Seismic Design and Construction of Precast Concrete Buildings in New Zealand

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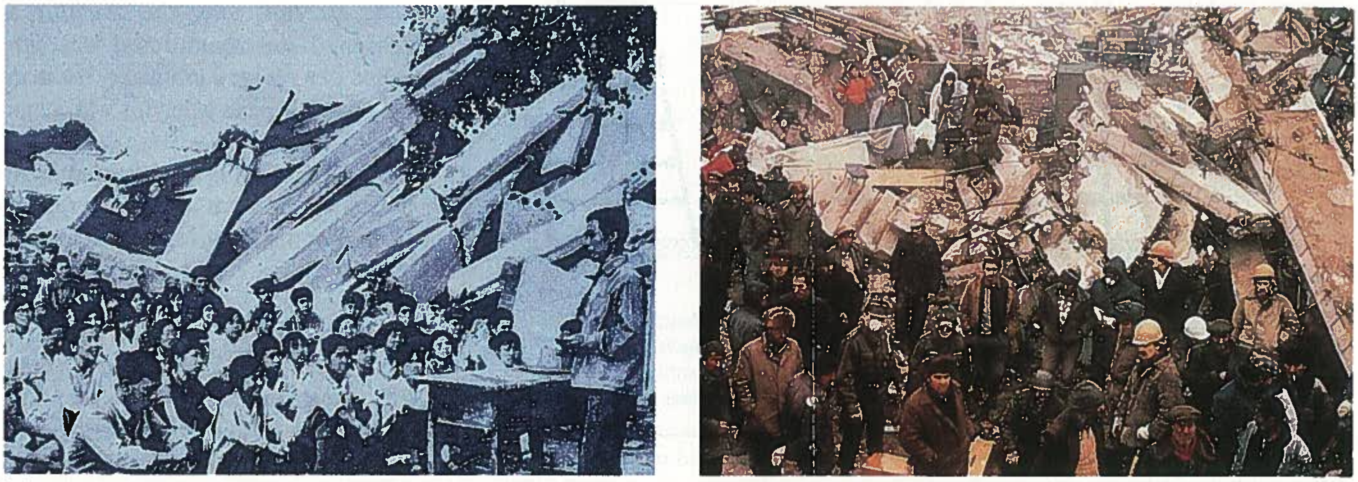
Trends and developments in the use of precast reinforced concrete in New Zealand for floors, moment resisting frames and structural walls of buildings are described. Currently, almost all floors, most moment resisting frames and many one- to three-story structural walls in buildings are constructed incorporating 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 since that is where the major difficulties exist in using precast concrete in New Zealand. Confidence in the use of precast concrete in an active seismic zone has required the use of an appropriate design philosophy and the development of satisfactory methods for connecting the precast elements together.

Since the early 1960s in New Zealand, there has been a steady increase in the use of precast concrete for structural components in buildings. The use of precast concrete in flooring systems has become commonplace since the 1960s, leaving 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. This came about because the incorporation of precast concrete elements has 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 demand for new building space in New Zealand in the mid 1980s, the advantage of speed gave precast concrete a distinct edge in cost.

Contractors readily adapted to pre-



(a) Tangshan, China, 1976 (b) Leninakan, Armenia, 1988

Fig. 1. Examples of damage to precast concrete buildings caused by major earthquakes.

cast concrete and the new construction techniques resulting from on and off-site fabrication of building components. Also, the availability of tall, high capacity cranes and other equipment have made precast erection more efficient.

New Zealand is in a zone of high to moderate earthquake activity and the use of precast concrete in seismic regions requires special provisions for design and construction. It is of interest that moment resisting frames and structural walls incorporating precast concrete elements have been observed in some countries to perform poorly in earthquakes.

The observed failures have been mainly due to brittle (non-ductile) behavior of poor connection details between the precast concrete elements, poor detailing of components and poor design concepts (see Fig. 1). As a result, the use of precast concrete in frames and walls was shunned in New Zealand for many years.

Confidence in the use of precast concrete in moment resisting frames and structural walls in the 1980s in New Zealand required the development of satisfactory methods for connecting the precast elements together. The then current New Zealand concrete design standard, NZS 3101:1982,[1] like concrete design standards 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. Hence, the increase in the use of precast concrete in the 1970s required a good deal of innovation.

The design methods introduced for the connections between precast elements of moment resisting frames generally aimed to achieve behavior as for a monolithic concrete structure (cast-in-place emulation). The design methods for structural walls aimed at behavior as for either a monolithic structure or a jointed structure with relatively weak joints between elements.

A Study Group of the New Zealand Concrete Society, the New Zealand National Society for Earthquake Engineering and the Centre for Advanced Engineering of the University of Canterbury was formed in 1988 to summarize and present data on precast concrete design and construction, to identify special concerns, to indicate recommended practices, and to recommend topics requiring further research. The outcome of the deliberations of the Study Group was the publication of a manual entitled "Guidelines for the Use of Structural Precast Concrete in Buildings," which was first printed in August 1991.

A second edition incorporating research undertaken in the first half of the 1990s was published in December 1999.[2] A revision of the New Zealand concrete design standard was published in 1995.[3] This revision contains additional provisions for the seismic design of structures containing precast concrete based on that research.

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. It emphasizes design and construction for seismic resistance, since that is where the greatest difficulties exist in the connection of precast elements.

SEISMIC DESIGN CONCEPTS FOR PRECAST CONCRETE IN BUILDINGS

General Requirements

For moment resisting frames and structural walls incorporating precast concrete elements, the challenge is to find economical and practical means of connecting the precast elements together to ensure adequate stiffness, strength, ductility and stability. The designer should consider the loadings during the various stages of construction and at the serviceability and ultimate limit states during the life of the structure.

In common with other countries, the seismic design forces recommended for structures in the current New Zealand standard for general structural design and design loadings for buildings, NZS 4203:1992,[4] 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

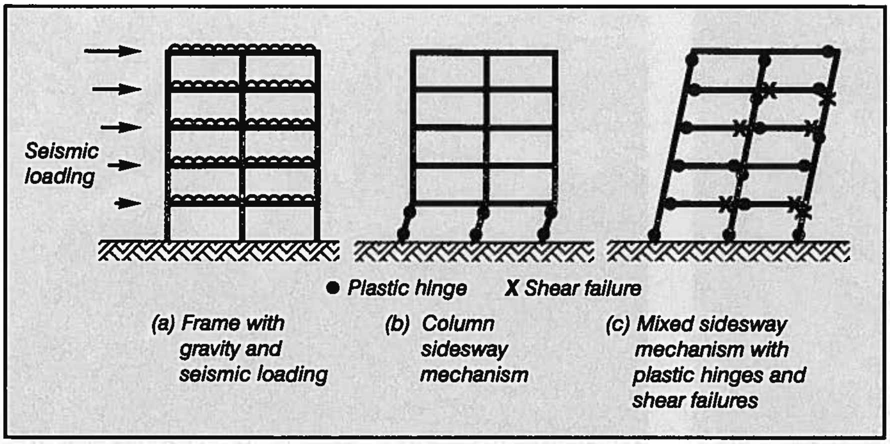


Fig. 2. Undesirable modes of behavior for tall seismically loaded moment resisting frames in the post-elastic range.

$\mu = \Delta_{max}/\Delta_y$, 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.

According to the New Zealand loadings standard,[4] structures may be designed as ductile or structures of limited ductility or elastically responding.

- For ductile structures, μ = 5 or 6 is used to determine the appropriate spectra of seismic coefficients and the design horizontal seismic forces at the ultimate limit state 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, μ = 2 or 3 is used and the design horizontal seismic force at the ultimate limit state typically varies between 0.03g and 0.39g.
- For elastically responding structures, μ = 1.25 is used.

Note that the design seismic forces for cast-in-place concrete structures and for structures incorporating precast concrete elements *of the same available ductility* recommended in the New Zealand loadings code[4] are identical.

Capacity Design

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.

As a result, when the structure as designed and constructed was subjected to a severe earthquake, the manner of post-elastic behavior 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 behavior of such structures in the post-elastic range was somewhat unpredictable.

For example, for monolithic moment resisting frames, overstrength of the beams in flexure or understrength of the columns in flexure could result in column sidesway mechanisms (see Fig. 2b) (soft story behavior). Also, the flexural overstrength of members leads to increased shear forces when plastic hinges form, which could result in shear failures (see Fig. 2c). These undesirable failure modes could cause catastrophic collapse of the frame.

In the case of monolithic cantilever structural walls, there are a range of possible undesirable modes of behavior. Overstrength of the wall in flexure could cause failure modes in either diagonal tension shear, sliding or hinge sliding (see Figs. 3b, 3c and 3d) which have limited ductility.

There is no doubt that the confidence of New Zealand designers that adequate ductility may be achieved in structures, either totally cast-in-place or incorporating precast concrete elements, has come about mainly as a result of the introduction of the *capacity design approach.* The capacity design approach is the result of research and development in New Zealand.

The method was developed by discussion groups of the New Zealand National Society for Earthquake Engineering in the 1970s and by Park and Paulay.[5] The capacity design approach was first recommended by the New Zealand loadings standard in 1976 and by the New Zealand concrete design standard in 1982.[1] The capacity design approach is described in more detail by Park,[6] Paulay and Priestley,[7] and Park, Paulay and Bull.[8]

To ensure that the most suitable mechanism of post-elastic deformation does occur in a structure during a se-

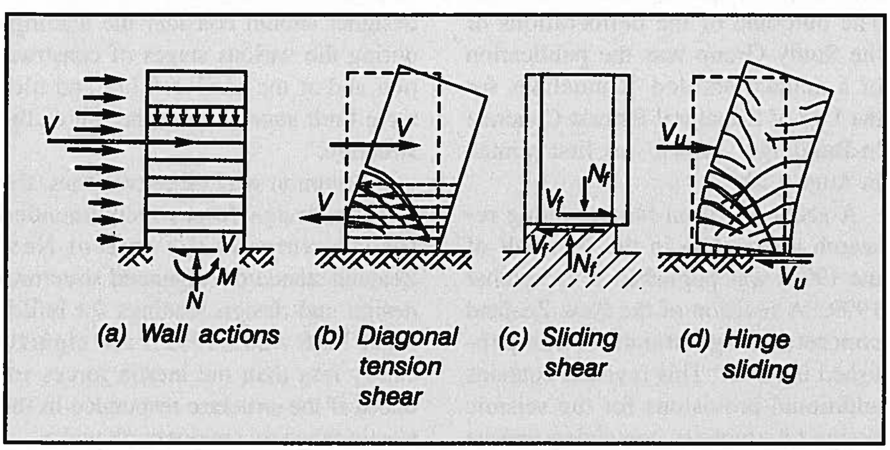


Fig. 3. Undesirable modes of behavior for seismically loaded cantilever structural walls in the post-elastic range.

vere earthquake, New Zealand design standards[3,4] require that ductile structures be the subject of *capacity design.* In the capacity design of ductile structures, the steps are:

1. First, appropriate regions of the primary lateral earthquake force resisting structural system are chosen and suitably designed and detailed for adequate *design strength* and ductility during a severe earthquake.

2. Next, all other regions of the structural system, and other possible failure modes, are then provided with sufficient *nominal strength* to ensure that the chosen means for achieving ductility can be maintained throughout the post-elastic deformations that may occur when the flexural *overstrength* develops at the plastic hinges.

The use of capacity design has given designers confidence that structures can be designed for predictable ductile behavior during major earthquakes. In particular, brittle elements can be protected. Yielding can be restricted to ductile elements as intended by the designer.

The steps of the capacity design approach according to the New Zealand concrete design standard,[3] as described above, require consideration of three levels of member strength, namely, design strength ϕS_n, nominal strength S_n and overstrength S_o as defined below.

Design strength, ϕS_n, 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. Note that 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 including the effect of confinement, and reinforcement area including any additional reinforcement placed for construction and otherwise unaccounted for in calculations.

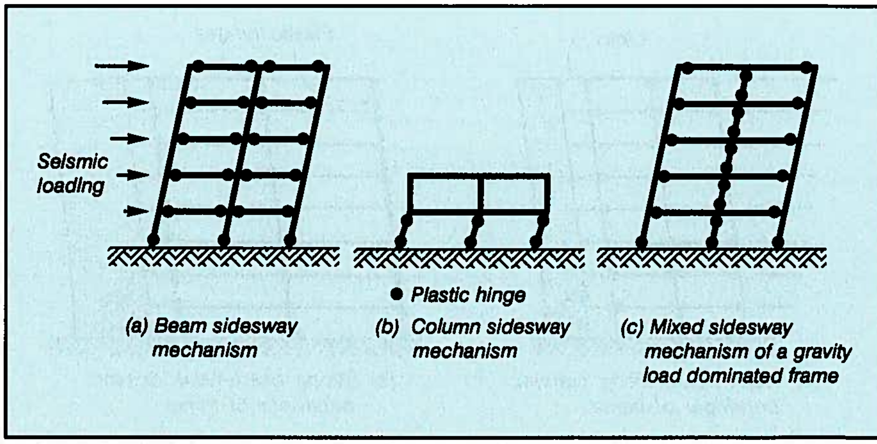


Fig. 4. Desirable mechanisms of post-elastic deformation of monolithic moment resisting frames during severe seismic loading, according to the New Zealand Standard.[3]

Recommended Mechanisms of Post-Elastic Behavior for Monolithic Moment Resisting Frames

For moment resisting frames of buildings, the best means of achieving ductile post-elastic deformations is by flexural yielding at selected plastic hinge positions, since with proper design and detailing the plastic hinges can be made adequately ductile.[5,6]

Significant post-elastic deformations due to shear or bond mechanisms are to be avoided since with cyclic loading they lead to severe degradation of strength and stiffness and to reduced energy dissipation due to pinched load-displacement hysteresis loops. Post-elastic deformations due to flexural yielding at well designed plastic hinge regions result in stable load-displacement hysteresis loops without significant degradation of strength, stiffness and energy dissipation.

The preferred mechanism for precast concrete equivalent monolithic moment resisting frames is a beam sidesway mechanism (see Fig. 4a). A beam sidesway mechanism occurs as a result of strong column-weak beam design.[5] The ductility demand at the plastic hinges in the beams and at the column bases is moderate for this mechanism and can easily be provided in design. A column sidesway mechanism is not permitted (except for the exceptions given below), since it can make very large demands on the ductility at the plastic hinges in the columns of the critical story.[5] Column

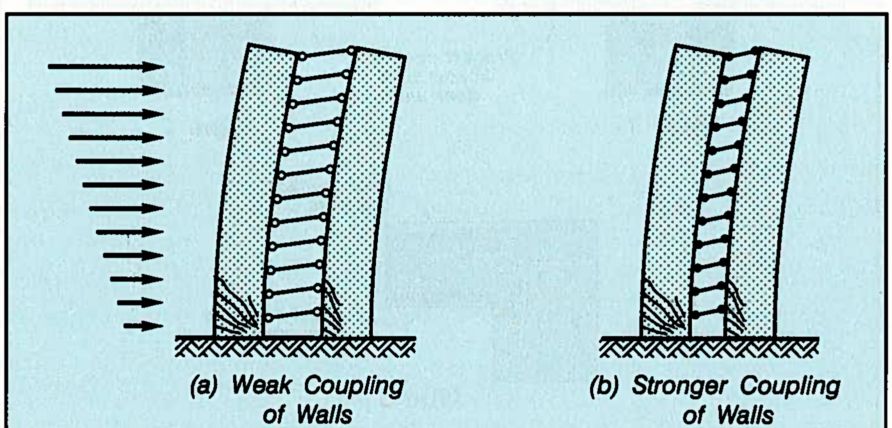


Fig. 5. Desirable mechanisms of post-elastic deformation of monolithic coupled structural walls.[8]

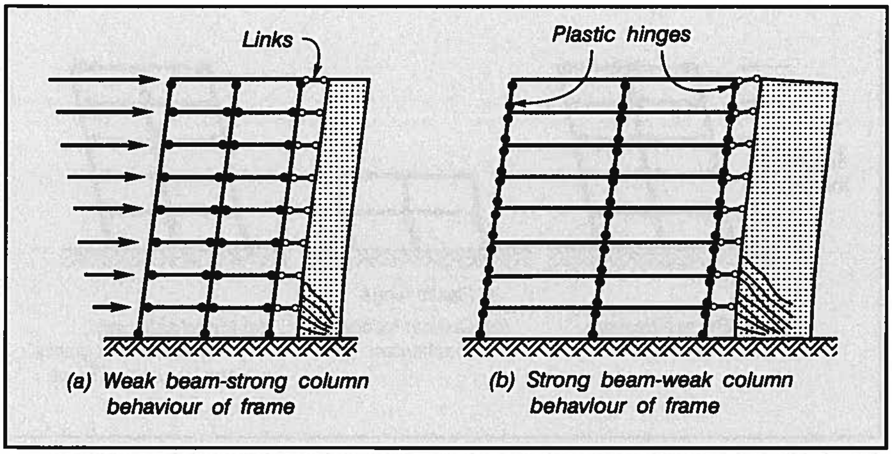


Fig. 6. Desirable mechanisms of post-elastic deformation of monolithic dual systems.[6]

sidesway mechanisms (soft stories) have often led to the collapse of buildings during earthquakes.

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 flexural overstrengths at the plastic hinges taking into account 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 yield strength used in design and additional longitudinal steel strength due to strain hardening at large ductility factors (the sum of these two factors is referred to as the steel overstrength). The flexural overstrength in New Zealand is taken as $1.25M_n$, where M_n is the nominal flexural strength.

To avoid column sidesway mechanisms, the design column bending moments need to be amplified (strong column-weak beam design) to take into account beam flexural overstrength, higher mode effects and concurrent earthquake loading.[3-8]

The New Zealand concrete design standard[3] recommends that the column bending moments at the center of beam-column joints derived by elastic structural analysis for the equivalent static design seismic forces acting in a principal direction of the frame be multiplied by an amplification factor of at least 1.9 for one-way frames or at least 2.2 for two-way frames. The amplified column bending moments so determined are to be resisted by the nominal flexural strength of the columns in uniaxial bending. These amplification factors take into account the possible flexural overstrength of the beams, the effects of higher modes of vibration of the frame and the effect of seismic loading acting along both principal axes of the frame simultaneously in the case of two-way frames.

The New Zealand concrete design standard[3] has only two exceptions to the requirement of the strong column-weak beam design approach:

1. For ductile frames of one- or two-story buildings (see Fig. 4b), or in the top story of multistory buildings, column sidesway mechanisms are permitted (that is, plastic hinges occurring simultaneously at the top and bottom of all the columns of a story). In such cases, the design seismic forces are those associated with a structure ductility factor $\mu = 6$ since the curvature ductility demand at the plastic hinges in the columns in such cases of low frames is not high as can be provided by proper detailing.

2. 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 cases, ductile frames three stories or higher may be designed to develop plastic hinges in any story simultaneously at the top and bottom ends of some columns, while plastic hinges develop in beams at or near the other columns in that story which remain in the elastic range. The columns that remain in the elastic range will prevent a soft story failure (see the mixed sidesway mechanisms in Fig. 4c). Such frames are required to be designed for the design seismic force associated with μ equal to 12 times the ratio of the total shear capacity of the columns remaining in the elastic range to the total story shear to be developed, but not more than $\mu = 6$.

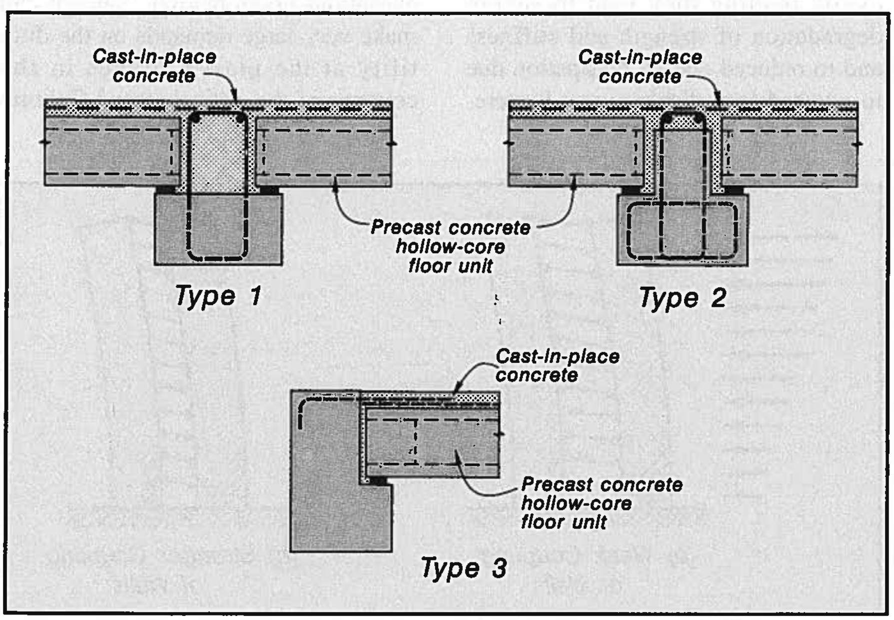


Fig. 7. Type of support of precast concrete hollow-core floor units by precast concrete beams used in New Zealand.[2]

It should be appreciated that the mechanisms of Fig. 4 are idealized in that they involve possible post-elastic behavior obtained from static "push-over" analysis with the frame subjected to code type equivalent static seismic forces. The actual dynamic situation may be different, due mainly to the effects of higher modes of vibration of the structure.

For example, the curvature ductility demand at the plastic hinges in the beams in the lower region of the frame may be greater than in the upper region. However, considerations such as those shown in Fig. 4 can be regarded as providing the designer with a reasonable feel for the situation. Nonlinear dynamic analyses indicate that mechanisms such as those shown in Fig. 4 do form.

Fig. 8. Collapse of a hollow-core floor as a result of the 1994 Northridge earthquake.

Recommended Mechanisms of Post-Elastic Behavior for Structural Walls

Ductile capacity of structural walls is required to be obtained by plastic hinge rotation as a result of flexural yielding, with a displacement (structural) ductility factor μ of 6 or less, depending on the height-to-length ratio of the wall, and in the case of coupled walls also depending on the ratio of the overturning moment resisted by the coupling walls to that resisted by the wall bases.[3]

The preferred mechanism for a monolithic cantilever structural wall involves a plastic hinge at the base. Fig. 5 shows desirable mechanisms of post-elastic deformation of monolithic structural walls during severe seismic loading with coupling beams between them.

If the coupling is weak (for example, only from floor slabs), the walls will act as individual cantilever walls connected by pin-ended links (see Fig. 5a). If the coupling beams are stiffer and have significant flexural strength, but not sufficient to cause shear failure of the walls, plastic hinging will also develop in the coupling beams (see Fig. 5b).

Jointed precast concrete wall construction (with relatively weak joints between the precast elements) could develop other mechanisms of post-elastic deformation and need to be de-

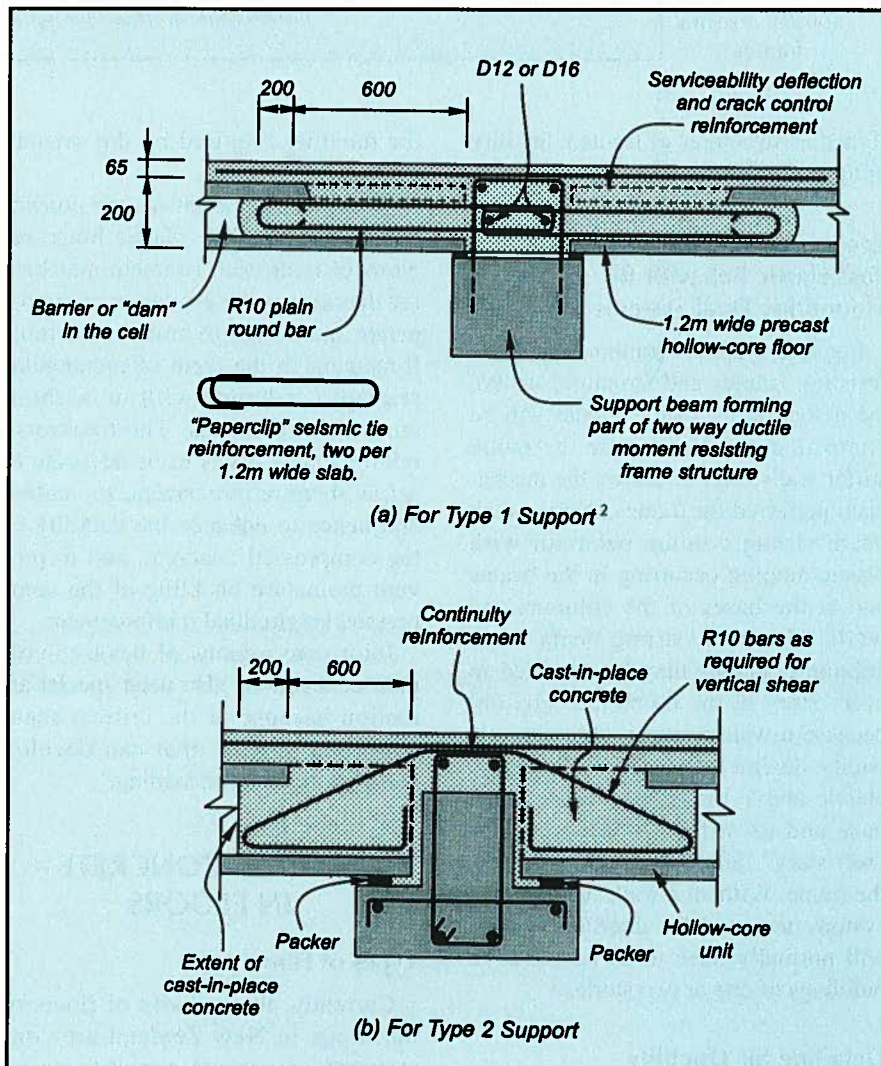


Fig. 9. Examples of types of support and special support reinforcement at ends of precast concrete hollow-core floor units.[2,13]

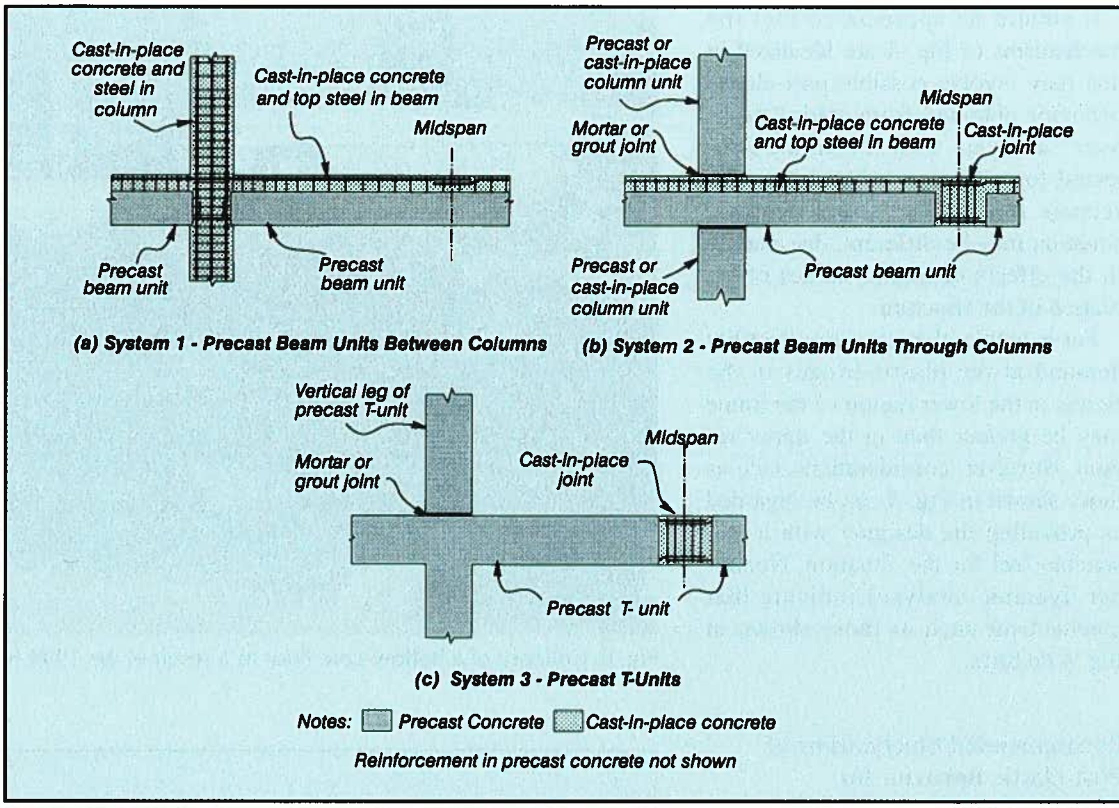


Fig. 10. Arrangements of precast reinforced concrete members and cast-in-place concrete for constructing reinforced concrete moment resisting frames.[2, 14-16]

signed as structures of limited ductility or to respond in the elastic range.

Recommended Mechanisms of Post-Elastic Behavior for Monolithic Dual Systems

For dual systems (combined moment resisting frames and structural walls), the deformations of the frames will be controlled and limited by the much stiffer walls. Fig. 6a shows the mechanism preferred for frames, that is, weak beam-strong column behavior with plastic hinging occurring in the beams and at the bases of the columns and walls. However, strong beam-weak column behavior may be admitted in every story of the frame (see Fig. 6b) because a wall proportioned using capacity design principles will remain elastic above the plastic hinge at the base and its stiffness will prevent a "soft story" failure from developing in the frame. Without a wall, such a frame system designed for ductile response will normally have to be restricted to buildings of one or two stories.[3]

Detailing for Ductility

Structures need to be detailed so as to possess sufficient ductility to match the ductility required by the seismic forces used in design.

The most important design consideration for detailing plastic hinge regions of reinforced concrete members for ductility is the provision of appropriate quantities of transverse reinforcement in the form of rectangular stirrups, or hoops with or without cross ties, or spirals. The transverse reinforcement needs to be adequate to 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.[3]

Joint core regions of beam-to-column connections also need special attention because of the critical shear and bond stresses that can develop there during seismic loading.[3]

PRECAST CONCRETE IN FLOORS

Types of Floor

Currently, the majority of floors of buildings in New Zealand are constructed of precast concrete units, spanning one way between beams or walls. The precast concrete units are either of pretensioned prestressed 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. Probably, the most common floors are constructed from precast concrete hollow-core floor units typically 200 mm (8 in.) deep, or deeper.

As well as carrying gravity loading, floors need to transfer the in-plane imposed wind and seismic forces to the supporting structures through diaphragm action. The best way to achieve diaphragm action when precast concrete floor elements are used is to provide a cast-in-place reinforced concrete topping slab over the precast units.

Where precast concrete floor units are used without an effective cast-in-place concrete topping slab, in-plane force transfer due to diaphragm action must rely on appropriately reinforced joints between the precast units. This may be difficult to achieve in some floors unless the connections between the precast units are specifically designed and constructed.

Support Details

The supports for precast concrete floor units may be simple or continuous.

Three types of support for precast concrete hollow-core or solid slab flooring units seated on precast beams, identified by the New Zealand Guidelines,[2] are shown in Fig. 7. The differences between these types are the depth of the supporting beam prior to the cast-in-place concrete being placed.

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 movements caused by earthquakes or other effects which reduce the seating length (see Fig. 8).

One source of movements during severe earthquakes, which could cause precast concrete floor units to become dislodged, is that beams of ductile reinforced concrete moment resisting frames tend to elongate when forming plastic hinges, which could cause the distances spanned by precast concrete floor members to increase.[2,9,10]

The elongation is due to the tensile yielding of the reinforcement associated with plastic hinge formation. Longitudinal extensions of beams in the order of 2 to 4 percent of the beam depth per plastic hinge have been observed in tests in which expansion was free to occur.[2,9] The compression induced in beams by restraint against this expansion can enhance the flexural strength of beams and cause cracking of the topping slab.

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

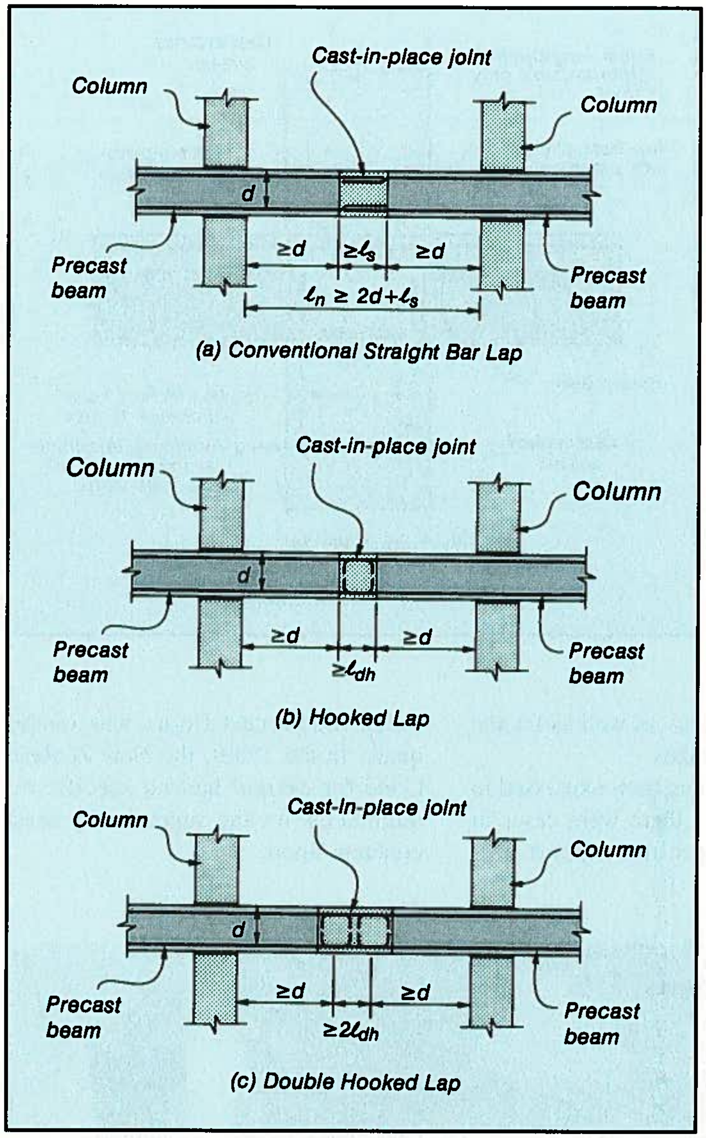


Fig. 11. Some details of mid-span connections between precast reinforced concrete beam elements.[2,14-16]

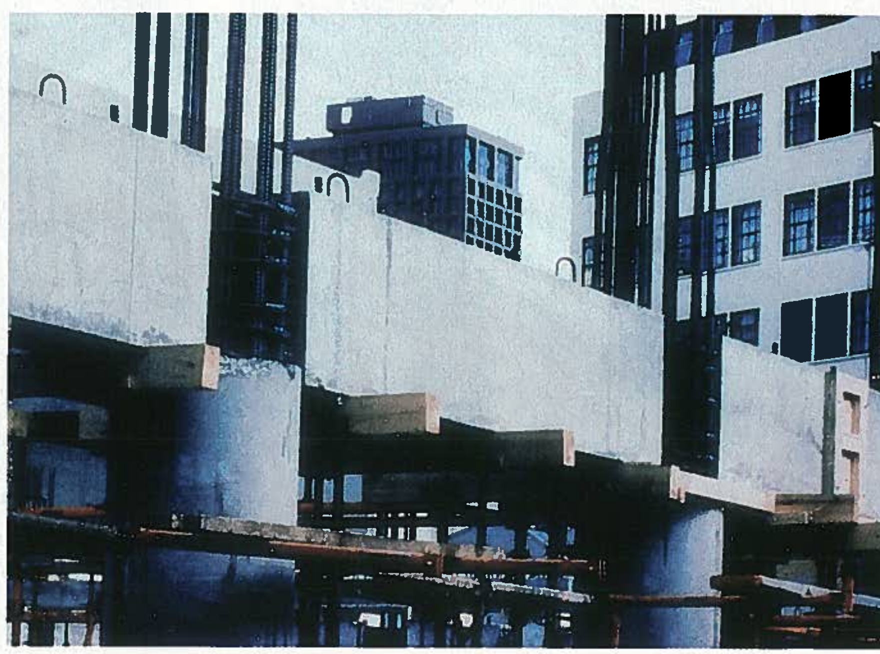


Fig. 12. Construction of a building frame using System 1 in New Zealand.

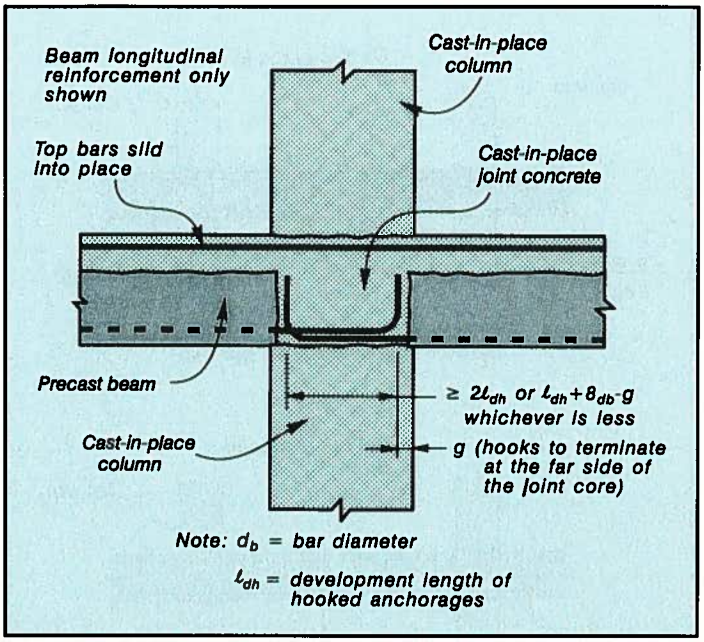


Fig. 13. Hooked lap of bottom bars within joint core for System 1.[2]

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 floors was inadequate. In the 1980s, the New Zealand Code for design[1] had no specific requirements for the support of precast concrete floors.

As a result, the current New Zealand concrete design standard, NZS 3101:1995,[3] provides 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, normally 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. However, if shown by analysis or by test that the performance of alternative support details is adequate, the above specified end distances need not be provided.

The above recommendation, which requires proven alternative support details unless the specified end distances are provided, is similar to the recommendation in the building code of the

Fig. 14. Construction of the 22-story perimeter frame of the Price Waterhouse-Coopers building using System 2 in Christchurch, New Zealand.

Fig. 15. Construction of the 152 m (499 ft) tall ANZ Tower using System 2 in Auckland, New Zealand.

American Concrete Institute, ACI 318-99.[11]

One method of providing the alternative details which permit smaller seating lengths is to use special reinforcement between the ends of the precast concrete floor units and the supporting beam, which can carry the 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,[2] and recommended earlier by the Precast/Prestressed Concrete Institute and the Fédération Internationale de la Précontrainte. For example, for precast concrete hollow-core units, the special reinforcement may be either placed in some of the cores which have been broken out at the top and filled with cast-in-place concrete or grouted into the gaps between the precast units.

Tests conducted at the University of Canterbury[12,13] on special reinforcement, placed in filled cores at the ends of hollow-core units and passing over precast supporting beams, have investigated a number of types of special support reinforcement. They were found to be able to support at least the service gravity loads of the floor, in the event of loss of end seating.

The plain round straight or draped reinforcement with hooked ends shown in Fig. 9 are favored.[2] Plain round end hooked reinforcement was found to perform better than deformed reinforcement since bond failure propagating along the plain round bars allowed extensive yielding along the bar, therefore allowing substantial plastic elongation before fracture.[12]

MOMENT RESISTING FRAMES INCORPORATING PRECAST REINFORCED CONCRETE ELEMENTS

Moment resisting frames incorporating precast reinforced concrete elements have become widely used in New Zealand. The design aim has been to achieve behavior of the frame as for monolithic cast-in-place construction (cast-in-place emulation).[14,15,16]

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 less 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 inte-

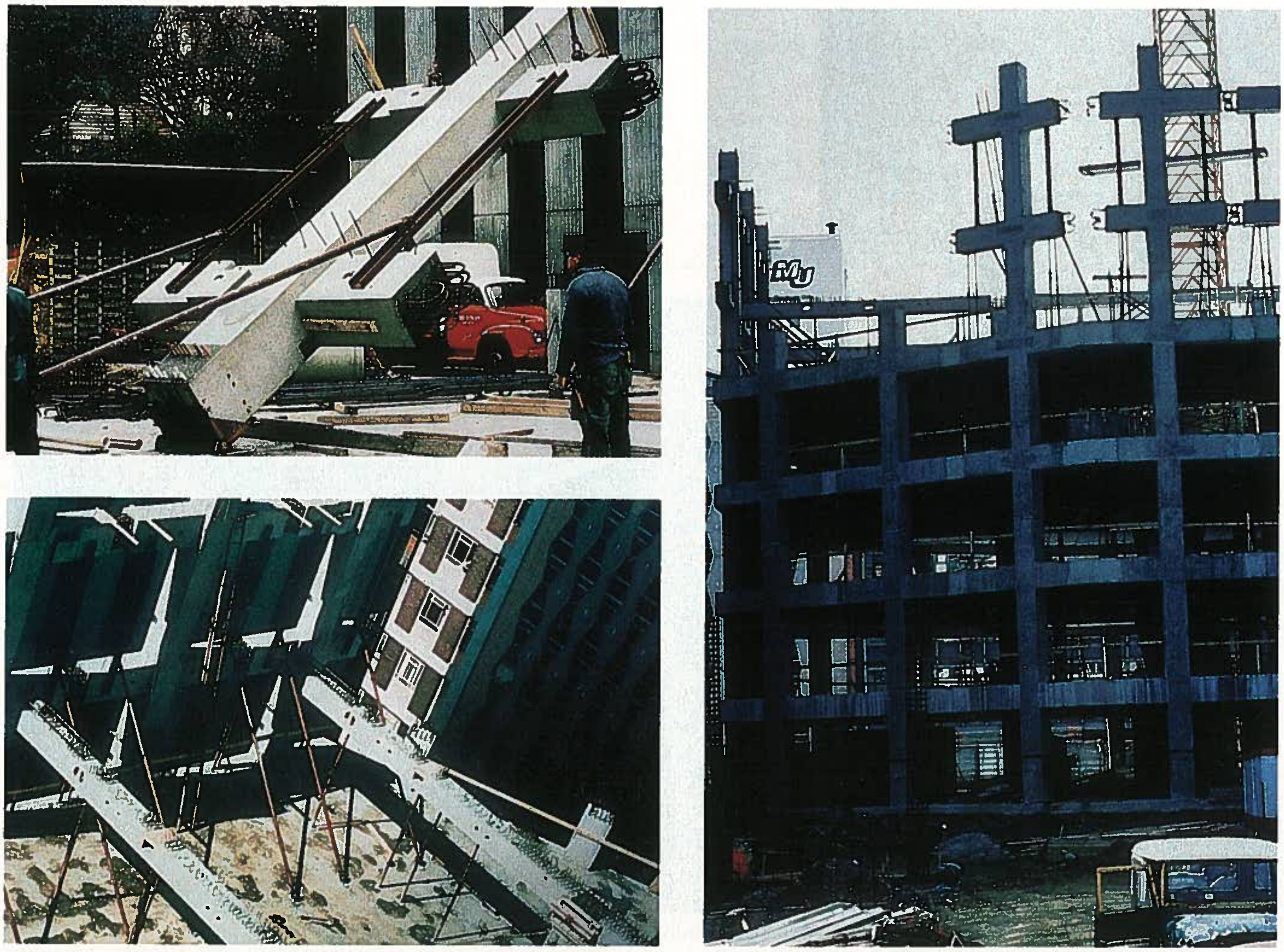


Fig. 16. Construction of the 13-story perimeter frame of Unisys House using two-story high cruciform-shaped precast units (System 3) in Wellington, New Zealand.

rior 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 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. References 17 to 22 give details of several buildings constructed in New Zealand which incorporate significant quantities of precast concrete in their frames and floors.

Arrangements of Precast Concrete Members and Cast-in-Place Concrete

The precast reinforced concrete frame elements are normally connected by reinforcement protruding into regions of cast-in-place reinforced concrete. Three arrangements of precast concrete members and cast-in-place concrete, forming ductile moment resisting multistory reinforced concrete frames, commonly used for strong column-weak beam designs in New Zealand, are shown in Fig. 10. Fig. 11 shows some midspan connection details used with Systems 2 and 3 of Fig. 10.

The precast concrete beam elements of System 1 of Fig. 10 are placed between the columns and the bottom longitudinal bars of the beams are anchored by 90-degree hooks at the far face of the cast-in-place joint core (see Figs. 12 and 13). Figs. 14, 15 and 16 show structures under construction using Systems 2 and 3 of Fig. 10.

For System 2, the vertical column bars of the column below the joint protrude up through vertical ducts in the precast beam unit (see Fig. 14), where they are grouted, and pass into the column above. The white plastic tubes over the bars in Fig. 14 are there to help pass the bars through the vertical ducts. The plastic tubes are then removed. Those vertical bars are connected to the bars in the column above by splices if the column is of cast-in-place concrete or by steel sleeves or ducts which are grouted if the column is of precast concrete (see Fig. 17).

The columns of the precast elements of System 3 are connected by longitudinal column bars which protrude into steel sleeves or ducts in the adjacent element and are grouted (see Figs. 16 and 17). The beams are connected using a cast-in-place joint at midspan.

It should be noted that the capacity design procedure for these three systems will ensure that yielding of the column bars at the connections is kept to a minimum. Fig. 18 shows a further system using pretensioned prestressed concrete U-beams and cast-in-place reinforced concrete.[23]

Many of the currently used connection details shown in Figs. 10 to 18 have had experimental verification.[10,23,24] The verification involved simulated seismic loading tests conducted on typical full-scale beam-to-column joint specimens, designed for strong column-weak beam behavior, to determine the performance of the hooked bar anchorage of the bottom bars of the beam in the cast-in-place concrete joint core in System 1 of Fig. 10, the performance of the grouted vertical column bars which pass through vertical ducts in the precast beam in System 2 of Fig. 10, and the performance of the composite beam shown in Fig. 18.

Simulated seismic loading tests have also been conducted to determine the performance of the cast-in-place midspan connections between precast beam elements shown in Fig. 11. The test results indicated performance as for totally cast-in-place construction.

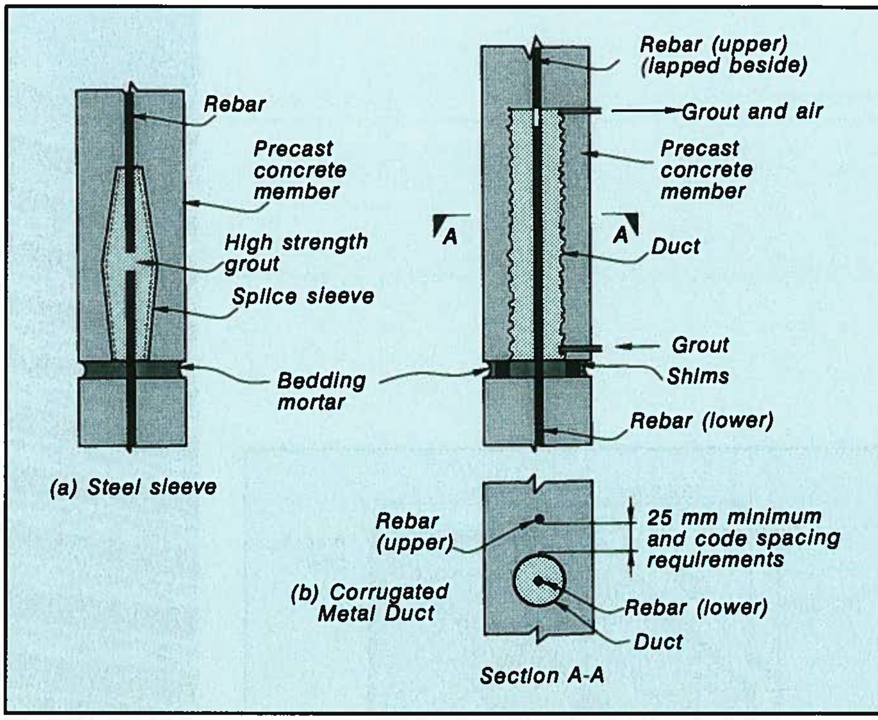


Fig. 17. Steel sleeve splices and corrugated metal ducts used for column-to-column and slab-to-slab connections.

Precast Reinforced Concrete Moment Resisting Frames and Cast-in-Place Reinforced Concrete Structural Walls

Structures comprising both reinforced concrete structural walls and flexible moment resisting frames can also be used to advantage. 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, being much more flexible than the walls, will be called on to resist only a small portion of the horizontal forces, the amount depending on the relative stiffnesses of the walls and frames. The columns of such frames in the building mainly carry the gravity loading.

When such systems are used in seismic regions, the frames can be designed for limited ductility, providing 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 building so designed in New Zealand is shown in Fig. 19. 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 reinforced concrete beams and the reinforced concrete columns were designed mainly for gravity loading.

PRECAST CONCRETE STRUCTURAL WALLS

Construction Details

Most structural walls for multistory buildings in New Zealand are of cast-in-place concrete, but significant use is made of precast concrete walls for smaller buildings (see Fig. 20). Precast reinforced concrete structural wall construction usually falls into two broad categories: "monolithic" or "jointed."

In monolithic wall construction, the precast concrete elements are joined by "strong" reinforced concrete connections which possess the stiffness, strength and ductility approaching that of cast-in-place concrete monolithic construction.

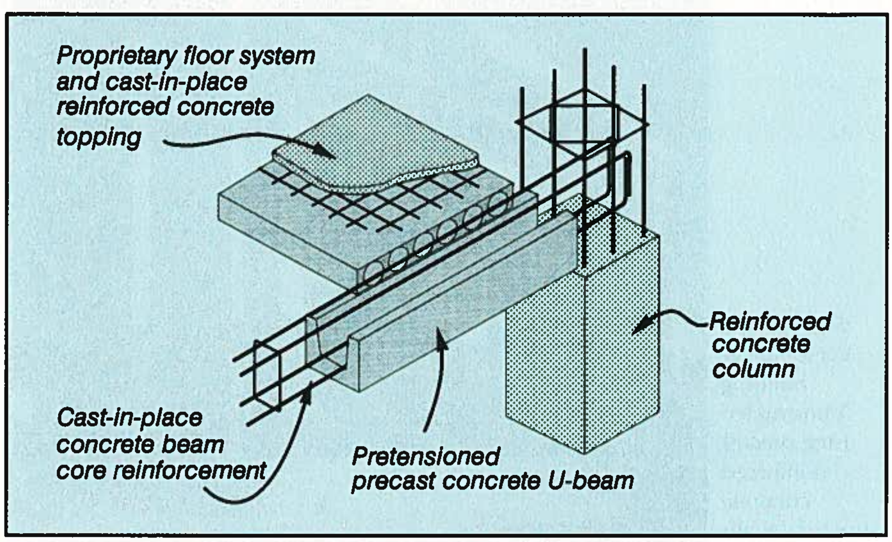
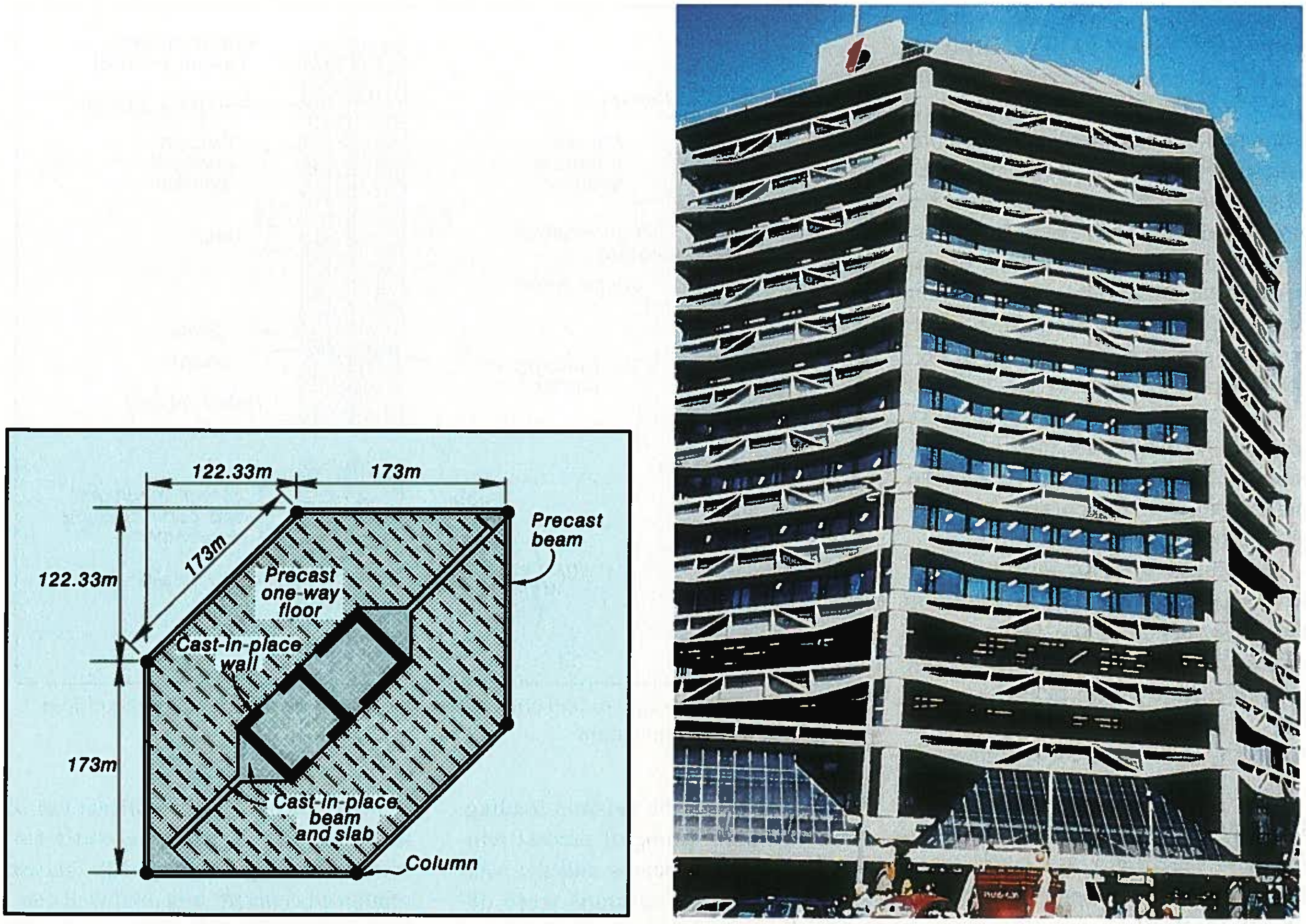


Fig. 18. A structural system involving precast pretensioned prestressed concrete U-beams and cast-in-place reinforced concrete.[23]



(a) Typical floor plan
(b) The perimeter structure

Fig. 19. The Westpac Trust building constructed with a precast reinforced concrete perimeter frame with seismic forces resisted by cast-in-place reinforced concrete interior structural walls in Christchurch, New Zealand.


Fig. 20. The West Fitzroy building constructed using precast reinforced concrete structural walls in Christchurch, New Zealand.

In jointed wall construction, the connections are "weak" relative to the adjacent wall panels and, therefore, govern the strength and ductility of the building.

Monolithic Wall Construction

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

At the horizontal joints between precast concrete wall panels or foundation beams, the ends of the panels are usually roughened to avoid sliding shear failure, and the joint is made using mortar or grout. The vertical reinforcement protruding from one end of the panel and crossing the joint is connected to the adjacent panel or foundation beam by means of grouted steel splice sleeves or grouted corrugated metal ducts (see Fig. 17).

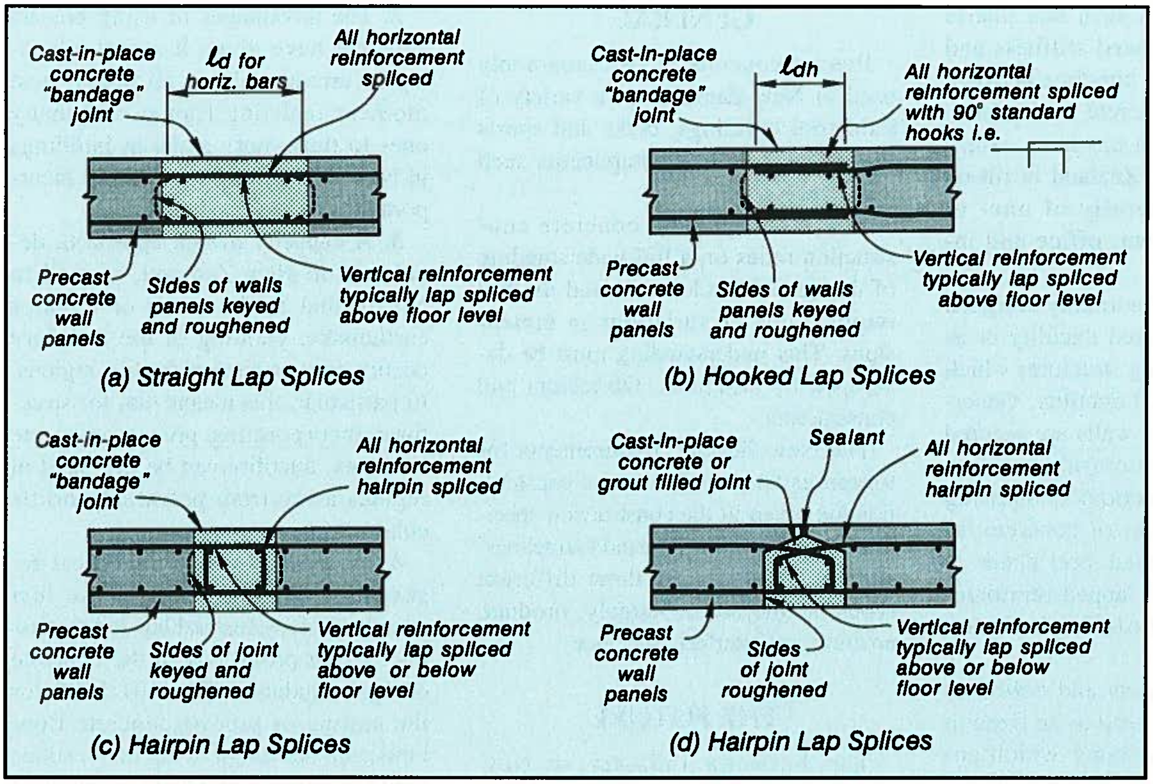


Fig. 21. Some vertical joints used for monolithic precast concrete wall construction.[2]

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. Fig. 21 shows some possible vertical joint details between precast wall panels that make use of cast-in-place concrete. The widths of the strips of cast-in-place concrete are determined by code requirements for lap lengths of horizontal reinforcement.

Fig. 21a shows a joint with sufficient width to accommodate the lap splice length of the straight horizontal bars that protrude from the precast wall panels. Fig. 21b shows hooked lap splices that enable the width of joint to be reduced. Figs. 21c and 21d show hairpin spliced bars, which may not be convenient to construct since once the lapping bars have been overlapped, the ability to lower the precast panels over starter bars is very restricted.

At exterior walls, support for precast floor units can be achieved in a number of ways — for example, on a steel angle anchored to the wall panel, on a concrete corbel, or on a recess in the wall panel. These connections are designed to transfer the floor inertia forces to the walls and to avoid loss of seating. Typical floor-to-wall connection details are described elsewhere.[2]

Jointed Wall Construction

In jointed wall construction, the connection of precast reinforced con-

(a) Lecture theatres

(b) Commercial buildings

Fig. 22. Typical low rise buildings constructed in New Zealand incorporating tilt-up precast concrete walls.

crete components is such that planes of significantly reduced stiffness and strength exist at the interface between adjacent precast concrete wall panels. Jointed construction has been extensively used in New Zealand in tilt-up construction, generally of one- to three-story apartment, office and industrial buildings.

The buildings are normally designed as structures of limited ductility or as elastically responding structures which require only nominal ductility. Generally, tilt-up concrete walls are secured to the adjacent structural 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.

The concrete inserts and bolted or welded steel plates need to be fixed to the concrete in a manner which ensures ductile yielding of the reinforcing bars, bolts or plates before a brittle pullout failure from the concrete occurs. This requires a capacity design approach for the fixing to ensure that the desired yielding of the steel occurs under the most adverse strength conditions.

Tilt-up construction has been used to build structures with complex geometric and appealing architectural features. Some photographs of typical low-rise buildings constructed using precast concrete walls are shown in Fig. 22.

Unfortunately, the current New Zealand concrete design code[3] does not have design recommendations covering all aspects of tilt-up construction. However, a research project has been in progress at the University of Canterbury,[25,26,27] which has the aim of cataloguing currently used connection details, assessing and testing them where necessary, and recommending appropriate details for tilt-up and jointed construction.

GENERAL

Precast concrete is also commonly used in New Zealand for a variety of industrial buildings, tanks and sports stadiums, including components such as stairways.

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.[28] The New Zealand Guidelines[2] suggests considering three different types of tolerances, namely, product, erection and interface tolerances.

THE FUTURE

The building industry in New Zealand has embraced the use of precast concrete. All indications are that precast concrete construction will continue to be used extensively in the future. The use of the capacity design approach and the development of appropriate methods for the detailing of connections and members have given designers the confidence that precast concrete can be used in an active seismic region such as New Zealand. The advantages of using precast concrete have given it a cost advantage.

CONCLUSIONS

Based on accumulated design and construction experience during the last three decades, the following conclusions can be made:

1. Confidence in use of precast concrete in structures in an active seismic zone such as New Zealand has required the application of appropriate design approaches and the development of satisfactory methods for connecting the precast elements together.

2. The advantages of using precast concrete have given it a cost advantage. Currently, almost all floors, most moment resisting frames and many one- to three-story walls in buildings in New Zealand are constructed incorporating precast concrete.

3. A capacity design approach, developed in New Zealand, is used to ensure that in the event of a major earthquake, yielding of the structure occurs only at chosen ductile regions. In particular, this means that for structures incorporating precast concrete elements, ductility can be provided in regions away from potentially brittle connections.

4. Experimental and analytical research conducted during the last decade in New Zealand has led to seismic design provisions in the concrete design standard NZS 3101:1995 for the seating of precast concrete floor units and the design of the connections between precast elements in moment resisting frames. Guidelines have also been written for aspects of the seismic design of tilt-up wall construction based on experimental and analytical research.

5. The future of precast concrete construction in New Zealand appears to be bright.

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