diaphragms in precast parking structures based on dual systems during the 1994 Northridge earthquake [Fleishman (1998), Wood (2000)]. This issue is addressed in detail in Section 6.10 of this report. Another issue that needs to be considered in the evaluation of the seismic response of dual systems is the effect of base flexibility, which arises from flexibility of the foundation itself or from soil deformations. A related problem, rocking of walls in dual systems, has also been observed in laboratory tests [Bertero (1985)]. In these tests, after a plastic hinge formed at the wall's base, the wall tended to rotate with respect to its base as a rigid body as depicted in Fig. 5-70. Such behaviour results from the outriggering action of the transverse frames. Several factors need to be considered as a result of this interaction:

1) Increasing ductility demands in beams of the frame at the tension side of the wall.
2) Development of tensile forces at the other end of transverse beams could arise due to the outriggering action of the frames.
3) Increase of axial compression in the wall will cause overstrength. A consequence of this is the increase in shear demand in the wall.

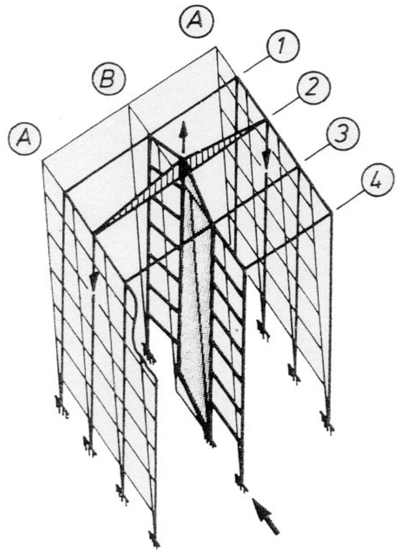


Fig. 5-70: Wall-transverse frame interaction [Bertero, (1985)]

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6 Diaphragms

6.1 Introduction

This chapter covers the seismic behavior and design of floor diaphragms in precast concrete building structures. The design of precast flooring units for gravity loads is covered by other ***fib*** documents [FIP (1994)] [FIP (1998)].

Floor systems play a key role in the lateral resistance of building structures by providing diaphragm action. Diaphragm action serves to:

(1) transfer lateral loads at each level to the lateral load-resisting elements (walls, frames); and

(2) unite individual lateral load-resisting elements into a single lateral load-resisting system.

These actions are crucial in allowing the lateral (load-resisting) system to perform as intended. Diaphragm action occurs by virtue of the in-plane stiffness inherent in the floor system. Accordingly, significant in-plane forces may develop in the diaphragm during lateral loading. In this regard, the paneled nature of floor systems in precast concrete construction can present challenges in the seismic design of diaphragms. As covered in this chapter, these design challenges may pertain to providing adequate strength, stiffness and in cases, ductility in the diaphragm and diaphragm connections.

It is important to note that seismic design procedures in-place for diaphragms, particularly precast concrete construction, are currently under reexamination, largely driven by recent performance of precast concrete diaphragms in earthquakes. While modifications to diaphragm design have occurred in recent code editions, e.g. 1997 UBC [ICBO (1997)], it is generally agreed among researchers and practitioners that current design guidelines require further improvement. In this regard the research community is working toward consensus on a viable seismic design approach for precast concrete diaphragms. Some information on these developments appears in the last section (6.10) of this chapter.

6.2 General concepts of diaphragm behavior

6.2.1 Diaphragm action

At each level in a structure, the floor diaphragm is responsible for transferring the inertial forces that develop within the floor system to the lateral load-resisting elements. In cases where this action dominates the diaphragms are referred to as "simple" diaphragms [Bull (1997)]. In certain regions of structures with dissimilar or discontinuous lateral system elements, however, the action of uniting the individual lateral-load resisting systems can dominate. Diaphragms in these regions are typically termed "transfer" diaphragms (Sec. 6.8.7). However, it is important to note that both actions occur to an extent in all diaphragms and a designer should consider their combination [Menegotto (2002)]. Regardless of whether the source is primarily floor system inertia or the relative displacement of vertical elements of the lateral system, diaphragm action involves the development of forces in the plane of the floor slab. These in-plane forces can be significant, particularly for transfer diaphragms or simple diaphragms with long spans between lateral system elements.

6.2.2 Standard analysis methods for diaphragm design

The magnitude of the inertia force at each floor level is estimated using a capacity design approach (Section 6.6.3.1) or a diaphragm design force distribution based on equivalent lateral loads (Section 6.6.3.2). A key design step is to transform the diaphragm design forces into internal actions. For regular floor plans, a "horizontal plate girder" analogy is typically used in design (Section 6.6.4.1). In this approach, diaphragm design forces are typically applied at each level as a distributed in-plane load along the length of the diaphragm. The resulting "girder" internal forces are used to determine the amount of flexural reinforcement (chord steel), shear (web) reinforcement, and lateral system reaction load path reinforcement (collector steel). An irregular mass distribution in plan may require the design forces to take on a "smeared" load configuration as the mass distribution with respect to diaphragm depth has relevance in determining the proper web reinforcement [Menegotto (2002)]. For more complex diaphragm configurations, a strut-and-tie method can be used (Section 6.6.4.2). However, there is compelling evidence that the strut-and-tie method accurately predicts the behavior of floor diaphragms in general [McSaveney (1997)] and may become the preferred method of diaphragm analysis.

6.2.3 Floor diaphragm configurations and diaphragm force paths

Building floor plans often adopt squat rectangular configurations in which the diaphragm may be considered to possess infinite in-plane stiffness (the so-called 'rigid' diaphragm assumption). Diaphragm load paths in such structures are typically straightforward. Buildings with wings or floor penetrations, however, create irregularly configured diaphragms. Care must be taken in establishing force paths through the diaphragm to the lateral system elements, and assessing force concentrations at corners and around openings (See Fig. 6-1). Irregular floor plans may also be susceptible to in-plane twisting or lateral loads from diagonal lines of attack. Openings in the floor or wings of sufficient length can also lead to diaphragms with considerable in-plane flexibility, which must be addressed in the design (Section 6.9.2).

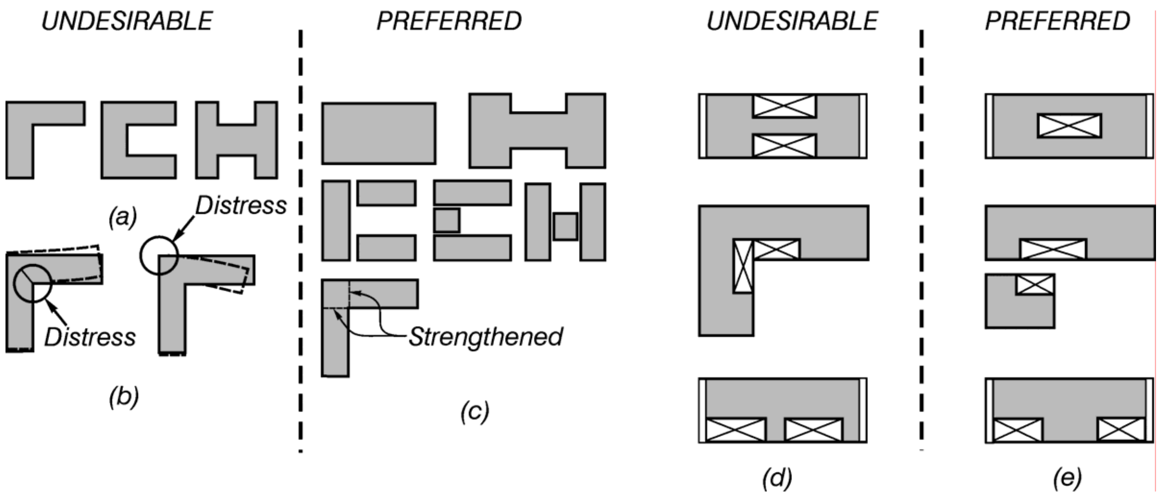

Fig 6-1: Building plan configurations (after Paulay and Priestley, 1992)

6.3 Precast floor systems

6.3.1 General

Precast floor slabs are typically designed as simply supported units because of the lower relative cost of providing midspan positive moment capacity in pretensioned floor units. Simple support suits long spans, heavily loaded structures, or top flange-supported units where providing end moment restraint is difficult. Diaphragm forces can be safely transferred to the lateral system elements from simply supported floor systems provided attention is paid to detailing for adequate corrosion protection of the reinforcement. However, since end continuity reinforcement is needed to resist diaphragm forces, this reinforcement may be also used to provide partial fixity to reduce midspan moments.

The precast floor system ultimately selected depends on factors related to the utility and form of the structure, availability and the seismic zone. Each system has characteristics unique to its form that drive its diaphragm detailing. These characteristics depend on construction method and type of precast unit, as described in the following.

6.3.2 Precast concrete diaphragm construction methods

Precast concrete diaphragm construction falls broadly within two categories: composite (topped) and non-composite (pretopped/untopped) construction.

6.3.2.1 Composite or topped diaphragm construction

Current U.S. provisions for precast floor diaphragms in high seismic zones require a cast-in-place topping slab to provide continuity in the floor [ICBO (1997)]. The Concrete Structures Standard (NZS 3101) for New Zealand [Standards New Zealand (1995)] also encourages the use of a topping slab and provides recommendations as to when composite action between the slab and precast units is to be relied upon. Concrete toppings required to resist seismic forces should be well bonded to adequately roughened precast elements. These elements should be cleaned prior to placing the topping concrete. Dampening of the surface has also been a common technique but this practice is controversial as dampening of the surface may weaken the newly cast concrete layer near the interface [Menegotto (2002)]. Steel reinforcing bars embedded in the slab serve as collector and chord steel. Current procedures recommend continuous web reinforcement within the topping slab to provide an uninterrupted shear path through the diaphragm [PCI (1999)]. Welded wire fabric provided within the slab for crack control will contribute to the shear resistance through shear friction across panel joints. In the past, some designers have used this fabric to provide the entire shear transfer [Bockemohle (1981)], a practice now viewed unfavorably. Alternatively, mechanical connectors between the precast units can supplement the shear transfer (See Fig. 6-2 a). A minimum structural topping thickness of 65mm is recommended to ensure adequate lapping of welded wire mesh, perimeter tie bars, continuity and chord reinforcement. Bars up to 16mm in diameter can be effectively lapped in this topping, but the use of larger bars requires verification by cyclic testing. Steel fibre reinforced concrete (SFRC), at a minimum thickness of 50mm [Oliver (1998)], can also be used as a structural topping layer to resist seismic actions (Section 6.7.4).

6.3.2.2 Non-composite or pretopped (untopped) diaphragm construction

In moderate or low seismic zones, pretopped systems are permitted. These types of floors are common in low seismic regions in the U.S. [Cleland (2001)]. Untopped precast concrete floors are not generally used for high seismic zones in New Zealand [Bull (1997)], but this is for economic reasons, not technical reasons. Certain advantages of untopped diaphragm construction should be recognized including lower weight and depth of the floor, and less vertical eccentricity between center of floor mass and the plane of diaphragm stiffness [Menegotto (2000)]. Chord forces in pretopped diaphragms are carried via mechanical connectors made continuous within the precast element, or by chord steel included within pour strips. Current procedures for pretopped diaphragms recommend securing precast units to internal beams to provide an uninterrupted shear path through the floor [PCI (1999)], with adequate accommodation of creep and shrinkage. In such dry systems, mechanical connectors embedded along the edge of the precast units provide this web reinforcement (See Fig. 6-2 b). Often, alternative design criteria such as traffic or thermal loads control the design of the web reinforcement. For this reason, it is common practice to provide standard mechanical connectors placed at uniform spacing throughout the structure.

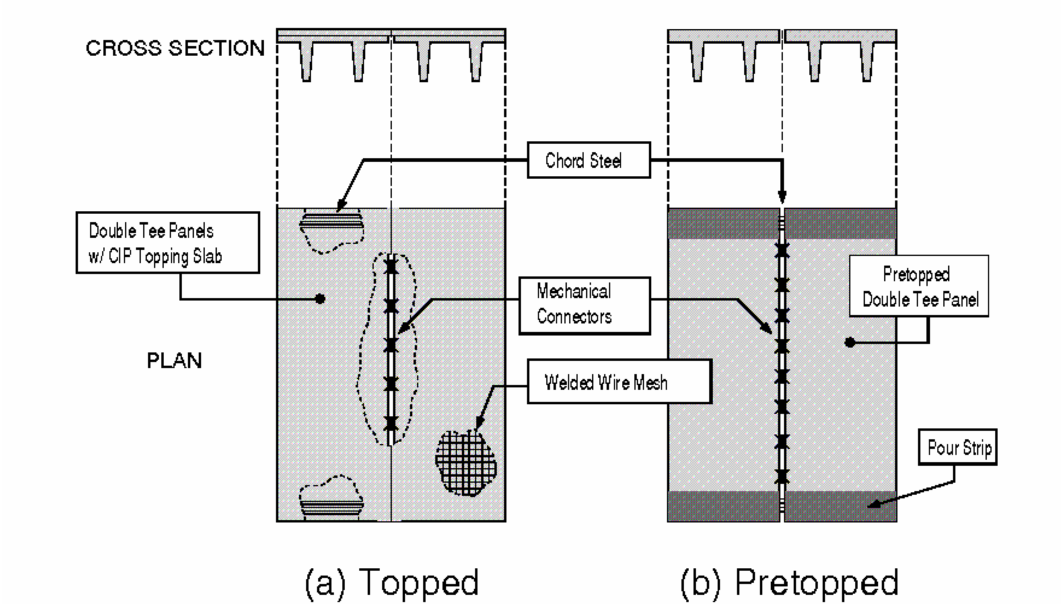

Fig 6-2: Precast joint details (a) topped diaphragm; (b) untopped diaphragm (after Farrow, 2002)

6.3.3 Types of precast concrete floors

6.3.3.1 Hollowcore slabs

Hollowcore slabs, manufactured by proprietary extrusion or slip-forming machines [FIP (1988), PCI (1985a)] are used in many earthquake prone countries. As indicated in Section 6.3.2, hollowcore slabs are typically used without a topping in moderate seismic zones. In high seismic zones, hollowcore slabs are used in conjunction with a cast-in-place topping reinforced with welded wire mesh, deformed bars or steel fibres.

In untopped hollowcore slabs, diaphragm forces are resisted by shear-friction reinforcement located in the shear keys between the precast floor units and across the ends of the slabs [Moustafa, (1981)]. Davies et al (1990) tested full-scale hollowcore diaphragm segments in pure shear, evaluating diaphragm shear stiffness and strength. They concluded that the shear resistance depends heavily on the ability of chord reinforcing to provide dowel action and recommended that this action be protected to allow these diaphragms to sustain their strength and stiffness during cyclic loading.

Menegotto (2000) developed a hollowcore unit with an undulated lateral profile at the shear key. This special feature mobilizes shear-friction mechanisms along slab edges without the need for reinforcing in the joint, though end reinforcement is still required. Full-scale experiments of untopped slabs with the corrugated shear key demonstrated improved seismic performance in terms of strength, ductility and repeatability at high amplitude reversals [Menegotto (1994)]. Finite element-based analytical models capturing the behavior have been introduced into structural analysis software to produce a diaphragm analysis computer program [Menegotto et al (1998)].

For topped hollowcore diaphragms, the top surface of the extruded slabs requires some roughness to enhance the bond of the topping concrete. For high shear regions in a floor associated with transfer diaphragms or adjacent to shear walls, reinforcement tying the topping to the precast slabs can be located in the shear keys between the precast units, or in holes cut into the voids. Transfer diaphragms will often require drag bars to distribute forces into the thin topping concrete.

Because hollowcore slabs contain no secondary reinforcement, their contribution in resisting horizontal in-plane diaphragm forces is usually ignored in high seismic zones. Though this assumption is conservative, further testing is required to determine the shear or compression levels where composite hollowcore floors cease to function in a ductile manner during in-plane cyclic loading. Regardless, hollowcore slabs do restrain the topping layer from buckling, allowing the full diaphragm force to be carried by the thin topping.

6.3.3.2 Solid planks

Solid planks with roughed top surface finish may be designed as composite slabs. Diaphragm forces can be wholly carried by the structural topping, or the solid planks can also be reinforced and detailed to carry their share of the diaphragm forces. For this reason they are often a preferred system for transfer diaphragms carrying high shear. Alternatively, untopped solid planks can provide adequate diaphragm action if connected by ductile welded or grouted connections, and provided with transverse bars at their ends.

6.3.3.3 Beam-block systems

Beam-block flooring systems typically use thin pre-tensioned concrete ribs, typically 150-200mm wide, ranging from 100 mm to 250 mm in height, spaced at 600 to 900 mm centres (Refer to Fig. 6-3c). Between the ribs, acting as permanent formwork, kiln fired clay blocks, lightweight concrete, plastic or timber planks are placed and an in situ topping is cast to complete the floor (with a thickness of 100 mm – 175 mm, typically). These systems require the composite structural topping to carry both gravity loads and diaphragm forces. The topping provides ample space to include diaphragm reinforcement, but the lack of shear reinforcement in some proprietary extruded rib sections warrants caution in detailing areas likely to be subjected to severe local seismic damage.

6.3.3.4 Double and single tees

Double or single tee pretensioned units are often used for long span floor or roof construction. They may be designed to act compositely with a cast-in-place topping layer or pre-finished in the factory for use without a structural topping. Multiple tees, with three or more legs are also used for shorter span applications.

6.4 Seating requirements for precast floor units

6.4.1 General

As part of the floor system, diaphragms must maintain gravity load carrying capacity while resisting lateral loads. In precast systems, a key component of this requirement is the preservation of adequate seating for the precast units (Refer to Fig. 2-9 for an example of unseating of precast diaphragm units). Because of this issue, precast floor systems are generally perceived to be more susceptible to the loss of gravity carrying capacity than monolithic floor systems. In any case, gravity should not be relied upon for providing friction and positive mechanical devices are needed for carrying horizontal forces.

6.4.2 Recommended minimum seating dimensions

Minimum seating dimensions for precast flooring systems must consider construction tolerances and the effects of system response to the earthquake [McSaveney (1997)]. In New Zealand, NZS 3101 (1995) specifies minimum end bearing lengths after a reasonable combination of unfavorable construction tolerances of 1/180 clear span and not less than 50mm for solid or hollow core slabs; and 75mm for beam or ribbed members. For lightweight concrete, 10 mm is added to nominal minimum seating dimensions. In the U.S., Section 16.6 of ACI 318 provides similar provisions [ACI (1998)]. These minimum seating requirements should be modified for potential reductions in seating associated with maintaining compatibility with the lateral system unless some alternative support mechanism is provided [Herlihy (1999)].

6.4.3 Tolerances

The standard construction details used in precast buildings [PCI (1985b), PCI (1985c)] have been found by experience to allow for the statistically likely combinations of manufacturing, construction and erection tolerances anticipated on most well run job sites. As a result, most precast manufacturers have developed standard details to accommodate out-of-tolerance support conditions [Yapp (1985)]. Supervision of construction will ensure the use of these details where required. On rare occasions however, accumulated tolerances may reduce safe seating limits. Thus, while the capacity of these remedial details under gravity load is easily verified in static load tests, the performance under seismic loads requires consideration of displacement ductility, verified in extreme cases by laboratory testing [Herlihy (1999)].

6.4.4 Displacement compatibility

Diaphragm designs must account for the displacements imposed on the diaphragm by the lateral system. For instance, during large ductility demands in moment resisting frames, elongation associated with the plastic hinge zones will cause column separation. For precast systems, this should not lead to a loss of support of the precast units provided the recommendations of Sec. 6.4.2 are met. However, wide cracks can form in the slab at the supports, or the topping slab can delaminate from the precast units causing failure of the welded wire mesh at the ends of starter or continuity bar laps [Bull (1997)]. In mixed frame/shear wall structural systems, the restraint of incompatible displacements between the two systems will impose forces on connections between precast floor units and their support beams. While these tie connections may be designed to yield in a ductile manner, the effect of this movement on seating lengths must be assessed [McSaveney (1997)].

Provisions for displacement compatibility are also essential for slab-to-column connections providing diaphragm action while subjected to rotations imposed by the vertical bracing system. These out-of-plane actions render the use of topping-to-column connections alone as unreliable. Instead, tie-beams should be supplement the force transfer. Alternatively, the full floor depth should be involved both in diaphragm action and connections [Menegotto (2002)]. Punching shear capacity and adequate curvature ductility can be detailed to prevent collapse, while ductile connectors ensure that diaphragm forces are transferred to the vertical elements [McSaveney (2000)]. Similarly, vertical motion of walls due to curvature or rocking will tend to lift adjacent floor regions as lateral loads are resisted. The articulation of the floor, in conjunction with the in-plane force paths, should thus be considered in design [Bull (1997)].

It is also important to assess the deformation imposed *by* the diaphragm. The UBC code [ICBO (1997)] prescribes deformation compatibility detailing requirements for non-seismic elements to be: "adequate to maintain support of design dead load plus live loads when subjected to the expected deformations caused by seismic forces…diaphragm deflections shall be considered"{UBC 1633.2.4}. The inclusion of these deformations is particularly important in the calculation of expected gravity system column drifts for structures with flexible diaphragms (See Sections 6.9.2 and 6.10.4)

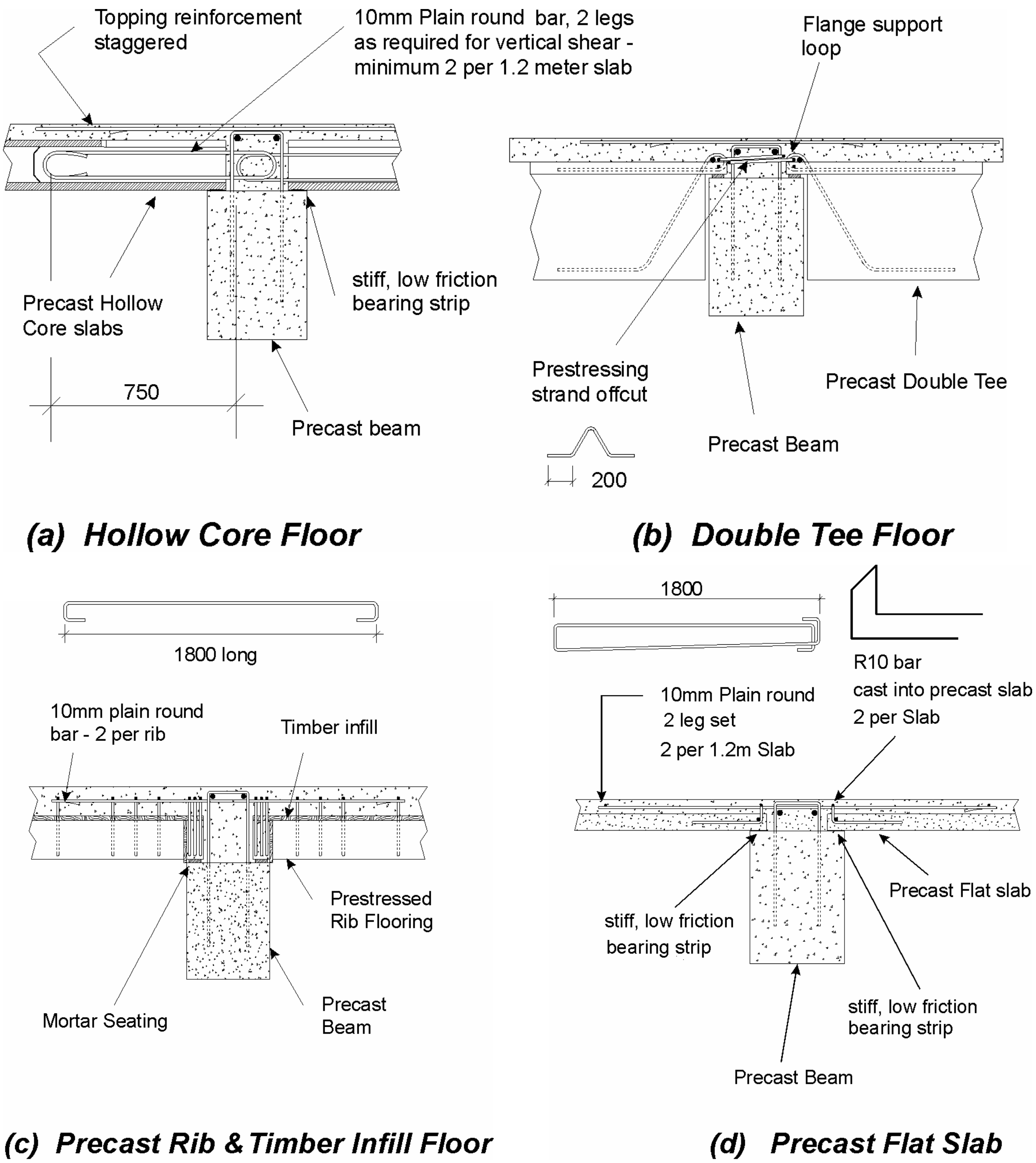

Fig 6-3: Details to prevent loss of support: (a) hollow core; (b) double tee; (c) rib; (d) flat slab (after McSaveney,1997) .

6.4.5 Detailing for loss of support

The New Zealand Concrete Structure Standard, NZS 3101 (1995), discusses options for ensuring vertical load transfer if supports are lost due to tolerances or earthquake effects. The intent is to keep the diaphragm relatively intact in a seismic event. For instance, hollowcore slabs allow for the easy installation of connecting bars to provide an additional robustness to accommodate seismic effects that cause loss of seating [Mejia-McMaster (1994)] [Herlihy (1999)] [Centre for Advanced Engineering (1999)]. Such details if properly designed have been found capable of supporting the floor's gravity load after a loss of seating [Mejia-McMaster (1994)]. For beam block systems, a secondary support mechanism is to be provided by dowels, hanger bars, or through membrane action in case the extruded rib loses its primary end seating [McSaveney (2000)]. Figure 6-3 shows details to prevent loss of support for different types of precast units.

6.5 Service load effects

6.5.1 General

Service performance of precast floor slabs is typically of more concern than for cast-in-place slabs, because of relatively thin concrete toppings and the presence of numerous joints between the precast units. Pretopped floors, joined together by mechanical connectors at intermittent locations, may be even more prone to undesirable service level performance [Bull (1997)]. Service load effects are discussed below in terms of both service and ultimate performance.

6.5.2 Serviceability requirements

Creep and shrinkage, temperature bowing and concrete durability are the main serviceability issues that affect the performance of precast diaphragms [McSaveney (1997)]. A detailed discussion of the serviceability aspects of precast concrete is given in recent guidelines [Centre for Advanced Engineering (1999)]. The principal serviceability requirements pertain to crack width control and deflection as it relates to durability of the structure, and utility and appearance of non-structural elements. For floors that will not be covered, or those exposed to the weather or to de-icing salts, the end supports must be detailed for crack control as continuity reinforcement may yield at service load conditions [McSaveney (1997)]. Internal forces due to creep, shrinkage and temperature effects are often accommodated through provisions of compliance built into the construction [PCI (1999)] or the assumption of redistribution due to limited yielding of the reinforcement. However, the effect on stiffness or load path by this localized yielding or cracking should be considered in the design [Bull (1997)].

6.5.3 Effect of service action on seismic behavior

Cracking, deflection and deterioration due to service actions may affect the ultimate behavior when seismic action occurs. Thus, additional or stricter requirements are needed. These requirements resemble those provided for serviceability limit states but strictly speaking are related to the ultimate limit states.

Long span pretensioned units undergo considerable creep and shrinkage shortening. For satisfactory seismic performance, welded or grouted diaphragm connections must be detailed to accommodate these movements without locking in high restraint forces at the connections to the lateral load resisting shear walls or frames. The failure of prestressed roof slabs during the 1964 Anchorage, Alaska earthquake were attributed to this restraint effect [Sutherland (1965)]. Providing ductile connections that can yield to accommodate creep and shrinkage effects, but are able to maintain their full strength capacity for earthquake forces, is an effective solution, but may be difficult to achieve in practice. If this approach is taken, the joints and connection hardware must be detailed for long-term durability as corrosion may reduce the strength or ductility of the diaphragm system [McSaveney (1997)].

6.6 Seismic design procedures for diaphragms

6.6.1 General

Seismic design procedures for diaphragms involve determination of diaphragm design forces (lateral loads); transforming these loads to internal in-plane forces in the diaphragm; and providing sufficient reinforcing at each location to sustain the combination of these in-plane forces with gravity load carried by the floor system. In the following, code procedures in New Zealand and the United States are reviewed. It is noted that differences exist between the recent editions of each of the major U.S. seismic codes (UBC, IBC and NEHRP) [ICBO (1997), ICBO (2000), BSSC (2000)]. For clarity, the UBC code will be used as the representative U.S. code in the following discussions.

6.6.2 Code approaches for diaphragm design

6.6.2.1 New Zealand code

In the NZ code, diaphragm actions are considered to remain within the elastic domain under design level earthquake actions [Bull (1997)]. Special studies are required when in an exceptional case ductile diaphragm response needs to be considered in the design. [Standards New Zealand (1995)] Diaphragms interacting with vertical elements of the lateral system must be designed for the actions that would occur in ductile response of the system. Lateral loads are determined as described in Sec. 6.6.3. These loads can then be transformed into internal forces as described in Sec. 6.6.4 to determine the dependable shear and flexure strength required in the diaphragm. The purpose of this approach is to ensure that no significant inelastic deformations and hence ductility demands are imposed on the diaphragms [Centre for Advanced Engineering (1999)].

Such an approach, demonstrated for frames, is found in a report of the New Zealand Concrete Society (1994). Using a capacity design approach, the story shear for the sway mechanism associated with a strong-column/weak-beam approach is limited by beam strengths including an appropriate overstrength. Because diaphragms as deep members are susceptible to shear demands, the report suggests that the design nominal strength be associated with a strength reduction factor $\phi = 0.75$.

6.6.2.2 United States code

Existing U.S. code provisions, while implying elastic diaphragm behavior, may not clearly accomplish this goal [Nakaki (2000)]. Current seismic designs for diaphragms are based on the ultimate limit state. However, while the code does account for lateral system overstrength for collector elements, this overstrength factor is not applied to the diaphragm flexure or shear reinforcing [ICBO (1997)]. Furthermore, until recently, design codes used the same strength reduction ($\phi = 0.9$) for the diaphragm as for other elements of the lateral-system [ICBO (1994)]. The inconsistency between code intent and realization raises uncertainty as to whether current designs will maintain elastic behavior for the Design Basis Ground Motion [Nakaki (2000)].

6.6.2.3 Recent modifications to the U.S. code

Immediately following the 1994 Northridge earthquake, a task group investigating parking structure performance proposed two code changes for precast diaphragms, later adopted by the 1997 Uniform Building Code [ICBO (1997)]:

(1) minimum spacing, cover, and transverse reinforcing for chords and collectors {UBC 1921.6.7.3};
(2) strength reduction factor of 0.6 {UBC 1909.3.4.2}.

Though not based on any conclusive results, the objective of the code changes were to improve the toughness of chords and collectors in diaphragms and attempt to ensure that these elements do not yield in tension [SEAOC (1994)]. Another design change involved the use of an overstrength factor (Ω_o) applied to collector reinforcement transferring the diaphragm reaction to the lateral-system elements [ICBO (1997)]. In that code edition, prescriptive requirements were also introduced which limit the span-to-depth ratio of diaphragms in buildings that contain precast gravity systems to no more than 3:1 {UBC 1921.2.1.7}. This provision is intended to minimize excessive diaphragm deformations under the Design Basis Ground Motion [Ghosh et al (1997)]. Clearly, the writers of these code sections were concerned that diaphragms, especially precast concrete diaphragms, could be the weak link in a structure if not properly designed and detailed [Nakaki (2000)].

The current U.S. code, perhaps in recognition that it does not provide a prescriptive elastic design, contains provisions intended to ensure the existence of some ductility capacity in diaphragm chords and collectors. In addition to the new UBC 1921.6.7.3 described above, the code requires diaphragms with high compressive stresses to possess confined collectors {UBC 1921.6.2.3}. Since collectors for lateral loads in one direction often serve as chords in the orthogonal direction, this requirement often produces confined chords as well. Additionally, since the required length of diaphragm chords is typically longer than those commonly available for reinforcing steel, splices in chords are commonplace; thus UBC 1921.6.7.3 also results in tied chords. In the absence of a rational elastic design, these measures are important, but may not necessarily assure good performance [Nakaki (2000)]. Thus, researchers and practitioners hope to create a cohesive seismic design approach for precast concrete diaphragms (See Section 6.10.2).

6.6.3 Code design forces

6.6.3.1 New Zealand code

In the NZ code, principles of capacity design are appropriate at the ultimate limit state to estimate the magnitude of lateral force including the effects of overstrength. However, some conservatism is justified in the selection of the lateral forces because of the potential for exceptionally large inertia forces at a particular level due to higher mode effects [Bull (1997)]. Such an approach is demonstrated for frames in New Zealand Concrete Society (1994), where a meaningful capacity design diaphragm force is proposed as the product of the design lateral force at a level, F^*_i, and the system overstrength factor Ω_{os}. For typical frame system overstrengths, this roughly translates into a diaphragm force twice the magnitude of the design lateral force at a level.

As an alternative, the N.Z. Concrete Structures Standard [Standards New Zealand (1995)] permits the use of numerical time history methods of analysis in lieu of the capacity design approach. These methods provide direct information on floor accelerations corresponding to appropriate earthquake records.

6.6.3.2 United States code

Currently, US codes use an equivalent lateral force approach for determining diaphragm seismic design loads [ICBO (1997)]. The diaphragm design loads at each level, F_{px}, are specified by a distribution (See Fig. 6-4a) based on the equivalent lateral forces, F_i. The values of F_{px} differ from the values of F_i because the diaphragm design forces need not occur simultaneously; they represent estimates of individual maximum values at each level. The vertical distribution of current code-prescribed diaphragm design forces, F_{px}, produces diaphragm strength that increases with floor level. Design minimum and maximum values based on a percentage of the floor weight [BSSC (2000)] limit these diaphragm forces, for instance the diaphragm force is not to exceed the floor weight amplified by the seismic coefficient and not to be less than half that value [ICBO (1997)]. These diaphragm design forces may require revisiting (See Section 6.10.3).

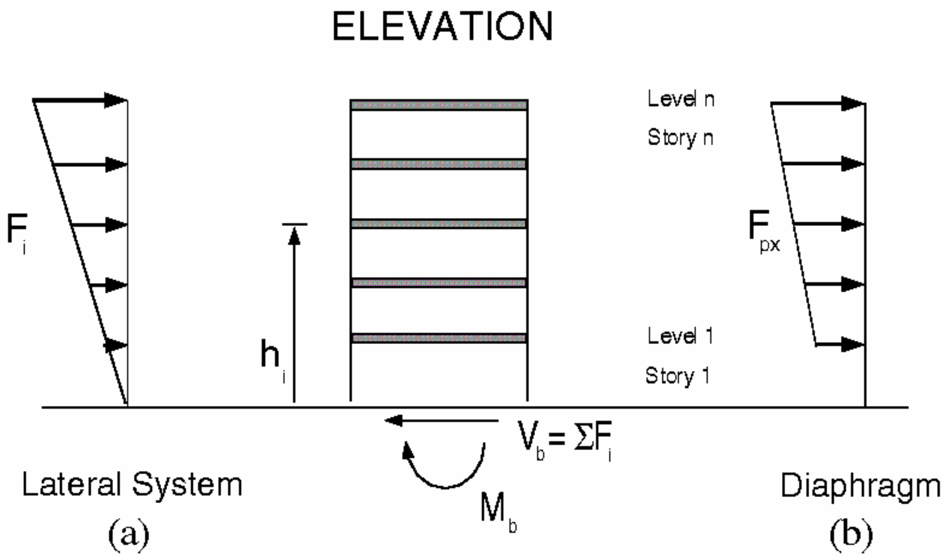

Fig 6-4: Equivalent lateral loads: (a) lateral system; (b) diaphragm

6.6.4 In-plane force analysis method

6.6.4.1 Horizontal plate girder analogy

For regular floor plans, a "horizontal plate girder" analogy is typically used in design [Gates (1981)]. The diaphragm seismic design loads are applied at each level as a distributed in-plane load along the length of the diaphragm as indicated in the floor plan schematic (See Fig. 6-4b). However, attention should be paid to where loads are applied along the depth of the girder, too, as it affects the tensile internal forces (See Section 6.2.2). Lateral system vertical elements serve as the beam supports and the in-plane beam shears and moments are calculated accordingly. In the procedure, it is common practice to design the chord steel to carry the entire calculated in-plane bending moment; and design the web reinforcement to carry the entire in-plane shear across panel joints parallel to the load. Joints in the floor transverse to the loading direction are designed in accordance with current detailing guidelines based on tributary shear, as described in Sec. 3.6.2 of the PCI Handbook [PCI (1999)]. This approach implicitly relies upon elements with plastic redistribution qualities, an important consideration that may have been overlooked in the past and will require attention in the detailing of web reinforcing (See Sec. 6.8.3).

6.6.4.2 Strut and tie method

An alternative and indeed more comprehensive method to the horizontal plate girder approach is the strut and tie method. The method is often employed when the presence of large openings in floor diaphragms may interfere with the simple beam action considered in the deep beam model [Bull (1997)]. For instance, the NZS 3101 [Standards New Zealand (1995)] requires a rational analysis be used to design and detail internal force paths for irregular diaphragm cases, and recommends the strut-and-tie approach. Though the approach is often associated with design of diaphragms with openings or irregular floor plans, it is a statically admissible method for determining possible load paths for regular floor plans (See Fig. 6-5). In fact, several researchers recommend this method in all cases, as it is easy to apply in cases of regular simple diaphragms [Menegotto (2002)]. Details of the strut-and-tie method are given in Appendix A of the Commentary on Section 9, NZS 3101 [Standards New Zealand (1995)]. A comprehensive treatment of the method is found in the classic paper by Schlaich et al (1987).

6.7 Diaphragm reinforcing

6.7.1 General

The presence of joints in the precast floor system reduces the in-plane capacity of the diaphragm associated with full nominal slab thickness. Thus, the critical details typically occur at the joints between precast units. This condition extends to topped precast floor systems because slabs sections over the joints serve as planes of weakness. These sections are often cracked due to service events or intentionally pre-cracked to mitigate shrinkage or temperature effects. The reinforcing crossing these joints is typically some combination of reinforcing bars, mechanical connectors, or welded wire fabric. Steel fibre reinforced concrete (SFRC) can also be used as a structural topping layer.

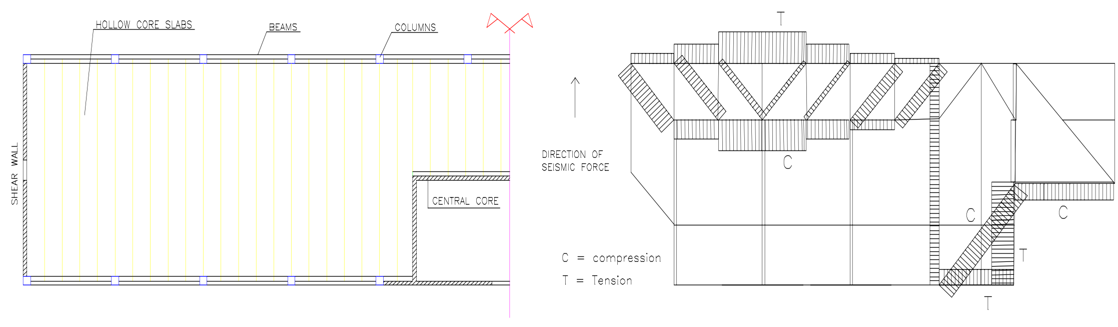

Fig 6-5: Example of strut-and-tie distribution of diaphragm forces in a hollow-core slabs floor with bracing core and walls

6.7.2 Mechanical connectors between precast elements

Pincheira et al (1998) described the load-deformation characteristics of mechanical connectors under shear, tension, and combined loading. These experimental studies showed that mechanical connectors can experience considerable shear capacity reduction if loaded simultaneously in shear and tension (cyclic or monotonic). This information is crucial in the assessment of diaphragm designs since tensile forces are currently ignored in the web reinforcing design, yet certain diaphragm joints attract force combinations of axial, shear and flexure [Fleischman et al (1998)]. Furthermore, the tests indicated tensile yield deformation of typical double tee flange connectors to be less than 1.3mm and failure deformation of approximately 7.5mm. In contrast, the yield deflection value for debonded mild steel crossing a joint between typical precast units is in the order of 6.4mm. As research has shown that compatibility-induced tension forces arise in the web reinforcement as the diaphragm flexural strength is developed [Wood et al (2000), Farrow and Fleischman (2002)], these findings imply that typical mechanical connectors will severely limit the elastic range of the diaphragm. In fact, Nakaki (2000) showed that while the use of closely-spaced mechanical connectors provides similar stiffness to a cast-in-place diaphragm with minimum reinforcing, the deformation capacity of the connectors severely reduces the moment allowable in an elastic diaphragm design. If however, mechanical connectors with tension compliance are used, the chord steel is able to develop fully without distress to the web reinforcing resulting in a diaphragm with both stiffness and strength commensurate with a cast-in-place diaphragm. Such improved mechanical connectors for precast concrete are under development [Oliva (1998)].

6.7.3 Welded wire fabric

Welded wire fabric is often used as diaphragm reinforcing. This reinforcement is intended for crack control and thus does not possess sufficient inherent ductility to meet any seismic demands involving plastic deformation [Herlihy (1999)]. The local ductility demand is increased through good bond, thus a smaller mesh matrix (e.g. 15.2x15.2mm vs. 25.4x25.4mm) will increase the chance of fracture under earthquake loading, particularly since these elements have likely endured shear cycles due to traffic.

6.7.4 Fibre concrete topping

Steel fibre reinforced concrete (SFRC) can be used as a structural topping layer designed to resist seismic actions. Research [Oliver (1998)] has confirmed the enhanced crack control, ductility and energy absorption of SRFC topping using suitable steel fibres. The steel fibre alternative also relaxes limitations on the minimum topping thickness for cover requirements to 50 mm, though other considerations such as electrical conduit embedment may control this dimension.

While SFRC can easily accommodate temperature/shrinkage stresses and can be relied on to transfer diaphragm forces, it is recommended that deformed bars be used in conjunction with steel fibres for support continuity and perimeter tie reinforcement. In cases where seismic flexural, shear or strut and tie actions are in excess of the SFRC capacity, or fire resistance continuity forces dictate, a combination of steel fibre and conventional reinforcement can be the ideal option.

The SFRC topping design strength is based on the post-cracking flexural toughness exhibited by a standard modulus of rupture test SFRC specimen, tested to failure. An "equivalent toughness" flexural strength is calculated from the approximation of the actual load-deflection curve to an idealised elastic-purely plastic curve. Certain fibre manufacturers (such as Bekaert, for Dramix fibres) have proposed mathematical functions for estimating the equivalent flexural strength of SFRC in terms of the fibre aspect ratio and weight dosage.

Seismic moments and axial tensile forces are resisted by an internal post-cracking tensile strength equal to a proportion of the equivalent flexural strength. Bekaert proposed 37% as this proportion, within a limitation of 1% of the ultimate tensile strain. Moment resisting capacities can then be calculated from the compression and post-cracking tensile stress blocks. Seismic shear stresses are resisted by a combination of concrete and steel fibre contributions, where the latter is found to be a function of fibre dosage and aspect ratio as well as the concrete strength.

Research in New Zealand [Oliver (1998)] has shown that the presence of Dramix steel fibres in topping layers over hollow core floor slabs greatly enhanced the seismic performance of the diaphragm at the slab supports. Hooked-end steel fibres, of 80 aspect ratio, at a dosage of 40 kg/m^3 could accommodate a displacement of 45 mm over the supporting beam before rupture.

6.8 Seismic detailing of diaphragms

6.8.1 General

Diaphragms are not generally considered to be part of a structure's primary energy-dissipating system. Ductility demands incurred therefore are likely unintended. However, ductility demands in certain regions of the diaphragm may be difficult to avoid (for instance at strut and tie node points, or adjacent to beam plastic hinge zones). Thus, while precast diaphragms are generally expected to remain within the elastic range during earthquakes, localised areas can require to be detailed for ductility [Bull (1997)]. Furthermore, evidence exists to show that the reliable assurance of global elastic diaphragm behavior may be difficult to attain [Rodriguez et al (2002)], particularly for severe ground motion [Fleischman et al (2001)]. In this case, the formation of an appropriate limit state is warranted to prevent a non-ductile diaphragm failure.

6.8.2 Detailing to accommodate localized displacements

Precast floor units must be detailed to accommodate the localized displacements that are likely to be imposed on them by the actions of the primary lateral load resisting system. The localized actions that can impact on the design of the precast elements include:

(1) Plastic hinge formation in the supporting beams of ductile moment resisting frames and the associated elongation that accompanies this. The formation of plastic hinges in beams will result in elongation because plastic hinge rotation occurs predominantly due to tensile yielding of the longitudinal reinforcement resulting in plastic strains. Longitudinal extensions of beams of the order of 2 to 4 percent of the beam depth per plastic hinge have been observed in tests where expansion was free to occur [Centre of the Advanced Engineering (1999)]. Elongation of beams can cause tearing away of the diaphragms in extreme events.
(2) Gapping joints in hybrid post-tensioned frames with unbonded tendons. The openings of joints will cause elongation of members.
(3) Strut and tie node points, where diaphragm forces pass around floor openings, irregularities in the floor plan, or zones of damage at plastic hinge locations; also sliding joints in vertically jointed wall systems.
(4) Transfer diaphragms where, for example, lateral forces from a tower structure are distributed into a lower level podium with different dynamic characteristics.
(5) Pinned, gravity-load beam connections in structures designed for large lateral displacements (an example is large area parking buildings with a perimeter lateral load resisting system.

Two procedures that can be used to accommodate these actions are:

(1) Isolate the precast components from any high displacement demands, with sliding supports and/or compressible joints. This method is preferred for relatively brittle extruded or slip-formed hollowcore sections, and for some of the more brittle beam and block flooring systems.

(2) Reinforce the precast units, and any composite topping concrete to provide adequate ductility to resist the required gravity loads during, and after, the imposed displacement.

6.8.3 Diaphragm strength limit states

The following basic strength limit states can be identified for precast concrete diaphragms:

(1) failure of collector or anchorage reinforcement at lateral system elements (Sec. 6.8.6);
(2) shear failure of web reinforcement;
(3) tensile failure of web reinforcement; and
(4) flexural failure of chord steel.

Of these, only the chord reinforcing flexural limit state is a desired outcome for the diaphragm in an overload situation as it is ductile, i.e. large inelastic deformations will be accommodated prior to a failure. Thus, an essential detailing requirement for precast concrete diaphragms is the elimination of non-ductile failure modes involving the web reinforcement and anchorage reinforcement.

6.8.3.1 Shear limit state of web reinforcing

For typical web reinforcing, the shear limit state is nonductile, i.e. it will involve local failure of the floor system. Such failure can cause loss of the diaphragm's ability to provide force transfer, and could lead to a progressive collapse of the entire structure. It should thus be avoided.

Wood et al (2000) determined that shear strength design equations based on inclined cracking are inconsistent for precast diaphragms due to the absence of monolithic action in the topping slab, which results in a significant decrease in shear strength. Furthermore, as current design of web reinforcement is based solely on shear transfer, any unaccounted tensile components at the joint may accelerate the occurrence of a diaphragm shear failure. Indeed, pushover analyses of parking structure diaphragms indicated that force combinations can produce failure at load levels below design strength for topped or pretopped diaphragms alike [Farrow and Fleischman (2002)]. However it is equally important to note that analyses of simple diaphragms absent of tensile components indicate that, while the shear limit state will not occur prematurely, it will occur prior to the diaphragm developing its flexural capacity. This result implies that a nonductile diaphragm failure is the likely outcome in an overload condition. For this reason, a capacity design for the web reinforcing relative to the chord reinforcing is suggested (See Section 6.8.4).

6.8.3.2 Flexure limit state of web reinforcing

Tensile deformation demand is imposed on the web reinforcing due to the strain-curvature compatibility associated with in-plane flexure of the diaphragm. These deformation demands concentrate at joints between precast units in high in-plane bending regions, and may be further concentrated at certain joints by the presence of spandrel beams.

As described in Section 6.7.2, the concentrated deformation may limit the allowable diaphragm forces due to the low elastic strain capacity of mechanical connectors. Insomuch as web reinforcing is typically smaller in diameter than the chord reinforcing, and development length in reinforcing bars is proportional to bar diameter, this conclusion likely holds for mild web reinforcing as well [Nakaki (2000)].

Furthermore, if no assurance exists that diaphragm loads will remain below design strength, the web reinforcement must also possess sufficient tensile deformation capacity to meet ductility demand compatible with the achievement of a flexural limit state in the chord reinforcing. In topped systems, the ductility of welded wire fabric will exhaust initially. Recent provisions for the use of 250 mm x 250 mm mesh instead of 150 mm x 150 mm mesh welded wire fabric [Cleland (2001)] improves but does not eliminate this situation. Typical mechanical connectors possess slightly better ductility characteristics and thus will fail at a larger tensile deformation demand. In this regard, while welded-wire fabric will supplement the mechanical connectors to produce a higher shear strength, little is gained in preventing deformation-driven failures as brittle elements in parallel simply fail successively without plastic force redistribution. Thus, mechanical connector tension limit states occur at essentially the same global diaphragm deformation, regardless of the presence of welded wire fabric.

In summary, though this reinforcing is intended simply for shear transfer, failure may occur from exhaustion of its tensile deformation capacity. It can be argued that loss of web reinforcement in the midspan regions of diaphragms has minimal consequence since this region attracts low shear force under transverse lateral loads. However, it should be recognized that lateral loading from any other seismic attack angle than purely transverse can produce significant shear forces in midspan regions of complex diaphragms [Farrow and Fleischman (2002)]. Similarly, these regions of diaphragms in dual systems may face the same vulnerability.

6.8.4 Detailing for web reinforcement

Current design employs the same strength reduction (ϕ) factors for the web reinforcement as the chord steel, a practice that fails to ensure a flexural limit state in an overload condition. A capacity design for the web reinforcement can thus be used to prevent a nonductile shear limit state by providing this reinforcement with sufficient shear capacity to develop the chord steel strength. This outcome can be realized by lowering the web reinforcement ϕ factor with respect to the chord steel ϕ factor. Farrow and Fleischman (2001) propose the use of $\phi = 0.6$ for the web reinforcement (and for the same reasons, the collector and anchorage steel), while restoring the ϕ factor for chord steel to 0.9. This added factor of safety applied to the web reinforcement provided sufficient overstrength to fully develop the flexural strength of precast diaphragms in pushover analyses, provided that tensile deformation compliance is provided as described next.

Wood et al. (2000) indicated that while current design strengths are adequate with respect to shear friction calculations, strain compatibility is not considered. By matching global demands from dynamic analyses with local ductility demands from pushover analyses, Farrow and Fleischman (2001) established that the welded wire fabric did not possess sufficient ductility to meet the maximum tensile deformation demands required of web reinforcement. Thus, it is suggested that, unless elastic design can be assured, the shear friction contribution of welded wire fabric not be included in high in-plane bending regions of the diaphragm.

Summarizing the strain compatibility issue, it seems more important that web reinforcement in high bending regions be detailed for tensile *deformation capacity* than tensile *strength*. Indeed, as mentioned in Section 6.7.2, mechanical connectors with improved tensile deformation compliance sufficient to meet these demands have been under investigation [Oliva (1998)]. Such connectors could be used in the high in-plane bending regions, particularly at column lines where spandrel beams terminate.

6.8.5 Detailing according to strut and tie model

In the strut and tie method, reinforcement must be placed where tension fields exist in the diaphragm and at tension chords along the perimeter. Shear transfer through compression struts is not typically a problem for cast-in-place construction; however care must be taken with topped precast diaphragms [Bull (1997)].

Special detailing is required at the strut and tie node points for precast diaphragms [McSaveney (1997)]. Connections to column tension field nodes may not be maintained in a seismic event because compressive fields cannot be relied upon in instances of large ductility demand. Extra tie bars, placed around the column at slab mid-depth (See Fig. 6-6) thus may be required details if the reinforcement in the topping is not sufficient. To ensure adequate anchorage, these bars should extend beyond the center of a column [Standards New Zealand (1995)]. This reinforcement should be sufficient to generate the total tension force required from the strut-and-tie analysis and able to meet the expected local ductility demands [Bull (1997)]. Beams can be used in the stead of the extra tie bars if they exist on the column lines (as shown by the dotted lines in Fig. 6-6). However, the impact of the additional tension forces must be assessed in terms of the gravity load-carrying capacity of the beams. If the beams exist in only one direction as often occurs, the additional tie steel is required.

Diaphragm node regions may undergo extensive tension yielding, followed by compression on the reversing cycle of the earthquake. Herlihy et al (1996) demonstrated that the topping slab was susceptible to buckling through a combination of reinforcement debonding and delamination from the precast unit. The buckling destroys the ability of the support to receive the force generated by the local compression field, rendering the zone only capable in tension [Bull (1997)]. This behavior may impact the effectiveness of the tie provisions discussed above.

To prevent this buckling, a positive connection is required. Embedment of connectors into mortared or grouted joints between precast elements without engaging some reinforcement in the joint is not considered effective anchorage [Bull (1997)]. McSaveney (1997) recommends an enhanced tie detail for the hollowcore units (See Fig 6-7) and suggests equal applicability of this concept to other precast construction (Refer to Figure 6-3). The term “paper clip” is common in New Zealand, and refers to a few versions of a 2 leg set reinforcing bar.

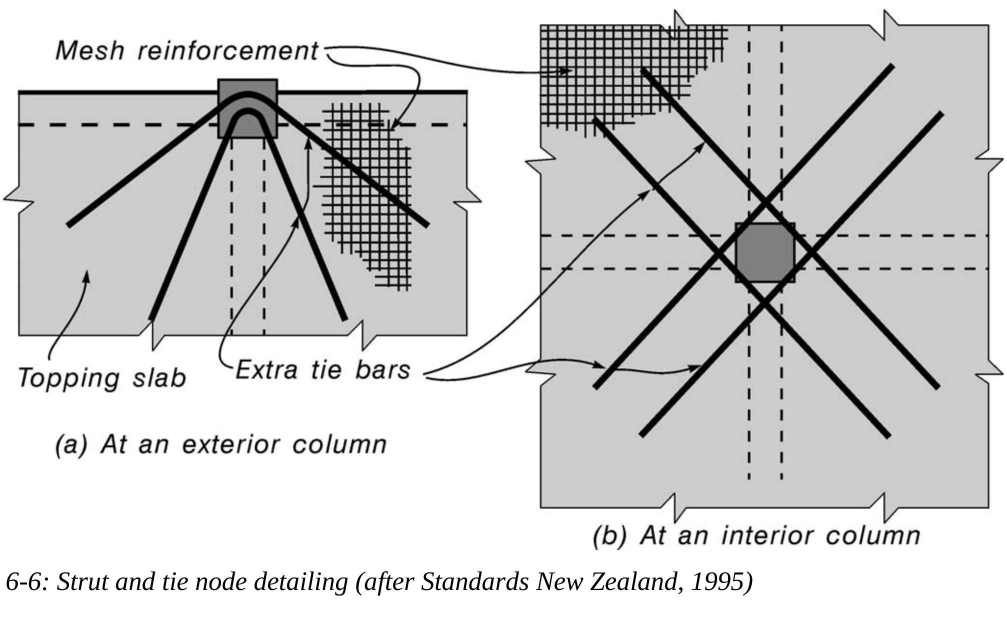

Fig 6-6: Strut and tie node detailing (after Standards New Zealand, 1995)

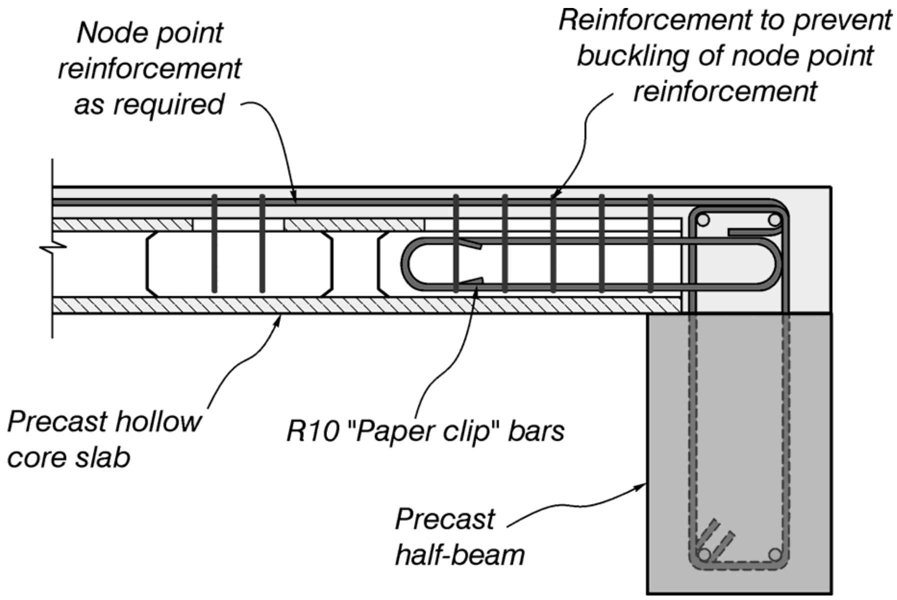

Fig 6-7: Enhanced tie detail for hollowcore units (after McSaveney, 1997)

6.8.6 Force transfer to vertical system

Regardless of the adequacy of detailing in the diaphragm itself, the transfer of lateral loads to the vertical elements of the lateral system must be maintained during the earthquake. This task involves the proper identification of force flow to the lateral system elements, sufficient anchorage elements and provisions for collector (or drag) reinforcement to allow sufficient development of the forces. Poor seismic performance of structures in several earthquakes has been attributed to inadequate anchoring of the diaphragm to the lateral system vertical elements [Bull (1997)]. U.S. design codes have recently included an overstrength factor applied in the design of collector reinforcing transferring the diaphragm reaction to the lateral-system elements (Section 6.6.2.3)

Often the regions of the floor slab near the vertical elements of the lateral load resisting system undergo localized damage due to plastic hinge formation. Thus, connections to the lateral system often prove most critical. This reinforcement must be adequately anchored both in the lateral system element and the diaphragm, and lapped with the main diaphragm reinforcement [Bull (1997)]. However, it is essential to understand that anchoring the topping alone is not proper anchorage (See Section 6.4.4), and the interaction of the topping with the entire precast unit or tie beams included in the floor system is of great importance to this end [Menegotto (2002)].

The NZ Concrete Structures Standard [Standards New Zealand (1995)] “structural integrity” requirements, similar to ACI 16.5 [ACI (1998)], are intended to ensure that minimum reinforcement is available to maintain integrity under the most adverse conditions. This requirement includes minimum reinforcement to tie walls and columns to the diaphragm to avoid tearing away of the diaphragms under extreme events (See Section 6.8.2).

6.8.7 Transfer diaphragms

Two common building situations require transfer diaphragms:

(1) a tower on a podium or a core structure in which the story shear near the base is transferred to a perimeter structural or foundation system; and

(2) a dual system.

Transfer diaphragms tend to generate exceptionally large forces because of the actions of one primary lateral load resisting system relative to another. This condition typically necessitates the use of diaphragms that are much thicker than simple diaphragms resisting inertia force alone.

For precast construction, significant seismic-induced shear forces must be transferred by the cast-in-place topping. Some doubt exists as to the effectiveness of bond in preventing the separation of the topping from the precast units in regions of high stress [Bull (1997)]. Accordingly, shear stress in excess of 0.3 $\sqrt{f'_c}$ (MPa) requires a positive connection to prevent buckling of the topping slab (See Sec. 6.8.5). In cases, the shear stresses may be too large to permit the use of precast floor units. Even in the presence of a topping slab, all connections between precast units, and to vertical elements of the lateral system, must be designed in consideration of these large forces. For this reason, precast concrete transfer diaphragms are not usually recommended [Bull (1997)]. In any case, special attention is required for transfer diaphragms, and precast concrete floor systems should not be used without a special study including the use of finite element analysis to model diaphragm stiffness and load development.

6.9 Diaphragm stiffness considerations

6.9.1 General

Building structures are typically designed using the assumption that the floor systems serve as rigid diaphragms between the vertical elements of the lateral force resisting system. Thus, gravity system columns are expected to undergo the same drifts as the vertical elements of the lateral force resisting system. A rigid diaphragm treatment is also used to determine the distribution of forces in plan to elements of the lateral force resisting system.

6.9.2 Flexible diaphragms

Diaphragm flexibility can produce large gravity system drifts (See Section 6.10.4) Diaphragm flexibility may also significantly affect the participation of certain vertical lateral force resisting elements in the portion of the total lateral force. These diaphragm flexibility issues may be an important design consideration for precast concrete structures because:

(1) precast construction is used effectively for long floor-span structures with isolated or perimeter lateral-system elements. These structures may possess flexible diaphragms by virtue of their configuration; and
(2) precast floor systems possess less inherent in-plane stiffness than monolithic cast-in-place decks.

6.9.3 Stiffness of precast diaphragms

The stiffness of the precast diaphragm is highly dependent on the reinforcing details crossing the joints [Nakaki (2000)]. Deformation compliance in the panel joints of precast diaphragms contributes to a decrease in both flexural and shear rigidity. Shear deformation within pre-cracked joints is non-negligible, accounting for up to 20% of the lateral displacement in topped diaphragms; and up to 50% in pretopped diaphragms [Farrow and Fleischman (2002)]. Thus, shear deformation contribution in precast diaphragms should be included in stiffness evaluations. Due to the absence of shear friction behavior, pretopped diaphragms are significantly more flexible than topped diaphragms at service level loads.

Equivalent strength designs for typical spans produce pretopped diaphragms that are approximately twice as flexible as topped counterparts [Gates (1981)].

6.9.4 Diaphragm stiffness calculations

Code editions classify diaphragm as flexible when their deformation is more than twice the interstory drift of the associated story [ICBO (1997)]. In this case, the design is to account for diaphragm flexibility. To evaluate flexible diaphragm structures, a deflection calculation is required. Nakaki (2000) provides effective stiffness calculations for precast (cracked joint) diaphragms that includes shear deformation. A gross section reduction for cracked joint behavior based solely on moment-curvature relationships is used, with shear stiffness reduced in the same proportion. Farrow and Fleischman (2002) provide a design calculation based on bending span coefficients and effective moduli, E_{eff} and G_{eff}, which produce reasonable bending and shear rigidity values for typical diaphragm thickness.

6.9.5 Current U.S. code requirements for seismic drift

Drift limits contained in the U.S. codes recognize both elastic and inelastic building deflections {UBC 1630.9.1}[ICBO (1997)]. However, it is currently unclear whether the calculated deflections are to include the diaphragm. Code requirements state that the structural model shall include all elements of the lateral-force-resisting system and represent the spatial distribution of the mass and stiffness. Elastic deflection calculations are also to include the effects of cracked-section behavior {UBC 1630.1.2}. Thus, the code seems to imply that the diaphragm deformations should be part of the elastic drift assessment [Nakaki (2000)].

For inelastic drift, the code specifies a Maximum Inelastic Response Displacement. This inelastic value is obtained by scaling the elastic deflection calculation by 70% of the force reduction factor, R. The inclusion of diaphragm deformation in the elastic drift calculation now seems in conflict since the code R value is related only to the expected ductility of the vertical elements of the lateral system [Nakaki (2000)]. As it is unlikely that the diaphragm is intended to possess similar ductility capacity as the lateral system, the code's handling of the diaphragm contribution to maximum inelastic response displacement seems inconsistent.

6.10 Proposed seismic design approaches for precast diaphragms

6.10.1 General

There has been considerable discussion recently regarding the approach to seismic design of diaphragms. Elastic response has traditionally been the preferred behavior for diaphragms [ACI (1992)]. The preference originates from recognition of the relationship between efficient diaphragm action and in-plane stiffness of the floor slab [Chopra (1995)]. However, for seismic design, the need for elastic diaphragms stems primarily from a desire to avoid non-ductile failure in the floor system, since this region of the structure is not typically provided with special detailing. Clearly, designs in which the diaphragm acts as the structure's weak link should be avoided [Wood et al (2000)] since the capacity reduction coefficients used in the seismic design are based on the expected ductility and energy dissipation of the lateral load-resisting system and thus will not be valid for a building with inelastic diaphragms [Nakaki (2000)].

Prescriptive elastic diaphragm behavior, therefore, seems an essential part of any seismic design code. However, the U.S. codes may not meet this objective [Nakaki (2000)]. In fact, studies indicate that current designs do not necessarily prevent inelastic diaphragm action in high seismic zones [Fleischman et al (2001)]. The consequences of such action can be severe as evidenced in the collapses of several parking structures during the Northridge earthquake [EERI (1994)]. Investigations of these failures indicated that a combination of insufficient diaphragm strength [Wood et al (1995)] and diaphragm stiffness [Fleischman et al (1996)] likely led to the collapses. In light of the role that inelastic diaphragm response is perceived to have played in poor structural performance, practitioners have advocated modifications to current codes to produce prescriptive elastic diaphragm designs [Ghosh (1999)].

A capacity design approach seems the most appropriate way to achieve the elastic diaphragm design [Nakaki (2000)]. The intent of a capacity design is to prevent nonductile behavior by designing a brittle component, not to an expected force, but instead relative to the strength of another (ductile) portion of the structure [Paulay and Priestley (1992)]. One could imagine using the equivalent lateral force (ELF) pattern to design the diaphragm stronger than the vertical elements of the seismic system, thus relying on yielding of the vertical seismic system to limit system response. However, evidence exists to show that the design code ELF values may in cases significantly underestimate diaphragm loads [Rodriguez et al (2002)] [Fleischman et al (2001)]. For wall structures in particular, extreme force events in the diaphragm are driven by modifications to the structure's dynamic properties after base hinge formation [Fleischman and Farrow (2001)]. As a result, even a capacity design that successfully initiates hinging in the shear wall is no guarantee of sustained elastic diaphragm behavior throughout the seismic event. Because of these issues, achieving a reliable capacity design is not as straightforward as it might appear.

Accordingly, two proposed approaches are presented for seismic design of diaphragms:

(1) an elastic design, and
(2) a design incorporating performance criteria.

The first approach is aimed at eliminating diaphragm detailing requirements; the latter approach is based on the opinion that diaphragm detailing is still necessary. Regardless, each approach supports the need for diaphragm design strength increases in the codes.

6.10.2 Proposed alternative approaches to precast diaphragm design

6.10.2.1 Elastic diaphragm design

Nakaki (2000) proposed an elastic diaphragm design procedure. The procedure designs against inelasticity in both the chord and web reinforcing, actions considered unacceptable due to incompatibility with behavior implicitly assumed in the design code. To obtain elastic behavior, the procedure employs a capacity design with respect to the lateral system by:

(1) including a system overstrength factor, $\Omega_0 = 2.8$, in the determination of the diaphragm design forces; and,
(2) designing to the diaphragm yield moment rather than an ultimate state.

Elastic limits are established through strain compatibility relationships using a strength reduction factor, $\phi = 0.9$. An elastic force design alternative is suggested for squat wall systems due to the difficulty in determining the overstrength. In this alternative, the diaphragm is designed with an R factor of unity.

The procedure also explicitly incorporates diaphragm flexibility effects through:

(1) the use of a period modified for diaphragm deformation; and
(2) the inclusion of elastic diaphragm deformation in the maximum inelastic response displacement calculation.

To eliminate inconsistency in the code handling of the diaphragm contribution, the diaphragm elastic deformation is not scaled by 0.7R in determining the diaphragm stiffness satisfying the inelastic drift limit. Stiffness-critical designs were found to occur, even for aspect ratios less than 3:1. Accordingly, diaphragm reinforcing quantities are controlled in the procedure by either the strength or stiffness requirement.

The approach intends to allow relaxation of prescriptive detailing requirements for the diaphragm by:

(1) eliminating the need to confine concrete in the compression chord;
(2) preventing chord reinforcing from buckling on load reversal;
(3) improving bar splice performance, and
(4) limiting inelastic diaphragm deformation.

6.10.2.2 Diaphragm design using performance criteria

Fleischman et al. (2001) focused on wall structures with long span simple diaphragms and proposed a design procedure based on diaphragm performance requirements. Analytical studies indicated that current diaphragm design strengths do not necessarily assure elastic diaphragm behavior for the Design Basis Ground Motion. While the studies indicated that only modest (20-60%) increases in current design strength were needed at this seismic hazard level, a 300% increase was required to assure elastic behavior in a maximum considered earthquake (MCE). Given that the diaphragm is a portion of the structure not typically considered for special detailing costs, such strength increases did not seem economically viable. Furthermore, elastic diaphragm designs were found to be no guarantee of adequate drift performance. For this reason, the design approach based on performance requirements was proposed.

Acceptance criteria for drift and damage were developed using FEMA-273 expected performance at life-safety and collapse-prevention structural performance levels (NEHRP, 1997). Accordingly, the following performance requirements were proposed:

(1) gravity-system elements remote to the lateral-system are to remain within life-safety drift limits for the design basis ground motion and collapse-prevention drift limits for a MCE; and
(2) elastic diaphragm behavior is targeted for the design basis ground motion while exhaustion of the diaphragm's probable deformation capacity is permitted in a MCE.

On the basis of time-history analyses, it was determined that an appropriate diaphragm design force level can be determined by selecting the larger of the values producing elastic design-basis behavior and acceptable collapse-prevention drift performance. The authors showed that diaphragms below a certain absolute flexibility are controlled by the former, while diaphragms with greater flexibility are controlled by the latter. Design equations were developed providing overstrength factors as a function of building height and diaphragm flexibility. These factors are to be used with constant diaphragm force patterns [Fleischman and Farrow (2001)]. For many configurations, the recent change in precast diaphragm strength reduction factor was found to be sufficient.

Clearly, this approach also requires the accommodation of inelastic diaphragm behavior during a MCE event. Accordingly, the overall diaphragm design includes provisions for estimating expected ductility demand. These demands are estimated for diaphragms controlled by flexural limit states, and thus the designs are only effective if provisions are made to prevent non-ductile limit states in the diaphragm (See Sec. 6.8.4).

6.10.3 Diaphragm design seismic loads

An aspect of diaphragm behavior not elucidated in the codes and thus possibly not clearly understood is the relationship of the design forces used in ELF procedures to the inertial forces that may actually occur in floor diaphragms during a seismic event. It has been traditionally considered that inertia forces in diaphragms are related to the magnitude and distribution of the lateral forces for designing the lateral force resisting system. However, evidence exists to show that the design code force values may in cases significantly under-estimate diaphragm loads. For instance, recorded maximum floor accelerations for 25 multistory buildings during the Northridge earthquake reached 4.6 times the peak ground acceleration [Hall (1994)]. Underestimation of these loads can lead to inelastic diaphragm behavior [Fleischman et al (1998)]. For this reason, various researchers have promoted the use of overstrength factors in the determination of diaphragm design loads [Nakaki (2000), Rodriguez et al (2002), Fleischman et al (2001)].

Rodriguez and Restrepo investigated the magnitude and distribution in height of absolute floor accelerations responsible for the development of inertia forces in the diaphragm [Rodriguez et al (2002)]. On the basis of a series of non-linear time-history analyses, it was concluded that:

(1) current building codes generally underestimate the magnitude of the inertia forces;
(2) floor accelerations are not greatly affected by the system's displacement ductility demand; and,
(3) ductility demand in the system primarily affects those actions contributed by the first translational mode of vibration.

To rationalize the findings of their research work, Rodriguez et al. (2002) proposed the First-mode reduced method (FMR) for evaluating inertia forces for the design of diaphragms. In this method, the absolute acceleration of the top most floor, A_n, is obtained from a modal combination rule in which only the spectral acceleration corresponding to the first mode of vibration, S_{a1}, is reduced by an R factor. They proposed the following equation for estimating A_n,

$$A_n = \sqrt{\left[\Gamma_1 \;\; \phi_n^1 \;\; \frac{S_{a1}}{R_1}\right]^2 + \sum_{q=2}^{r} \left[\Gamma_q \;\; \phi_n^q \; S_{aq}\right]^2} \qquad (6\text{-}1)$$

in which Γ_q is the participation factor for mode q, $\phi_n{}^q$ is the amplitude of mode q at level n, S_{aq} is the spectral acceleration corresponding to the translational mode q. The R factor, given by the coefficient R_1, accounts for the effect of ductility in the first mode of response. R_1 is also modified to account for the overstrength of the structure.

These researchers observed that absolute floor accelerations are not distributed similarly to the lateral forces for the design of the primary lateral force resisting system. A simplified version of the FMR for determining the diaphragm design forces, F_{di}, uses the design code spectrum. The diaphragm design force is then given by,

$$F_{di} = \lambda \; C_{di} \, W_{di} \qquad (6\text{-}2)$$

where factor λ takes into account the seismic hazard of the area and structure importance and C_{di} and W_{di} are the diaphragm's seismic design coefficient and the diaphragm's seismic weight, respectively. Coefficient C_{di} is given by,

$$C_{di} = \Omega_i \; C_{hg} \qquad (6\text{-}3)$$

where C_{hg} is the peak ground acceleration (in terms of acceleration of gravity) and Ω_i is a floor magnification factor based on vertical location of the floor and the influence of the higher modes of response.

The method was compared with non-linear dynamic time-history analyses of twelve-story buildings with emulative precast concrete and unbonded post-tensioned wall systems. In general it can be found that the FRM approach can predict the floor accelerations, and therefore the floor inertia forces with reasonable accuracy.

Fleischman and Farrow (2001) examined the load distribution for flexible diaphragm structures. Though diaphragm flexibility significantly affects the elastic force distribution and dynamic properties of both wall and frame structures, the greatest dissimilarity with the ELF pattern was found in inelastic wall structures due primarily to the dynamic response of shear walls possessing a base plastic hinge. These variant patterns have significance for diaphragm design because force magnitudes in wall structures are particularly sensitive to the vertical distribution of seismic load [Blakeley et al (1975)]. Principally, the base shear V_b required to develop the lateral-system's flexural strength M_b is inversely proportional to the effective centroid of the seismic load, H^*. For a wall structure following the formation of a base plastic hinge, instantaneous effective centroids consistently occur well below the centroid implied by the ELF pattern. This downward shift is due to the nature of higher modes in the instantaneous deformation state [Eberhard and Sozen (1993)]. The lower centroid indicates larger base shear and in turn larger diaphragm forces. Thus, the potential exists for larger than anticipated diaphragm forces during seismic response, including less frequent but extreme force events. The potential for these extreme forces is greater in lower levels of the structure, in opposition to current design distributions. The maximum diaphragm force amplification was found to increase with number of stories.

It should be noted that the high diaphragm forces are not necessarily unique to precast systems. However, the heavy demands produced from these conditions can be particularly problematic for paneled systems in which diaphragm forces must be carried across joints between precast units. Extreme diaphragm force values do not seem to develop in frame structures. However, diaphragm force distributions in frame structures do possess similar or slightly higher diaphragm forces in lower levels of the structure.

6.10.4 Flexible diaphragms

Fleischman et al. (2001) studied perimeter lateral-system structures with flexible diaphragms and determined that drift, deformation and force demands are higher in lower regions of the structure, particularly for wall lateral systems. Inelastic diaphragm response can exacerbate the effects of diaphragm flexibility including drift profiles that possess a severe concentration in the first story. Based on comparisons with expected behavior, it was concluded that perimeter lateral-system structures with flexible diaphragms can produce inadequate seismic performance due to: (a) exceedence of acceptable drift limits; or (b) large unintended ductility demand on the diaphragm. It was concluded that the interrelation of diaphragm flexibility and diaphragm strength is important in describing the seismic performance of these structures.

The following design recommendations are made for flexible diaphragm structures in high seismic regions:

(1) a constant strength pattern based on the top-level lateral force;

(2) an overstrength factor Ω based on diaphragm flexibility and story height for wall structures. For frame lateral systems, $\Omega = 1$ seems adequate if the first criterion is met;

(3) the inclusion of the elastic contribution of the diaphragm in lateral-systems that are drift controlled, as is typical of frame structures; and,

(4) certain configurations, comparable to the longest span parking structures of recent construction, should not be built as no design strength increase exists to realize adequate drift performance.

Results for zones of moderate seismicity indicated that current diaphragm design strength levels are most likely adequate and the longest span parking structures are likely viable.

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7 Gravity load resisting systems

7.1 Introduction

This chapter covers precast concrete systems which are subject to sway but do not provide significant horizontal load resistance. These elements and subsystems carry gravity loads and/or are part of the cladding and partitioning systems. Examples include:

- Precast gravity load carrying columns
- Precast gravity load carrying beams
- External precast concrete wall panels and spandrels
- Internal precast concrete partitions that may or may not carry gravity loads.

These elements and subsystems require to be designed and detailed so that they retain their gravity load carrying capacity during and after major seismic induced lateral deformation of the main horizontal load resisting structure by either:

- Remaining in the elastic range when deflected laterally
- Deforming in a ductile manner when deflected laterally in the post-elastic range
- Being separated from the structure so that they do not deflect into the post-elastic range.

As with cast-in-place construction, precast elements that do not form part of the primary horizontal loading resisting structure should not unduly influence the seismic response of the structure. Thus important considerations in their design must include relative stiffnesses, structural separation, continuity and capacity design.

The chapter covers in general terms design considerations, and discusses some specific details as illustrations of considerations that require to be taken into account in design and detailing.

7.2 Continuity

7.2.1 General requirements

Continuity of floor systems is often necessary to maintain diaphragm action of slabs so that they can transfer floor loads to the horizontal load resisting structural systems. Otherwise continuity of precast beams over supports and precast floors over supporting beams is usually only necessary to restrain members from sliding off seatings and to avoid opening of joints to the extent that architectural finishes are damaged. Adequate shear capacity to transfer dead and live load to support structures may also require continuity.

Flexural continuity of precast slab systems across supports is also desirable in order to avoid abrupt localised changes in slope which may result in excessively wide cracks in insitu toppings at precast slab element supports.

7.2.2 Beams

Where structural floors or toppings to precast floors are cast-in-place it is normal practice for the cast-in-place concrete to be cast continuously over beams. The precast beams will generally be cast to the level of the soffit of the slab. Stirrups will thus protrude from the top of the precast beam units. It is normal practice to add longitudinal top steel to these beams when placed and to cast the stirrups and top steel in with the slab pour. Thus, continuity of beams across supports, and slabs across the beams is assured. The shear capacity of precast beams prior to casting the toppings should be carefully considered especially in relation to anchorage of stirrups. Fig. 7-1 shows a typical precast beam section with cast-in-place concrete providing the top section of the beam and slab topping.

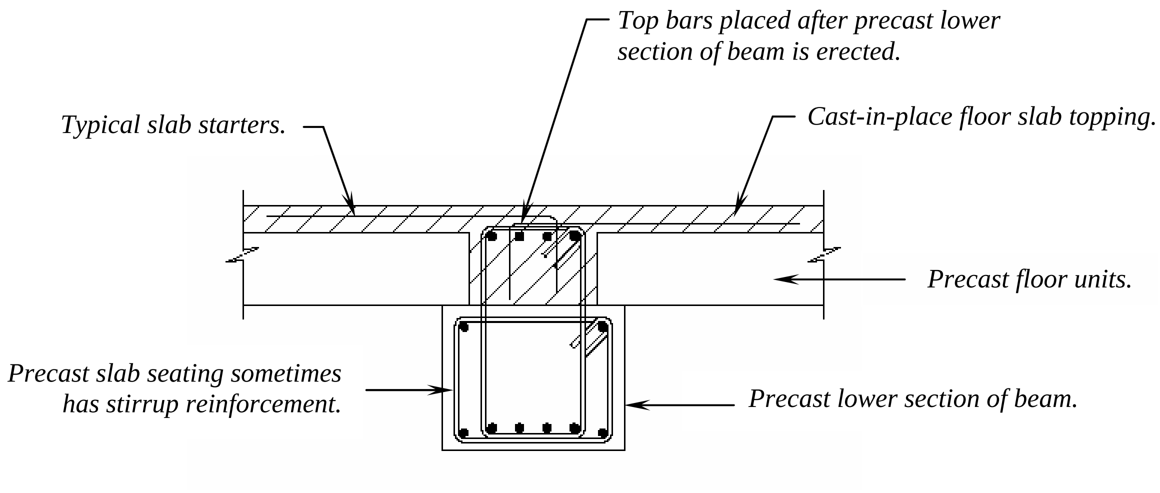

Fig. 7-1: Typical precast beam with cast-in place slab topping

With precast beams, particularly those bearing on corbels or other discontinuous shear connections, normal practice is to anchor the bottom beam reinforcement by turning it up at the ends of the beams. Where precast beams are placed on precast or cast-in-place column edges outside the column reinforcing cages it has been found that the appropriate detailing is to leave the bottom steel in the beams projecting so that it can be cast into the cast-in-place beam-column joint. It is generally accepted that the protruding bottom steel does not require a full tension anchorage into the beam-column joint. In some circumstances the bars have been purposefully debonded so that they act only as dowels in shear, rather than providing continuity into the beam-column joint of the bottom steel. It is anticipated that when the structures sway the high rotations occurring at the beam column joint will fail the beam bottom bars in bond in the joint. Provided the precast beams and floor systems have adequate continuity so that they cannot slide off their bearing area on the columns this local bond failure is not considered a problem.

It is further suggested that precast gravity load resisting beams are better not fully continuous with the supporting columns, particularly slender columns, resisting gravity loads only. The moment transferred from beams into these columns should be kept as low as possible. Such columns are likely to be under large axial compression and thus moments in the columns from sway mechanisms should be avoided or at least reduced as much as possible.

Designers need to consider carefully the moments and shears being transferred into columns from precast beams, and how these actions are transferred, to ensure that unpredictable failure mechanisms in the gravity load bearing system under earthquake sway conditions are avoided. In all cases the criteria of capacity design should be considered.

7.3 Beam supports

7.3.1 General types

There are many types of supports for precast concrete beams. The support systems fall into two main categories. Beams supported directly on columns or within beam column joints are one category which can be extended to beams supported by walls. The second category is beams supported on other beams.

7.3.2 General requirements

Supports must be designed so that the integrity of the load path from the precast member through the support to the parent structure is maintained. Particular care needs to be taken to ensure that shear at the end of precast beams is appropriately transferred, particularly where the transfer mechanism does not directly engage the soffit of the supported beam.

Most precast beam supports do not provide longitudinal continuity as cast-in-place construction normally does. Therefore the effective rotation of the support relative to the beam end needs to be addressed in the design and detailing of the interfaces. For example, horizontal shear stresses induced as a beam rotates and potentially slides across a horizontal bearing surface can produce longitudinal tension, particularly in corbels, which needs to be designed for.

The assumption that rotation is free to take place in some details may not be correct and again secondary stresses set up due to relative rotation should be a consideration in the design. Details that “lock up” as a result of accumulated debris during construction or in service also need to be considered in design.

7.3.3 Corbels

The design of corbel type beam supports are adequately covered in most National Codes. Less well covered are support details similar to corbels which incorporate structural steel inserts such as short lengths of structural steel sections protruding from the beam end. Refer to Fig. 7-2 for a typical detail of such a corbel. These types of details usually require a transfer mechanism of shear from the steel insert to the level of the bottom reinforcing in the beam. The localised shear (hanging load) can be high, which requires careful detailing. The overall design of the structural steel inserts considers them acting as short beams in their own right, and ensuring that the reactions for the short beams are adequately dealt with within the precast beam end region.

7.3.4 Rocking and sliding supports

Most precast beam supports are designed as sliding where relative movement of the support and the precast element is required. In particular circumstances rocking supports are required. Stability of the rocking supports over the entire range of movement both in the serviceability and ultimate limit states must be considered. Sliding supports do not normally present a stability problem.

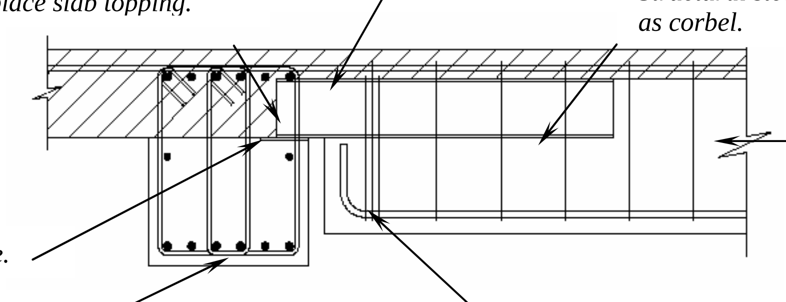

Fig.7-2: Typical structural steel corbel detail for precast beams

7.3.5 Bearing

Localised bearing stresses adjacent to the contact areas in bearing type supports require consideration. If high bearing stresses are required consideration can be given to increased local concrete strengths, or the use of inserted steel plates, or providing confinement in the bearing area, or other ways of enhancing bearing strength. Bearing details that are located within cover concrete should be avoided unless the bearing and shear stresses in the cover concrete are low. Supplementary reinforcement within the cover concrete can be used to enhance the integrity of the bearing area.

7.3.6 Eccentricity and torsion

Care should be taken in the design of precast concrete supports which include eccentricity and/or torsional effects resulting from known design actions or random effects resulting from earthquake sway mechanisms. Significant torsional effects result from unsymmetrical loading on beams, particularly edge beams. Precast concrete floor systems combined with precast concrete beams with no continuity in their support from the primary structure, are of particular concern. Analysis will show that certain configurations require positive detailing to ensure that beams cannot rotate sideways off their supports.

Falling into this category of potential problems are beams that are curved in plan. These require special design considerations both within the precast element and at the supports. Corbel type supports are usually unsuitable for this type of loading. End continuity of the beams to the primary structure may be the only effective way of restraint.

Similarly the effect of unsymmetrical high bearing loads from precast concrete elements on primary structural members must also be considered. This is particularly the case where secondary gravity load beams are supported by primary beam elements of the structure using noncontinuous support details.

7.4 Column connections

7.4.1 General

Precast columns that do not participate in the lateral load resisting mechanism of a structure do not usually require significant continuity with floors and other gravity load bearing elements. The only loads likely to cause tension in columns are generated by vertical accelerations. There is little evidence from earthquake damage of gravity load bearing elements loosing vertical continuity with columns as a result of vertical acceleration. In some connection systems for precast concrete columns some minor instantaneous separation can be tolerated as the vertical deflections will be small and hence the structures supported by the columns will return to their original location.

Column deflection compatibility within a storey is achieved through the floor diaphragms, and hence the forces associated with the compatibility deformation must be taken account of when designing the connections between the floor diaphragms and precast columns. This is covered in more detail in Chapter 6.

7.4.2 Load paths

The load paths through column to beam and column to slab connections should be as direct as possible. Connections through weld plates and other devices that do not act in direct bearing should be avoided.

The ideal load path is by direct bearing across the entire end area of the column. Undersized bearing areas that leave part of the column end area unloaded should be avoided accept in specially designed “hinged” type connections.

7.4.3 End region rotation

An advantage of some precast column end connections is that they are practically hinges, and act as such when that structure undergoes earthquake induced sway.

An example is the connection detail where column bases are finished square and are located in the beam-column or slab-column joint by a single or pair of short dowels. Such a detail is shown in Fig. 7-3. The dowels serve only to locate the column base and transfer some shear. The holes for the dowels are usually drilled in the beam or slab concrete prior to erection of the column which helps to locate the precast concrete column accurately. The end of the column bears on a bedding of mortar. This type of connection attracts very little moment and thus when the structure sways it acts as a restricted hinge where the flexural strength of the column is limited by the end geometry of the column and not by the flexural capacity of the column section.

Rocking connections as described above need to be designed to ensure that the localised compression in the end region of the column is controlled. The rotation will be governed by the inter-storey drift under earthquake loading. Thus when large drifts occur some spalling and crushing of the column concrete at the interface with the bedding can be expected. To control this the longitudinal reinforcing in the precast concrete should finish as close as possible to the end of the member and lateral ties or confinement should start as close as practicable to the end. Excessive cover should be avoided. In some cases special spirals or similar may be required at the ends of the precast members. The end region of the column may need to be designed to lose all the cover concrete.

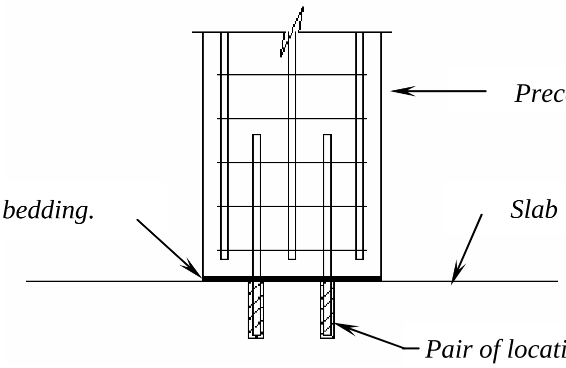

Fig. 7-3: Typical dowel located precast concrete column base detail

Special care needs to be exercised where columns bear on thin slab toppings where beams are the same width as the column. When the structure sways transverse to the axis of the beam consideration must be given to the integrity of the sides of the beam under the high local column stresses.

7.4.4 Shear

Shear transfer between the ends of precast columns and the bearing area of slabs or beams must be considered in design of the connections. To achieve true capacity design so that shear failure at the connection (or indeed in the column) cannot occur under sway, the maximum possible moment that can be reasonably induced in both ends of the column needs to be considered. The shear resulting from dividing the sum of the top and bottom moments factored by appropriate over strength factors, by the clear length of the column, must be used unless it can be shown that a lesser shear is only possible because of the flexibility of the column. Column flexibility should be conservatively calculated in this case. Realistic post-elastic inter-storey drift at or close to the ultimate limit state will govern this calculated required shear capacity.

All precast elements must be designed and detailed to avoid shear failure just as cast-in-place elements are designed and detailed. Capacity design criteria will facilitate this.

7.4.5 Flexure

The required flexural capacity of precast columns will usually be governed by the capacity of the end connections. The potential to form plastic hinges in precast columns should be avoided wherever possible.

Like all columns P-delta considerations and accidental eccentricity must be considered in design. Beams may apply eccentric loads because they bear on column edges in precast construction.

7.4.6 End bearing

Compatibility of concrete strength between the precast columns and beams or slabs that form the bearing area is desirable although some variation is acceptable. Because of larger plan area than the column, biaxial stress conditions, and other such considerations, bearing areas may have lower strength concrete than the precast column.

It is important that the stress condition in the load path through beams and slabs is direct and taken into account in their design and detailing.

7.4.7 Typical bedding details

As discussed in Section 7.4.3, a common bedding detail for precast columns consists of one or two location dowels and a bedding mortar under the column butt end.

Another detail consists of bedding columns into preformed pockets, again onto a bedding of mortar. This detail allows very positive shear transfer at the base of the column provided the socket is suitably reinforced to resist transverse load.

Details using rebar connectors and mechanical splices are available. These systems tend to produce load transfer similar to fully cast-in-place construction.

Two types of precast column base details are shown in Fig. 7-4.

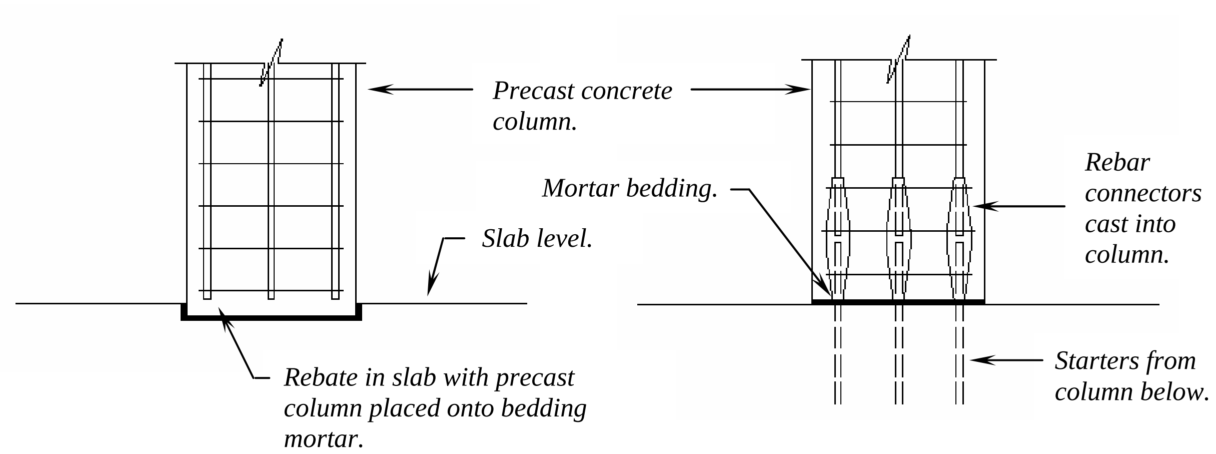

Fig. 7-4: Types of precast column base detail

Precast columns located prior to forming cast-in-place joints in beams have been successfully used but care must be taken to ensure concrete cast under the column end is properly vibrated to eliminate trapped air. Also shrinkage needs to be avoided and the accumulation of concrete latence.

7.4.8 Hinges

Artificial concrete hinges are possible at precast connections. When detailing these hinges limited shear transfer is still necessary and localised high bearing stresses must be considered.

7.5 Walls

7.5.1 General

This section discusses both internal and external walls which do not provide significant horizontal load resistance. Such walls subdivide into external precast concrete wall panels and spandrels, and internal precast concrete partitions. Walls of this latter type are used relatively infrequently. Both the internal and external wall panels may provide gravity load carrying capacity.

Precast structural concrete walls can be used as the main horizontal load resisting structure but are discussed elsewhere in Section 5.

7.5.2 Separation

Both gravity and nongravity load bearing walls must be detailed to ensure that they do not inadvertently participate in the horizontal load resistance of the structure. If they do then the wall may well be overstressed even at relatively low horizontal loads, or it may substantially alter the predicted horizontal load resistance of the main structure to such an extent that premature failure in various members can occur. There is little difference between external and internal precast concrete walls and partitions in their design and detailing. Part height walls such as spandrel panels and concrete walls to window sill level must be separated from the adjacent columns so that the free length of the column is not shortened by the presence of a stiff part-height panel. Lack of separation in the plane of the wall will lead to two serious effects during horizontal loading of the main structure. The spandrels or part height walls will suffer premature damage, and adjacent columns may be subject to unrealistically high shear stresses thus precipitating premature shear failure.

Full height walls must also be separated from adjacent columns as partial failure of these walls can induce high shear stresses in columns precipitating shear failures. Similarly the walls are likely to be subjected to high shear stresses and suffer critical damage during horizontal loading.

Thus there are two fundamental criteria for separation of part height and full height walls. They must be separated to avoid participation in horizontal load resistance of the structure, and they must be detailed to avoid shortening the shear span of adjacent columns.

In structures which are stiff, for example shear wall structures, separation of precast walls not designed as part of the horizontal load resistance is not so much a critical issue. Designers need to consider the elastic and post-elastic deformation of the structure as a whole and then consider the effect of these deformations on any individual nonseismic load resisting wall element.

7.5.3 Gravity load resisting walls

Base details of gravity load resisting walls can be dowelled, cast into slots, or connected by mechanical connectors of various descriptions. It is unusual to provide sliding connections at the base of a wall. However there are some occasions when such details are preferable to sliding connections at the top. Appropriate detailing is required.

Connections of tops of walls to the structure above fall into two categories. The first is gravity load bearing connections, and the second is non-gravity load bearing connections.

Gravity load bearing connections can be sliding or fixed. Sliding connections are often required where the main structure is relatively flexible and the wall element is a stiff element. A sliding connection for loads in the plane of the wall is therefore necessary. Fixing the wall rigidly or semi-rigidly to the structure for loads at right angles to the plane of the wall is permissible. Walls are usually very flexible at right angles to their plane and thus do not participate significantly in the horizontal load resistance of the structure in that direction. Even walls of considerable length are usually of insignificant stiffness compared to the main horizontal load resisting structure. A commonly required detail is for sliding in the plane of the wall and fixing out of the plane of the wall. There are various proprietary fixing details but in nearly all cases shear-sliding connections in the plane of the wall are required. This has been achieved in the past by various low friction inserts, flexible rubber bearings, or horizontally flexible set-in items. A common detail is a plate set across the axis of the wall so that the wall is relatively rigidly fixed across its axis but the plate is free to flex along the axis.

Various proprietary precast wall systems resisting gravity loads have been used. One of the most famous is the slotted wall which reduces the longitudinal stiffness of the wall dramatically so that it acts as numerous relatively thin columns both longitudinally and laterally. Particular architectural detailing is required to allow this wall to be incorporated into structures.

7.5.4 Walls not required to resist gravity loads

Detailing of walls not required to resist gravity loads is similar to gravity load resisting walls except the top details are much simpler. Wall bases are generally grouted or connected with mechanical fixings. Top details require only resistance to out of plane loads. Thus provided the top detailing stops the wall from out of plane movement no particular consideration needs to be given to in-plane restraint. In fact in-plane restraint is undesirable and should be avoided. All the arguments associated with separating the wall from horizontal load resistance are similar to those for gravity load resisting walls.

Part height walls and spandrels are often fixed at their four corners only, and sometimes along the base for out of plane loads. Such walls fixed to columns should be by individual connections which allow horizontal sliding. In the design of these connections consideration should be given to the effect of corrosion which, overtime, locks up some mechanical fixings.

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8 Foundations

8.1 Introduction

This chapter briefly treats two topics concerning foundations of particular interest to designers of precast concrete buildings and foundations. The topics covered are socket foundations and prestressed precast concrete piles.

8.2 Socket foundations

8.2.1 General

Use of the socket base connection is the simplest way to connect precast concrete columns to foundation beams. This type of connection is often used for precast concrete low-rise buildings, bridge piers and other light facilities like promenades with roofs supported by precast concrete columns. Fig. 8-1 shows an example of application of such socket base connection. The connection is grouted at the column is placed.

8.2.2 Design of the socket connection

In Germany, calculation methods for the steel sectional area required for reinforcement of embedded column base foundations are specified in DIN 1045. [Deutscher Beton-Verein E.V. (1981)] In the ACI Building code [ACI 318 (2002)] general design and construction concepts are mentioned. In Japan, there is no standard for such type of connection design, but recommendations can be found for the design and fabrication of tubular structures in steel [AIJ, 1990]. Thus, the socket base connection is used all over the world but the design and construction practices are normally based on the engineering judgment and common sense rather than the code specifications.

When the column axial force, N, and horizontal force, P, due to gravity load and earthquake or wind load act on the precast concrete column, reactions due to overturning moment and shear force are generated at the connection part as shown in Fig. 8-2 [Osanai et al. (1996)]. In the figure, N and R notate the axial load and the reaction force acting on the column, respectively. Also, F_1, F_2, F_3, C, and C_{22} are the interface frictions and horizontal reactions induced by N and P, respectively. The key points for the design of the socket base connection are to provide sufficient stiffness and strength, comparable to the monolithically constructed connection. The experimental study by Osanai et al. [Osanai et al. (1996)] led to the conclusion that the socket base connection behaved as rigid as a monolithic one when the embedment depth of precast column was equal to or more than 1.5 times the column section depth, D. Also concluded in the study was that, when adequate shear keys were provided on the column and socket surfaces, the connection could achieve satisfactory performance comparable to the monolithic one even in the case of the embedment depth of precast column of 1.0 times D. When the foundation beam depth can be reduced by the use of shear keys with the reduction of the required embedment length of the precast column, construction cost can be significantly saved. This is because the cost of ground excavation for the additional depth of 0.5 D without using shear keys is normally much higher than the cost for providing shear keys. The results of this study were applied to several cases of low-rise buildings and facilities constructed by Oriental Construction Co. Ltd. in Japan.

Fig. 8-1: *Application of the socket base connection in New Zealand (Courtesy of J Restrepo)*

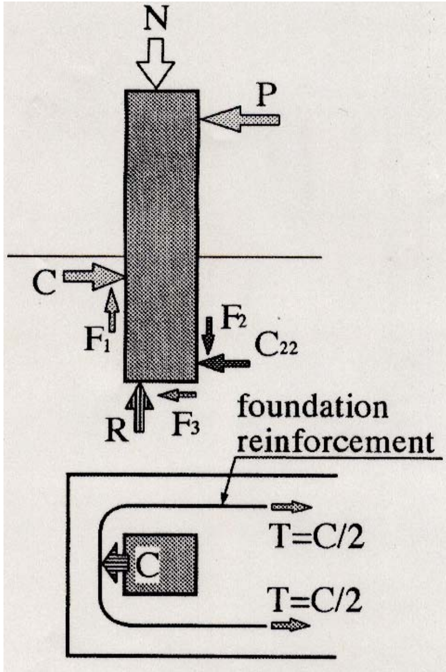

Fig. 8-2: *Actions in the socket [Osanai et al. 1996]*

The reason why the socket type connection is not normally used for high-rise buildings is that extra countermeasure needs to be provided when high axial tension due to overturning moment is induced in the precast column. One of the solution for this problem is the use of anchorage bolts shown in Fig. 8-3 [Okamoto (2000)]. Thus, the socket base connection can be used for high-rise buildings when anchorage devices are provided.

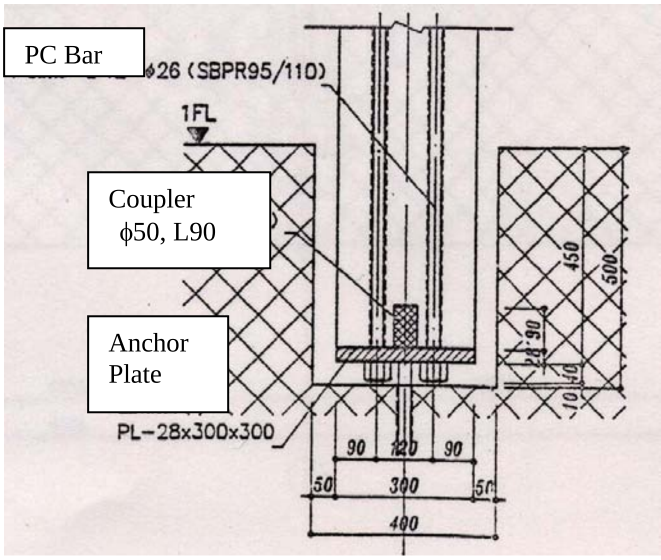

Fig. 8-3: *Anchorage of column with high axial tension [Okamoto, 2000]*

Fig. 8-4: *Undamaged socket foundation (Kocaeli earthquake, Turkey 1999)*

After the Kocaeli Earthquake in Turkey in 1999, damage investigation was conducted for a warehouse with precast concrete frame which was under construction. No damage was found at the socket base connections while yielding or failure of precast concrete columns were observed as seen in Fig. 8-4.

8.3 Pile foundations

8.3.1 Seismic design of prestressed/precast concrete piles

Prestressed precast concrete piles are sometimes preferred to reinforced concrete piles because of such advantages as

(1) high durability due to the use of high-strength concrete,
(2) easiness of handling, transportation and pitching,
(3) ability to take hard driving and to penetrate hard soil layers,
(4) high load carrying capacity for moment with or without axial load and
(5) suitability to produce longer length of pile, etc.

However, when the seismic design of piles is based on the ductile behaviour concept, prestressed/precast concrete piles may need to be designed more carefully than ordinary reinforced concrete piles.

Damage of piles observed in recent earthquakes, particularly in the 1995 Kobe Earthquake, indicated the necessity of an innovative pile design which takes account of the distribution of lateral displacements of a pile along the soil layers in addition to the seismic loads transferred from upper structures to the pile cap [Tokimatsu and Asaka, 1998]. Under strong earthquakes, foundation piles embedded in soft and variable soil strata may undergo significant lateral displacements. Based on the observations made in the past of earthquake damage to piles, Ben C. Gerwick, Jr. stated that the lateral displacement of the pile might attain 500mm or more at the pile cap [Gerwick,1982]. Typical damage to pretensioned spun concrete piles in the 1995 Kobe earthquake can be seen in Fig. 8-5. Damage at the middle portion of the pile and the pile tip was found in various locations by an integrity sonic test or television observation [Tokimatsu et al. 1996, Ohoka et al. 1996].

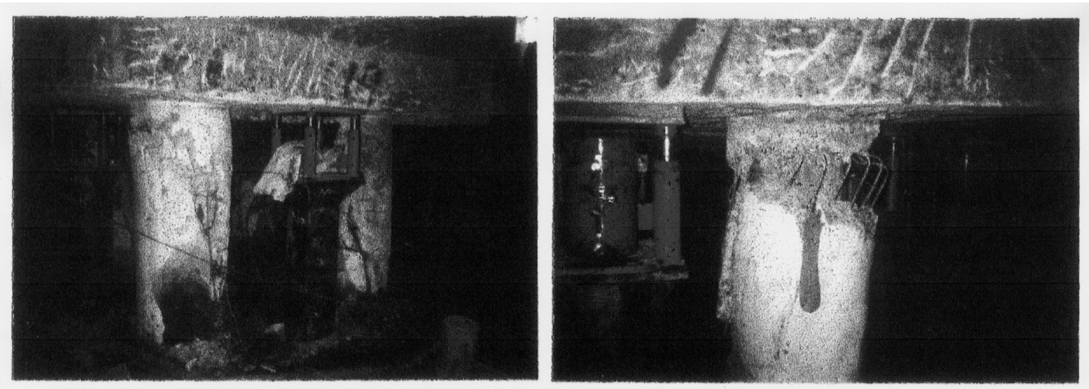

Fig. 8-5: Damage of pretensioned spun concrete piles in 1995 Kobe earthquake

When large seismic lateral forces and overturning moments act at the pile cap, failure of pile may occur mainly near the pile cap as shown in Fig. 8-6(a). At the same time, a large relative lateral displacement may take place between the ground surface and the bearing stratum when soft and/or liquefied soil layers existing. As a result, the pile may fail not only at the pile cap but also near the pile tip just above the bearing stratum as shown in Fig. 8-6(b). Therefore, piles need to be designed from the top to the bottom all through their length and thus reinforcing details must be determined. The methods to estimate the distribution of lateral displacements along a pile have been developed in several studies [Unno et al. (1978), Nishimura (1978)].

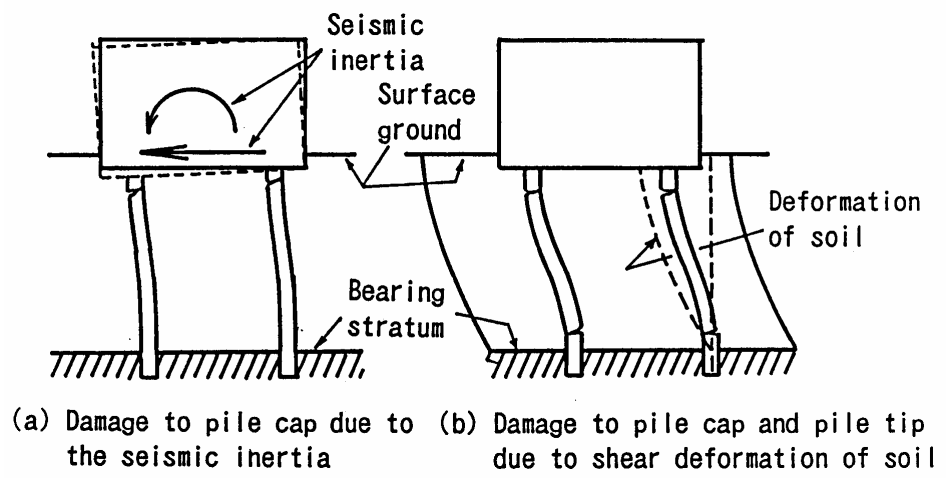

Fig. 8-6: Schematic figure showing damage to pile

8.3.2 Comparison between steam cured and autoclaved piles

Pretensioned prestressed concrete piles in Japan have been produced by the unit mould method with spinning compaction of concrete. After compaction of concrete, the pile is steam-cured under the maximum temperature of 75° Celsius. Then, the prestressing force is introduced by releasing the end anchorage of prestressing steel bars and then the steel mould is removed. These steam-cured piles normally have concrete compressive strength of about 50 MPa. To achieve a higher strength of concrete, such as 80 MPa, autoclaved-curing method with temperature of 180° to 200° Celsius is also used for spun concrete piles. However, the elastic modulus and fatigue strength of concrete are significantly reduced in autoclave-cured concrete piles in comparison with steam-cured ones [Muguruma (1997,1982)]. This is because microcracks are formed when the piles are quickly cooled down immediately after the autoclave-curing.

It is also noted that significant prestress loss takes place during the autoclave-curing process. This is because excessively large shrinkage of concrete and large relaxation in prestressing steel occur during autoclave-curing under the high temperature of 180° Celsius [Muguruma (1982, 1997)]. Results obtained from trial tests on prestress transfer conducted by Muguruma showed that it was hard to introduce a prestress of 14 MPa or more into an autoclave-cured pile. Also shown was that excessive loss in prestress could be prevented when steam-cured high-strength concrete was used because of lower curing maximum temperature, such as 75° Celsius.

In recent years, it has become unusual to produce piles by post-tensioning method because of the extra work required for grouting. However, the post-tensioning method has the excellent advantage that precast pile units can be connected easily with satisfactory stiffness.

The greatest advantage of introducing larger prestress is the increase of the ultimate flexural strength of the pile. For example, the ultimate flexural moment of a pile with an effective prestress of 20.9 MPa attained was twice that of a pile prestressed at 8.4 MPa in the tests conducted by Muguruma [Muguruma (1982, 1997)]. Also shown in the tests was that the initial flexural cracking and ultimate flexural moments increased linearly with an increase of effective prestress.

8.3.3 Flexural ductility enhancement of piles

It is usual that for prestressed concrete members to have a smaller ductility than ordinary reinforced concrete members. Therefore, ductility enhancement is a key point when designing prestressed concrete members in a seismic area. The basic factors to obtain high ductility of prestressed concrete members are

(1) to prevent shear failure,
(2) to increase the ductility of the concrete in the compression zone and
(3) to increase the steel strain at the peak stress of prestressing bar or strand by improving steel quality.

Past experimental studies on the ductility enhancement of prestressed concrete members indicated that the use of high yield strength lateral confining reinforcement having a yield strength of 600 to 800 MPa will result in a very stable inelastic deformation after peak load, particularly in high-strength concrete members [Muguruma et al (1979, 1982, 1990)]. Muguruma et al. showed that ductility of a pretensioned pile with hollow section can also be enhanced by confinement using high yield strength lateral reinforcement [Muguruma et al. (1987)]. However, in the case of tubular piles, confining effects of spiral or circular reinforcement are significantly reduced when the thickness of tube wall is considerably small compared with the diameter of the pile [Kohashi et al. (1999)]. Hence, large ductility cannot be always expected for tubular piles even if a large amount of transverse reinforcement is provided.

When a pile reaches a flexural deformation with large ductility by providing adequate lateral confinement, fracture of prestressing steel eventually occurs. The flexural tests by Muguruma et al [Muguruma et al., 1987] have shown the importance of having an adequate maximum strain in uniform elongation of the prestressing steel, which corresponds to the strain at peak stress of prestessing steel. Based on the test observation, it was concluded that the maximum strain in uniform elongation of the prestressing steel was the available limit of prestressing steel strain in ductile design for prestressed concrete members and the piles. Thus, flexural ductility of piles may be enhanced by improving the characteristics of steel, that is, by increasing the maximum strain in uniform elongation of steel.

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9 Modelling and analytical methods

9.1 Introduction

As discussed in the previous chapters, recent developments in the research of precast/prestressed concrete structures for seismic areas have resulted in the experimental validation of different innovative typologies of ductile connections for moment resisting frame and wall systems, either in accordance with or alternative to the “emulation of cast-in-place concrete” approach, typically required by major current design codes. As a result, a wide range of alternative arrangements for connections of precast structural members is available, in the form of a continuous set of solutions from monolithic to pure precast jointed connections.

Significant differences in the behaviour at both local and global level are expected, depending on the adopted solutions for the longitudinal reinforcement (prestressed, post-tensioned, mild steel or combination of the above), on the characteristics of bond conditions (from fully bonded to fully unbonded) as well as on other structural details in the anchorage solutions (mechanical, lap splice, end-hooks).

In this chapter the efficiency and accuracy of alternative modelling approaches and analytical methods in representing (in either a design or assessment phase) the seismic behaviour of different precast/prestressed connections/systems will be critically discussed.

Within an overview of alternative solutions, at different levels of complexity, particular attention will be given to simplified approaches, based on section analysis procedure and lumped plasticity models, which have been shown, though extensive analytical-experimental comparative investigations, to represent viable tools for reliable prediction of the response of subassemblies as well as of whole frame or wall systems.

In particular, a procedure for the definition of the monotonic moment-rotation curve for connections with unbonded reinforcement (e.g. jointed ductile or hybrid solutions, see Section 3.2), which violate strain compatibility between steel and concrete, will be presented. Based on a member compatibility concept, the method relies on an analogy with equivalent cast-in-place solutions, named “monolithic-beam-analogy”. The cyclic behaviour of a general hybrid connection can ultimately be defined as an adequate combination of self-centering or energy dissipation contributions, modeled with rotational springs in parallel with appropriate hysteresis properties. Similar approach is suggested to be extended and adopted for prestressed connections/systems with fully bonded reinforcement.

It is worth underlining that the scope of this chapter is to discuss critical aspects and issues in modelling and analysing the seismic behaviour of precast/prestressed concrete buildings, addressing the need of simplified and viable tools to deal with the aforementioned wide range of solutions/arrangements available at the moment. Well-known concepts related to material behaviour or section analysis modelling typical of ordinary cast-in-place concrete (thus suitable for those solutions based on the “emulative” approach) will be only partially covered in an appendix of this chapter, while adequate references to other specific contributions available in literature will be provided.

An overview of alternative modelling approaches (macro-models, fibre or lumped plasticity approaches) and analytical methods (linear elastic or inelastic, static or dynamics) is given. Issues related to recently raised issues, indeed not peculiar of precast/prestressed systems but also expected in cast-in-place solutions, as a) beam elongation effects, b) diaphragms flexibility and inelastic behaviour as well as displacement compatibility with lateral-load resisting systems will be briefly addressed.

9.2 Alternative connection solutions

9.2.1 General

A classification of alternative types of connections between precast concrete elements in moment resisting frames and structural walls has been given in Section 3.2, while Chapter 5 has provided a more detailed discussion on the expected response characteristics of lateral load resisting systems using different connection arrangements.

As mentioned, a fundamental difference between "emulative" monolithic and "jointed" ductile connections/systems response is related to the modality of accommodating the inelastic demand. In jointed ductile solutions, precast elements are typically connected through unbonded post-tensioning techniques; the inelastic demand is accommodated within the connection itself (beam-column, column to foundation or wall-to-foundation critical interface), while reduced level of damage, when compared to equivalent cast-in-place solution, is expected in the structural precast elements, which are basically maintained in the elastic range.

The "emulative" approach has been widely adopted in New Zealand construction practice [Park (1990), Centre for Advanced Engineering (1991), Restrepo (1993)] and, later on, broadly developed in Japan [Priestley (1992); Watanabe (2000)]. Typical arrangements for precast concrete beam-column subassemblies according to the emulation approach are shown in Fig. 9-1.

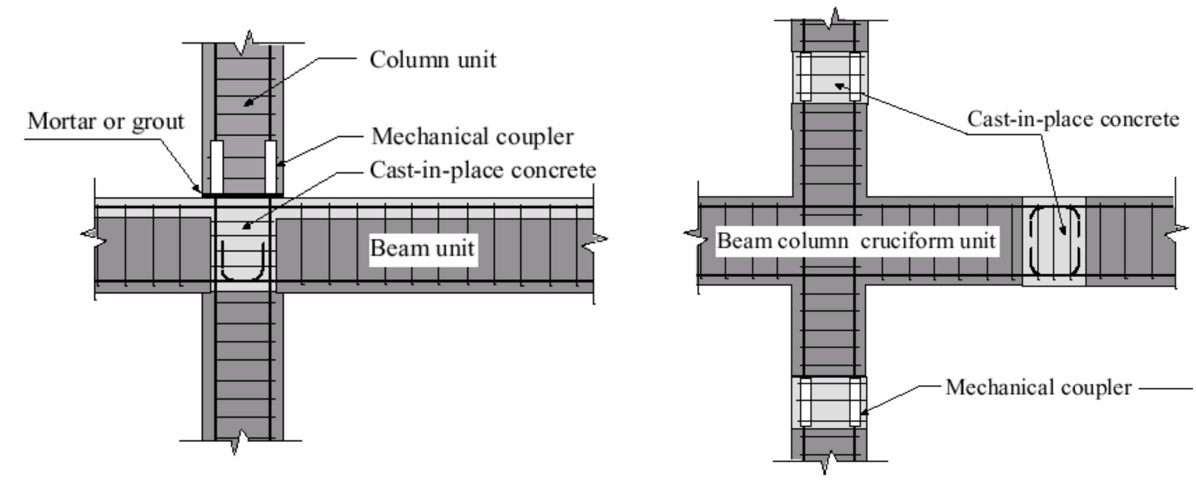
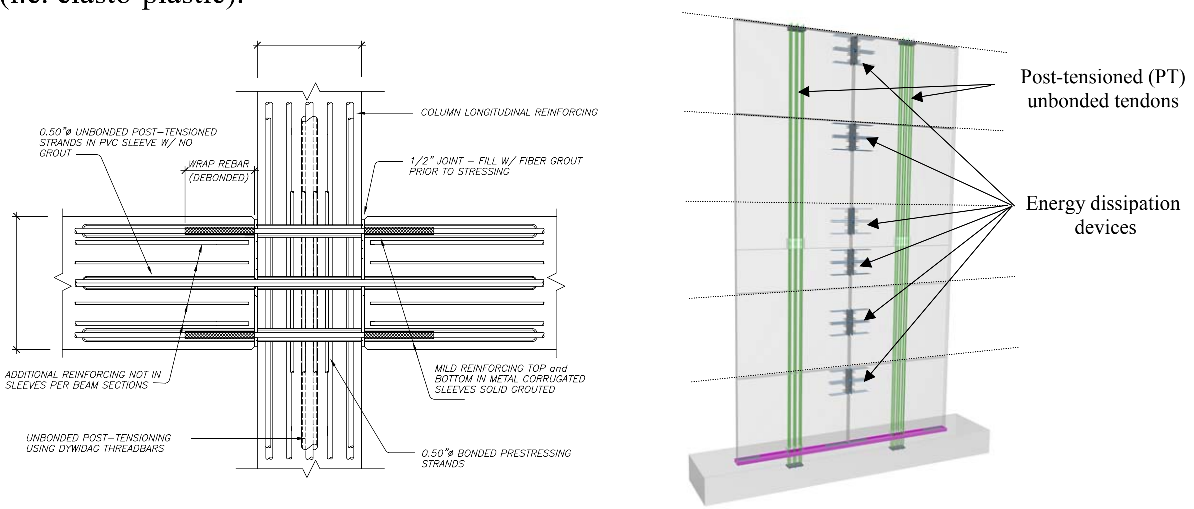


Fig. 9-1: Typical arrangements of precast units and cast-in-place concrete [after Watanabe (2000)]

9.2.2 Hybrid systems

Within the alternative solutions to the emulative approach, a particularly promising efficiency and high flexibility have been shown by the hybrid systems [Stanton (1997)], where unbonded post-tensioning tendons/bars with self-centering properties [Priestley and Tao (1993)] are adequately combined with longitudinal mild steel (e.g. beam,-column connection in frame systems) or additional dissipation devices (e.g. vertical connectors in jointed wall systems), which can provide an appreciable energy dissipation.

Typical hybrid solutions adopted in the PRESSS Program [Priestley *et al.* (1999)] are shown in Fig. 9-2 for a beam-column subassembly and a wall system, respectively. The inelastic demand is concentrated at the critical interface through opening and closing of an existing gap at the interface. A sort of "controlled rocking" motion of the beam or wall panel occurs, while the relative ratio of post-tensioning and mild steel governs the hysteretic "flag-shape" behaviour. Fig. 9-3 shows the ideal hysteretic behaviour of a hybrid system as a combination of a self-centering Non-Linear-Elastic (NLE) rule and an energy-dissipating rule (i.e. elasto-plastic).



a) b)

Fig. 9-2: Hybrid systems developed under the PRESSS program for:
a) frame system [Nakaki et al. (1999)]
b) all system (courtesy of Mrs. Nakaki)

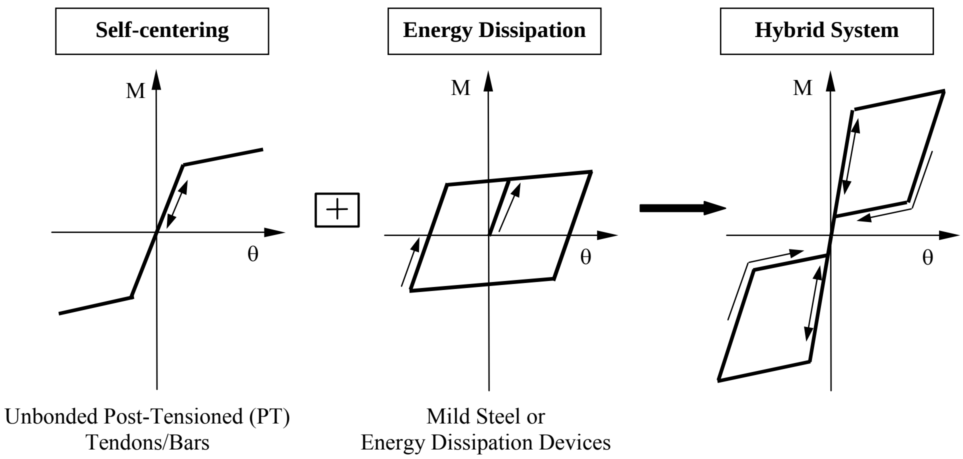


Fig. 9-3: Idealized flag-shape hysteretic rule for an hybrid system

Provided an adequate amount of energy dissipation capacity is given to the system, the seismic behaviour of the hybrid systems (whose concept has recently been extended to steel moment resisting frame, [Christopoulos et al. (2001)]) has been shown to be at least as satisfactory as equivalent monolithic solutions [Pampanin *et al.* (2000); Christopoulos *et al.* (2002)]. Furthermore, recent investigations within a framework for a performance-based seismic design and assessment approach have emphasized the role of residual deformation in the assessment of structural (and non-structural) damage, underlinig the positive effects of the inherent self-centering capacity of an hybrid system. [Pampanin et al. (2002)].

As anticipated, in the following paragraphs particular attention will be given to critical issues related to the modelling of jointed ductile systems, assuming that the behaviour of equivalent monolithic systems can be, by definition, satisfactorily described using well-known traditional approaches available in literature for cast-in-place reinforced concrete [CEB-FIP, Model Code 90 (1992); *fib* "Structural Concrete" (1999)].

9.3 Alternative modelling approaches

9.3.1 Bonded or unbonded conditions

The modelling of precast frame and wall connections/systems significantly depends on the bond conditions adopted for the longitudinal reinforcement. Recent analytical-experimental studies presented in literature on prestressed beam-column subassemblies [Adachi and Nishiyama (2000)] confirmed the large influence of bond property on the load capacity as well as on the hysteresis loop characteristics. Fully bonded or fully unbonded reinforcement would represent the lower and upper limits of a general case, where bond deterioration due to high level of inelastic cyclic demand is expected. When dealing with unbonded post-tensioning tendons, partially ungrouted longitudinal mild steel bars, or combination of the above (as typical of the hybrid system proposed in the PRESSS program and shown in Fig. 9-2) relatively substantial modifications to the modelling procedure are required when respect to the "bonded" case, at both local and global level. In the critical section, the strain compatibility assumption, typically required for a section analysis approach, is violated. As a result, traditional section analysis method, well known for characterizing monolithic or precast bonded concrete members, can not be directly applicable to these systems.

In the following paragraphs, an overview of alternative analytical approaches, at different levels of complexity, to characterise the behaviour of precast/prestressed connections/systems will be given, referring to the most general case of a connection where the "unbonded" concept is utilized (i.e. either jointed ductile or hybrid solutions).

9.3.2 Fibre element model

A finite element model based on fibre elements should be able to directly incorporate global relationship (equilibrium, compatibility) involving single members or the whole specimen, thus accurately predicting the behaviour of either bonded or unbonded connections/systems. The reliability of fibre element models in modelling the behaviour of monolithic reinforced concrete members under cyclic loading, including bar-slip phenomena due to bond deterioration has been extensively demonstrated and recognized in literature [Monti and Spacone (2000)].

Recently, analytical investigations on the seismic response of jointed unbonded post-tensioned frames and wall systems using fibre element model have been presented by El-Sheikh at el. (1999) and Kurama et al. (1999). Typical interior precast concrete unbonded post-tensioned beam-column subassembly and wall system are illustrated in Fig. 9-4 with the corresponding fibre models. The unbonded length of the precast elements (beams or wall panels) close to the critical interface, where inelastic demand is expected to concentrate, is modelled with a fibre element, while elastic one-dimensional elements are utilized to model the elastic regions of the precast members. Truss elements model the Post-Tensioned (PT) steel.

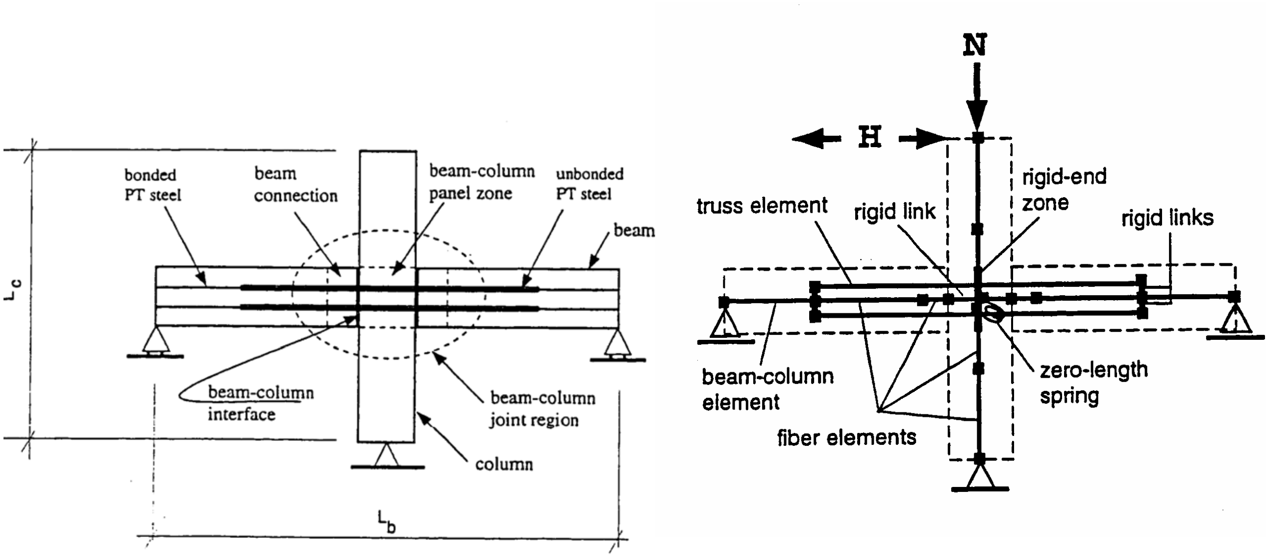


Fig. 9-4: Partially unbonded post-tensioned beam-column subassembly: scheme and fibre element model [El-Sheikh et al., (1999)]

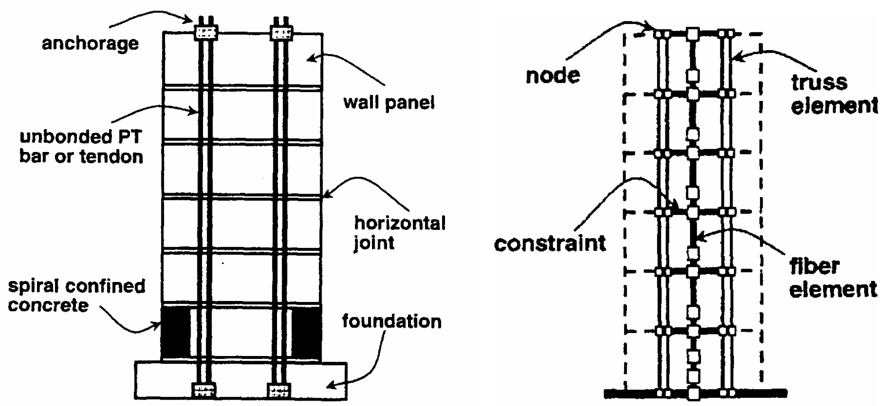


Fig. 9 5: Unbonded wall system: scheme and fibre element model [Kurama et al., (1999)]

In the beam-column subassembly, panel zone shear deformations are modeled by zero-length spring element, while rigid links and rigid ends are used to model the panel zone flexural deformations. Rigid links connect the truss element end zones to the adjacent fibre nodes at the locations of the PT steel anchorages. The Post-Tensioned steel tendons/bars are modelled with truss elements whose end nodes are slaved to the adjacent concrete nodes through rigid links. This solution should guarantee member compatibility between the concrete and the steel deformations, pending correct assumptions on the behaviour of the opening and closing of the gap at the critical interface. In fact, the Bernoulli-Navier hypothesis of "plane sections remain plane" could no longer be valid in the region close to the interface section. This local behaviour cannot be taken into account by the fibre model, which assumes a linear strain variation over the cross-section height. On the other hand, the opening of the gap can be captured as an accumulation of tensile deformations over a certain length of the beam. By integrating, at each level of the section, the concrete fibre tensile strains over the unbonded length, the width of the gap can be evaluated.

Analytical-experimental comparison of fibre models for unbonded connections has in general shown quite satisfactory results [El-Sheikh et al. (1999); Kurama et al. (1999)], although some inaccuracies in the prediction of ultimate deflection, yielding point and hysteretic energy dissipation, underline the necessity for further investigations and refinements to the model parameters. If alternative simplified approach were available, as those presented in the following paragraph and suggested as more viable tools, the complexity of a fibre element would no longer be justified.

9.3.3 Lumped plasticity model: moment-curvature/rotation

9.3.3.1 General

A lumped plasticity model relies on the assumption that the main inelastic demand is accommodated within discrete critical sections (i.e. beam-column, column-foundation or wall-foundation interfaces). While in monolithic connections (thus precast/prestressed emulating the cast-in-place concrete solutions), flexural cracking is expected to develop into the structural elements, the peculiar mechanisms of precast jointed systems guarantee a concentration of the inelastic demand at the interface while the structural members remain essentially elastic and suffer limited damage. In the latter case, thus, the seismic response of jointed precast/prestressed frame or wall systems can be accurately predicted using one-dimensional beam elements, representing the members, with concentrated inelastic behaviour at the critical sections (i.e. beam-to-column or column-to-foundation interfaces).

Due to the opening and closing of a single crack at the interface, an infinite curvature is developed at the critical section: therefore a moment-rotation relationship has to be preferred to a traditional moment-curvature when characterizing the section behaviour. Rotational inelastic springs, with appropriate hysteretic behaviour, can be assigned to represent the inelastic action at the beam-column interface, while elastic elements are used to represent the structural members.

9.3.3.2 Combination of hysteretic rules in parallel

The concept of hybrid system, so far related to jointed precast concrete solutions, can be suggested to be extended to describe the behaviour of prestressed concrete connections. In general, in fact, an hybrid system can consist of either a combination of unbonded post-tensioned steel and mild steel or a combination of prestressed steel and mild steel.

In the late 1970’s Park and Thompson conducted a series of experimental tests to investigate the reversed cyclic loading response of prestressed concrete beam-column interior joints [Park and Thompson (1977)] with different ratio of prestressed and non-prestressed steel reinforcement. Within the presented results, they proposed [Thompson and Park (1980)] an idealization for the moment-curvature characteristics of partially prestressed concrete members (ranging between fully prestressed and reinforced concrete members) under reversed cyclic loading by combining the hysteresis responses of the prestressed concrete system $M_p(\phi)$ with the idealized response of the reinforced concrete system $M_r(\phi)$ (see Fig. 9-6):

$$M(\phi) = \alpha M_r(\phi) + \beta M_r(\phi) \tag{9-1}$$

where $\alpha = M_{ru}/M_u$

$\beta = M_{pu}/M_u$

$\alpha + \beta = 1$

and M_u is the ultimate moment capacity of the partially prestressed concrete section; M_{ru} and M_{pu} are the moment contributions of the non-prestressed steel and the prestressed steel, respectively (taken about the centroid of the concrete compression forces in the section at ultimate condition). The moment-curvature idealization adopted for prestressed concrete was a modification of the curve proposed by Blakeley and Park (1973), while an idealization based on the Ramberg-Osgood curve was adopted to represent the reinforced concrete behaviour. A few modifications to the hysteretic rule proposed by Thompson and Park for prestressed concrete sections were suggested by Nishiyama and Watanabe (1996) to overcome some inconsistencies on the pinching behaviour and flexural cracking observed experimentally.

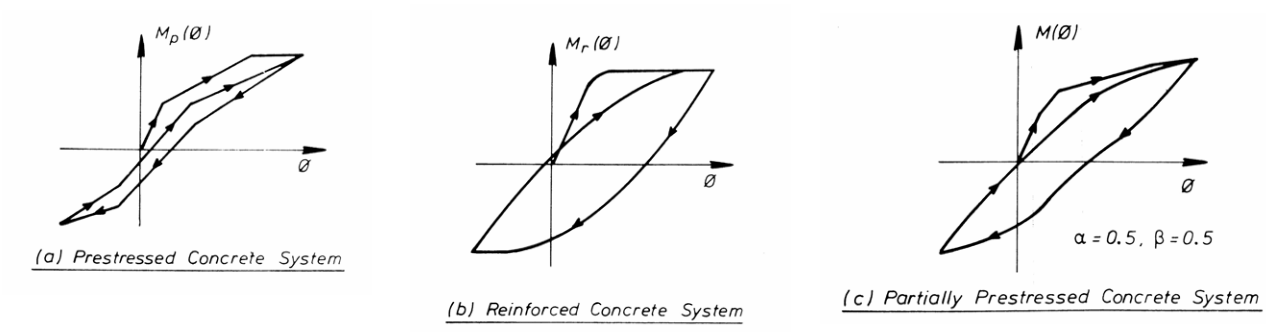


Fig. 9-6: Moment-curvature idealization for prestressed concrete as proposed by [Thompson and Park (1980)]

Recently, a similar approach has been independently proposed by Pampanin et al. (2000, 2001) to represent the moment-rotation cyclic behaviour of a critical section in precast jointed hybrid connections/systems. As previously mentioned and shown in Fig. 9-3, the hysteresis loop of a jointed hybrid system can ideally be modelled as a combination of a Non-Linear Elastic (NLE) hysteresis curve to represent the moment contribution of the unbonded tendons, M_{PT}, (self-centering characteristic), and of an appropriate dissipating rule (i.e. Elasto-Plastic, Takeda-type, Friction or Viscous-elastic) to represent the moment contribution of longitudinal mild steel or of alternative typologies of energy dissipation devices, M_s, (i.e. in the form of vertical connector between adjacent rocking wall panels):

$$M(\theta) = M_{PT}(\theta) + M_s(\theta) \tag{9-2}$$

The moment contribution of unbonded tendons and mild steel are calculated at each level of rotation about the centroid of the concrete compression forces in the section.

It is worth noting that, when dealing with wall systems, the axial load provides a self-centering contribution which should be either included in, or separately added to, the NLE behaviour associated to the contribution of the unbonded tendons.

As illustrated in Fig. 9-7, different combination of post-tensioned vs. mild steel (or dissipating devices) moment contribution ratio, M_{PT}/M_s (corresponding to the target rotation/drift level) will directly influence the hysteresis shape of the moment-rotation curve, for a given monotonic behaviour (e.g. required flexural strength at different level of member rotation or interstorey drift). Lower and upper bounds are given by a precast connection/system with only unbonded tendons (maximum self-centering, minimum energy dissipation) and by an emulated monolithic structure with only mild steel (maximum energy dissipation, minimum self-centering).

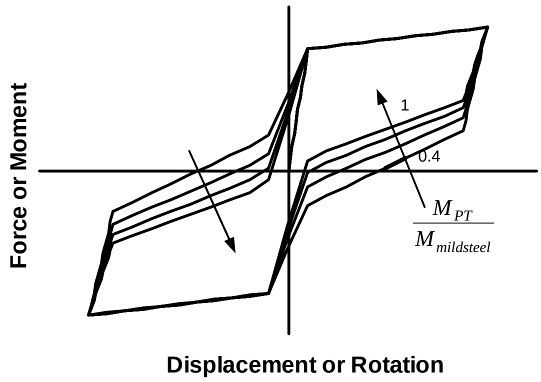


Fig. 9-7: Influence of the PT steel/mild steel moment contribution ratio on the hybrid moment-rotation hysteresis shape

Static (maximum feasible) residual deformation/displacement and equivalent viscous damping of the hysteretic rule can be adopted as main design or assessment parameters of a connection or a whole system. Using simplified charts, as those shown in Fig. 9-8, an adequate ratio of PT steel and mild steel, M_{PT}/M_s , can be defined in a design phase in order to satisfy the desired requirements or, vice versa, the expected influences of this ratio on the overall behaviour can be predicted in an assessment procedure [Pampanin *et al.* (2000)].

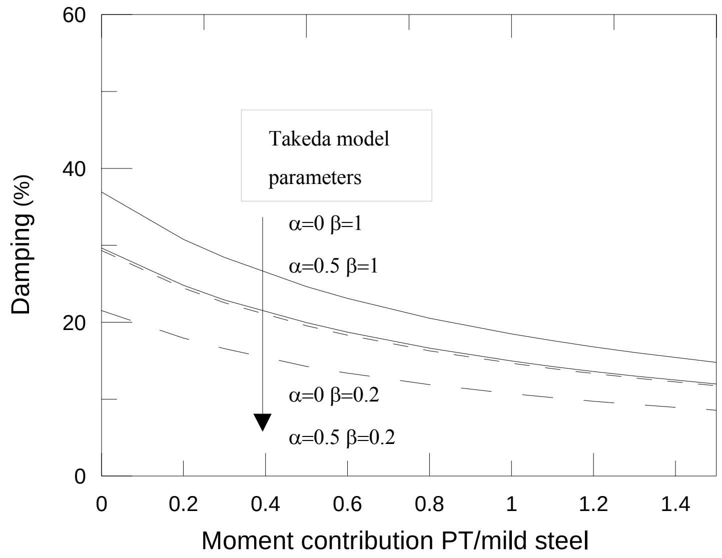
The aforementioned approach to represent the cyclic behaviour of a precast/prestressed connection through an adequate combination of different well-defined hysteresis contributions has, in principle, a wide range of applicability. A critical role is played by the possibility of correctly defining the monotonic behaviour in the form of a moment-curvature or a moment rotation curve, for a general solution, ranging from fully bonded to unbonded case, from post-tensioning to prestressed solutions (with variable amount of mild steel). A simplified procedure to define the monotonic moment-rotation behaviour when strain compatibility at a section level is violated by the use of "unbonded" reinforcement is therefore crucially needed. When section strain compatibility applies (fully bonded case, thus emulative solutions), a standard procedure to define a moment-curvature behaviour at the critical connection can be used, as in the case of cast-in-situ concrete.

9.3.3.3 Member strain-compatibility

An alternative simple procedure to develop a complete moment-rotation section analysis in presence of local strain-incompatibility has been recently proposed by Pampanin et al. (2001) and conceived as a viable tool for any typology of connection characterised by "unbonded" concepts:

- partially bonded or unbonded tendons/bars;
- unbonded length in mild steel;
- hybrid (combination of the above) connection.

Based on member compatibility considerations, an additional condition on the member global displacement can be obtained by through of an analogy with equivalent cast-in-place solutions, named as "Monolithic Beam Analogy". In this contribution, a summary of the general scheme is given referring to the general case of a hybrid beam-column connection with unbonded tendons (assumed to be at mid-height of the section) and mild steel unbonded for a short length at the beam-column interface. Simple modifications can be easily adopted for other particular cases. Further details can be found in Pampanin et al. (2001). Extension to the modelling of jointed wall system is given in subsequent paragraphs of this section.



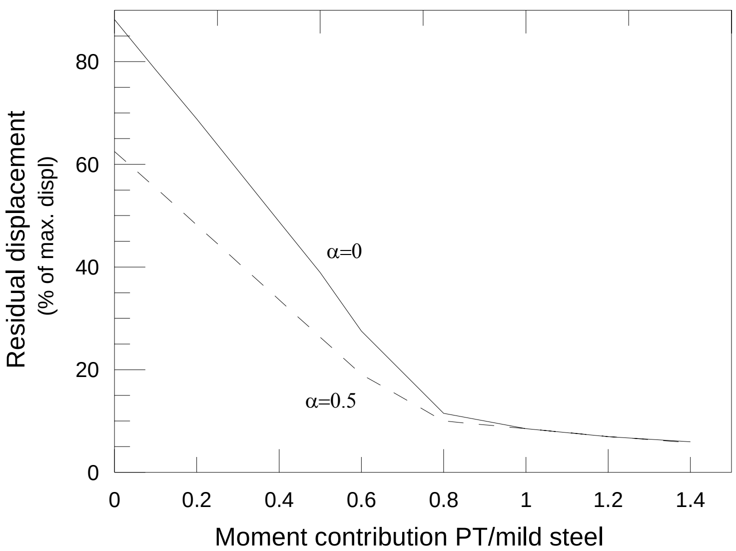


Fig. 9-8: Influence of the PT steel/mild steel moment contribution ratio on the on key parameters of an hybrid hysteresis loop (equivalent viscous damping and maximum feasible residual displacement)

As shown in Fig. 9-9 the conceptual skeleton of the proposed moment-rotation procedure is intended to reflect that typically used for section analyses of monolithic solutions: for a given rotation, the actual position of the neutral axis, *c*, corresponds to the unique solution respecting either equilibrium equations at a section level and compatibility conditions at a member level (member compatibility).

Referring to the peculiar mechanism (gap opening and closing at the critical interface) of the hybrid beam-column connection of Fig. 9-9, the strain levels in the unbonded post-tensioned tendons, ε_{pt}, and in the mild steel, ε_s,(with a short unbonded length at the critical section to avoid premature fraction), can be evaluated as follows:

$$\varepsilon_{pt} = \varepsilon_{in} + \frac{n \cdot \Delta_{pt}}{l_{ub}}; \qquad \varepsilon_s = \frac{(\Delta - 2\Delta_{sp})}{l'_{ub}} \qquad (9\text{-}3)$$

where ε_{in} is the initial strain in the PT tendons, n is the number of total joint openings along the beam (at beam-column interfaces); l_{ub} and l'_{ub} are the unbonded lengths in the tendons and in the mild steel, respectively; Δ_{pt} and Δ are the total elongations at the level of the tendons and of the mild steel, respectively; Δ_{sp} is the elongation due to strain penetration of the mild steel (assumed to occur at both ends of the unbonded region).

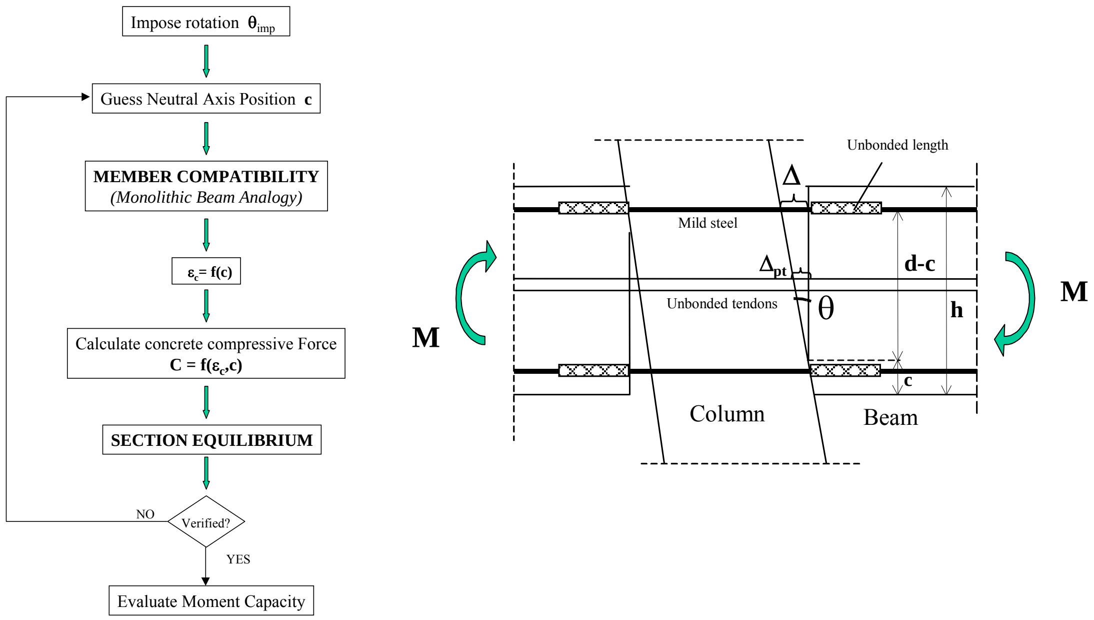


Fig. 9-9: Schematic flow chart of the moment-rotation procedure as proposed by Pampanin et al. (2001) and mechanism of a beam-column hybrid connection

The member compatibility condition is introduced in the form of an analogy (named Monolithic Beam Analogy, see Fig. 9-10), in terms of global behaviour (beam-edge displacement), between a precast connection and an equivalent monolithic one. Main assumption is that equivalent cantiliver beams (same geometry and reinforcement) following a precast jointed or a cast-in-place emulative solution would develop analogous total deflection. Being the elastic contributions equal, plastic contributions should be equal although resulting from different sources of mechanisms: in the precast case the inelastic deformation is localised at the interface, in the monolithic one is distributed along a plastic hinge.

By introducing an analogy with an equivalent monolithic solution, ultimate and yielding curvature (ϕ_u and ϕ_y) concepts can be utilised, having assumed appropriate value for the plastic hinge length (although the results are not sensible to significant variation of this parameter). After a few simple algebraic simplification a simple and familiar relationship between concrete strain, εc, and neutral axis position, c, is derived as follows, which satisfy member compatibility condition:

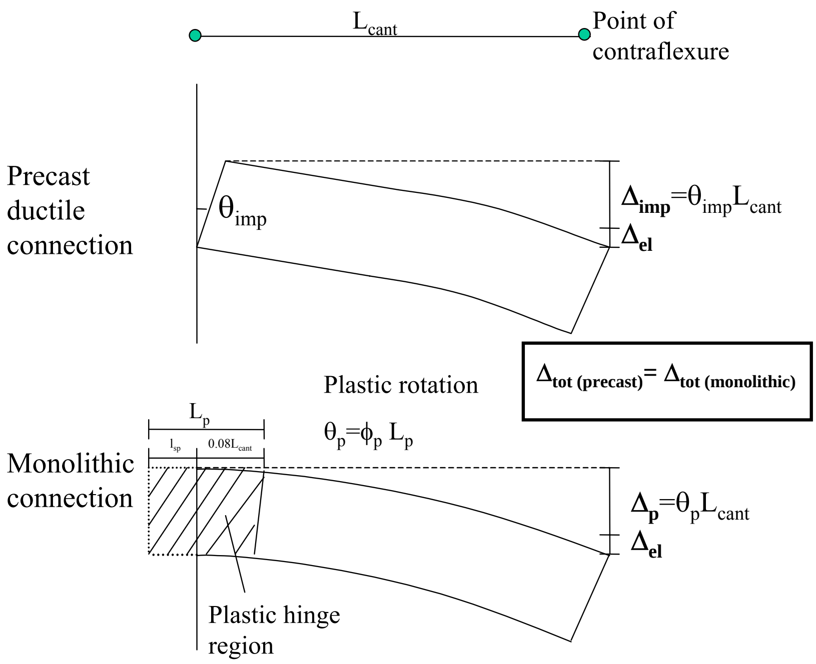


Fig. 9-10: Monolithic Beam Analogy [Pampanin et al. (2001)]

$$\varepsilon_c = \left[\frac{(\theta \cdot L_{cant})}{\left(L_{cant} - \frac{L_p}{2}\right)L_p} + \phi_y \right] \cdot c \qquad (9\text{-}4)$$

where L_p is the plastic hinge length, L_{cant} is the distance between the column interface and the point of contraflexure (length of the beam cantilever scheme).

Note that the main scheme of the aforementioned moment-rotation procedure (Fig. 9-9) can be used to evaluate the full moment-curvature behaviour of a fully bonded prestressed concrete sections, as typically done for gravity load limit states design. In this case, the strain values in the tendons and in the mild steel will simply be linearly related to the compression strain in the concrete, which represents the section strain compatibility condition and Eqs. 9-3 and 9- 4 will thus be no longer necessary.

9.3.4 Modelling of jointed wall systems

9.3.4.1 General

A brief description of the fibre element approach to model the behaviour of precast jointed wall system has been given, referring to the work done by Kurama et al. (1999).

However, as mentioned, the proposed section analysis procedure to develop a complete moment-rotation relationship in presence of strain incompatibility can be directly extended and modified for the analysis/design of general hybrid wall systems where vertical unbonded post-tensioned are combined with additional sources of energy dissipation in the form of alternative vertical connections between the precast panels [Nakaki et al. (1999); Kurama et al. (2000)] or longitudinal mild steel or external steel dissipators at the base sections [Rahman and Restrepo (2000)]

The gap mechanism at the base panel-to-foundation interface, due to the rocking motion of the cantilever wall system, is analogous to the one developed for the "cantilever" portion of a precast beam in a beam-column ductile connection, which undergoes an equivalent "rocking-type" of motion at the beam-to-column interface. Appropriate values of the plastic hinge length L_p should be chosen for wall systems, neglecting the strain penetration term l_{sp} if only unbonded tendons are adopted as vertical longitudinal reinforcement crossing the base section. The restoring force should include the additional contribution due to gravity load. If no vertical connector is utilized to couple adjacent wall panels, the whole system can easily be analysed and designed, provided minimum attention to the aforementioned details, as in the case of an unbonded beam-column connection without any additional mild steel.

9.3.4.2 S.D.O.F. model

Since the first mode of vibration is predominant, information on the gross system behaviour can be obtained, at a preliminary stage, from a simple equivalent S.D.O.F. model consisting of an elastic (uncracked section structural properties) beam element with a concentrated mass at the equivalent height (Fig. 9-11a). All the nonlinear behaviour would be concentrated at the base and represented by a rotational spring. The characteristics of the Moment-Rotation monotonic behaviour can be determined according to the proposed section analysis method. When modelling the cyclic behaviour, appropriate hysteretic rules (or combination of the above) should be chosen, depending on the conceptual analogy to any basic scheme (i.e. Non Linear Elastic, Tension Compression Yielding, Coulomb Friction, etc).

In the case of unbonded precast wall panels without vertical connections, the rotational spring at the base should be able to correctly model both the self-centering behaviour (NLE rule), provided by the unbonded tendons and the gravity load, and the non-linear behaviour (Takeda rule) of the concrete. If the moment is taken about the centroid of the concrete and the whole gap-behaviour modeled with a NLE spring, the energy dissipation provided by the concrete has to be taken into account, when performing a non-linear time-history or cyclic analysis, by adding an appropriate value of the equivalent viscous damping ξ. Experimental tests on unbonded beam-column connections as well as on wall systems have shown that the energy dissipation provided by pure unbonded systems is contained in the range $\xi \approx 5\text{-}10\%$ [Priestley (1996), Rahman and Restrepo (2000)].

9.3.4.3 Modelling of vertical connections contributions

When vertical connections are adopted between adjacent panel the additional contributions in terms or flexural (overturning moment) strength and energy dissipation should be taken into account. The Axial-Force vs. Vertical Relative Displacement relationship of the vertical connector (supposed to be known) can be simply transformed into a Moment-Rotation contribution at the base section level. Thus, as in the case of hybrid beam-column systems, the global flexural resistance given by the moment contributions of the unbonded tendons (and axial load) and the energy dissipation devices (in general vertical connectors) can be modeled by using two springs in parallel with appropriate hysteretic rules.

When calculating the moment contribution around the concrete centroid, a NLE rule is suggested to represent the contribution of the restoring force, while elasto-plastic with hardening rules or rigid-perfectly plastic can be adopted, for example, in the case of U-shape Flexural Plates devices or Friction-type vertical connectors.

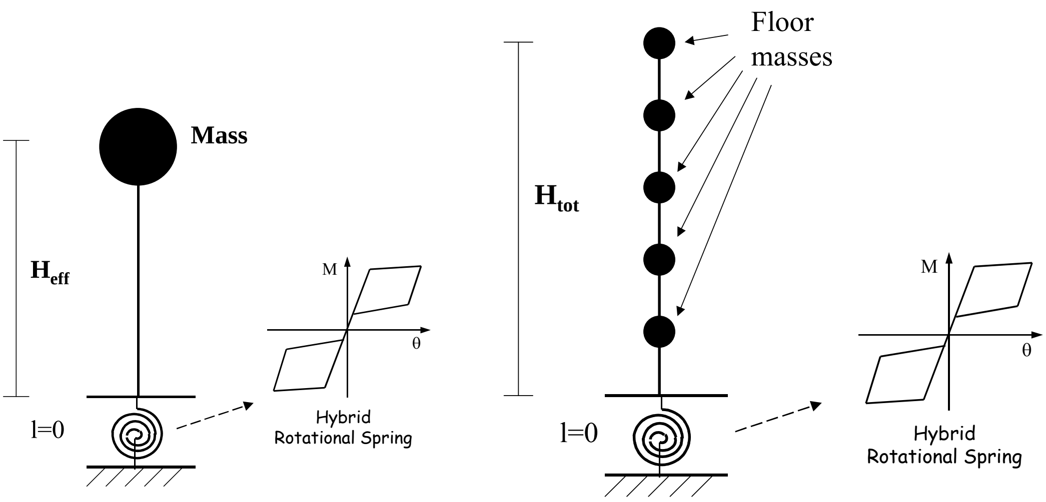


Fig. 9-11: Unbonded wall system: a) S.D.O.F. and b) M.D.O.F. simplified models [Pampanin (2000)]

9.3.4.4 M.D.O.F. analytical models

Experimental evidences have confirmed that the displacement response of precast jointed ductile wall systems, capable of rocking motion at the base and with a relatively low number of storeys, is dominated by first mode deformation pattern, with displacements at each floor levels being approximately proportional to the height. However, storey shear forces and floor force might be significantly affected by higher mode effects since only the first mode is significantly modified by ductility. Higher modes remain elastic and their contribution in the displacement response is limited by their large stiffness. Due to its simplicity, the S.D.O.F. system described above can therefore be extremely useful in the design phase and preliminary assessment analyses.

When time-history non-linear analyses are performed, still a good estimation of the maximum displacement/drift, reached under an input ground motion, can also be reliably predicted. On the other side higher mode effects influencing the storey shear forces and floor forces levels cannot be captured and might lead to critical under-designing of important structural components (i.e. diaphragms or diaphragms-to-seismic resisting system connections).

Starting from the characteristic of the equivalent S.D.O.F. presented above, a simplified M.D.O.F. model based on a lumped plasticity approach can consist of a multi-mass elastic mono-dimensional element with a non-linear flag-shape rotational spring at the critical base section modelling the inelastic behaviour concentrated at the base through a rocking motion (Fig. 9-11b).

The re-centering contribution provided by the vertical unbonded tendons as well as the axial load will be modelled with a NLE rule, while appropriate hysteresis rules shall be used to model the alternative sources of energy dissipation (i.e. elasto-plastic, rigid-plastic or viscous, depending on the use of mild steel or steel flexural plates, friction or viscous devices). The performance of each vertical connector along the elevation can be well controlled, since it depends on the relative displacements between adjacent panels, thus on the lateral displacement pattern of the whole system (which can be correctly predicted).

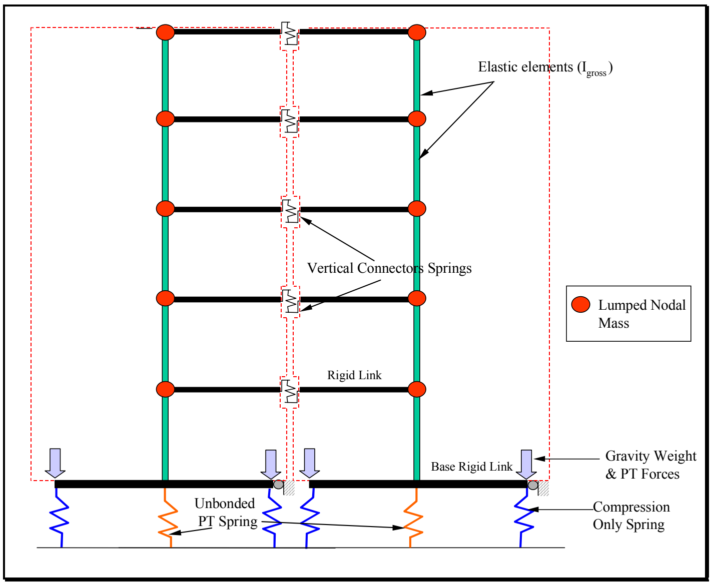


Fig. 9-12: Unbonded wall system: simplified model developed at UCSD [Conley et al. (1999)]

An alternative simple and reliable M.D.O.F. system can be obtained as illustrated in Fig. 9-12, which refers to the model developed at UCSD during the test of the PRESSS Five-Storey precast concrete buildings [Priestley et al. (1999), Conley at a. (1999)]. The different flexural contributions provided by unbonded PT steel tendons/bars, concrete and special vertical panel-connectors are independently considered and modelled with springs of appropriate hysteretic characteristics. In this case, thus, approximate assumptions on the neutral axis position has to be made to define the position of the concrete spring. The panels of the two adjacent walls are represented by elastic elements. The base section interface is represented by rigid horizontal beams. The restoring action of the unbonded post-tensioned bars, which run all through the height of the wall systems and anchor the panels to the foundation, are modelled with non linear inelastic spring located under the wall elements. The concrete behaviour is modelled with compression-only springs located at the calculated compression centroid at each wall end under rocking response. Preloading of these springs is obtained by forces representing the gravity load and the initial unbonded prestressing force at the wall base.

Adjacent walls are connected by vertical inelastic springs, representing the vertical connectors, (i.e. U-shape Flexural Plate energy dissipation devices), acting between horizontal links extending at each floor level.

9.4 System displacement compatibility issues

9.4.1 General

In the previous paragraphs of this chapter, the critical aspects in modelling prestressed/precast connections for wall and frame systems have been discussed. When assuming rigid floor diaphragms and neglecting torsional effects, a 2-D model has been implicitly suggested as more than satisfactory viable analytical tool.

Under these hypotheses, the seismic response of dual systems can thus be predicted simply imposing similar lateral floor displacements.

Main issues related to the effects of inelastic torsional response of new-designed seismic resisting buildings (either relying on walls, frames or dual systems as lateral resisting structures) have been recently discussed by Paulay (1997, 2000) and extensive analytical investigations have been carried out to develop simplified design procedures to reduce undesirable response [Castillo *et al.*, (2002); Castillo (2003)].

However, a part from torsional response, further issues related to global system response deserve to be herein mentioned, which may be taken into account in a refined 2-D model analyses:

a) beam elongation;
b) diaphragm/lateral-load-resisting-system interaction.

9.4.2 Modelling of beam elongation effects

9.4.2.1 General

As anticipated in Section 5.2.3.3, “Beam elongation” is a term used to describe an increase in distance between column centerlines at one or more levels of a reinforced concrete (RC) or prestressed concrete frame. This occurs in frames in which flexural deformation causes plastic hinging and cracking in a RC beam or gap opening at the beam-column interface of a prestressed jointed frame under significant lateral displacements.

Possible effects of this phenomenon on frame response, have been briefly discussed, based on analytical and experimental investigation findings [Fenwick and Fong (1979); Douglas (1992); Fenwick and Megget (1993); Restrepo et al. (1993); Christopoulos, (2002); Matthews et al. (2003a,b)], in terms of:

- damage to and interaction with the floor system;
- increase of column curvature and, thus, flexural and shear demand;
- increase of beam moment capacity due to the increase of bam axial force;
- increase of residual local deformations.

9.4.2.2 Multi-spring model

A refined analytical model for beam-column joint connection able to take into account the effects of beam-elongation in the analyses can consist of multi-springs at the beam-column interface representing discrete contact region. Rigid body relative rotation (i.e. gap opening) of beam and column sections is modeled by use of rigid link. A similar model has been recently proposed by Kim (2002) after validation on experimental tests of precast prestressed jointed beam column subassemblies [Kim (2002)] as well as on reinforced concrete monolithic subassemblies [Zerbe and Durrani (1990)].

In this model the joint itself was assumed to remain rigid, while beams and columns elements are assumed to remain elastic. Inelastic action was supposed to concentrate in the grout (at the beam-column interface) and mild steel bars. An idealization of the connection and the proposed model is given in Fig. 9-13. Truss elements were used to model the reinforcing steel, while a prestressing element was used to prestress the joint together and 9 gap elements were used over each side to represent the grout behaviour. Further details on the model as well as complete results on beam elongation effects from analytical investigations (using the computer program DRAIN-2DX) on multistorey frame systems can be found in Kim (2002).

It is worth noting that the proposed analytical model represents a viable but relatively complex tool to accurately estimate the effects of beam elongation on the overall response of the building under seismic excitation. The use of multi-springs as contact elements (typically adopted in the past to represent rocking behaviour) can allow to follow the variation of the neutral axis position corresponding to the gradual opening of the gap as well as the possible softening behaviour due to local damage of the concrete in the compression region. Simplified hand methods to evaluate the potential consequences of such phenomenon as well as practical recommendation for design are under development [Kim (2002)].

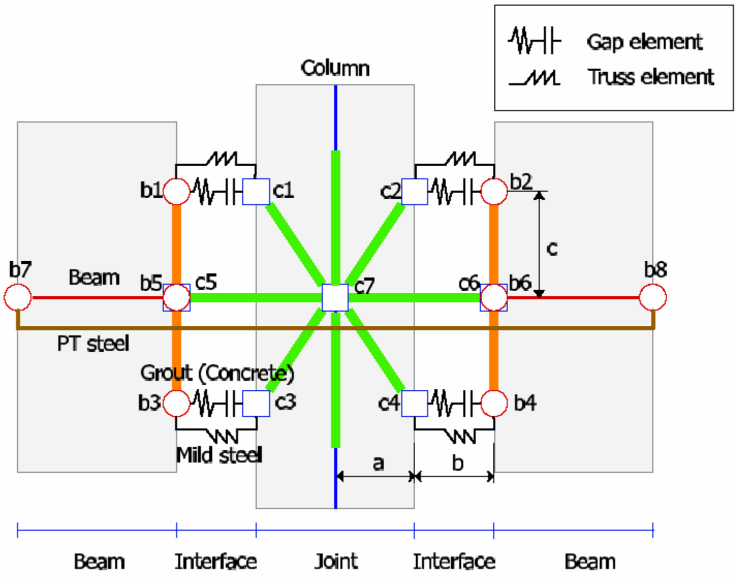


Fig. 9-13: Idealized model for beam-column joint to account for beam elongation

It should also be noted that these analytical studies were based on a model calibrated to the behaviour of test units without slabs. According to experimental tests on frame systems with slab (Lau and Fenwick, 2002) the interaction (displacement compatibility) between floor and lateral-load-resisting system can reduce the effects of beam elongation. At higher level of drift and cyclic loading, inelastic behaviour and damage in the slab might occur (as discussed in the following paragraph) leading to increase level of elongation as well as reduction of system strength at a specific drift.

Furthermore, recent experimental investigations on a 3-D full scale specimen conducted at the University of Canterbury, Christchurch (NZ) [Mattews et al. (2002)], have shown that beam elongation may result in unexpected out-of-plane deflection of the central column (instead of, or in addition to, lateral deflection of exterior columns).

9.4.2.3 Section analyses approach

The analytical procedure proposed in the previous paragraph to define a complete moment-rotation behaviour according to a section analysis approach can be simply modified to account for the beam-elongation effects and the axial restrain provided by a real frame system (Christopoulos, 2002). The increase of additional compression force in the beam (thus at the beam-column interface) can has non-negligible effects on the behaviour of hybrid post-tensioning systems (i.e. increase of the post-yielding stiffness as well as of the localized damage).

A series of springs can be adopted to capture the effect of beam elongation in a restraining frame system. Fig. 9.14 shows, as example, the proposed model for of a two bays, three columns, one storey subassembly.

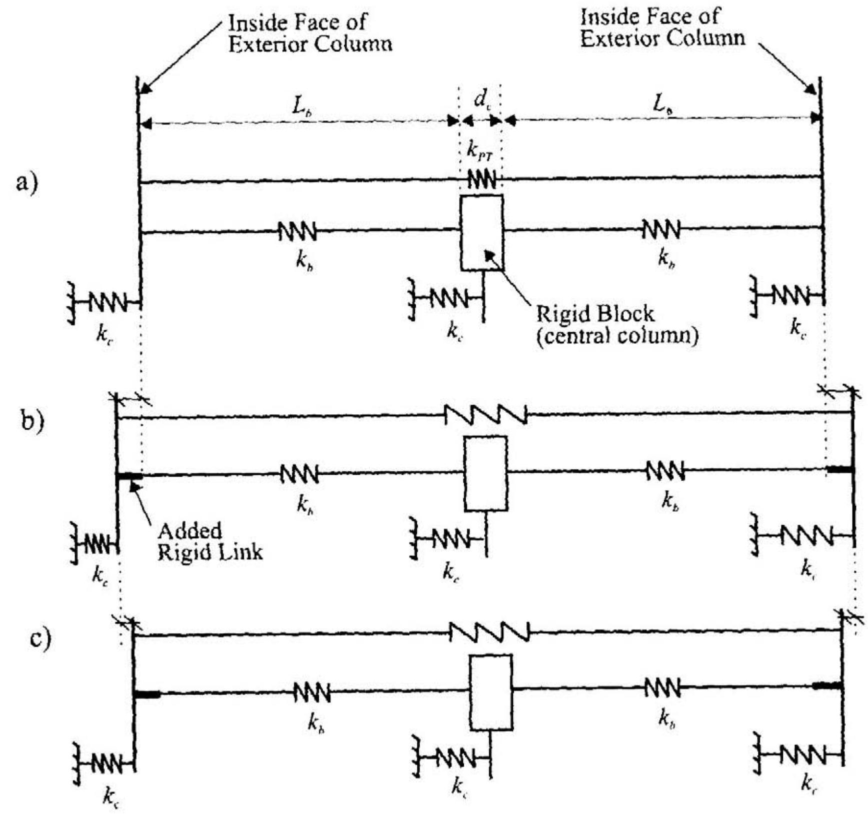


Fig. 9-14: Spring Model of Assembly Elongation: a) System at Rest, b) Imposed Elongation and c) Final Deformation (Christopoulos, 2002)

The effects of beam-elongation on the strain increase in the post-tensioning steel (ε_{pt}), are suggested to be taken into account simply multiplying Eq. 9.3 by the factor $\left(1-\frac{1}{\Omega}\right)$, leading to the following Eq. 9-5:

$$\varepsilon_{pt} = \varepsilon_{in} + \frac{n \cdot \Delta_{pt}}{l_{ub}}\left(1-\frac{1}{\Omega}\right) \tag{9-5}$$

where Ω is an indicator of the restrain effects:

$$\Omega = \frac{k_b}{k_c + 2k_{pt}} + 1 \tag{9-6}$$

where k_b is the axial stiffness of one beam, k_c is the bending stiffness of one column at the level of the beam-to-column connection and k_{pt} is the axial stiffness of the PT tendons spanning the entire subassembly.

It is worth underlining that the column restraining effects in a multi-storey frame system has to be reduced along the elevation, assuming different values of equivalent column lateral stiffness coefficient k_c for the first storey and for the stories above the first.

Within this simplified model, the variation of shear forces induced into the column by the increase in beam axial force can be taken into account by adding, at the maximum drift level, the contribution caused by system elongation, which, according to the assumptions made, is drift-dependent.

9.4.3 Analytical modelling of precast diaphragms

9.4.3.1 General

As mentioned, building structures are typically designed assuming that floor systems act as a rigid diaphragm between the vertical elements of the lateral load-resisting system. However, long floor span structures with perimeter lateral laod resisting systems possess diaphragms which behave quite flexibly, which can modify [Ju and Lin (1999)] or control the dynamic response [Tena-Colunga and Abrams (1996)], leading to unexpected seismic demand including large structural drifts, diaphragm deformation and forces. Inelastic diaprhragm response could occur execerbating the effects of diaphrams flexibility.

Design issues and analytical techniques used in diaphragms design (horizontal plate girder analogy, strut and tie method) have been discussed in Section 6.6.4.

Analyses used to investigate the seismic behaviour of precast structures may in cases require modelling of the floor diaphragms to accurately determine the dynamic characteristics of the structure, the diaphragm loading, or diaphragm failure modes. It is often not practical to capture all aspects of the behaviour in one model (e.g. dynamic behaviour, nonlinear response, three-dimensional representation). Thus, depending on the analysis objectives, the modelling technique used may focus on certain aspects, as described in this section. Analytical techniques used in diaphragm design (horizontal plate girder analogy, strut and tie method) are covered in Section 6.6.4.

9.4.3.2 Nonlinear static models of precast diaphragms

Static analyses of individual diaphragms are useful in determining diaphragm stiffness, strength and failure modes. These analyses reduce the size and complexity of the overall problem by: (1) isolating the diaphragm from the remainder of the structure; and (2) eliminating the dynamic aspects of the behaviour. Often, out-of-plane motion is ignored as well, reducing the analysis to two-dimensions (the floor plane). In this case, it is only necessary to model those portions of the floor system creating the high in-plane stiffness (precast flange, topping, etc.).

Continuous elastic models can indicate “hot spots” and smeared crack models can approximate softening effects. However, for most precast floor systems, diaphragm behaviour is dominated by the joint regions, whether it be elastic compliance or concentrated inelastic deformation demand in the reinforcing acting between relatively stiff and strong precast units. This behaviour can occur in topped systems and pretopped systems alike, since shrinkage cracks or troweled joints in the topping can produce a similar condition for the former case. Accordingly, a key aspect in the modelling of precast diaphragms is adequately capturing the paneled system behaviour. This behaviour can be modeled through discretization of the panel joints separate from the panel. For well-behaved cases (long simple diaphragms under transverse seismic loads), this has been accomplished successfully with fibre models [Rhodes et al. (1997)]. However, for most situations, the squat geometry, boundary conditions, seismic loading direction, and floor plan irregularity render finite element (FE) models as the most facile technique [Fleischman et al. (1998)]. The FE discretization of a typical joint in a topped precast diaphragm may include non linear spring elements to model the chord steel, wire mesh and mechanical connectors. Nonlinear springs can model axial (tensile) or shear resistance individually (e.g. chord steel, dowel action), or coupled elements can be used in cases where shear-axial interactions are important (e.g. mechanical connectors, shear friction). Force-deflection properties for reinforcement can be provided by established bond stress-slip relationships, [e.g. Alsiwat and Saatcioglu (1992)]. Shear-friction characteristics for the steel reinforcement and properties of the mechanical connectors can be provided through empirical data, [e.g. Porter and Sabri (1990), Pincheira et al (1998)]. Contact pseudo-elements [ANSYS (1997)] are necessary to provide the compression transfer across the closed joint in order to obtain a proper neutral axis. For a pretopped diaphragm model, capturing the shear compliance in the joint region is crucial; it may be necessary to include the elastic resistance provided by secondary elements (e.g. spandrel beams).

The diaphragm models can be loaded in two primary ways. First, with fixed supports representing the vertical elements of the lateral load-resisting system, inertial loading of the diaphragm can be generated by body forces representing uniform acceleration. It is important in this case to include the mass of the entire floor system, though it can be assumed to act within the projection of the topping slab and precast flange. Secondly, the diaphragm's response to differential motion of vertical elements of the lateral system can be determined by imposing relative displacements at the supports. It should be recognized that the actual loading will be a combination of these actions; an elastic three-dimensional dynamic analysis could lend insight on the relative importance of each effect. A third approach is to perform modal analyses on the elastic model in order to determine complex diaphragm deformation modes under general excitation [Fleischman et al. (1998)]. An nonlinear "pushover" analysis of the diaphragm can be obtained by subjecting the two- dimensional model to incrementally increasing levels of inertia loading until failure is incurred [Farrow and Fleischman (2002)]. This pushover analysis provides the load-deformation response of individual floor diaphragms, including estimates of the initial stiffness and ultimate strength. Additionally, the internal detail of the finite element model allows determination of: (a) force paths within the diaphragm; (b) the sequence and occurrence levels of limit state events; and, (c) relationships between global diaphragm deformation and local ductility demand. It should be noted that precast diaphragms often contain brittle elements that will pose difficulties in a force-controlled pushover analysis. Arc-length methods [ABAQUS (1992)] have been employed with mixed results. A brute force method is possible in which a piecewise pushover envelope is created by removing the failed elements and re-running the analysis with the updated model [Farrow and Fleischman (2002)].

Several significant assumptions are associated with the static two-dimensional finite element analysis including: (1) specified loading patterns or relative displacements; (2) the absence of the superimposed gravity loading; and (3) no out-of-plane diaphragm motion. The last assumption likely leads the diaphragm model to undergo lower out-of-plane forces in the vicinity of vertical elements of the lateral load resisting system, but also forces the diaphragm model to deform more in-plane than is likely to occur in reality.

9.4.3.3 Nonlinear dynamic models of flexible diaphragm structures

Dynamic analysis of full structure models can be used to determine the loading on the diaphragm and the effect of diaphragm flexibility on the structure response. Three-dimensional FE models can be used in elastic modal analyses [Ju and Lin (1999)]. However, for nonlinear transient dynamic analyses, three-dimensional FE analyses are likely too computationally intensive. For this case, simplified multi-degree-of-freedom (MDOF) representations of the structures, verified through comparison to a limited set of three-dimensional finite element models, are valuable. These reduced-DOF representations have been employed successfully in investigations of flexible diaphragm structures [Costley and Abrams (1995), Fleischman et al (2002)].

In its simplest representation, each level of the structure is described by two DOFs: one DOF representing strong-axis translation of the lateral-system and one DOF representing horizontal translation of the diaphragm. In this case, the diaphragm behaviour is confined to symmetric in-plane response to a horizontal earthquake influence vector orthogonal to the diaphragm weak axis. Generalized coordinates [Chopra (1995)] are used to describe the diaphragm as a single-degree-of-freedom oscillator. MDOF models of slightly more detail have been used to investigate the dynamic response of more complex floor systems, e.g. five DOF per floor for three-bay parking structures [Fleischman et al. (1996)].

The nonlinear time-history analyses described above were performed using DRAIN-2DX [Prakash et al (1993)]. Diaphragm DOFs were connected to the lateral system by zero-length bilinear springs with stiffness-degrading hysteretic properties [Wu (1995)]. A polynomial curve-fit of the fundamental shear/flexure deformation mode from FE analyses provided the shape functions used to determine diaphragm generalized mass and stiffness [Fleischman and Farrow (2001)]. The post-yield stiffness ratio was determined from the diaphragm pushover analyses.

Rayleigh damping was specified for the first and last significant structural modes. It is noted that this approach results in underdamped intermediate modes [Tena-Colunga and Abrams (1996)]. Indeed, flexible diaphragm structures have sufficient higher mode participation to make this point an issue, however, frequency domain spacing of the modes is such that the effects of underdamping are negligible [Fleischman and Farrow (2001)].

Several significant assumptions are associated with the MDOF analyses including: (1) the diaphragm deformation modes are predetermined; (2) the mass distribution between the diaphragm and lateral-system DOFs is assumed to remain constant during response; and, (3) the diaphragms are assumed able to achieve a fully-ductile flexural limit-state.

9.4.3.4 Floor/lateral-load resisting system displacement incompatibility

Further concerns on the global system behaviour has been raised by recent experimental investigations on a full scale 3-D frame structure including hollow core slab, performed at the University of Canterbury, Christchurch (NZ) [Matthews et al. (2002)] underlined the possibility of critical damage in the floor system due to interaction (displacement compatibility) with the lateral-load resisting system. Alternative detailing solutions to allow relative deformations and minimizing floor damage (e.g. through discrete mechanical couplers as used in the PRESSS Five-Storey Precast Concrete Building Test, Priestley et al. 1999) are currently under investigations. Simplified analytical methods to evaluate the possible effects of this interaction as well as design strategy to minimize the floor damage are under development. It is worth nothing that, based on the observed behaviour, possible concerns on displacement incompatibility between floor and lateral load resisting frame are not expected to involve only precast systems. Indeed, a natural extension of the concept of “jointed” ductile system to the floor-to-frame or wall-to-frame connections, with the inelastic demand being concentrated (thus more easily controlled) in discrete regions, can be anticipated as an extremely efficient solution when compared to monolithic (cast-in-place slab or over-reinforced topping) equivalent solutions, where higher level of damage is expected to spread in the floor system.

9.5 Analytical methods

9.5.1 General

As typically recognized, the analysis method should be appropriate for the type of evaluation or design being conducted. Similarly to what discussed for the alternative modelling approaches, different level of complexity of the analytical methods (i.e. pushover analyses or time-history non linear analysis) can be adopted depending on the information needed, the stage of the design or assessment (preliminary or final evaluation). Furthermore, according to the adopted design philosophy and approach (i.e, force-based or displacement/deformation based) alternative analysis methods can be required.

In this view, suggestions for the "most appropriate" combination of model and analytical method is not herein given, mostly relying on designer experience and choice in judging case by case situation, being well aware of the limits of the reliability of the results when considering also the uncertainty on the input as well as on the mechanical properties of the considered structures. Probabilistic analyses accounting for these uncertainties can obviously be, in some cases, a viable choice.

In this paragraph, an overview of alternative analytical methods will be given underlining pros and cons when dealing with precast/prestressed concrete building structures.

9.5.2 Alternative analytical methods

A formal distinction of available alternative analytical methods can be proposed as followed according to FEMA356 (2001) notation:

- Linear Static Procedure (LSP);
- Linear Dynamic Procedure (LDP);
- Non-linear Static Procedure (NSP);
- Non-linear Dynamic Procedure (NDP).

Any of these methods may be applied to 2-D or 3-D structural models. However, as expected, as the ability of the method to represent a greater number of aspects of structural behaviour increases, the information required and computational effort also tend to increase.

The Linear Static Procedure (LSP) is an elastic static analysis in which a set of lateral forces are applied to the structure. Such a procedure has been extensively used in literature for the majority of short structures throughout the world, being the basic tool of traditional force-based design approach. The simplicity of the methods relies on a correct assumption of the stiffness properties of the structure, which however, as recently recognized (Priestley, 1998) as main draw-back of force-based design approaches, can lead to significant un-conservative unconsistenties. In the particular case of precast/prestressed structures the use of initial (uncracked section properties up to the decompression point) stiffness would not be appropriate and a "yield" point (thus stiffness secant to yielding) can be not well defined.

While the LSP does not model the inelastic structural behaviour, design methods have been developed in conjunction with this analysis method allowing designers to estimate with some approximation the expected dynamic behaviour [i.e. Newmark and Hall (1982) equal displacement assumption, which is however a trend, not a rigorous rule].

It should be furthermore recognized that the ability to obtain satisfactory structural performance using the LSP is greatest for short structures with mass and stiffness regularity which are expected to have small inelastic demands.

The Linear Dynamic Procedure (LDP) involves either an elastic time history analysis or an elastic response spectrum analysis. Again the uncertainties on the estimation of the correct stiffness to use may lead to significant unconservative estimation of the seismic demand. Information about the distribution of mass, as well as that of the stiffness properties, is needed and effects of different modes of building vibration are captured. In the case of elastic time history analysis, a suite of earthquake records is required to estimate the likely demands. LDP has been used for most modern regular tall structures in the world in regions of moderate to high seismicity. However, since the analysis is elastic like the LSP, it does not estimate the inelastic behaviour well.

The Non-linear Static Procedure (NSP) involves either a monotonic push (pushover) to a specified lateral displacement with a predetermined lateral force regime, or a cyclic push-pull analysis to specified displacements with a predetermined lateral force regime. Both member strength and stiffness information are required, but since the structures is displaced in the inelastic regime, depending on the target (or code-based) drift/ductility level, the uncertainties on the initial stiffness will not critically affect the result.

Recent advances in computational capability and the availability of computer programs have meant that the NSP monotonic push is becoming used more commonly by design professionals for specific structures. One advantage of this approach is that it allows designers to obtain some idea of the possible structural mechanism. However, there are a number of presently contentious issues relating to the displacements to which the frame should be pushed, the lateral force distribution used for the push analysis (i.e. triangular or uniform distribution or adaptive pushover), and how member force demands should be estimated from the analysis.

The Non-linear Dynamic Procedure (NDP) involves inelastic dynamic time history analysis of a structure subjected to a set of earthquake records. A full description of the masses, strengths, stiffness and damping values of the structure are required and appropriate integration time steps are needed to describe the model structure behaviour as it is deforms during the earthquake motion.

This method should be used with the other analysis methods described above to ensure that the results are reasonable. Presently this analysis method is most commonly used by researchers, however it is likely to become used more by practitioners in the future.

It is worth noting, in fact, that the higher complexity of a NDP is only apparent and can be easily overcome following the simplified (plasticity concentrated) modelling approach described in this chapter.

Simple section analyses calculations can provide the moment-curvature (or moment rotation according to Section. 9.3.3) monotonic behaviour at the critical sections, as indeed required from a more commonly used pushover analysis. As previously discussed, once the non-linear envelope curve is given, the cyclic behaviour of the connection/system can be evaluated with simple assumptions (e.g. Section 9.3.3.2), depending on the relative contribution of the different reinforcement (prestressed, post-tensioned, mild steel or energy dissipating devices).

Furthermore, the availability and simple accessibility, nowadays, of well-defined set of accelerograms (recorded or synthetic), related to local seismic area, can allow to perform with slightly higher effort, extensive detailed analyses (maybe in a final stage) at different level of seismic intensity , according to a performance-based approach.

9.5.3 Analytical methods and design procedures

Significant advances have been accomplished in the last decade in the seismic protection of structures, based on the introduction and refinement of innovative conceptual approaches. A discussion on the design philosophy and approach adequate for precast/prestressed concrete structures has been extensively given in Chapter 4. The main drawbacks and inconsistencies related to traditional force-based design approach have been recognized as well as the critical role of displacement or deformation demand in characterizing the structural damage and thus the performance of the system when subjected to different level of ground motion intensity. A common generalized trend of developing and adopting displacement/deformation-based approaches within a performance-based design philosophy has resulted.

Common trend of current design and assessment procedure, either force-based or displacement/deformation-based is in fact to rely on the use of alternative analytical methods or combination of them. A discussion on the design philosophy and approach adequate for precast/prestressed concrete structures has been extensively given in Chapter 4. In this paragraph a brief discussion on the conceptual skeleton of commonly adopted design procedure will be given in order to emphasise the various refined adoption of alternative analytical methods within a more general and complex design/assessment procedure.

It is indeed clear that a critical discussion on the pros and cons of each analytical procedure, as independent tool (as given in the previous paragraph), should better refer to the efficiency of the global comprehensive design procedure, where the peculiarities of different analytical methods are adequately combined. Within a same class of design approaches included in code-provisions or recently proposed in literature (i.e. force-based, displacement/deformation-based or -focussed) significant differences in the analytical methods adopted as well as in the fundamental assumption of structural parameter (i.e. mass, stiffness, damping) can be noted. Few examples will be given in the following pages.

In the direct DBD procedure suggested by Priestley (1998), adequate elastic displacement spectra are entered with the target displacement Δ_d. The design base shear as well as the structural "effective" period (corresponding to "effective" stiffness secant to target displacement) are determined depending on the assumed level of energy dissipation capability (in the form of equivalent viscous damping). Once the seismic demand for the whole structure is determined, the each member is designed according to the internal forces obtained by a sort of LSP (the stiffness of element are not uncracked-section one, but adequately reduced) distributing the base shear along the elevation.

In the deformation-controlled procedure proposed by Fardis and Panagiotakos (1997), the peak deformation demands during the inelastic seismic response are estimated through a simplified linear-elastic analysis. A simple "equal displacement" rule is proposed to be adopted for the estimation of the peak inelastic displacements and deformations: the interstorey drifts and chord rotations, resulting from nonlinear dynamic response, are assumed to be equal to their elastic counterparts.

The procedure can be summarized as follows:

- an elastic static analysis with inverted triangular distribution of lateral forces is performed, the magnitude of the forces being arbitrarily defined);
- the so-computed (elastic) story drifts are used to calculate, through the Rayleigh quotient formula, the first mode period T and the "work-equivalent" peak displacement (u_e) of the equivalent SDOF system.
- As final step, the elastic displacement spectrum (5% damped) is entered with T and a spectral displacement S_d of the equivalent SDOF is determined. The structural displacements and deformations computed from the preliminary linear-elastic analysis are scaled by the ratio S_d/u_e and considered representative of their inelastic counterparts during the nonlinear dynamic analysis.

Several recent methods for the seismic design and assessment of structures, as the ones included in FEMA 356 (FEMA, 2001), ATC 40 (ATC, 1996), and Japan provisions (Otani et al, 2000), as well as the N2 method (Fajfar, 2000), combine the pushover analysis (NSP) of a multi-degree-of-freedom (MDOF) model with the response spectrum (either elastic or inelastic) analysis of an equivalent single-degree-of-freedom (SDOF) system.

With the exception of FEMA 356, these methods are formulated in the acceleration - displacement (AD) format. In this format, the capacity of a structure is directly compared with the demands of earthquake ground motion on the structure. The graphical presentation allows a visual interpretation of the procedure and of the relations between the basic quantities controlling the seismic response (Fig. 9-15).

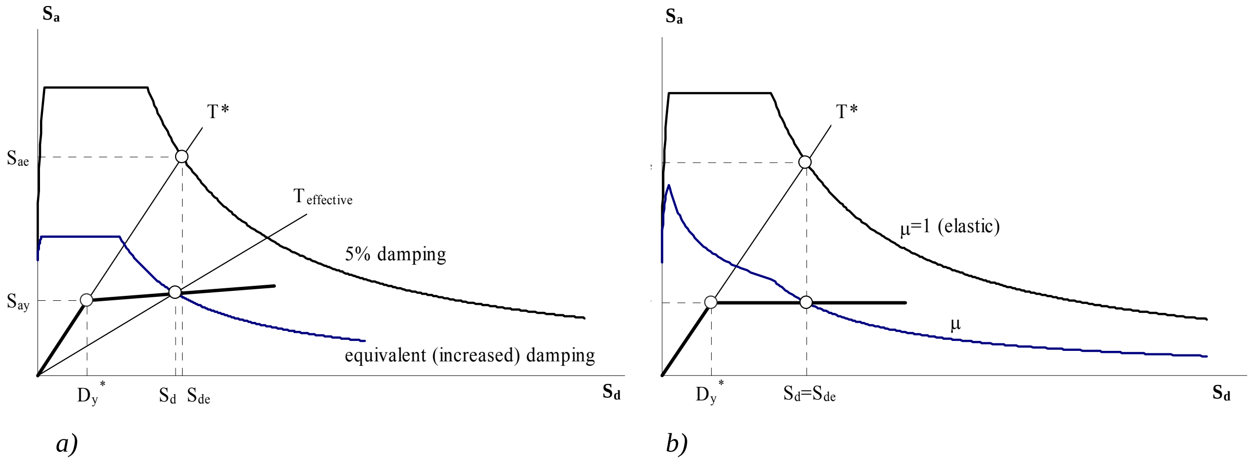


Fig. 9–15: a) Elastic, equivalent elastic and b) inelastic demand spectra versus capacity diagram (Fajfar, 1999)

The capacity of the structure is represented by a force-displacement curve, obtained from a non-linear static (pushover) analysis (NSP). The base shear forces and roof displacements are converted into spectral accelerations and spectral displacements of an equivalent single-degree-of-freedom (SDOF) system, respectively. These spectral values define the capacity diagram. The definition of seismic demand spectrum represents the main difference between different methods. In all cases, the intersection of the capacity diagram and the demand spectrum provides an estimate of the inelastic acceleration (strength) and displacement demand.

When dealing with "equivalent" elastic spectra the estimation of an adequate equivalent viscous damping is a critical step. According to the concept of hybrid system, introduced for both prestressed and post-tensioned system with additional mild steel, the energy dissipation capacity (thus equivalent viscous damping) will depend on the ratio of self-centering and dissipating properties, e.g. by using design charts as those presented in Fig. 9-8).

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Appendix A: Modelling of material behaviour

A.1 General

Modelling of material behaviour is essential for modelling section and frame behaviours, and analyses. Whether a fibre element model or a lumped plasticity approach is adopted, basic stress-strain relationship (or linearized approximations) has to be correctly defined and adopted. As mentioned, modelling of concrete and mild steel behaviours have been well documented elsewhere in the past and will not herein reported. In this section modelling of prestressing steel and bond stress-slip relation between prestressing steel and concrete will be summarized.

A.2 Prestressing steel

a) General

The stress-strain curve of a high-strength steel, such as prestressing steel, is different from that of mild steel in that the yield point is not well-defined. A proof stress or offset yield stress is often used as the stress causing 0.1% or 0.2% residual strain. High-strength steel does not have a yield plateau but strain hardening occurs causing a tangent stress-strain modulus of about 1/1000 of that in the elastic portion. Elongation at fracture of high-strength steel is generally less than that of mild steel.

Prestressing steels are usually delivered in the form of wires, strands and bars. Mechanical properties, such as tensile strength and 0.1% proof-stress, modulus of elasticity, ductility (elongation), relaxation and fatigue behaviour, are slightly different among these three types of prestressing steel

The two methods commonly used to produce high-strength steel are patenting and cold drawing. Patenting involves hot-rolling a non-alloyed high carbon steel heated to about 800 degree Celsius then cooling it slowly. Cold-drawing involves pulling the steel through successively smaller dies to increase the strength, High frequency induction heat treatment to low carbon steel is also used. The stress-strain behaviour depends on the manufacturing method.

b) Monotonic stress-strain behaviour

Several idealizations of stress-strain curve for prestressing steel have been proposed. All of them is based on the characteristics of high-strength steel.

The CEB-FIP Model Code 1990 gives force-strain relations for wires and strands as shown in Fig. A-1. It also gives a simplified schematic diagram to be used for calculation purposes (Fig. A-2).

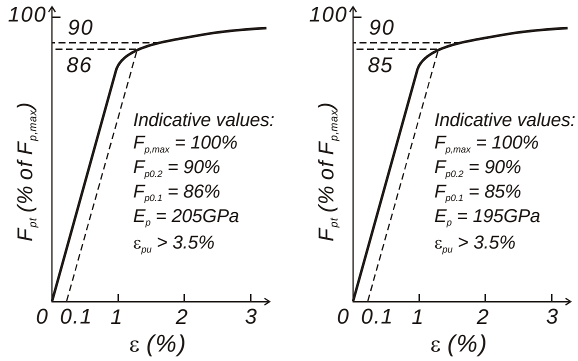

Fig. A-1: Force-strain diagram: a) for cold-drawn stress-relieved wires; b) for stress-relieved strands

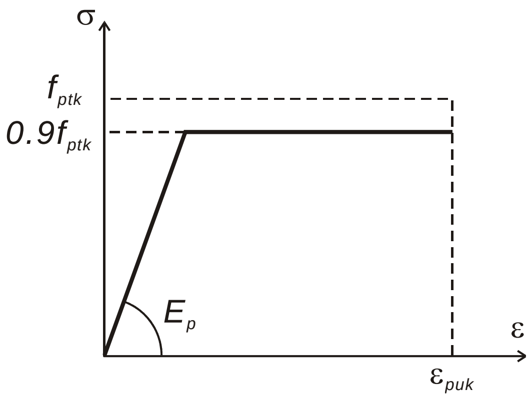

Fig.A-2: Schematic stress-strain diagram for prestressing steel

A monotonic stress-strain relation was proposed by Blakeley and Park (1973) to form an envelope for repeated loadings of the same sign and is illustrated in Fig.A-3. The curve is defined piece-wise as an initial linear portion, a hyperbolic curved portion, and an upper linear branch. The following input data is required to describe the monotonic stress-strain behaviour of the tendons:

ε_{pa} = strain at limit of proportionality

ε_{pb} = strain at beginning of upper linear branch

ε_{pu} = ultimate strain

f_{pa} = stress corresponding to ε_{pa}

f_{pb} = stress corresponding to ε_{pb}

f_{pu} = ultimate stress at strain ε_{pu}

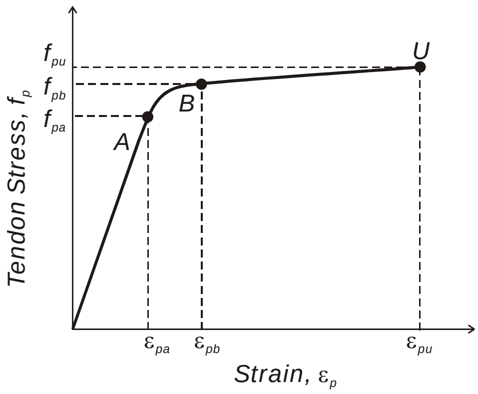

Fig. A-3:Stress-strain model proposed by Blakeley and Park (1997)

The stress-strain relations corresponding to the monotonic loading curve in Fig.A-3 are defined piece-wise by the following equations.

$$f_p = \varepsilon_p E_p \text{ for } \varepsilon_p \le \varepsilon_{pa} \tag{A-1}$$

$$f_p = \frac{\left(f_{pb}\varepsilon_{pb} - f_{pa}\varepsilon_{pa}\right)}{\left(\varepsilon_{pb} - \varepsilon_{pa}\right)} + \frac{\varepsilon_{pa}\varepsilon_{pb}\left(f_{pa} - f_{pb}\right)}{\varepsilon_p\left(\varepsilon_{pb} - \varepsilon_{pa}\right)} \text{ for } \varepsilon_{pa} \le \varepsilon_p \le \varepsilon_{pb} \tag{A-2}$$

$$f_p = f_{pb} + \left(f_{pu} - f_{pb}\right)\cdot\frac{\left(\varepsilon_p - \varepsilon_{pb}\right)}{\left(\varepsilon_{pu} - \varepsilon_{pb}\right)} \text{ for } \varepsilon_{pb} \le \varepsilon_p \le \varepsilon_{pu} \tag{A-3}$$

where
$\varepsilon_p =$ tendon strain
$f_p =$ tendon stress
$E_p =$ initial Young's modulus $= f_{pu}/\varepsilon_{pu}$

Mattock (1979) proposed the following equation for the algebraic representation of the stress-strain curve based on the form used by Menegotto and Pinto (1973) for mild steel reinforcing bars. His modelling was developed for the use with a small calculator and is rather simple.

$$f_s = \varepsilon E\left[Q + \frac{1-Q}{\left\{1+\left(\frac{\varepsilon E}{K f_{py}}\right)^R\right\}^{1/R}}\right] \tag{A-4}$$

where
Q is a post yield stiffness ratio such that

$$Q = \frac{f_{pu} - Kf_{py}}{\varepsilon_{pu} E - kf_{py}} \tag{A-5}$$

K = a coefficient
f_{py} = specified yield strength
E = modulus of elasticity

The coefficient R is determined by solving Eq. B-5 for the condition $f_s = f_{py}$ when $e = 0.010$. Here, f_{pu} and ε_{pu} are the specified tensile strength and the corresponding strain, respectively. The solution of Eq. B-5 to obtain R is carried out using Newton's method, which involves an iterative procedure.

K is determined if the complete stress-strain curve for the steel is available. The stress corresponding to the point of intersection between the two linear parts of the stress-strain curves is Kf_{py}. If the complete stress-strain curve is not available, a reasonable value to assume for K in the case of seven-wire strand is 1.04.

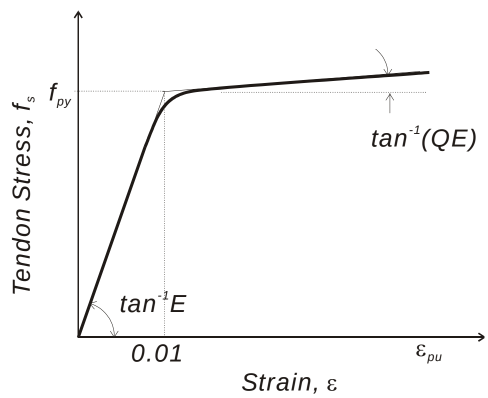

Fig. A-4: Stress-strain model proposed by Mattock (1979)

c) Cyclic stress-strain behaviour

Thompson and Park (1980) proposed cyclic loading behaviour of prestressing steel. Load reversal in the elastic range is governed by Eq. A-6. For load reversal in the inelastic range the stress-strain behaviour is illustrated in Fig. A-5. Here, (e_o, f_o) are the coordinates of the reversal point. Unloading occurs along a curve with initial slope ϕE_p, where ϕ is a modification factor to allow for softening effects which occur with increasing levels of strain. The stress-strain relationship is obtained by the following equation:

$$(\varepsilon_p - \varepsilon_o)\phi E_p = (f_p - f_o)\left[1 + \left\{(f_p - f_o)/(f_{ch} - f_o)\right)^{r-1}\right\}\right] \tag{A-6}$$

where

$$\phi = 58.27\varepsilon_m^{\ 2} - 7.506\varepsilon_m + 1.043 \quad (\text{with } \phi \le 1.0) \tag{A-7}$$

$\varepsilon_m =$ current value of maximum imposed strain

The stress-strain envelope for reversed loading is taken as the original monotonic envelope shifted along the strain axis as shown in Fig.A-5.

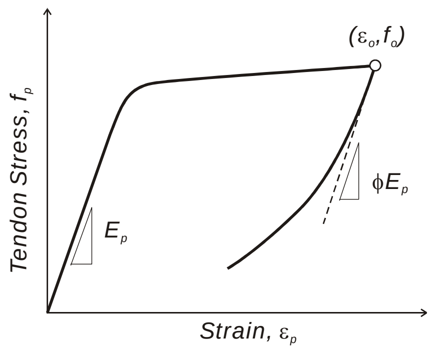

Fig. A-5: Load reversal in the inelastic range

Tagawa et al. (1997) carried out loading tests on coupons produced from prestressing steel bar. Two types of bars were used: patenting and cold drawing. In the coupons manufactured by patenting stress-strain relation in tension was almost the same as in compression. However, in the coupons manufactured by cold-drawing small yield plateau was observed in tension but not in compression. Based on the tests he proposed a modelling for each of prestressing steel bars produced by patenting and cold drawing (see Figure A-6).

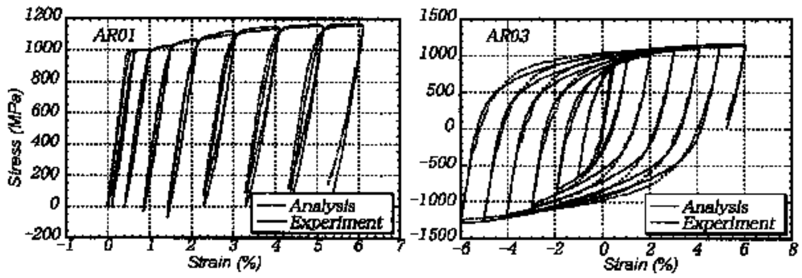

Fig. A-6: Comparison of stress-strain model proposed by Tagawa et al. (1997) with experimental results

A.3 Bond stress-slip relationship

While many studies have been carried out to obtain bond stress-slip relationship of reinforcing steel [*fib* Bulletin 10, "Bond of Reinforcement in Concrete", (2000); CEB-FIP Model Code 90, (1992)], only a few experimental studies are available for the bond stress-slip relationship of prestressing steel. The bond stress-slip relationship is influenced by a considerable number of factors such as bar roughness, bar shape (wires, strands or bars), concrete strength, grout mortar strength, position and orientation of the bar during casting, state of stress, boundary conditions and concrete cover.

The bond stress-slip curve is usually idealized by two portions: The first ascending curve characterized by adhesive action, local crushing and micro-cracking. The descending branch indicated the bond resistance reduction due to the occurrence of concrete splitting cracks along the bars or crushing of grout mortar.

CEB-FIP Model Code 90 (1992) specifies the bond stresses between concrete and reinforcing bar calculated as a function of the relative displacements according to Equations A-8 to A-11 (see Fig. A-7)

$$\tau = \tau_{\max} \left(s/s_1 \right)^{\alpha} \qquad \text{for } 0 \le s \le s_1 \qquad \text{(A-8)}$$

$$\tau = \tau_{\max} \qquad \text{for } s_1 < s \le s_2 \qquad \text{(A-9)}$$

$$\tau = \tau_{\max} - \left(\tau_{\max} - \tau_f \right) \left(\frac{s - s_2}{s_3 - s_2} \right) \qquad \text{for } s_2 < s \le s_3 \qquad \text{(A-10)}$$

$$\tau = \tau_f \qquad \text{for } s_3 < s \qquad \text{(A-11)}$$

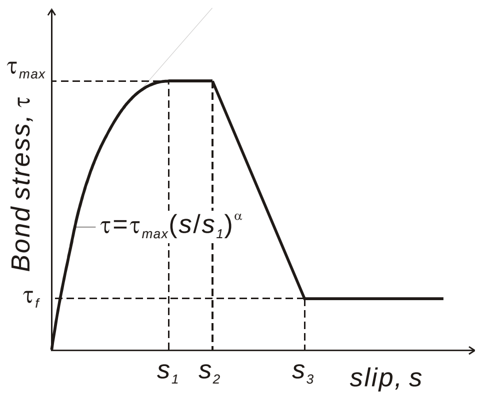

Fig. A-7: Bond-slip curve according to CEB-FIB Model Code 90

The parameters defining the mean bond stress-slip relationship are summarized in Table A-1.

	Column 2	Column 3	Column 4	Column 5
	Unconfined concrete*		Confined concrete**	
	Good bond conditions	All other bond conditions	Good bond conditions	All other bond conditions
s1	0.6mm	0.6mm	1.0mm	1.0mm
s2	0.6mm	0.6mm	3.0mm	3.0mm
s3	1.0mm	2.5mm	Clear rib spacing	Clear rib spacing
α	0.4	0.4	0.4	0.4
τ_{max}=1.4	2.0√fck	1.0√fck	2.5√fck	1.25√fck
τf	$0.15\tau_{max}$	$0.15\tau_{max}$	$0.40\tau_{max}$	$0.40\tau_{max}$

* Failure by splitting of the concrete
** Failure by shearing of the concrete between the ribs

Table A-1: Parameter for defining the mean bond stress-slip relationship

The actual bond stress-slip relation is highly nonlinear and depends on the considered length. Often, an average 'local bond stress' relationship is used for the anchorage of prestressing tendons, especially in pretensioned members.

For the calculation of crack widths and of the distribution of internal stresses, CEB-FIP Model Code 90 gives a reduction factor η_p, which considers smaller bond stress for a post-tensioned tendon than that for an ordinary bar.

η_p = 0.2 for smooth prestressing steels
η_p = 0.4 for strands
η_p = 0.6 for ribbed prestressing steels

The design bond strength for prestressing tendons for anchorage in pretensioned members is given in CEB-FIP Model Code 1990 as follows,

$$\eta_{bond} = \eta_{p1}\eta_{p2} f_{ctd} \qquad \text{(A-12)}$$

where:

$f_{ctd} = f_{ctk}(t)/1.5$ is the lower design concrete tensile strength; for the transmission length the strength at the time of release, for the anchorage length the strength at 28 days;

η_{p1} takes into account the type of prestressing tendon; $\eta_{p1} = 1.4$ for indented and crimped wires and $\eta_{p2} = 1.2$ for 7-wire strands;

η_{p2} takes into account the position of the tendon; $\eta_{p2} = 1.0$ for all tendons with an inclination of 45°-90° with respect to the horizontal during concreting; $\eta_{p2} = 1.0$ for all horizontal tendons which are up to 250 mm from the bottom or at least 300mm below the top of the concrete section during concreting, and $\eta_{p2} = 0.7$ for all other cases.

Scribner and Kobayashi (1984) conducted bond tests on seven-wire strands under repeated and reversed inelastic pullout. The main parameters of the tests were concrete compressive strength, strand diameter, lateral confining pressure and slip of strand relative to surrounding concrete. Their tests provided basic qualitative information about the effect of the four variables. However, because of a small number of specimen tested, no attempt was made to quantitatively describe the bond-slip relationship for the strands. The test results showed that concrete strength and confining stress were the two most important factors governing overall bond behaviour after initial bond stress had been overcome.

Adachi and Nishiyama (2000) carried out bond tests on prestressing strands. The concrete compressive strength was 36.7 MPa and the grout compressive strength was 29.0 MPa. 12.7mm SWPR-7B strand was used. The following conclusions were derived:

(1) The maximum bond strength attained was 2.98 MPa in average.
(2) The average slip at the maximum bond strength was 0.09 mm.
(3) Bond behaviour under cyclic loading was presented as shown in Fig.A-8

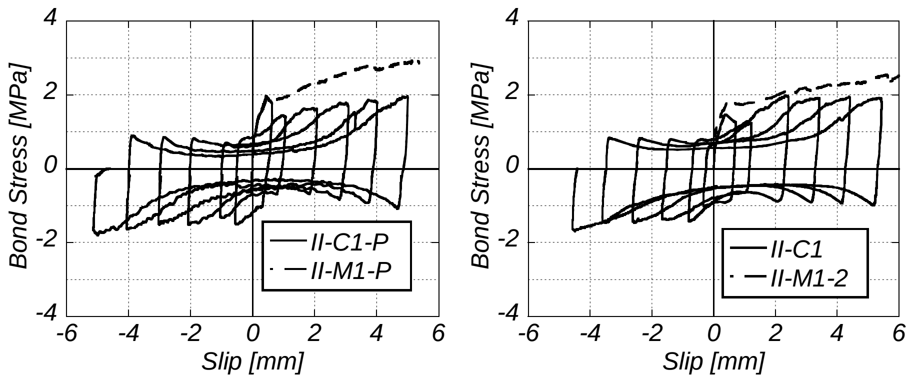

Fig. A-8: Bond behaviour of prestressing strand under cyclic loading

Based on experimental results and a model proposed by Morita and Kaku (1975), Adachi proposed a bond-slip model for prestresseing strand. For monotonic loading, due to the stress reduction observed soon after bond yield stress, Point A (6.0 $S_{\tau y}$, 0.65 τ_y) shown in Fig. A-9 was chosen as a characteristic point. Point B (50.0 $S_{\tau y}$, 1.0 τ_y) is a reference point used for the ascending segment following Point A. the envelope for cyclic loading is shown in Fig. A-10.

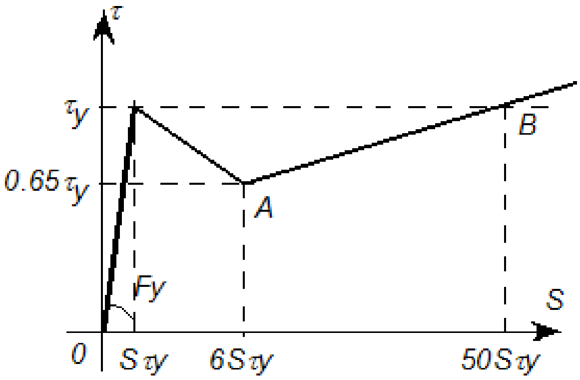

Fig. A-9: Bond-slip model for monotonic loading

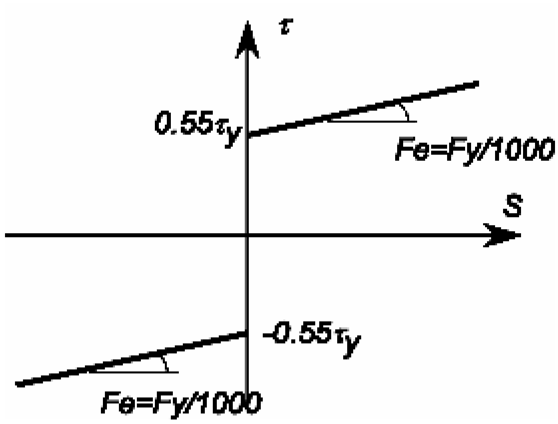

Fig. A-10: Envelope for reversed cyclic loading

10 Miscellaneous elements and structures

10.1 Introduction

Precast concrete elements have been widely used for the construction of special structures such as shells or folded plate roofs. These structures typically emulate cast-in-place construction details to achieve seismic resistance.

Brittle precast elements that are not designed to contribute to earthquake resistance must be protected from damage during seismic displacement of the primary lateral load resisting structure. These elements must also be prevented from contributing significant lateral load stiffness to parts the structure, and adversely altering its seismic performance. Examples include precast stairs, ramps and non-structural cladding panels.

10.2 Shells and folded plates

Shells and folded plates are used for roof construction in some earthquake prone countries. Zhenqiang (1983) reports very good performance of these roofs in the Tangshan (China) earthquake in 1976. A special feature of their design is detailing the supports for serviceability limit state temperature and shrinkage effects with limited restraint, while restraining the supports for ultimate limit state lateral forces.

Anchorage of connection hardware in the thin shell or plate sections is a key requirement. These connections must be designed for the full ductility demand imposed on them by the ultimate limit state actions. Zhenqiang [Zhenqiang (1992)] refers to the use of “semi-rigid” connections from the folded plate roof elements to folded plate supporting walls.

10.3 Stairs

10.3.1 General

Stairs are essential for the post-earthquake evacuation of buildings, but are often unintentionally built into the structure in such a manner that they act as rigid bracing elements. This bracing effect results from the floors moving horizontally relative to each other during a major seismic event. Stairs with landings are not normally reinforced to act as compression braces and can be expected to fail in a very brittle manner. Such behaviour was observed in the earthquakes of San Fernando (1971), Nicaragua (1972), Lima (1974), El Asnam (1980), Guam (1993) and Northridge (1994). The overload of the stairs, and in some cases, the supporting structures (landings and intermediate height connections of stairs to columns and walls) led to loss of gravity carrying capacity of the stairs and supporting structures.

Of particular concern is a variety of stairs that incorporated a mid-height landing (e.g. Fig. 10-1).

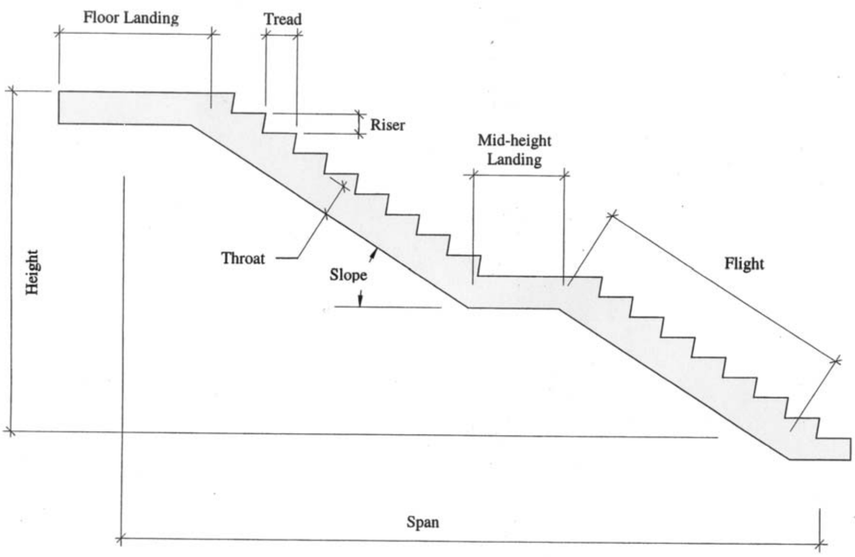

Fig. 10-1: Example of a stair flight with a mid-height landing

10.3.2 Detailing the supports

If the precast stair is built in at both ends and restricts the interstorey drift of the floors supporting the stair flight, then stair collapse is a possibility through either:

- the landings at the top or bottom of the stair being damaged
- if the stair has a mid height landing, the knee of the landing may rupture or a plastic hinge may form in the lower flight of the stair [Simmons (2000)].

It is recommended that one end of the stair be fixed to either the top or the bottom landing. The other end is permitted to slide. This configuration accommodates the interstorey drifts that result from a major earthquake.

The easiest way to build a precast concrete stair into the structure is to connect the stair at its upper end. This is because the precast stair unit is lifted in through the stair well penetration on the floor that is being constructed. The fixed end of the stair can be cast in as part of that process. The lower, sliding end is supported on the completed floor/landing below.

Detailing the stair to slide on its support is preferable to designing the stair-to-landing joint for the magnitudes of the bracing or propping effect or for ductility in the connections. The stair may require to be used in darkness, or emergency lighting conditions. Hinging at the landing junction, while technically feasible, does create a significant hazard that should be eliminated.

The connections required for fixity should act as a supporting beam, spanning across the end of the stair, framing the edge of the floor/landing [eg, Fig. 10-2 (a) and (b)].

The preferred sliding end detail is shown in Fig. 10-2(c). The gap details (for sliding) of Fig. 10-2 (d) and (e) are sized to accommodate the expected interstorey drift. However, these details are more complex to build and hence more costly than the detail Fig. 10-2 (c). Figure 10-2 (c) is favoured because it avoids a recurring problem with the cover plates, which are often removed so the gap can be filled to allow for easier fixing of carpet or vinyl floor coverings. Once the gap is filled (usually with some sort of cement mortar), the stair flight acts as a very effective compression brace.

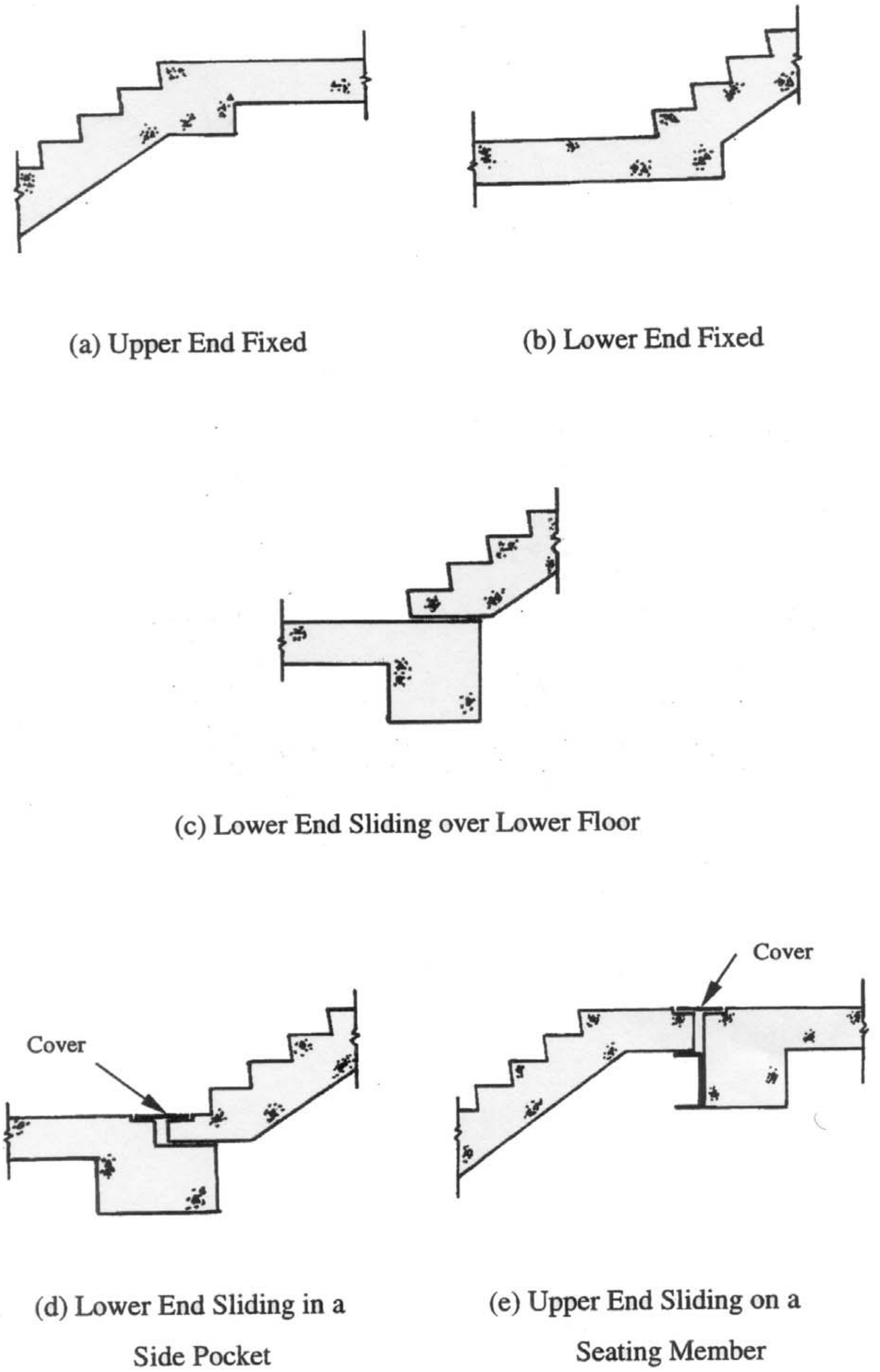

Fig. 10-2: End connection details for precast stairs

10.3.3 Reinforcement details

In New Zealand, it is common practice to develop a bending moment envelope that assumes full fixity or continuity at the built-in support. By doing so, the maximum amount of tension reinforcement required along the top of the stair, as well as the maximum shear demand is found. Further, a bending moment assuming the stair acts as a simply supported beam is selected to maximise the tension reinforcement in the bottom of the stair. The actual bending moment demand lies in a narrower envelope that depends on the extent of fixity at the built-in support. This continuity should not be relied upon especially after a major seismic event due to the likely damage to the supporting beam/landing.

It is recommended by Simmons (2000) that transverse reinforcement (ties) be placed at the knee to resist the bursting forces (see Figure 10-3) and to reduce the buckling of any longitudinal reinforcement, should cracking through the knee occur. Buckling of the reinforcement, coupled with spalling of the concrete tread and landing, leads to significant reduction of internal lever-arms to resist bending and creates a hazard for people evacuating the stairwell. The ties are sized through consideration of equilibrium of forces. Further, the design actions (flexure, shear and axial) that result from either tension or compression, induced by friction at the sliding end, should be combined in accordance with an appropriate Limit State Design Code, and be designed for.

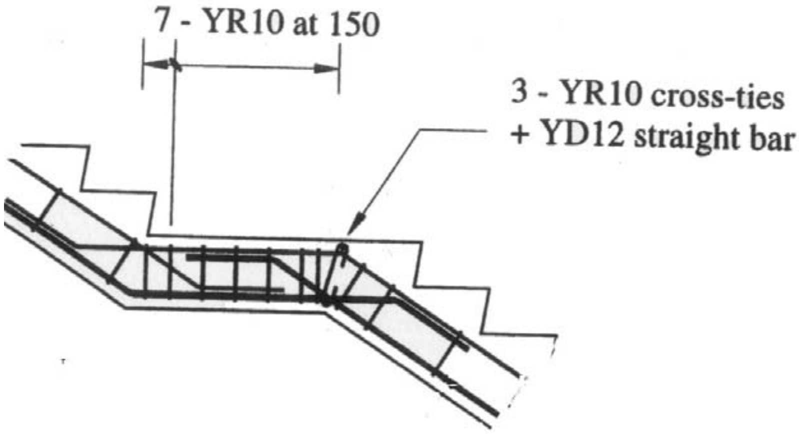

Fig.10-3: Transverse reinforcement to resist bursting at the knee of a landing

10.4 Architectural cladding panels

While architectural panels can provide part of the earthquake resistance of the structure [Giaotti et al (1992)] more commonly they are designed to act independently from the lateral load resisting system. The designer of the structure will have determined drift limitations that are imposed by building code requirements, or the proximity of other structures. Panel connections must accommodate this drift without failure. Failure of the connections can cause panels to fall from the building, as has occurred in many earthquakes. Failure of panel connections during earthquakes has been attributed to:

- brittleness induced by corrosion due to water penetration or condensation on the back of facade panels;
- lack of ductility in the connections or their anchorage;
- lack of ability to accommodate the required sliding movement.

Cladding panel connections must be detailed for adequate corrosion protection for the anticipated building life, or must be accessible for maintenance. Connections exposed to chlorides (parking structures) or chemical attack (pulp and paper factories) may require regular inspection, or special protection.

Connections that are detailed for ductility must be able to resist the serviceability limit state actions, while sliding freely or deforming elastically but must have the ability to yield for the full relative displacement expected at the ultimate limit state of the design earthquake. The deformation capacity of well-designed connectors can be used to protect the cladding panels from seismic forces. Anchorage details, both in the precast panel and in the supporting structure, must be adequate for the full over-strength capacity of the yielding connector and welding procedures (if used) must ensure ductile welds. These lessons have been learned in some regions, Iverson and Hawkins [Iverson et al (1994)] report that no damage to precast cladding panels or to their connections was observed following the 1994 Northridge Earthquake.

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11 Conclusions

11.1 General

This state-of-art report discusses the practice of using precast and prestressed concrete in countries in seismic regions. The report has contributions from 30 contributors from nine different countries.

The main sections of the report are: state of the practice in various countries; advantages and disadvantages of incorporating precast reinforced and prestressed concrete in construction; lessons learned from previous earthquakes; precast construction concepts; design approaches; primary lateral load resisting systems (moment resisting frames and structural walls including dual systems); diaphragms of precast concrete floor units; modelling and analytical methods; gravity load resisting systems; foundations and miscellaneous.

Design equations are presented where necessary but the emphasis of the report is on principles. Ordinary cast-in-place concrete construction is not considered in the report.

The object of the report is to present existing practice, to recommend good practice, and to discuss current developments.

11.2 State of the practice in various countries

Precast reinforced and prestressed concrete elements are widely used in the structural systems of buildings in many seismic zones of the world; for example in Asia; Europe; North, South and Central America and New Zealand. Particularly significant use of precast concrete is made in some European countries and in Japan and New Zealand for floors, moment resisting frames and structural walls.

11.3 Advantages and disadvantages of incorporating precast concrete in construction

The main advantages of incorporating precast reinforced and precast concrete in construction are the possible increase in speed of construction, the high quality of precast concrete units and the improved durability, the reduction in site labour, and the reduction in site formwork.

The main disadvantages are that economical and effective means need to be developed for joining precast concrete elements together to resist seismic actions, the construction techniques for the joints between precast concrete elements may be unfamiliar and need to be conducted with good quality control, relatively small tolerances may need to be worked within, and enhanced craneage may be required to lift heavy precast concrete units.

11.4 Lessons learned from previous earthquakes

Precast reinforced and prestressed concrete has had significant and successful application in earthquake resisting structures in many parts of the world. Experience of earthquakes and laboratory testing gives confidence that precast reinforced and prestressed concrete elements can be used very successfully in structures designed for earthquake resistance providing careful attention is paid to design and construction. Poorly designed and constructed precast reinforced concrete structures have performed badly in some major earthquakes due to brittle (non-ductile) behaviour of poor connection details between the precast elements, poor detailing of the elements, and poor design concepts. This has resulted in the use of precast concrete in earthquake resisting structures being regarded with suspicion in some countries. However, experience has shown that structures incorporating precast concrete elements which are well designed and constructed for seismic resistance will perform well in earthquakes.

11.5 Types of connections between precast concrete elements used in the construction of moment resisting frames and structural walls

11.5.1 Broad categories of construction

The construction of moment resisting frames and structural walls incorporating precast concrete elements fall into two broad categories, either “equivalent monolithic” systems or “jointed” systems. The distinction between these two types of construction is based on the design of the connections between the precast concrete elements.

11.5.2 Equivalent monolithic systems

The connections between precast concrete elements of equivalent monolithic systems (cast-in-place emulation) can be subdivided into two categories.

(a) Strong connections of limited ductility

Strong connections of limited ductility of equivalent monolithic systems are designed to be sufficient strong so that the connections remain in the elastic range when the building is satisfying the ductility demand imposed by the earthquake. That is, the yielding occurs elsewhere in the structure. Precast reinforced concrete elements have protruding longitudinal bars which are connected either by lap splices in a cast-in-place concrete joint or by non-contract lap splices involving grouted steel corrugated ducts, or by splice sleeves, or by welding, or by mechanical connectors. These connections are not designed to be ductile and hence may have limited ductility if subjected to cyclic yielding. In moment resisting frames and structural walls these connections are protected by a capacity design approach which ensures that flexural yielding occurs away from the connection region.

(b) Ductile connections

Ductile connections of equivalent monolithic systems are designed for the required strength and with longitudinal reinforcing bars or grouted post-tensioned tendons in the connection region which are expected to enter the post-elastic range in a severe earthquake. Yield penetration may occur into the connection and region. The plastic hinge region may extend a distance along the end of the member as in cast-in-place construction.

11.5.3 Jointed systems

In jointed systems the connections are weaker than the adjacent precast concrete elements. Jointed systems do not emulate the performance of cast-on-place concrete construction. The connections between precast concrete elements of jointed systems can be subdivided into two categories.

(a) Connections of limited ductility

Connections of limited ductility of jointed systems are usually dry connections formed by welding or bolting reinforced bars or plates or steel embedments and dry-packing and grouting. These connections do not behave as if part of monolithic construction and generally have limited ductility. An example of a jointed system with connections of limited ductility involving structural walls is tilt-up construction. Generally such structures are designed for limited ductility or nominally elastic behaviour.

(b) Ductile connections

Ductile connections of jointed systems are generally dry connections in which unbonded post-tensioned tendons are used to connect the precast concrete elements together. The non-linear deformations of the system are concentrated at the interfaces of the precast concrete elements where a crack opens and closes. The unbonded post-tensioned tendons remain in the elastic range. These connections have the advantage of reduced damage and of being self-centering (i.e. practically no residual deformation) after an earthquake.

Hybrid systems have dry connections which combine both unbonded post-tensioned tendons and longitudinal steel reinforcing bars (tension/compression yield) or other energy dissipating devices (e.g. flexing steel plates or friction devices). The post-elastic deformations of the system during an earthquake are again concentrated at the interfaces of the precast concrete elements where a crack opens and closes.

11.6 Tolerances and erection

The design and construction of precast concrete structures to resist seismic actions requires an appreciation of product, erection and interfacing tolerances. The designer and erector must possess knowledge of the capabilities and limitations of precast element production processes, and an understanding of how the structure will be safely assembled.

11.7 Design approaches

The required performance criteria for structures incorporating precast reinforced concrete elements adopted in seismic design are generally similar to those for cast-in-place construction. Generally for seismic design at the ultimate limit state force-based design is used along with the capacity design approach.

Primary lateral force resisting systems can be designed for nominally elastic, limited ductility or ductile response. The magnitude of the design lateral forces decreases, whilst the complexity in the detailing of the reinforcement in the critical regions increases, as the design approach goes from nominally elastic to a fully ductile response. For structures, incorporating precast concrete elements emulating ductile cast-in-place construction the detailing rules are as for monolithic structures. The post-elastic mechanism of deformation should involve flexural yielding at plastic hinges. A capacity design procedure should be used to ensure that the mechanism can be maintained during the design earthquake.

11.8 Primary lateral force resisting systems

11.8.1 General

In general, primary lateral force resisting systems built incorporating precast concrete elements are either “equivalent monolithic” or “jointed” moment resisting frames or structural walls. Dual systems, in which moment resisting frames and structural walls are combined to provide lateral force resistance in the same direction, are also used, but to a much lesser extent. However it is common practice, particularly in regions of low and moderate seismicity, to combine moment resisting frames or structural walls with frames carrying mainly gravity loading.

11.8.2 Moment resisting frames

(a) Ductile equivalent monolithic reinforced concrete moment resisting frames incorporating precast concrete elements

Four equivalent monolithic ductile reinforced concrete construction systems are widely used in Japan and New Zealand. In System 1 the precast concrete beam elements are placed between columns and the beam-column joint core is cast-in-place. In System 2 the precast concrete beam elements are placed over the columns below with the vertical column bars passing through and grouted in the beam unit. The beams are connected by a cast-in-place concrete joint at midspan. In System 3 T-shaped or cruciform shaped precast concrete elements are connected vertically and horizontally. In System 4 pretensioned prestressed concrete U-beams and cast-in-place reinforced concrete are used. In all systems a precast concrete floor system is placed seated on the top of the precast concrete beams.

The precast concrete column elements of Systems 2 and 3 can be spliced either at the end above the beam or at mid height. Either grouted steel sleeves or non-contact lap splices involving grouted corrugated steel ducts are used to connect the vertical column bars. The precast concrete beam elements in Systems 2 and 3 are connected by a cast-in-place concrete joint at midspan in which the horizontal beam bars are connected by either bar laps or by mechanical couplers or by grouted steel sleeves or by welding.

(b) Jointed reinforced moment resisting forces of limited ductility incorporating precast concrete elements

Some jointed reinforced concrete frame systems that have relatively weak connections between precast elements, achieved mainly by welding or bolting and dry packing, have been attempted. Such systems should be used with caution since they can have very limited ductility.

(c) Ductile equivalent monolithic moment resisting frames incorporating bonded post-tensioned or pretensioned tendons

Ductile equivalent monolithic construction incorporating grouted post-tensioned tendons is widely used in Japan and has been used in New Zealand since the 1960s. The two main systems used have precast concrete beam elements placed between the columns either with temporary end seating before post-tensioning or with seating on concrete corbels. Alternatively pretensioned precast concrete beams can be placed on columns as for reinforced concrete System 2.

(d) Ductile jointed moment resisting frames incorporating unbonded post-tensioned tendons and non-prestressed reinforcement

In ductile jointed systems the precast concrete elements are connected by unbonded post-tensioned tendons. Hybrid systems are jointed systems which combine unbonded post-tensionsed tendons with longitudinal non prestressed bars which provide the energy dissipation. This system has the advantage of reduced damage and of being self centering after an earthquake.

11.8.3 Structural walls

(a) Equivalent monolithic reinforced concrete structural walls incorporating precast concrete elements

At horizontal joints between precast concrete wall panels the vertical reinforcement protruding from one end of the panel and passing through the mortar of the joint can be connected to the adjacent panel or foundation beams by means of either grouted steel splice sleeves or grouted corrugated metal ducts. Vertical joints between precast concrete wall panels are typically strips of cast-in-place concrete into which horizontal reinforcement from the ends of the adjacent panels protrude and are lapped. Support for precast concrete floor units at walls can be achieved using steel angles or a concrete corbel or a recess in the wall.

(b) Jointed reinforced concrete walls of limited ductility

In tilt-up construction precast reinforced concrete wall panels generally are secured to the adjacent elements using jointed connections comprising various combinations of concrete inserts, bolted or welded steel plates or angle brackets, and lapped reinforcement splices within cast-in-place joining strips. Tilt-up construction is generally designed for elastic or limited ductile response.

(c) Ductile hybrid structural walls

Ductile hybrid structural walls which contain unbonded post-tensioned tendons with non-prestressed longitudinal steel reinforcement and/or other energy dissipating devices.

11.9 Diaphragms

Floors and roofs need to be designed to transfer by diaphragm action the in-plane imposed seismic forces to the supporting structure as well as gravity loading. Diaphragm action can be achieved either by providing a cast-in-place reinforced concrete topping slab over the precast floor units or by appropriately reinforced joints between the precast floor units. Support of the floor units must be maintained throughout the movements caused by the earthquake (for example due to elongation of plastic hinges of supporting beams). Hence seating must be adequate and special support reinforcement may need to be provided.

11.10 Modelling and analytical methods

Models based on fibre elements or lumped plasticity can be devised for use in structural analysis to account for the behaviour of precast concrete systems at both local and global levels including the characteristics of the joints, type of longitudinal steel, bond conditions and anchorage.

11.11 Gravity load resisting systems

Elements of gravity load resisting systems need to be designed and detailed to retain their gravity load carrying capacity during the lateral displacements of the primary lateral force resisting system during an earthquake.

11.12 Foundations

Precast concrete columns are conveniently connected to foundation beams using a socket-base connection. A significant use of precast concrete for foundations is for piles suitably detailed for ductility. Pretensioned prestressed concrete piles are often preferred.

11.13 Miscellaneous

Precast concrete elements have been widely used for various non-structural elements such as stairs, cladding panels and partitions. They need to be constructed so as to permit unrestricted movement of the primary lateral force resisting system during an earthquake.

11.14 Summary

Precast and prestressed concrete can be used successfully in structures designed for earthquake resistance, providing careful attention is given to conceptual and detailed design and to fabrication and erection. The ***fib*** state-of-the-art report on the seismic design of precast concrete building structures is intended to assist designers and constructors to provide safe and economical applications of structural precast concrete and at the same time to allow innovation in design and construction to continue.