A Perspective on the Seismic Design of Precast Concrete Structures in New Zealand

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The author describes trends and developments in the use of precast concrete in New Zealand for floors, moment resisting frames, and structural walls of buildings. Currently, almost all floors, most moment resisting frames, and many one- to three-story walls in buildings are constructed incorporating precast concrete elements. Aspects of design and construction, particularly the means of forming connections between precast concrete elements, are discussed. The paper emphasizes seismic design; the aim of the design methods for frames is to emulate monolithic construction. Examples of recent precast concrete buildings using the above discussed methods are presented.

In addition to his duties as a faculty member in the Department of Civil Engineering, Professor Park is Deputy Vice-Chancellor of the University of Canterbury and Chair of the Concrete Design Committee of Standards of New Zealand. During his long professional career, he has received many awards and honors, including PCI's Martin P. Korn Award (twice) and ASCE's T. Y. Lin Award. Under his leadership, the University of Canterbury is well known throughout the world for its pioneering research in the seismic design of structures.

Since the early 1960s, there has been a steady increase in the use of precast concrete for structural components in buildings in New Zealand. The use of precast concrete in flooring systems has been commonplace since the 1960s, making cast-in-place floor construction generally uncommon. Also, precast concrete non-structural cladding for buildings has been widely used.

During the boom years of building construction in New Zealand in the mid to late 1980s, there was also a significant increase in the use of precast concrete in moment resisting frames and structural walls. Precast concrete elements have the advantages of high quality control, a reduction in site formwork and site labor, and increased speed of construction. In particular, with high interest rates and pressure for new building space in the mid 1980s, the speed advantage gave precast concrete a distinct cost advantage. Contractors adapted to precast concrete construction with increased crane capacity, new construction techniques, and off-site fabrication.

The increase in the use of precast concrete in the 1980s required considerable innovation because of New Zealand's location in an active seismic zone; the seismicity of most of New Zealand is similar to that of California. At the time, the New Zealand concrete design standard, NZS 3101:1982,[1] like the concrete codes of many countries, contained comprehensive provisions for the seismic design of cast-in-place concrete structures but did not have seismic provisions covering all aspects of precast concrete structures. The design ultimate seismic forces used for the design of ductile moment resisting frames in the most seismically active parts of New Zealand are very similar to those recommended by the Uniform Building Code[2] in Zones III and IV of the United States.

In the past, some framed structures incorporating precast concrete elements have performed poorly in earthquakes in many countries because of poor connection details. As a result, precast concrete in moment resisting frames was excluded in New Zealand for many years. Confidence in the use of precast concrete in moment resisting frames required the development of satisfactory methods for connecting the precast elements together. The design methods that were introduced in New Zealand in the 1980s for frames of buildings incorporating precast concrete elements generally aimed to achieve behavior equivalent to that of a completely cast-in-place concrete structure. That is, the objective of the design method is to emulate monolithic construction.

With the increase in the innovative use of precast concrete elements in buildings in New Zealand came an increasing concern that some of the design solutions should be more fully researched. Even if there were no reason to doubt the validity of extrapolating the results of design and construction procedures that were originally developed for cast-in-place concrete, the large number of important buildings employing precast concrete for seismic resistance demanded that more research and testing be done to justify confidence in the structural systems.

In February 1988, a seminar at the University of Canterbury, attended by designers, researchers, fabricators and constructors, highlighted a growing need to investigate and verify aspects of the performance of precast concrete in building structures designed for seismic resistance. Following the seminar, a Study Group, jointly funded by the New Zealand Concrete Society, the New Zealand National Society for Earthquake Engineering, and the Centre for Advanced Engineering at the University of Canterbury, was formed with the following objectives:

1. Summarize data on precast concrete design and construction

2. Identify special concerns

3. Indicate recommended practices

4. Recommend topics requiring further research

The outcome of the deliberations of the Study Group during 1988-91 was the publication of a manual authored by the members of the Study Group titled "Guidelines for the Use of Structural Precast Concrete in Buildings," which was first printed in August 1991.[3]

A new revision of the New Zealand concrete design standard, NZS 3101: 1995,[4] is being published in 1995. This revision contains more provisions for the seismic design of structures incorporating precast concrete than its predecessor.

This paper describes aspects of the design and construction of buildings in New Zealand incorporating precast concrete structural elements in floors, moment resisting frames, and structural walls. Design and construction for seismic resistance are also emphasized because that is where the greatest difficulties lie in the connection of precast concrete elements.

SEISMIC DESIGN CONCEPTS FOR PRECAST CONCRETE IN BUILDINGS

General Requirements

For moment resisting frames and structural walls constructed incorporating precast concrete elements, the challenge lies in finding an economical and practical means of connecting the precast concrete elements together to ensure adequate stiffness, strength, ductility, and stability. The design should consider the loadings during the stages of construction and at the serviceability and ultimate limit states during the life of the structure. The design should ensure that the structure performs satisfactorily in the service load range, has a reasonable margin of safety before the ultimate load is reached, and will not fail catastrophically at the ultimate load.

In common with other countries, the seismic design forces recommended for structures in the current New Zealand loadings standard for general structural design and design loadings for buildings, NZS 4203:1992,[5] 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 structure ductility factor $\mu = \Delta_{max}/\Delta_y$, where Δ_{max} is defined as the maximum horizontal displacement that can be imposed on the structure during several cycles of seismic loading without significant loss in strength, and Δ_y is defined as the horizontal displacement at first yield assuming elastic behavior of the cracked structure up to the design seismic force.

In the New Zealand loadings standard, NZS 4203:1992,[5] for ductile structures, $\mu = 5$ or 6 is used to determine the appropriate spectra of seismic coefficients from the elastic response spectra. The design ultimate horizontal seismic forces typically vary between $0.03g$ and $0.20g$, depending on the seismic zone, the soil category, the importance of the structure and the fundamental period of vibration of the structure.

For structures of limited ductility, $\mu = 3$ is used and the design ultimate horizontal seismic forces typically vary between $0.03g$ and $0.39g$. The design ultimate seismic forces recommended in the New Zealand loadings standard[5] for cast-in-place concrete structures and for structures incorporating precast concrete elements of the same available ductility, are identical. Note that as an alternative to ductile structures, designers can design structures of limited ductility with higher design ultimate seismic forces but with less stringent requirements for detailing for ductility.

The exact characteristics of the earthquake ground motions that can occur at a given site cannot be predicted with certainty and it is difficult to evaluate all aspects of the complete behavior of a complex structure when subjected to a major earthquake. Nevertheless, it is possible to design the structure to ensure the most desirable behavior. The rational approach for achieving this objective in the design for earthquake resistance is to choose the most suitable mechanism of post-elastic deformation for the structure and to ensure, by appropriate design procedures, that yielding of structural members will occur only in the chosen manner during a major earthquake and that the available ductility is adequate.[4,5]

If all the elements of the structure resisting seismic forces are detailed for ductility in accordance with the seismic provisions of the concrete design standard,[4] adequate ductility is considered to be provided.

Capacity Design

To ensure that the most suitable mechanism of post-elastic deformation occurs in a structure during a major earthquake, New Zealand standards NZS 4203:1992[5] and NZS 3101:1995[4] require that ductile structures be the subject of capacity design. In the capacity design of structures, elements of the primary lateral earthquake load resisting systems are suitably designed and detailed for adequate strength and ductility for a major earthquake. All other structural elements and 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.[4,5]

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. 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 forces likely to be imposed on the structure should be calculated from the probable maximum flexural strengths at the plastic hinges. This is done by taking into account all the possible factors that may cause an increase in the flexural strength of the plastic hinge regions.

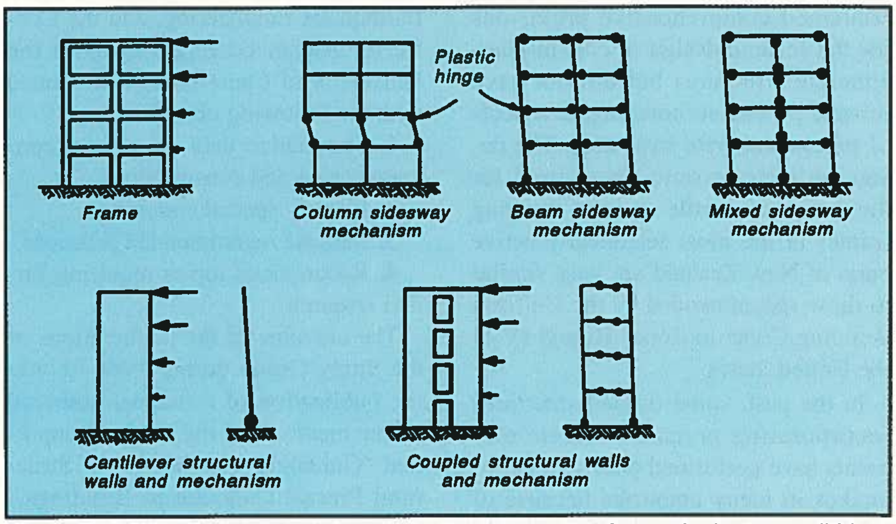


Fig. 1. Possible mechanisms of post-elastic deformation for equivalent monolithic moment resisting frames and structural walls of buildings during severe seismic loading.

These factors include an actual yield strength of the longitudinal reinforcing steel, which is higher than the lower characteristic yield strength,* and additional longitudinal steel strength due to strain hardening at large ductility factors. Due to these two factors, the steel overstrength in New Zealand is taken to be 1.25 times the lower characteristic yield strength when calculating the probable maximum flexural strength in the plastic hinge regions.

As a result, the shear reinforcement in the plastic hinge regions, and all flexural reinforcement in parts of the structure away from plastic hinge regions, will need to be designed for shear forces and bending moments that are at least $1.25/\phi$ times the shear forces and bending moments associated with the design bending moments of the plastic hinge regions. This ensures that non-ductile failures do not occur elsewhere, where 1.25 is the steel overstrength factor and ϕ is the strength reduction factor used for designing the flexural reinforcement at the plastic hinge, taken as 0.85 in New Zealand.[4] If plastic hinges in columns of moment resisting frames are to be avoided (that is, strong column-weak beam behavior is sought), the design column bending moments may need to be amplified by much more than $1.25/\phi$ in order to guard against higher mode effects and concurrent earthquake forces as well as the overstrength of steel in beams.[4]

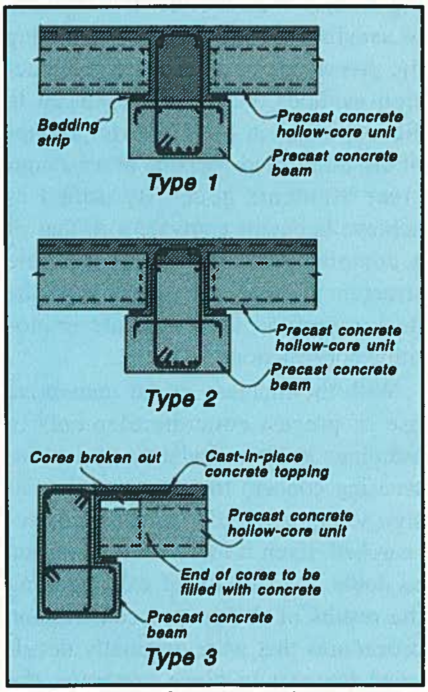


Fig. 2. Types of support using precast concrete beams for hollow-core floor units (Ref. 3).

The use of capacity design has given designers confidence that structures can be designed for predictable behavior during major earthquakes. In particular, brittle elements can be protected and yielding can be 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 behavior, because

* The lower characteristic yield strength is defined as the value of the yield strength below which not more than 5 percent of production tests in each size fall.

any brittle connections between elements can be designed to remain in the elastic range during a major earthquake.

Preferred Modes of Post-Elastic Deformation

Fig. 1 shows mechanisms of post-elastic deformation that could occur in equivalent monolithic moment resisting frames and structural walls due to the formation of plastic hinges during a severe earthquake. These mechanisms are idealizations in that they involve behavior under the typical equivalent static seismic forces recommended by codes, which are based mainly on the first mode of vibration. The actual dynamic situation for moment resisting frames and coupled structural walls can be different due to the effects of higher modes of vibration that can result in the plastic hinges in the beams forming in a few stories at a time and moving in waves up the structure during the earthquake. Nevertheless, the static mechanisms of Fig. 1 give designers a reasonable sense for the actual situation.

As shown in Fig. 1, if yielding begins in the columns of a moment resisting frame before it begins in the beams, a column sidesway mechanism can form. In the worst case, the plastic hinges may form in the columns of only one story because the columns of the other stories are stronger. Such a mechanism can make very large curvature ductility demands on the plastic hinges of the critical story,[6] particularly for tall buildings.

On the other hand, if yielding begins in the beams before it begins in the columns, a beam sidesway mechanism will develop, which makes much more moderate demands on the curvature ductility required at the plastic hinges in the beams and at the column bases. Therefore, a beam sidesway mechanism is the preferred mode of post-elastic deformation, particularly because ductility can be more easily provided by reinforcing details in beams than in columns.

As a result of the above considerations, the New Zealand concrete design standard[4] requires that columns of multistory ductile moment resisting

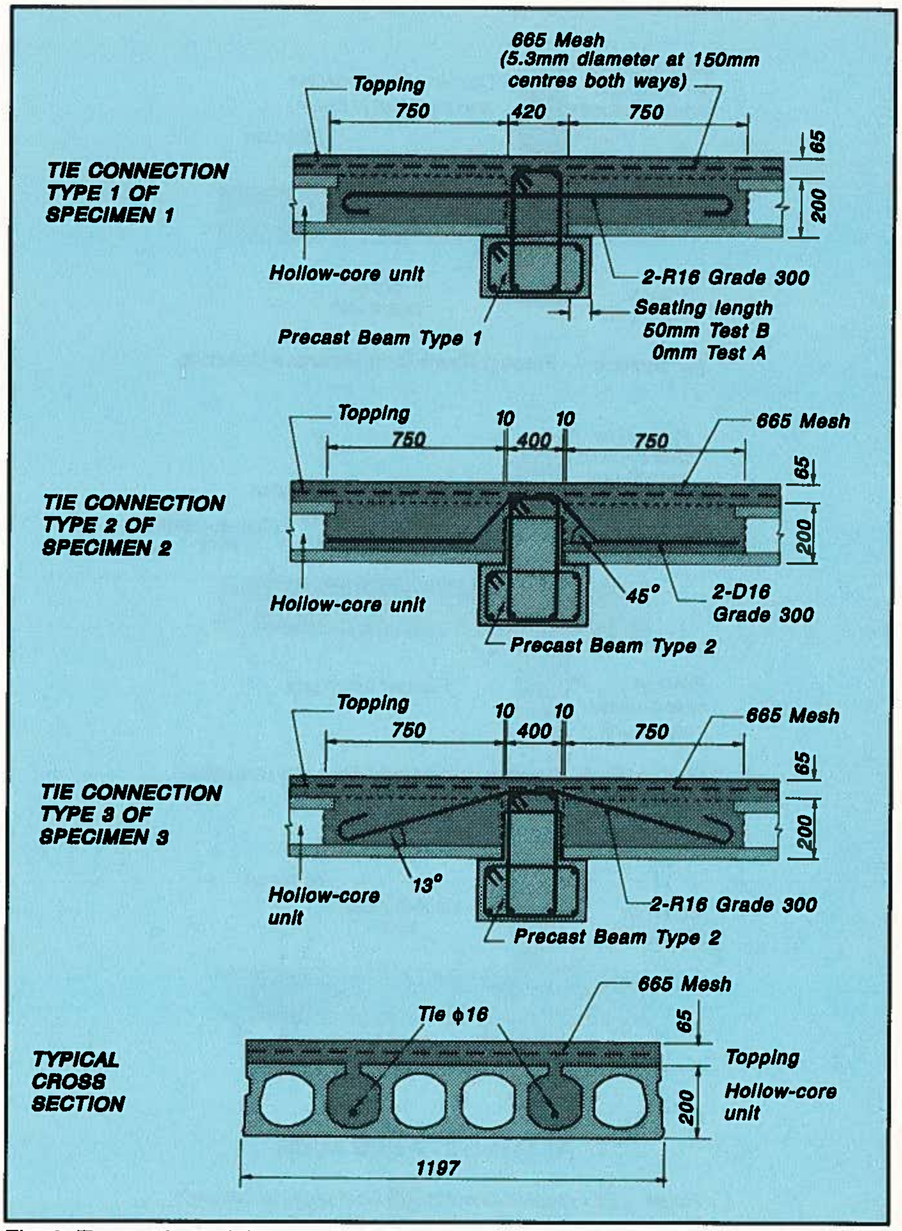


Fig. 3. Types of special support reinforcement at the ends of hollow-core floor units tested at the University of Canterbury (Ref. 13).

frames should have sufficient flexural strength to avoid the formation of column sidesway mechanisms as far as possible. Thus, a strong column-weak beam approach to design is advocated.

The New Zealand concrete design standard[4] has two exceptions to this rule:

1. In some buildings in areas of low seismicity and/or where beams have long spans, the gravity load considerations may govern and make a strong column-weak beam design impracticable. In such a case, the interior columns of gravity load dominated ductile frames three stories or higher may be designed to develop plastic hinges in any story simultaneously at the top and bottom ends, while plastic hinges develop in some beams only, typically only in the beams at or near the exterior columns (see the mixed sidesway mechanism in Fig. 1). Such frames are required to be designed for design seismic forces which may be higher than for frames with beam sidesway mechanisms.

2. For ductile frames of one- or two-story buildings, or in the top story of a multistory building, the New Zealand standard permits column sidesway mechanisms (that is, a strong beam-weak column approach) because the curvature ductility demand at the plastic

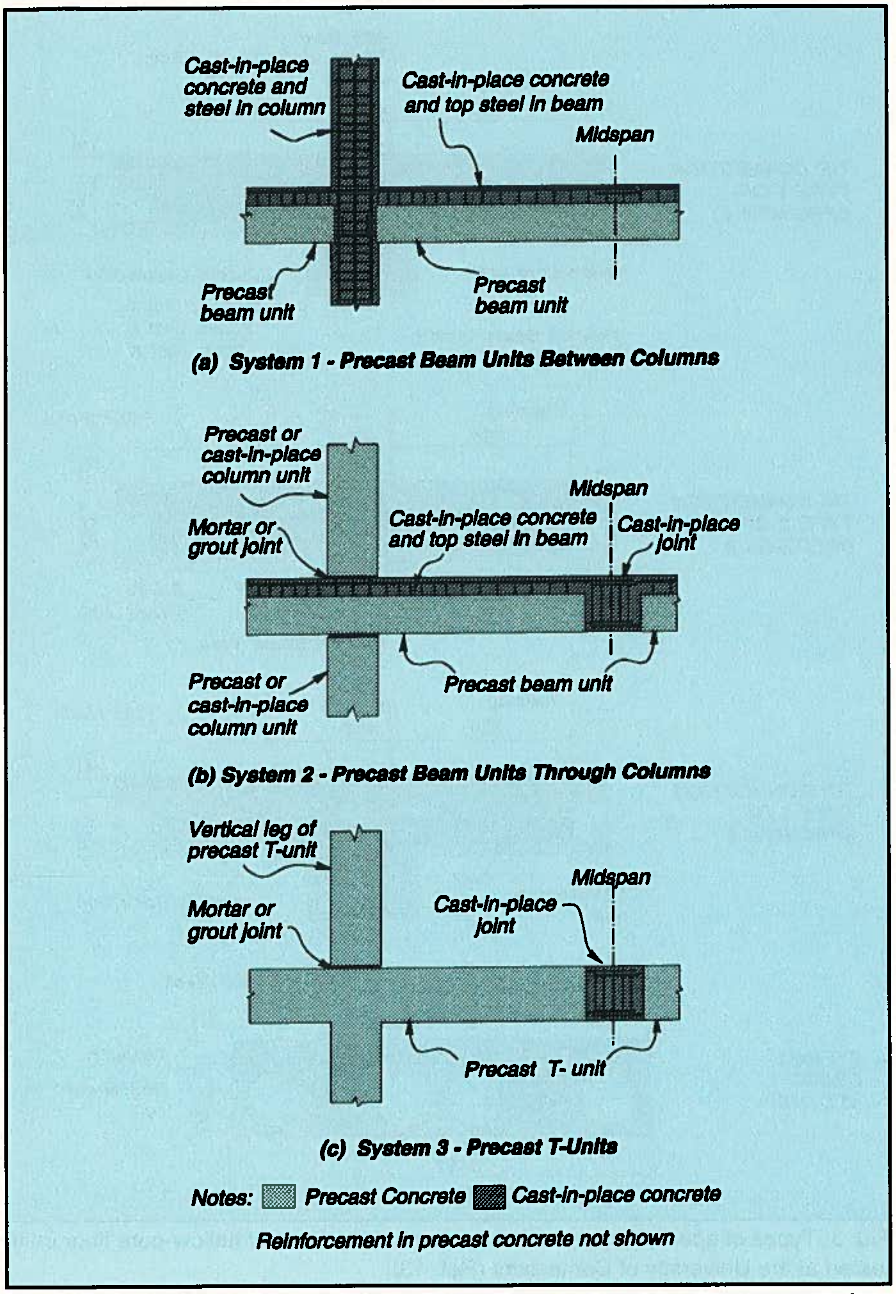


Fig. 4. Arrangements of precast concrete members and cast-in-place concrete for constructing reinforced concrete moment resisting frames (Refs. 14 and 15).

hinges of the columns in such cases of low frames is not high and can be provided by proper reinforcing detailing.

The preferred mechanisms of post-elastic deformation for structural walls are also shown in Fig. 1. For cantilever walls, a plastic hinge forms at the base of the wall. For coupled structural walls, yielding also occurs in the coupling beams and, ideally, the beams should yield before the wall bases.

In summary, a capacity design approach is used in New Zealand to ensure that, in the event of a severe earthquake, flexural yielding of members at the chosen plastic hinge position controls both the strength and post-elastic deformation capacity of the structure. When the connections between the precast concrete elements are placed in critical (potential plastic hinge) regions, the design approach in New Zealand ensures that the behavior of the connection region approaches that of a monolithic cast-in-place structure (equivalent monolithic); thus, monolithic construction is emulated.

Possible brittle connections between members are made over-strong in order to not be in critical regions. Reinforcing details and structural configurations can be arranged to ensure that the plastic hinging occurs away from the jointing faces of precast concrete members and cast-in-place concrete joints, but plastic hinging in regions including the jointing faces is permitted if appropriately designed.

Detailing for Ductility

The most important design consideration for ductility in the plastic hinge regions of reinforced concrete members is the provision of adequate longitudinal compression reinforcement as well as tension reinforcement, and the provision of adequate transverse reinforcement in the form of rectangular stirrups, or hoops overlapping or with cross ties, or spirals.

This ensures they act as shear reinforcement, to confine and, hence, to enhance the ductility of the compressed concrete, and to prevent premature buckling of the compressed longitudinal reinforcement. A center-to-center spacing of transverse bars not exceeding six longitudinal bar diameters in plastic hinge regions is considered necessary to control bar buckling.[4]

Failure modes to be prevented are those due to diagonal tension or diagonal compression caused by shear, excessive plastic hinge rotation of heavily loaded columns, sliding shear along construction joints or other jointing faces or in plastic hinge regions, buckling of compressed longitudinal reinforcement, and bond failure along lapped splices or at anchorages. All of these undesirable failure modes lead to premature strength degradation and reduced ductility. They can be avoided by use of the capacity design procedures.[4]

Joint core regions of beam-to-column connections need special attention because of the critical shear and bond stresses that can develop there during seismic loading.[4,6]

PRECAST CONCRETE IN FLOORS

As is common in many countries, floors in New Zealand buildings in early years were mainly of cast-in-place reinforced concrete construction. Significant use of post-tensioning was also made in cast-in-place concrete floors in the 1950s and 1960s. However, since the 1960s, precast concrete elements have become widely used in floors in New Zealand.

Currently, the majority of floors in New Zealand buildings are constructed of precast concrete units, spanning one-way between beams or walls. The precast concrete units are made of either pretensioned, prestressed concrete or reinforced concrete (solid slabs, voided slabs, rib slabs, single tees or double tees) and generally act compositely with a cast-in-place concrete topping slab of at least 50 mm (2 in.) thickness and containing at least the minimum reinforcement required for slabs.

Alternatively, precast concrete ribs spaced apart with permanent formwork of timber or thin precast concrete slabs spanning between are used acting compositely with a cast-in-place concrete slab. The most common floors are constructed of precast concrete hollow-core floor units. The most frequently used depth of hollow-core unit is 200 mm (7.9 in.) plus a 65 mm (2.6 in.) thick cast-in-place concrete topping. Typical spans are 8 or 9 m (26 or 30 ft) long.

This trend of precast concrete floors in New Zealand has come about because of the reduction in site costs resulting from reduced site labor and fast erection, and also because most precast concrete floors are lighter than cast-in-place floors, resulting in smaller dead loads and seismic forces. The current New Zealand practice of using mainly precast concrete floors contrasts markedly with the practice of most overseas countries that use mainly cast-in-place concrete floors.

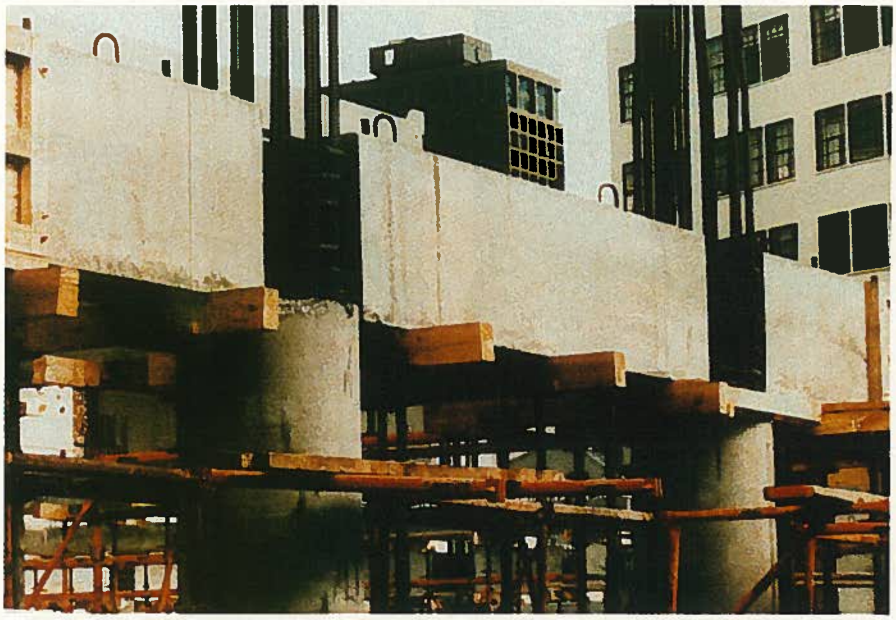


Fig. 5. Reinforced concrete building frame incorporating precast concrete elements in the beams between columns as used in System 1.

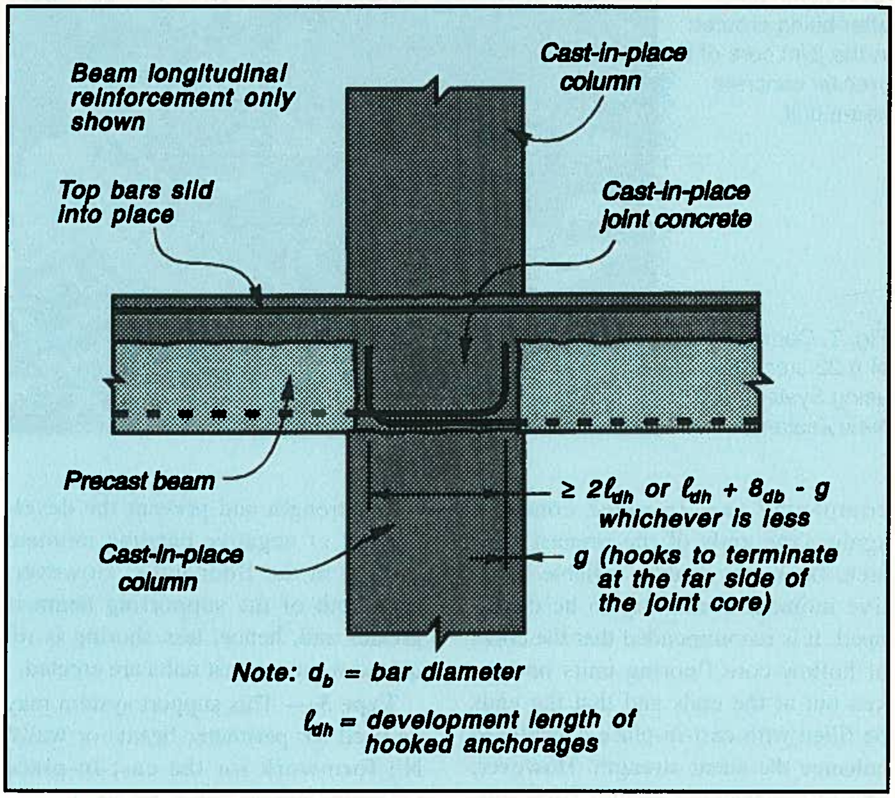


Fig. 6. Hooked lap of bottom bars within joint core for System 1.

Support Details for Precast Concrete Floors

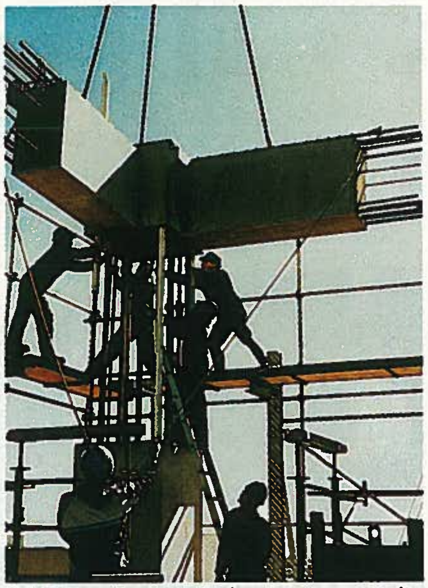
The supports for precast concrete floor units may be either simple or continuous. Both supports have their advantages in different applications. Simple support suits long span or heavily loaded structures where it would be difficult and costly to provide the required degree of negative moment restraint at the supports. Support of precast concrete flooring with moment fixity at the ends suits the more general commercial and residential type of construction, but requires attention to detail in order to ensure that the required degree of continuity can be achieved.

The types of support for precast concrete hollow-core or solid slab flooring units seated on beams, identified by the New Zealand Guidelines,[3] can be divided into the three groups shown in Fig. 2. The difference among these types of support is the depth of the supporting beam prior to placement of the cast-in-place concrete.

Some aspects of these three types of support are:

Type 1 — The presence of well

(a) Structure of reinforced concrete perimeter frame



(b) Precast concrete beam corner unit being lowered into place using temporary plastic tubes as guides

(c) Column bars after being grouted in the joint core of a precast concrete beam unit

Fig. 7. Construction of a 22-story building using System 2 in New Zealand.

compacted cast-in-place concrete against the ends of the precast concrete floor unit enables reliable negative moment continuity to be developed. It is recommended that the cores of hollow-core flooring units be broken out at the ends and that the ends be filled with cast-in-place concrete to enhance the shear strength. However, due to the reduced depth of the supporting beam at the stage when the precast floor units are erected, more shoring is generally required than with the other support types.

Type 2 — If the vertical gaps between the supporting beam and the floor units are too small, there may be difficulty in compacting cast-in-place concrete both in the gaps and in the recesses of the hollow-core units for this type of support. This can reduce the shear strength and prevent the development of negative bending moment actions in the floor units. However, the depth of the supporting beam is greater and, hence, less shoring is required when precast units are erected.

Type 3 — This support system may be used for perimeter beams or walls. No formwork for the cast-in-place concrete topping slab is required.

Adequate support of precast concrete floor units is one of the most basic requirements for a safe structure. It is essential that floor systems do not collapse as the result of imposed movement caused by earthquakes or other effects that reduce the seating length. One source of movements during severe earthquakes that could cause precast concrete floor units to become dislodged is the tendency of the beams of ductile moment resisting frames to elongate when forming plastic hinges. This can cause an increase in the distances spanned by precast concrete floor members.[3,7]

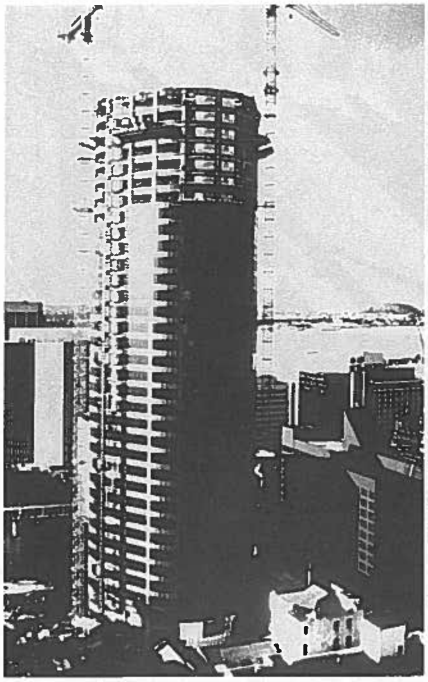
In the design of the length of the seating in the direction of the span, allowances must be made for tolerances arising from the manufacturing process, the erection method, and the accuracy of other construction. Also, allowances must be made for the long-term effects of volume changes due to concrete shrinkage, creep and temperatures effects, as well as for the effects of earthquakes.

Some concern has been expressed in New Zealand that there were cases in construction where the support provided for precast concrete floors was inadequate. The New Zealand standards for design and construction in the 1980s[1,8] had no specific requirements for the support of precast concrete floors.

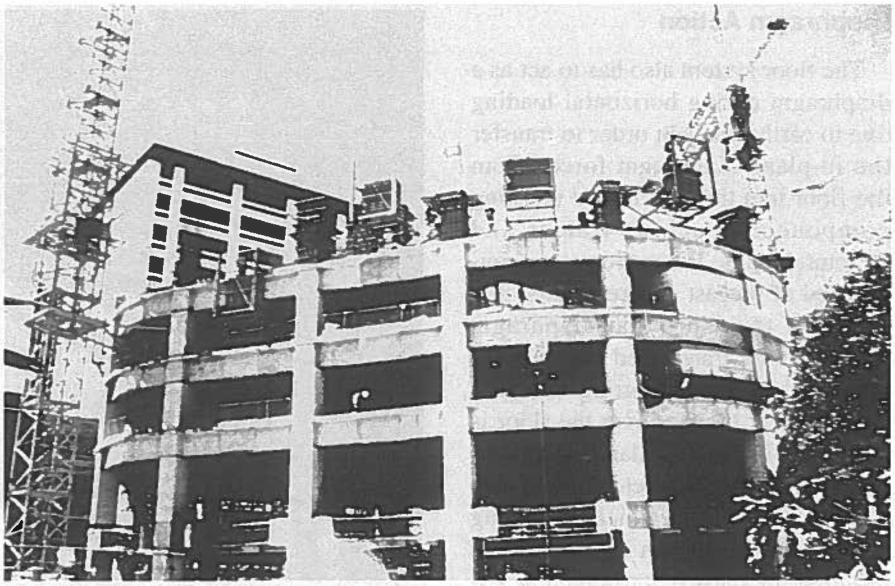
As a result, the revised New Zealand concrete design standard NZS 3101:1995[4] recommends that for precast concrete floor or roof members, with or without the presence of a cast-in-place concrete topping slab and/or continuity reinforcement, *unless shown by analysis or test that the performance of alternative details at the supports will be acceptable,* each member and its supporting system shall have design dimensions selected so that, under a reasonable combination of unfavorable construction tolerances, the distance from the edge of the support to the end of the precast member in the direction of its span is at least $^{1}/_{180}$ of the clear span but not less than: 50 mm (2 in.) for solid or hollow-core slabs or 75 mm (3 in.) for beams or ribbed members.

The above recommendation requiring proven alternative support details, unless the specified end distances are provided, is similar to that being considered by ACI Committee 318 for the revision of the current ACI Building Code.[9] The above end distances are similar to those recommended by ACI-ASCE Committee 550.[10]

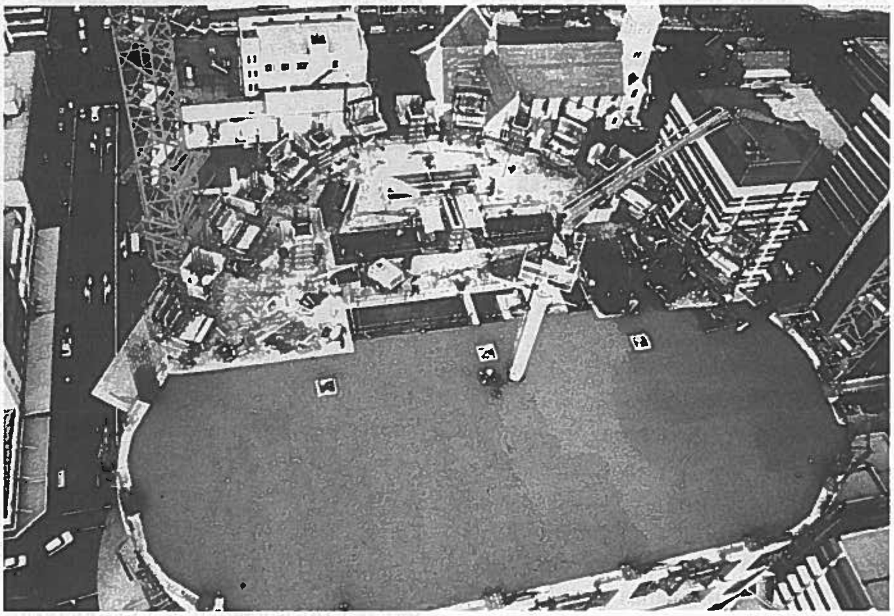
One method of providing the alternative details that permit smaller seating lengths is to use special reinforcement between the ends of the precast



(a) Construction overview of building



(b) Construction of the perimeter frame (Ref. 21)



(c) Construction of a floor

Fig. 8. Construction of 152 m (499 ft) Coopers and Lybrand Tower using System 2 in New Zealand.

concrete floor units and the supporting beam that can carry vertical load in the event of the precast concrete floor units losing their seating. The special reinforcement should be able to transfer the end reactions by shear friction across the vertical cracks at the ends of the units if the crack widths are relatively narrow or by kinking of the reinforcement crossing the cracks if the crack widths are large.

This reinforcement can be in the form of hanger or saddle bars, or horizontal or draped reinforcement, as recommended by the New Zealand Guidelines,[3] the Precast/Prestressed Concrete Institute,[11] and the Fédération Internationale de la Précontrainte.[12] This special reinforcement passes over or is anchored to the supporting beam. For example, for precast concrete hollow-core units, the reinforcement may be either placed in some of the cores that have been broken out at the top and filled with cast-in-place concrete or grouted into the gaps between the units. Note that reinforcement in a cast-in-place topping slab alone cannot be expected to provide an adequate load path to support the units, should the seating be lost, because the topping slab may split away from the precast concrete units.

Recent tests conducted at the University of Canterbury[13] on special reinforcement, placed in filled cores at the ends of hollow-core units and passing over precast supporting beams, have investigated the three types of special support reinforcement shown in Fig. 3. All three types were able to support at least the service gravity loads of the floor, in the event of loss of end seating, when no significant horizontal displacement of the floor occurred.

However, the plain round straight or draped reinforcement with hooked ends shown as tie connection Types 1 and 3 in Fig. 3 are favored, because it was found that they could undergo substantial plastic elongation when the precast concrete units were pulled horizontally off their 50 mm (2 in.) wide seating and subjected to significant vertical displacement. Plain round end hooked reinforcement was found to perform better than deformed reinforcement because bond failure propagating along the plain round bars allowed extensive yielding along the bar, thus allowing substantial plastic elongation before fracture.[13]

Diaphragm Action

The floor system also has to act as a diaphragm during horizontal loading due to earthquakes in order to transfer the in-plane diaphragm forces from the floor into the lateral load resisting components, such as frames and structural walls. When floors are constructed of precast concrete units, it is essential to ensure that diaphragm forces can be transferred between the units and to the supporting structure so that shear transfer over the floor is achieved. In New Zealand, a cast-in-place reinforced concrete topping slab at least 50 mm (2 in.) thick containing at least the minimum reinforcement required for slabs is considered an excellent means for transferring the in-plane diaphragm forces. Some limited use of precast concrete elements in floors without a cast-in-place topping slab has also been made, but with adequate shear connection between the elements.

(a) Double cruciform-shaped precast concrete frame unit being lifted (Ref. 16)

(b) Construction of the reinforced concrete perimeter frame

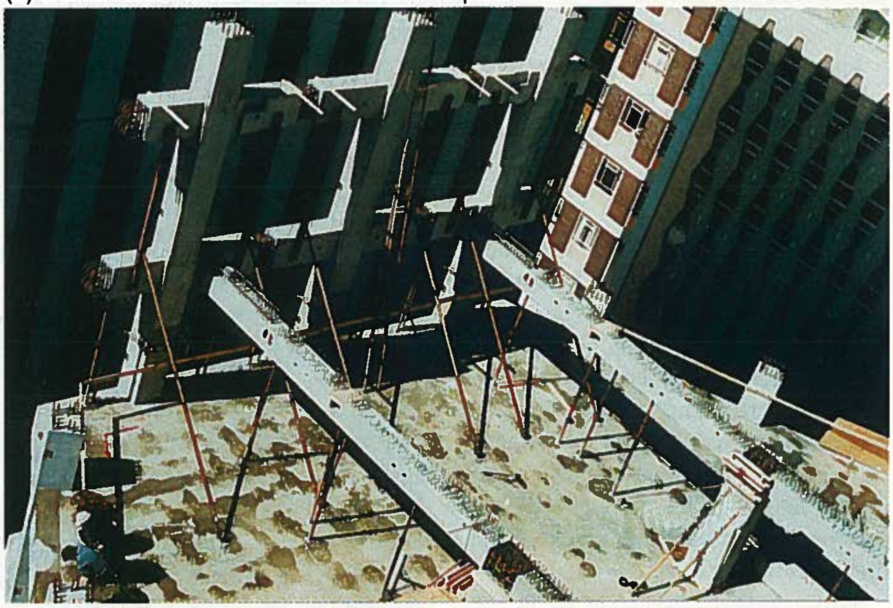


Fig. 9. Construction of a 13-story building using System 3 in New Zealand.

MOMENT RESISTING FRAMES WITH PRECAST REINFORCED ELEMENTS

Moment resisting frames incorporating precast reinforced concrete elements are widely used in New Zealand. The main challenge in the design of such structures is finding an economical and practical method for connecting the precast concrete elements together. In New Zealand, if the connections between the precast concrete elements in frames are placed in critical regions, such as potential plastic hinge regions, the approach is to design and construct connections that possess stiffness, strength, and ductility similar to that of cast-in-place concrete monolithic construction.[14,15] In other words, monolithic construction is emulated.

The general trend in New Zealand for multistory buildings with moment resisting frames is to design the perimeter frames with sufficient stiffness and strength to resist most of the horizontal seismic loading. The more flexible interior frames will be called on to resist only a small proportion of the horizontal forces, the exact amount depending on the relative stiffnesses of the perimeter and interior frames.

If the perimeter frames are relatively stiff, the columns of the interior frames will carry mainly gravity loading. Also, the interior columns can be placed with greater spacing between columns. For the perimeter frames, the depth of the beams may be large without affecting the clear height between floors inside the building and the columns can be at relatively close centers. The use of one-way perimeter frames avoids the complexity of the design of beam-to-column joints of two-way moment resisting frames.

Note that if the perimeter frame beams are fairly deep, and the columns are close and small, it may be difficult to ensure strong column-weak beam behavior. Hence, the relative dimensions of the beams and columns in tall ductile frames should be such that strong column-weak beam behavior can be achieved. Details of several buildings in New Zealand constructed in the late 1980s and early 1990s that incorporate significant quantities of precast concrete in their frames and floors are described elsewhere.[16-21]

Several possible arrangements of

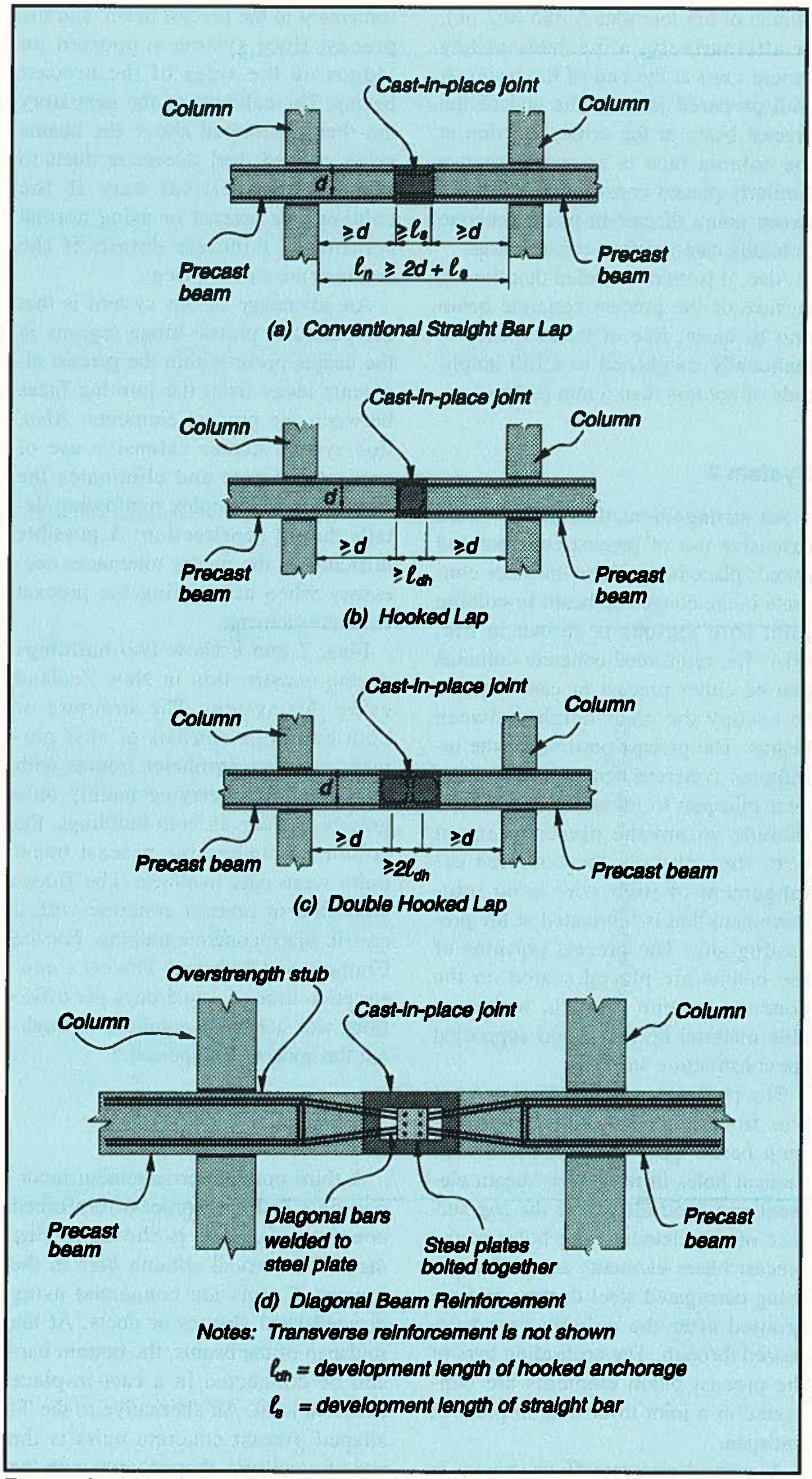


Fig. 10. Some details for midspan connections between precast reinforced concrete beam elements that have been used in New Zealand (Ref. 3).

precast reinforced concrete members and cast-in-place concrete forming ductile moment resisting multistory reinforced concrete frames have been identified.[14,15] Arrangements commonly used in New Zealand for strong column-weak beam designs are shown in Fig. 4. These three arrangements can also be used in a modified form when one- or two-story frames with strong beam-weak column design are permitted. The objective in design of the systems is to achieve behavior emulating a monolithic structure. The three arrangements are described below.

System 1

An arrangement involving the use of precast reinforced concrete elements to form the lower part of the beams is shown in Fig. 4(a). The precast beam elements are placed between columns, seated on the cover concrete of the previously cast-in-place reinforced concrete column below, and supported under the precast elements (see Fig. 5). In some cases, there may be two precast beam elements per span with a cast-in-place joint at midspan where the longitudinal beam bars are spliced. A precast concrete floor system is placed seated on the top of the precast beam elements and spanning between them. The reinforcement is then placed in the top of the beam, the topping slab over the floor system, the beam-to-column joint core, and the next story height of column. Lastly, the cast-in-place concrete is placed. The frame can be designed using the provisions for totally cast-in-place concrete structures.

This system leads to a large reduction in the quantity of site formwork necessary. A difficulty with the connection detail is that the bottom longitudinal bars of the beams, protruding from the precast beam elements, need to be anchored in the joint cores (see Fig. 6). Hence, the column dimensions need to be reasonably large to accommodate the required development length and to reduce the congestion of the hooked reinforcement.

Another possible problem is that the critical section of the potential plastic hinge region in the beam occurs at the column face where there is a vertical joint between the cast-in-place concrete of the joint core and the end of the precast beam. This is permitted by the New Zealand standard, NZS 3101:1995.[4]

To make the transfer of vertical shear possible, it is recommended that either the end of the precast beam should be clean, free of laitance and intentionally roughened to a full amplitude of not less than 5 mm (0.2 in.), or alternatively, a mechanical key should exist at the end of the beam. A well prepared joint at the end of the precast beam at the critical section at the column face is no worse than a similarly placed construction joint between pours of cast-in-place concrete in totally cast-in-place construction.

Also, it is recommended that the top surface of the precast concrete beam unit be clean, free of laitance and intentionally roughened to a full amplitude of not less than 5 mm (0.2 in.).

System 2

An arrangement that makes more extensive use of precast concrete and avoids placement of cast-in-place concrete in the congested beam-to-column joint core regions is shown in Fig. 4(b). The reinforced concrete columns can be either precast or cast-in-place to occupy the clear height between beams. The precast portions of the reinforced concrete 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 that is fabricated at the precasting site. The precast portions of the beams are placed seated on the concrete column beneath, with suitable material between, and supported for construction stability.

The protruding longitudinal column bars from the reinforced concrete column below pass through preformed vertical holes in the precast beam element and protrude above the top surface of the element. The holes in the precast beam elements are preformed using corrugated steel ducting and are grouted after the column bars have passed through. The protruding bars of the precast beam elements are connected in a joint to be cast in place at midspan.

A precast concrete floor system is placed seated on the precast beam elements and spanning between them. The reinforcement is then placed in the top of the beam and the topping slab, and the cast-in-place concrete is placed. Alternatively, the total depth of the beam can be precast, including the top and bottom longitudinal reinforcement in the precast beam, and the precast floor system supported on ledges on the sides of the precast beams. The columns of the next story are then positioned above the beams using grouted steel sleeves or ducts to connect the vertical bars if the columns are precast or using normal reinforced concrete details if the columns are cast in place.

An advantage of this system is that the potential plastic hinge regions in the beams occur within the precast elements away from the jointing faces between the precast elements. Also, this system makes extensive use of precast concrete and eliminates the fabrication of complex reinforcing details during construction. A possible difficulty is the tighter tolerances necessary when assembling the precast concrete elements.

Figs. 7 and 8 show two buildings during construction in New Zealand using this system. The structure of both buildings consists of stiff moment resisting perimeter frames with interior frames carrying mainly only gravity loading. In both buildings, the columns between the precast beam units were cast in place. The floors consisted of precast concrete with a cast-in-place concrete topping. For the Coopers and Lybrand Tower, a construction time of 4 to 5 days per office floor was achieved regularly throughout the construction period.[21]

System 3

A third possible arrangement incorporating T-shaped precast reinforced concrete elements is shown in Fig. 4(c). The vertical column bars in the precast T units are connected using grouted steel sleeves or ducts. At the midspan of the beams, the bottom bars can be connected in a cast-in-place concrete joint. An alternative to the T-shaped precast concrete units is the use of cruciform-shaped units with the joints between columns occurring at the midheight of the stories. Precast concrete floor systems can be used as with the other systems.

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

Fig. 9 shows a perimeter frame of a 13-story office building constructed using precast concrete cruciform-shaped units with columns two stories in height and two levels of beam stubs. Reinforcement projects from the beam stubs to be incorporated in cast-in-place hooked splices at the midspan of each beam. The column joint between the precast units consists of an epoxy grouted bedded joint and grouted steel sleeves. Note the long spans of the beams of the more flexible interior frames.

Midspan Connections Between Precast Concrete Beam Elements

Some details for cast-in-place midspan connections in beams that have been used are illustrated in Fig. 10. The New Zealand concrete design standard, NZS 3101:1995,[4] requires that no portion of any lap splice of the longitudinal reinforcement in the beam be located within a length of one effective depth of beam from the critical section of the potential plastic hinge region. This normally means that lap splices in beams cannot commence closer than one effective beam depth from the column face.

For short span beams of perimeter frames, a straight lap splice in the midspan region may be too long to meet this requirement. In this case, the conventional straight bar lap of Fig. 10(a) can be shortened using hooked laps, as shown in Figs. 10(b) and (c).

The double hooked lap of Fig. 10(c) is the most convenient hooked lap to construct because the protruding ends of the reinforcement from the precast concrete beam elements do not overlap, and therefore, the beam elements can be positioned during construction without difficulty. The lap is made using "drop in" bars, which consist of short lengths of bar with a hook at each end.

Diagonal reinforcement has been used where the shear forces in the beams are large [see Fig. 10(d)]. The design and detailing of this connection detail require particular care. The ends of the diagonal bars are welded to steel plates that are bolted together at midspan during construction to make the connection. Significant vertical ties are required between the bends in the diagonal reinforcement to resist the vertical component of the force in the diagonal bars. Also, bearing failure of the concrete should not occur under the bends of the diagonal reinforcement.

Grouting of a Beam-to-Column Joint

Fig. 11 illustrates a typical arrangement of a precast concrete beam element placed on a cast-in-place or precast concrete column, as used in System 2 of Fig. 4(b). The beam elements can be seated on leveling shims. The column bars pass through corrugated metal ducts, cast in the precast concrete beam. The diameter of the duct should accommodate the tolerances, plus a recommended additional 10 mm (0.39 in.) clearance between the duct wall and bar surface to allow grout to flow between the duct wall and bar. Typically, the duct diameters range from two to three times the nominal diameter of the bar. The two principal methods of grouting a precast concrete beam-to-column joint are as follows:[3]

Method 1 — The horizontal joint at the beam-to-column interface is first sealed around the outside and then grout (typically non-shrink cement-based) is pumped in at an inlet port (or tube) at one corner of the horizontal joint to displace air progressively across the interface (see Fig. 11). If the grout has a high viscosity, it may start to flow up the open ducts, starting at the duct closest to the grout inlet. It is recommended that outlet ports be provided at the other three corners of the interface. These are progressively plugged once grout without air bubbles flows out.

When all the outlets are plugged, further pumping of grout will result in the ducts being filled upwards from the bottom. The duct nearest the inlet should fill first while the one furthest away (opposite corner) may require topping up by use of a tremie tube, or by pouring-in from a dispenser in such a way that grout runs down in contact with the reinforcing bar to avoid airlocks. The inlet tube or port is plugged once injection is completed.

Method 2 — The horizontal joint at the beam-to-column interface is first sealed around the outside and then grout is poured in from a dispenser down one corner duct. Progressively, the grout will flow across the interface and up the remaining ducts. It is recommended that, as with Method 1, outlet ports be used to confirm the progress of the grouting of the interface. Topping off of ducts remote from the filling position may be neces-

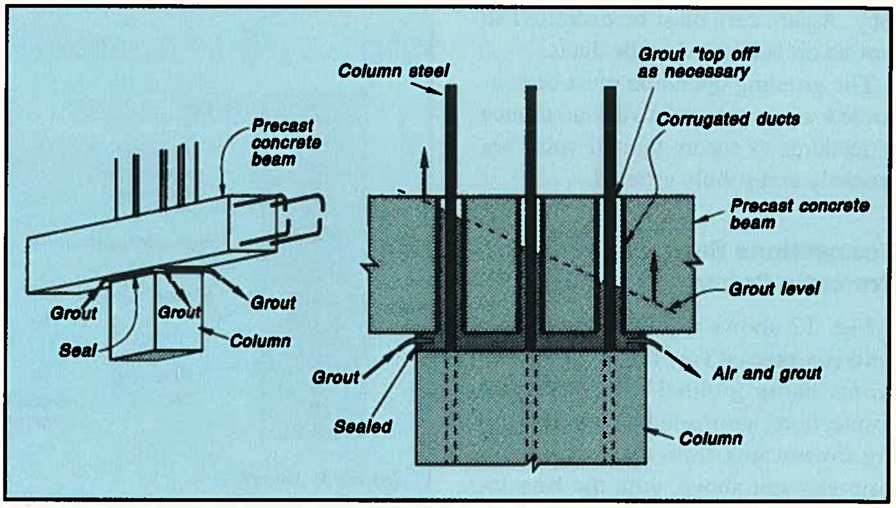


Fig. 11. Grouting of a beam-to-column joint (Ref. 3).

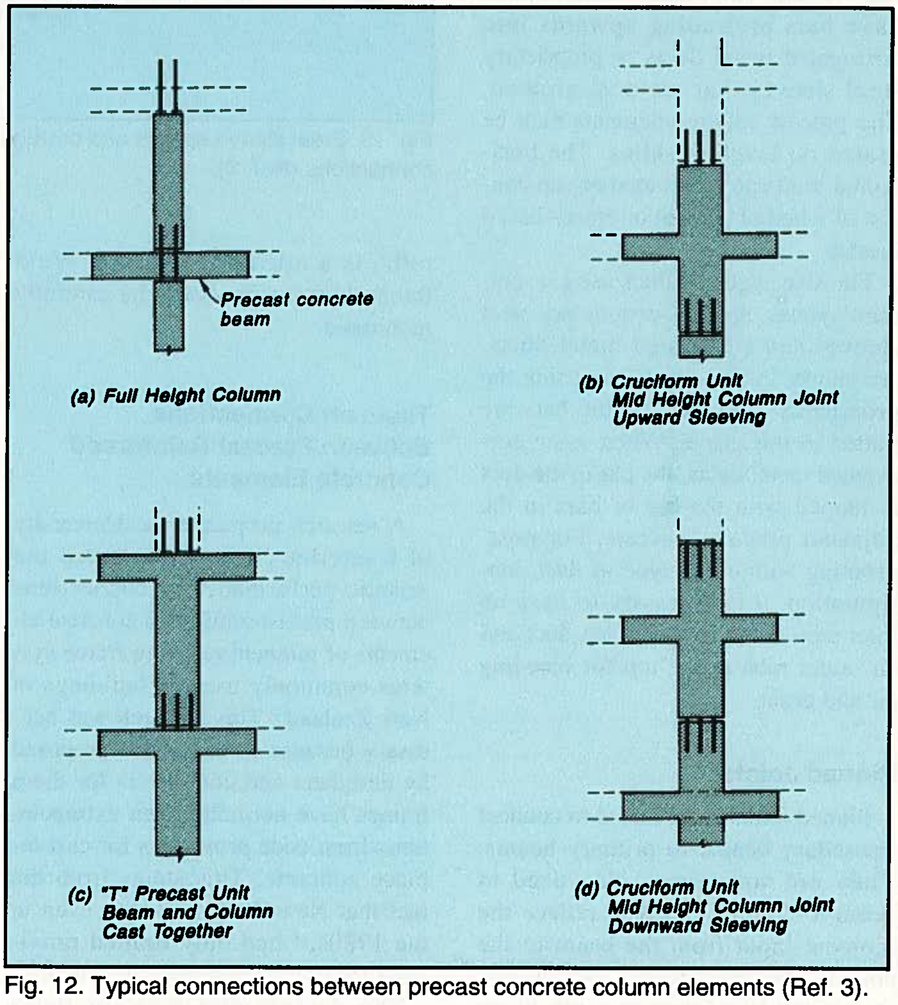


Fig. 12. Typical connections between precast concrete column elements (Ref. 3).

sary. Again, care must be exercised so that no air is trapped in the ducts.

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

Connections Between Precast Concrete Column Elements

Fig. 12 shows typical connections between precast concrete column elements using grouted splices.[3] Some connections are made by bars protruding downwards from the precast column element above, with the bars located in pregrouted corrugated metal ducts or proprietary steel sleeves [see Fig. 12(d)]. The other configurations have bars protruding upwards into corrugated metal ducts or proprietary steel sleeves that are post-grouted. The precast column elements may be seated on leveling shims. The horizontal joint can be grouted or can consist of a bed of cement or epoxy-based mortar.

The two types of duct used in column splices, namely, proprietary steel sleeves and corrugated metal ducts, are shown in Fig. 13. When using the proprietary steel sleeves, the bars are butted in the sleeve. When using corrugated metal ducts, the bar in the duct is lapped with the bar or bars in the adjacent precast concrete. For post-grouting with either type of duct configuration, it is necessary to have an inlet tube at the base of each duct and an outlet tube at the top for bleeding air and grout.

Pinned Joints

Pinned joints can be used to connect secondary beams to primary beams. They are sometimes also used at beam-to-column joints to reduce the moment input from the beam to the column. An example of a pinned joint at a secondary beam to main beam connection is shown in Fig. 14. The typical 20 mm (0.79 in.) tolerance gap, between the end of the secondary beam and the side of the main beam, implies that the precaster and contractor are required to work to very stringent tolerances. Also, the welding of reinforcing bars to rectangular hollow steel sections, or to other steel details, is a critical operation. Weld throat thicknesses should be carefully monitored.

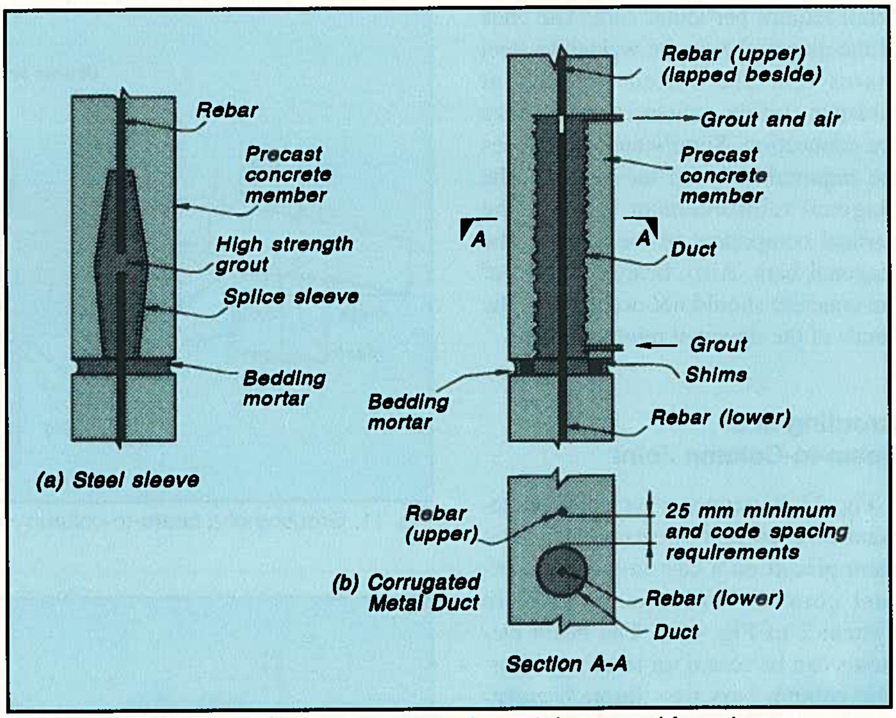


Fig. 13. Steel sleeve splices and corrugated metal ducts used for column connections (Ref. 3).

Tests on Connections Between Precast Reinforced Concrete Elements

A research project at the University of Canterbury has investigated the seismic performance of connections between precast reinforced concrete elements of moment resisting frame systems commonly used in buildings in New Zealand.[7] This research was necessary because the solutions proposed by designers and contractors for these frames have normally been extrapolations from code provisions for cast-in-place concrete. This stems from the fact that New Zealand codes, even in the 1980s,[1,8] had only limited provisions for precast concrete construction.

The design objective for these frames incorporating precast concrete elements is to achieve behavior as for totally cast-in-place concrete structures. That is, monolithic construction is emulated. The research project investigated whether the frames could achieve stiffness, strength and ductile behavior similar to completely cast-in-place concrete frames when subjected to cyclic loading in the post-elastic range, which simulated the effects of severe earthquakes.

Six full-scale subassemblies of frames were subjected to simulated seismic loading.[7] Two of the subassemblies were of cruciform shape and had precast concrete beam elements connected in two different ways at the beam-to-column joint, representing Systems 1 and 2 of Fig. 4.

At the vertical construction joints of the System 1 subassembly, which interfaced precast and cast-in-place concrete, the ends of the precast beams were clean and free of laitance, but with only a small amount of roughness, approximately 1 mm (0.04 in.) full amplitude, to represent the worst conditions normally encountered in construction practice. The precast concrete beam of the System 2 subassembly had corrugated metal ducts in the joint region through which the vertical column bars passed and were grouted. The columns of both subassemblies were cast-in-place concrete. During the cyclic horizontal loading, horizontal interstory drifts of up to at least ±3 percent occurred. (Horizontal interstory drift is horizontal displacement given as a percentage of the story height.) Both subassemblies performed with no significant difference in behavior from monolithic construction.[7]

Another four full-scale subassemblies were of H shape and were tested subjected to simulated seismic loading[7] to investigate the performance of the four details for midspan connections between precast concrete beam elements shown in Fig. 10. The details shown in Fig. 10(a), (b), and (c) were found to perform very satisfactorily in that the beams showed no significant difference in behavior from monolithic construction. The tests confirmed that the splice could begin at a distance of one effective beam depth from the critical section of the beam at the column face. This finding means that beams with relatively small span-to-depth ratios can be used, which is often a desirable feature in the configuration of moment resisting perimeter frames.

The test on the detail shown in Fig. 10(d) indicated that a problem could exist in the regions of the bends of the diagonal beam bars, if the design of that reinforcement does not consider the possible bearing stresses on the concrete at the inside of the bend. Also, the three-dimensional effects caused by the arrangement of the longitudinal reinforcement in that region requires the presence of significant tie reinforcement in the form of closed stirrups. However, it was found that the detail can be designed to perform satisfactorily.[7]

An interesting aspect of the test results from the six subassemblies[7] was the large permanent elongation of the beams that was measured after yielding of the longitudinal reinforcement occurred. This was due to residual plastic tensile strains in that steel during cyclic loading.

The elongation of the beams gradually increased during the loading cycles. At horizontal displacements corresponding to interstory drifts of about ±2 percent, the total length of the beams of the subassemblies had increased by 20 to 30 mm (0.8 to 1.2 in.) as a result of the deformations of the longitudinal bars in the two plastic hinge regions of each subassembly.

A practical consideration is that this beam growth during a severe earthquake could result in loss of support of precast concrete floor systems due to an increase in the span between supporting beams. Hence, there must be special support reinforcement at the ends of precast concrete floors, as shown in Fig. 3, unless seating lengths are adequate.

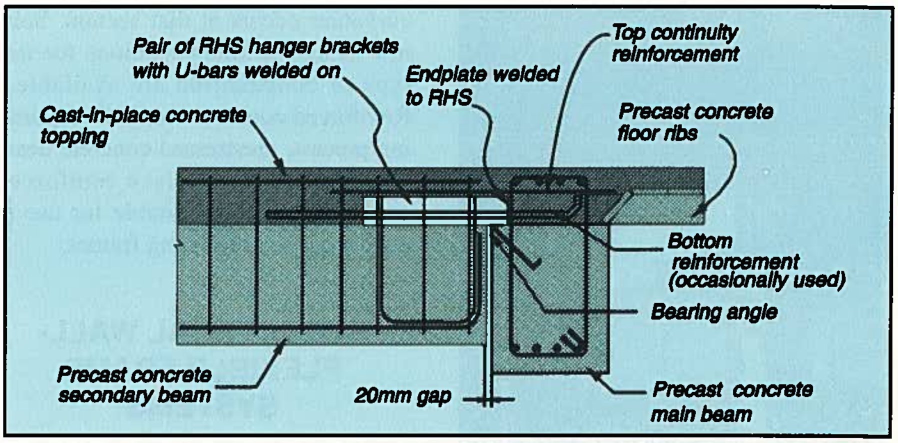


Fig. 14. Example of secondary beam to main beam connection using a rectangular hollow steel (RHS) section seated on a steel angle (Ref. 3).

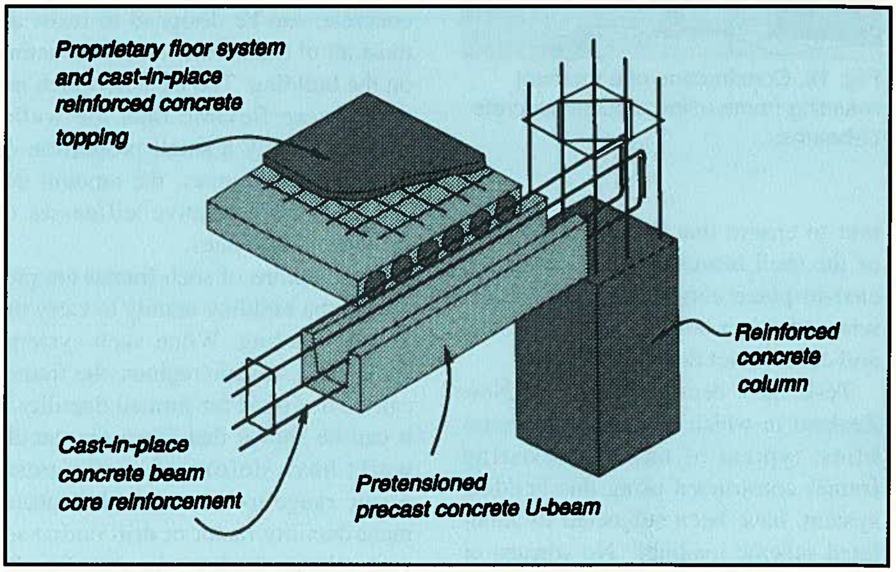


Fig. 15. Construction details of a structural system using precast, prestressed concrete U-beams and cast-in-place reinforced concrete (Ref. 22).

MOMENT RESISTING FRAMES WITH PRECAST, PRESTRESSED ELEMENTS

Another building system that has become popular in New Zealand involves the use of precast concrete beam shells as permanent formwork for beams (see Fig. 15). The precast beam shells are typically pretensioned, prestressed concrete 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. A building under construction is shown in Fig. 16.

The precast concrete 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. During construction, it is very impor-

Fig. 16. Construction of a moment resisting frame using precast concrete U-beams.

tant to ensure that the inside surfaces of the shell beams are clean when the cast-in-place concrete is cast, otherwise sufficient bond between the shell and core cannot develop.

Tests have been conducted in New Zealand in which full-scale subassemblies, typical of moment resisting frames constructed using this building system, have been subjected to simulated seismic loading.[22] No stirrups or ties protruded from the precast concrete U-beams. The resistance to seismic forces was designed to come from the cast-in-place reinforced concrete core of the U-beams. The tests were conducted because doubts had been expressed by some designers and building officials concerning the ability of this form of composite construction to perform as ductile moment resisting frames. It was felt that cracking may concentrate in the beam at the column face at the discontinuity caused by the end of the precast concrete U-beam.

However, the tests[22] demonstrated 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 some breakdown of bond. Hence, the plastic hinge rotation does not concentrate in the beam at the column face and, as a result, no undesirable concentration of curvature occurs at that section. Seismic design recommendations for this type of construction are available.[22] Reinforced concrete beams incorporating precast, prestressed concrete beam shells and cast-in-place reinforced concrete cores are suitable for use in ductile moment resisting frames.

STRUCTURAL WALL-FLEXIBLE FRAME SYSTEMS

Structures comprising both reinforced concrete structural walls and frames offer advantages. The structural walls, normally of cast-in-place concrete, can be designed to resist almost all of the horizontal forces acting on the building. The frames, which are much more flexible than the walls, will resist only a small proportion of the horizontal forces, the amount depending on the relative stiffnesses of the walls and frames.

The columns of such frames are present in the building mainly to carry the gravity loading. When such systems are used in seismic regions, the frames can be designed for limited ductility if it can be shown that when the ductile walls have deformed in the post-elastic range to the required displacement ductility factor or drift during severe seismic loading, the ductility demand on the frames is not large.

A New Zealand building employing this design is shown in Fig. 17. The central cast-in-place reinforced concrete walls, forming the service core of the building, were designed to resist the seismic loading. The perimeter frame of precast concrete beams (formed in the shape of trusses for lightness) and the columns (formed using precast concrete tubes infilled with cast-in-place concrete) were designed mainly for gravity loading.

STRUCTURAL WALLS WITH PRECAST CONCRETE ELEMENTS

Structural reinforced concrete walls in buildings have long been recognized in New Zealand as efficient structural systems for resisting horizontal forces due to earthquakes. Properly designed walls have a large inherent strength and their ample stiffness means that displacements during severe earthquakes are reduced, thus providing a high degree of protection against damage to structural and nonstructural elements.

Comprehensive design provisions exist for cast-in-place reinforced concrete structural walls.[4] In New Zealand, it is considered that well proportioned ductile cast-in-place reinforced concrete coupled walls form the best earthquake resisting structural system. The recent trend towards moment resisting frames, rather than structural walls, in New Zealand has been mainly due to the preference of architects for the more open spaces of floors when walls are not present.

Most structural walls for multistory buildings in New Zealand have been made of cast-in-place reinforced concrete, but there has been significant use of precast concrete walls for smaller buildings.

Precast reinforced concrete structural wall construction usually falls into two broad categories:[3] monolithic or jointed. In monolithic wall construction, the precast concrete elements are joined by "strong" reinforced concrete connections that possess stiffness, strength, and ductility approaching that of cast-in-place concrete monolithic construction. In jointed wall construction, the connections are "weak" relative to the adjacent wall panels and, therefore, govern the performance of the building.

Monolithic Precast Concrete Structural Wall Systems

Monolithic precast reinforced concrete structural wall systems are designed according to the code requirements of cast-in-place concrete construction.[3]

Horizontal joints between precast concrete wall panels are usually grouted connections. The vertical reinforcement is usually connected there using either grouted steel sleeve splices or a lap formed by grouting a bar extending from the end of one precast panel into a corrugated metal duct in the matching panel. Some typical details of monolithic horizontal joints are shown in Fig. 18.[3]

When corrugated metal ducts are used, the starter bars that project into the ducts are usually designed for a full lap length. In general, central starter bars are lapped with pairs of smaller bars, one on each face of the precast concrete wall section. Alternatively, all of the main flexural reinforcement is lapped on the precast concrete wall centerline and some additional basketing cover reinforcement is provided.

The horizontal joint between precast concrete panels is usually roughened to avoid a sliding shear failure.

Vertical joints between precast concrete wall panels are typically vertical strips of cast-in-place concrete. Horizontal reinforcement from the ends of the adjacent panels protrude into the joint zone and are lapped. The width of the cast-in-place concrete joint zone is determined by the code requirements for lap lengths of horizontal reinforcement. Typical details of monolithic vertical joints are shown in Fig. 19.

Vertical joints shown as Types D and E need to be detailed with extreme care. During construction, once the lapping bars have been overlapped, the ability for lowering the wall panels over the starter bars is very restricted. These details will typically work only when grouted steel splice sleeves are used to splice the vertical flexural reinforcement and when the laps of the vertical bars in the joints are made near floor level.

As an example, Fig. 20 shows part of the construction of a two-story building that uses full height precast concrete structural wall panels to provide support for the precast concrete floor system and the roof. The precast concrete walls, which provide the lateral load resistance of the building, were designed as cantilevered structural walls of limited ductility to the seismic requirements of what was the current New Zealand concrete design code.[1]

The connection detail between the walls and foundations was designed to withstand the larger seismic forces corresponding to the elastic response of the structure to a severe earthquake. The vertical joints between panels,

(a) Perimeter structure

(b) Typical floor plan

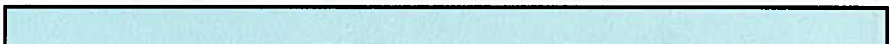


Fig. 17. Building with a precast reinforced concrete perimeter frame with seismic forces resisted mainly by cast-in-place reinforced concrete interior structural walls.

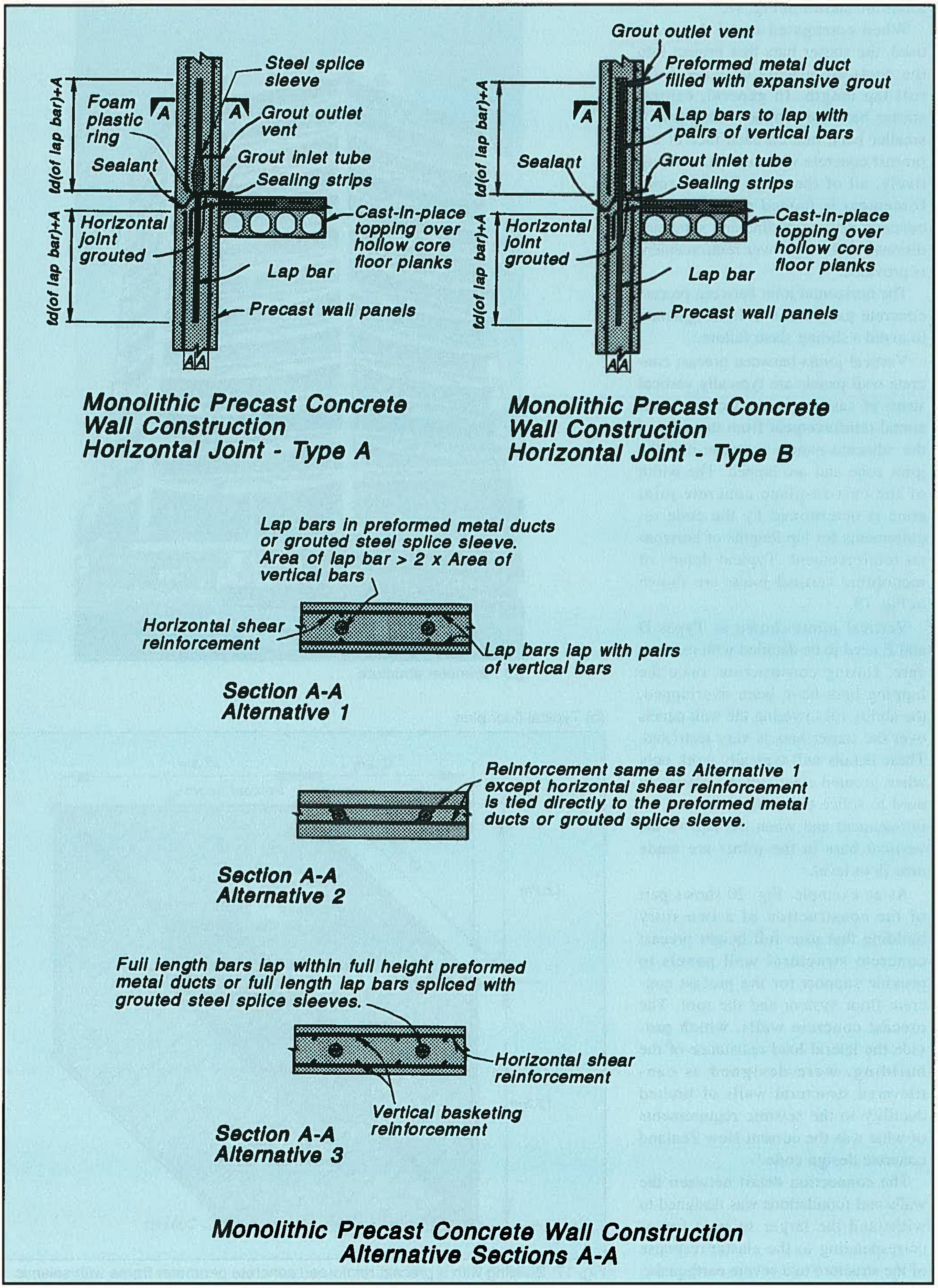


Fig. 18. Details of horizontal joints in monolithic precast reinforced concrete wall construction (Ref. 3).

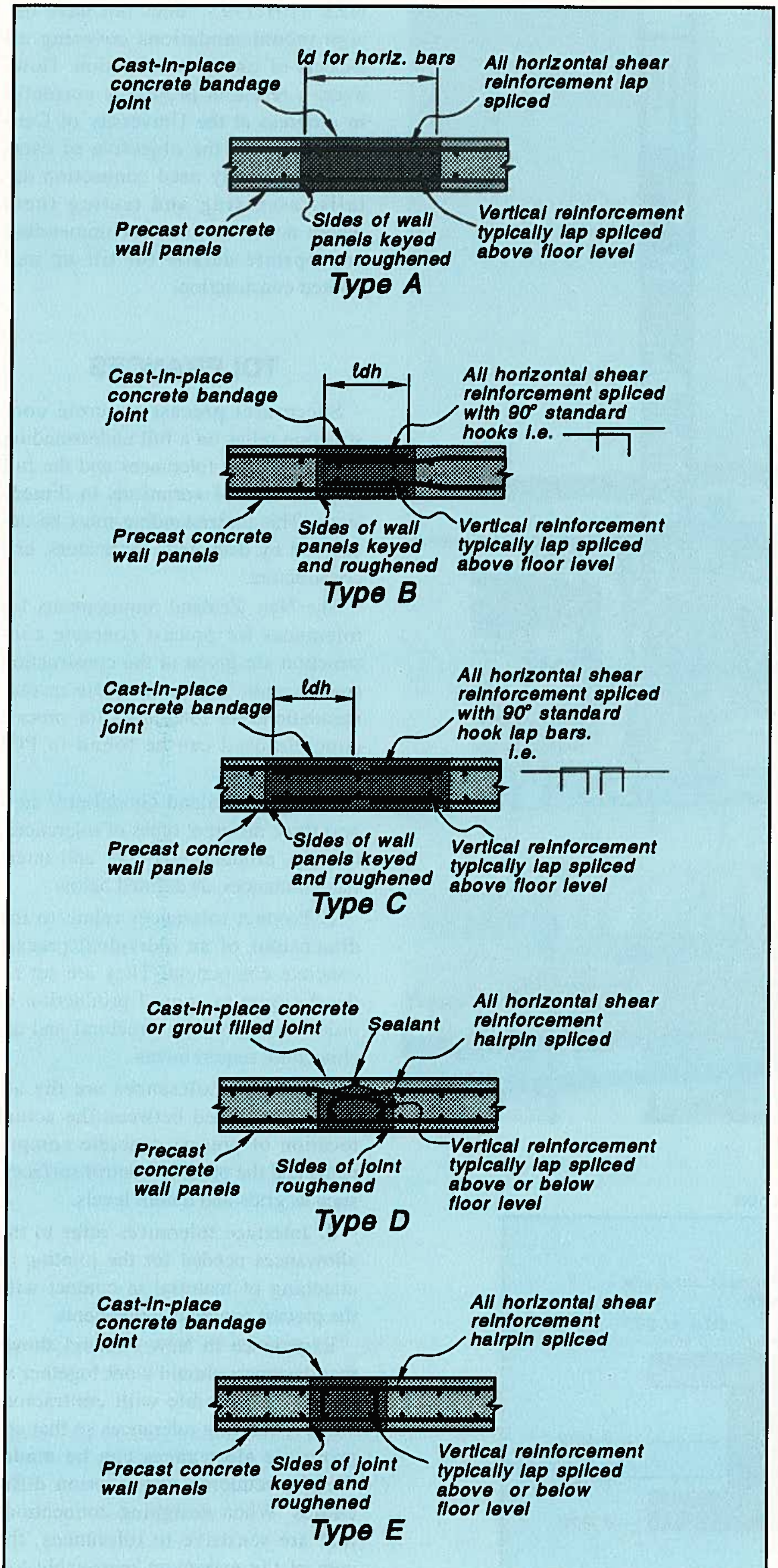


Fig. 19. Some details of vertical joints in monolithic precast reinforced concrete wall construction (Ref. 3).

shown during construction in Fig. 20(a), consist of horizontal overlapping hairpin-shaped reinforcement that projects from each of the wall panels and cast-in-place concrete creating a monolithic joint of Type E, as shown in Fig. 19.

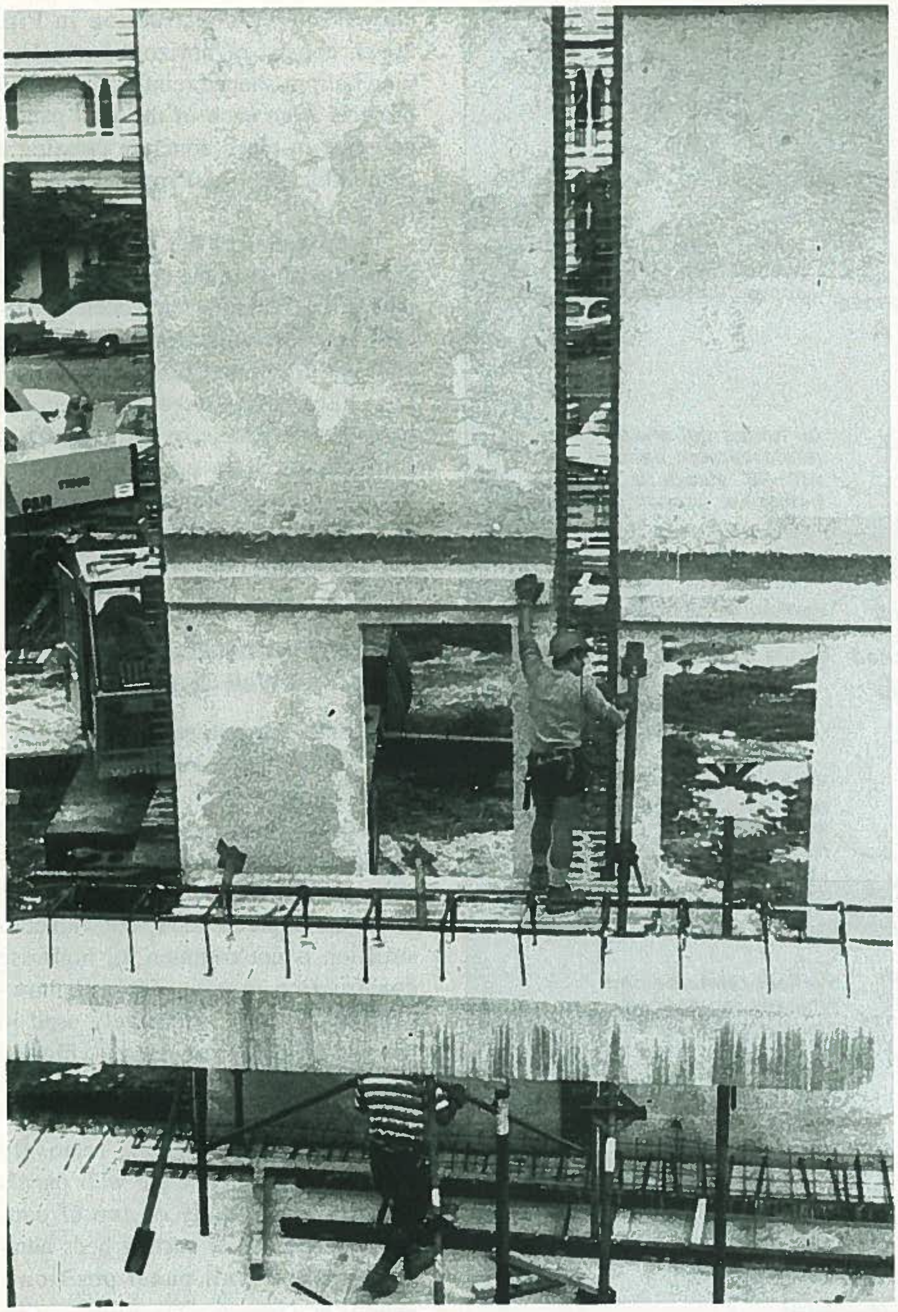
A vertical steel bar was placed in the space between the ends of the hairpins prior to casting the concrete. Details of the connection between the wall and the foundation are shown in Fig. 20(b). Holes were formed in the bases of the panels so that horizontal reinforcing bars could be placed through to resist the design horizontal shear forces and tension forces resulting from overturning moments.

Jointed Precast Concrete Structural Wall Systems

In jointed construction, the connection of precast reinforced concrete components is such that sections of significantly reduced stiffness and strength exist at the interface between adjacent precast concrete wall panels. This type of precast concrete wall construction is not common for high rise construction in New Zealand; however, it has been extensively used in the tilt-up construction of typically one- to three-story apartment, office and industrial buildings.[3]

For tilt-up construction, relatively large reinforced concrete wall panels are cast horizontally on top of concrete floor slabs or casting beds adjacent to final wall panel positions. When the concrete has gained sufficient strength for the wall panels to remain uncracked during lifting operations, the walls are tilted up and lifted into their permanent positions. Generally, tilt-up walls are secured to the adjacent structural elements with jointed connections consisting of various combinations of concrete inserts, bolted or welded steel plates or angle brackets, and lapped reinforcement splices within cast-in-place joining strips. Such walls are designed as structural walls of limited ductility. That is, the design seismic forces are on the order of twice those used in the design of ductile walls.

Unfortunately, the revised New Zealand concrete design standard,



(a) Precast concrete panel during erection

(b) Detail of wall/foundation junction

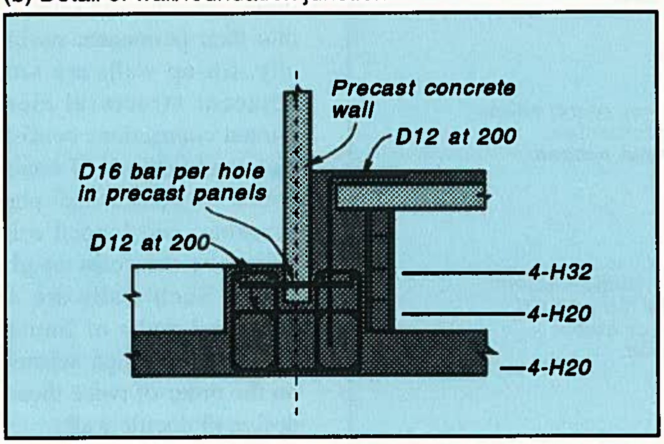


Fig. 20. Construction of a two-story building incorporating full height precast concrete wall panels (Ref. 3).

NZS 3101:1995,[4] does not have design recommendations covering all aspects of tilt-up construction. However, a research project is currently in progress at the University of Canterbury[23] with the objective of cataloging currently used connection details, assessing and testing them where necessary, and recommending appropriate details for tilt-up and jointed construction.

TOLERANCES

Successful precast concrete construction relies on a full understanding of the need for tolerances and the full implications of variations in dimensions. This understanding must be developed by designers, fabricators, and constructors.

The New Zealand requirements for tolerances for precast concrete construction are given in the construction specification.[8] More complete recommendations for tolerances for precast concrete used can be found in PCI publications.[24,25]

The New Zealand Guidelines[3] suggest three different types of tolerances, namely, product, erection, and interface tolerances, as defined below:

1. Product tolerances relate to the dimensions of an individual precast concrete component. They are set by the designer to control production in order to achieve the structural and architectural requirements.

2. Erection tolerances are the allowances needed between the actual location of precast concrete components and the primary control surfaces, such as grids and datum levels.

3. Interface tolerances refer to the allowances needed for the jointing or attaching of material in contact with the precast concrete components.

Experience in New Zealand shows that designers should work together as closely as possible with contractors when specifying tolerances so that appropriate allowances can be made, thereby reducing construction difficulties. When designing connections that are sensitive to tolerances, the sum of the maximum reasonable tolerances can be used to define the worst design case.

CONCLUSIONS

Based on the preceding discussion, the following conclusions can be drawn:

1. Precast concrete floor systems, normally spanning one-way, have been commonplace in New Zealand since the 1960s. Also, non-structural precast concrete has been widely used for the cladding of buildings.

2. The building boom in New Zealand in the mid-1980s produced a significant increase in the use of structural precast concrete, particularly for moment resisting frames, because of the advantages of high quality factory made units, speed of construction, and the reduction of site formwork and labor. This required developments in all aspects of the use of structural precast concrete as designers and contractors sought increasingly effective methods of design and construction.

3. The design approach for moment resisting frames incorporating precast reinforced concrete elements in New Zealand is similar to that for totally cast-in-place reinforced concrete structures. That is, the methods used for connecting the precast concrete elements together are generally aimed at achieving behavior equivalent to that of monolithic construction.

4. For moment resisting frames, the structural arrangements include precast reinforced concrete beam elements spanning between columns, precast reinforced concrete beam elements passing through columns, and T-shaped and cruciform-shaped precast reinforced concrete elements. Structural continuity between precast concrete elements is generally achieved using cast-in-place reinforced concrete. A structural system utilizing precast, prestressed concrete U-beams with a cast-in-place reinforced concrete core is also in use.

5. Significant use of precast reinforced concrete wall elements is also being made for low rise structural wall construction, particularly for tilt-up walls.

6. The successful use of precast concrete requires close cooperation between designers, precasters and contractors.

7. New Zealand design codes have traditionally contained extensive provisions for cast-in-place reinforced concrete but are now being extended to include design provisions for precast concrete.

8. Laboratory tests, involving the simulated seismic loading of full-scale subassemblies incorporating precast concrete elements, have been conducted in New Zealand.[7] These tests give confidence in the design and construction of a range of connections between precast concrete members that, when first developed, went beyond the codes of the time.

ACKNOWLEDGMENTS

The author gratefully acknowledges the helpful discussions of many members of the design and construction profession in New Zealand. Particularly appreciated are the members of the Study Group of the New Zealand Concrete Society and of the New Zealand National Society for Earthquake Engineering, which produced the "Guidelines for the Use of Structural Precast Concrete in Buildings," published by the Centre for Advanced Engineering of the University of Canterbury in August 1991. Thanks are also due to R. G. Wilkinson of the Holmes Consulting Group, and to A. J. O'Leary of Kingston Morrison Ltd., for providing material. The efforts of graduate students J. I. Restrepo and J. C. Mejia are also gratefully acknowledged.

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