Design Considerations for Precast Prestressed Concrete Building Structures in Seismic Areas

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Long-time PCI Professional Member Alfred A. Yee has been in engineering practice since 1953, specializing in the design of precast and prestressed concrete structures. Most of these structures have been located in the Pacific Rim — one of the most seismologically active areas in the world. In recognition of his innovative work in advancing concrete technology, he was awarded an Honorary Doctorate Degree in engineering from Rose-Hulman Institute of Technology in 1976. That same year Dr. Yee was elected to the prestigious National Academy of Engineering. He obtained his bachelor’s degree in civil engineering from Rose-Hulman Institute of Technology and his master’s degree from Yale University. Dr. Yee has lectured all over the world and is the author of numerous published papers including some that have appeared in the PCI JOURNAL.

Discusses the major design considerations necessary in the successful construction of precast, prestressed concrete building structures situated in seismic areas. The importance and interaction of stiffness, strength, toughness and, especially, fail-safe connections are emphasized. Such connections are needed not only to transfer loads but to provide continuity and overall monolithic behavior in the entire structure. Suggestions are given on how to increase the stiffness of the structure as well as advice on good and poor design practices. Examples of precast concrete buildings drawn from the author’s 40 years design experience are shown and commented upon.

Sound principles, attention to detail and good judgment are necessary in designing precast, prestressed concrete structures in seismic areas. These ingredients, of course, apply also to the design of all types of structures, whether they are made of concrete, steel or composite materials. However, in the case of precast and prestressed concrete structures, particular attention must be given to stiffness, strength and toughness requirements, and especially to connection details. Their function is not only to transfer loads but to develop continuity and monolithic behavior in the entire structure.

Strength, stiffness and toughness minimize second order effects resulting from drift, deflection and rotation.

It is important that movements and rotations be restrained in an earthquake because they can cause both structural and nonstructural damage to buildings and in severe cases can induce collapse. Shear walls combined with each other through deep beams or deep wall girders or trusses to form giant moment frames is one good method to increase the overall stiffness of a structure.

Soft floors, seismic joints and eccentricities (non-symmetry) in building frames should be avoided regardless whether they can be justified by mathematical calculations. If seismic joints become absolutely necessary due to unusual structure size, it is suggested that such joints be semi-restrained by a system of tension ties in
combination with the insertion of a cushioning material between building components to reduce possible damage from pounding action.

The purpose of this paper is to present the major design considerations that are required in the safe construction of precast, prestressed concrete building structures situated in seismic areas. Another purpose of the paper is to present real life examples of buildings that have withstood major earthquakes and to discuss some of the design features and practices that do and do not work well in seismic areas.

A list of articles pertinent to this paper is given in References 1 to 12.

This paper is drawn from the author’s 40 years experience designing precast, prestressed concrete structures. Most of these structures are situated in the Pacific Rim which is one of the most seismically active areas in the world.

**CONNECTION DETAILS**

The single most important ingredient in the design of precast concrete structures is the connection details. Connections between precast building elements such as columns, beams, slabs and shear walls must effectively integrate the individual structural components in full continuity with each other so that the overall building structure behaves monolithically. In this manner, the structural analysis and behavior of a building frame would be identical to that for a cast-in-place structure except that the framing system now uses precast concrete components which are assembled together to act monolithically.

As an example, take the case of simple precast bearing wall panels supporting precast concrete floor slabs. The most effective system we have found uses composite in-place connections to develop full continuity between adjacent slab spans as well as the precast concrete wall panel units. Fig. 1a illustrates a precast bearing wall panel supporting a precast floor or roof slab. The joint detail is arranged so that dimensional errors in construction would have little influence on the structural integrity and performance of the composite system.

The precast floor slab panels are integrated with a composite in-place structural concrete topping which provides a uniform leveling surface and a horizontal structural diaphragm to resist seismic forces. Utilities such as electric, telephone and television conduits can be buried within this topping. The precast concrete slab panels are detailed with extended reinforcing bars bent in a diagonal pattern, crisscrossing over the precast bearing wall panels. Negative moment steel is added in the composite topping to develop continuity between adjacent slab spans. The precast concrete wall panel of the upper floor is integrated in full continuity with that of the lower floor by means of grout-filled steel sleeve mechanical connections.

This particular joint does not require a specified minimum bearing area between the precast floor slab and wall panel in order to achieve adequate vertical support. The extended diagonal reinforcing steel from the ends of the precast concrete slab panels is designed to provide full shear resistance to the anticipated vertical dead and live loads on the slab span. Therefore, no reliance is placed on the compression bearing area between the floor slab panel and the top of the precast wall panel after this joint is in place.

The wall panel thickness or top of the wall need not be widened to provide support tolerance for the precast concrete slab. Negative moment steel in the topping not only develops full continuity between slab spans but also serves as a tie to resist horizontal tension between the slab units. In effect, the reinforcement prevents the slabs from pulling apart over the wall support when the concrete undergoes volume changes or if an earthquake occurs.

In the event that this particular joint is severely damaged during an earthquake and the joint deteriorates such that the concrete in the top of the wall panel as well as the bearing ends of the slab panels are spalled off, the floor slab will not be in any danger of collapse. This is because the extended overlapping diagonal bars in the bearing ends of the slab are designed to fully resist anticipated vertical dead and live loads. No reliance is placed on a bearing element or shear in the concrete. Therefore, this detail is practically speaking fail-safe.

![Fig. 1a. Precast bearing wall panels support precast concrete slabs with in-place composite structural topping.](image-url)
Fig. 1b. Precast slab panel erroneously manufactured short of wall support.

Fig. 1c. Extended reinforcement from slab panels diagonally crossing over wall supports to provide slab shear resistance. Supplementary negative moment steel in the topping area will develop full continuity between adjacent composite slab sections.

Fig. 2a-2b. Connection detail for interior and exterior beam.

Fig. 3a to 3e illustrate some of the more common types of mechanical connections.

Fig. 3a is a steel sleeve connection that utilizes a special high strength
Fig. 3a-3e. Typical mechanical connections.

Fig. 3f. Butt and lap weld connections.

**STRUCTURAL TESTS ON FULL SIZE COLUMNS WITH NMB SPlice SLEEVES**

(Test Report NPD-024)

Fig. 3g. Summary of results of tests on full size columns with splice sleeves.

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non-shrink cement grout matrix which forms a grout wedge anchoring the reinforcing steel to the external sleeve. This type of mechanical connection has been tested extensively for repetitive loads in both tension and compression and is widely used throughout the world.

Fig. 3b shows a splice connection utilizing a metallic matrix which engages the deformations of the reinforcing steel bars to a serrated inner surface of the sleeves.

Fig. 3c shows a threaded steel coupler engaging threads cut into the ends of the reinforcing bars. In some couplers the surface deformations of the bars are used as threads which in turn engage the coupler.

In Fig. 3d, the threaded coupler is tapered to accommodate the tapered threaded ends of the steel bars. By tapering the threaded ends of the steel bars, insertion into the threaded coupler is facilitated.

In Fig. 3e, the ends of the reinforcing bars are engaged by a steel sleeve hydraulically swaged under pressure to the surface deformations of the reinforcing bars.

Typical butt and lap welded connections are shown in Fig. 3f.

Fig. 3g is taken from a report on repetitive loading tests on columns with splice sleeve connections using a special grout. In this test the steel sleeves are located at the floor level of a column-to-beam specimen. The assembly is loaded so that bending moments attain a maximum intensity in the specimen. The column-beam connection is then wracked repeatedly by a jacking arrangement to simulate seismic forces on a column-to-beam frame. Results of these tests show that the mechanical connections enabled the precast concrete to perform in both strength and ductility equal to that of monolithically cast columns with fully continuous reinforcing steel.

In a "blind" connection, the mechanical connection is basically a tapered steel sleeve which envelopes the

with adequate grouting in the joint area, the vertical precast concrete column unit can be joined at the floor line or at any other convenient location along the height of the column to develop full continuity.

It should be noted that the mechanical connection between column units can be "blindly" executed; that is, access openings are not needed, thus eliminating the need for patching and grouting after the connection is installed. The grout-filled steel sleeve connection shown in Fig. 4 is classified as a "blind" connection that can achieve full continuity in the steel. A thin layer of grout inserted between the column units, together with the continuity provided in the reinforcing steel by such connections, will result in connected column units that act monolithically as a fully continuous member.

In a "blind" connection, the mechanical connection is basically a tapered steel sleeve which envelopes the...
ends of the connecting reinforcing bars as shown in Fig. 3a. This sleeve is made large enough to allow reasonable tolerance for erection and is completed by filling with a special grout to develop the required anchorage of the reinforcing steel.

Threaded couplers or sleeves filled with metallic matrix can also be used but these require access pockets to execute the connection and therefore cannot be considered as “blind” connections. These threaded couplers or metallic matrix sleeves have been used mainly for cast-in-place construction since the tolerances required would be extremely close and access to the sleeves must be provided before concrete pouring.

The joint location between the precast concrete beam and column can be at a point of highest moment adjacent to the column or at some distance away for convenience in precasting and erection. In the case of a horizontal joint, the joint area is cast-in-place to develop full continuity. The mechanical connection is totally exposed before installation with in-place concrete and, therefore, the reinforcing bars can be spliced by grout-filled sleeves, welding or other methods.

Welding is labor intensive and time consuming. The heat generated from welding can cause damage to bond in the steel bars and cracking in the adjacent precast concrete. Furthermore, high quality welding requires close supervision and inspection whereas mechanical connectors using sleeves with grout can be installed quickly and reliably without the need for special skills and supervision.

Grout-filled sleeves have sufficient built-in tolerances and can easily be installed at normal temperatures, thus avoiding any damage to the reinforcing steel or adjacent concrete. In situations where epoxy coated reinforcing steel is used, grout-filled sleeves are particularly advantageous because no heat or damage to the epoxy would result. Sleeves with a metallic matrix or threaded couplers are difficult to apply in precast concrete work since they would require extreme precision in bar placement to have all corresponding reinforcing steel properly aligned.

To develop continuity between the beam and column elements, the longitudinal steel in the connection must have sufficient anchorage in the column. Equally important, confinement steel must envelop both the column vertical steel and the horizontal bars of the beam or girder in the high moment region adjacent to the column. High strength concrete sufficiently plasticized to completely fill all void areas between the reinforcing steel and precast concrete contact surfaces will produce a fully continuous joint integrating the beam and the column in monolithic frame action.

Fig. 5 shows the connection between precast concrete shear wall panels with boundary members. The grouted joint between the precast panel elements in combination with properly spaced mechanical connections to develop continuity in the reinforcing steel can produce a shear wall frame acting monolithically throughout its height.

The precast concrete shear wall reinforcement is detailed in exactly the same way as a cast-in-place shear wall. Vertical wall steel spaced regularly throughout the length of the wall panel is required to develop shear friction along the entire contact face between the precast components. Boundary elements are detailed to provide the required tension and compression resistance as would be analyzed in a monolithic cast-in-place structure.

Fig. 6a. Overall view of Ramon Magsaysay Building, Manila, Philippines.

Fig. 6b. Lower portion of Ramon Magsaysay Building, Manila, Philippines.

This 18-story (including basement) reinforced concrete structure utilizes precast, prestressed concrete joists and composite in-place floor slabs. The structural system to resist lateral forces due to seismic or wind loads is a shear wall system. The shear walls are symmetrically clustered about the center of the building, thus eliminating eccentric forces. This building experienced severe earthquake forces in 1968 and 1972 (Richter Scale 7.2) and in July 1990 (Richter Scale 7.7) without suffering any structural damage.
STRUCTURAL FRAMING SYSTEMS

In the overall performance of a structural concrete frame, primary consideration should be given to the development of stiffness, strength and toughness.

In the past, some design engineers believed that pure moment resistant frame systems were more desirable than shear wall systems for high rise buildings in seismic regions. The justification for this philosophy is that moment resistant frames are flexible and therefore can move, deflect and absorb more energy, thus reducing the required calculated base shear forces. It was also thought that, in the case of shear wall structures, the base shear forces would be greatly increased due to the stiffness of such framing and therefore would be considered undesirable for seismic areas.

However, our experiences during the past 20 years have shown that flexible building structures that incur excessive deflection or rotation will suffer considerable damage in an earthquake. This damage could result from repetitive stresses at the joints between framing members, which causes progressive deterioration in stiffness. This, in turn, would cause a further increase in deflections and rotations, resulting in considerable damage to non-structural partitions, facades, utility lines, building contents and other building elements — including damage to the structural frame itself. With progressive joint deterioration, such building structures can become statically unstable and collapse completely. Increasing the lateral stiffness of the structure will result in a decrease in detrimental second order (P-Delta) effects and thus increase the critical load capacity and overall stability of the building.

In the case of shear walls, these elements produce stiffness and, although they may crack under repeated loads, they are able to maintain a large degree of their stiffness without the significant deterioration displayed by moment frame structures. Earthquakes in the Philippines, Chile, Mexico and elsewhere have repeatedly confirmed the advantages of the stiff shear wall building frame concept.

Figs. 6a and 6b show a high rise office building in Manila which utilizes precast, prestressed concrete joists integrated with a cast-in-place structural topping. Connections of the joists are similar to that shown in Fig. 2b. The main lateral resisting element is a central concrete shear wall core which was cast-in-place. Main vertical reinforcing steel in the shear wall core was joined by mechanical connections using a metallic matrix.

In August of 1968, this building experienced a major earthquake with epicenter immediately east of Manila.
Fig. 8. Cast-in-place reinforced concrete structure with stiff upper floors and a soft moment frame ground floor showed partial collapse of the flexible floor columns. The photograph on left shows the overall building, while the view on the right shows a closeup of the column failure.

at a magnitude of 7.0 on the Richter Scale. The only damage observed was cracking in the nonstructural concrete hollow block partitions of the toilet and stairwell areas. Some loosening of screws attaching metal partitions in the toilets were also noticed.

The exterior surface of the building is clad with marble slabs embedded to precast panels which in turn are compositely attached to the building frame by extended reinforcing bars. No cracks or damage occurred on this façade despite reports from the night security personnel that the building did undergo substantial accelerations and movement during the earthquake.

Other nearby buildings suffered severe damage to both structural and nonstructural elements. In some instances, partial or total collapse occurred, resulting in a loss of more than 300 lives. Virtually all moment frame buildings of both structural steel or reinforced concrete suffered major structural or nonstructural damage during the earthquake. Nonstructural elements, such as marble or stone claddings, nonbearing partitions, ceilings and utility ducts, were badly damaged or collapsed and required extensive repairs.

During this earthquake in Manila, a hotel building structure (shown in Fig. 7) utilizing wide columns and deep spandrel beams demonstrated its capability of maintaining stiffness even after cracking under repeated loads. The inherent stiffness of the framing system prevented the collapse of the building.

In another instance, a reinforced concrete structure with stiff upper floors and a soft ground floor moment frame (shown in Fig. 8) suffered a partial collapse due to the flexibility of the column beam framing system on the lower floor.

A similar magnitude earthquake occurred in Manila in 1972 and the high rise office building shown in Fig. 6 again experienced only minor damage to the nonstructural concrete block walls.

In the devastating earthquake of July 16, 1990 (magnitude 7.7 on the Richter Scale epicenter north of Manila), at least 25 major buildings were heavily damaged in Manila, with a loss of 34 lives. Again, this high rise office building experienced only minor damage to the nonstructural concrete block walls in the toilet and stairwell areas. No other structural damage was observed.

In 1970, a 38-story building using precast, prestressed concrete floor slabs and precast concrete column-tree frames was built in Hawaii (see Figs. 9a-9d). The lateral resisting framing system comprised a cast-in-place shear core and precast concrete column-tree frames. In addition to supporting vertical loads, the column-tree frames functioned as a main lateral re-
Moana Hotel, Honolulu, Hawaii. This 38-story, 1260 room hotel was constructed using cast-in-place shear walls with precast concrete moment frames supporting precast prestressed concrete floor slabs. Built in 1970, today the building is in very good condition and has required minimal maintenance over the past 20 years.

Fig. 9a. Ala Moana Hotel, Honolulu, Hawaii. Precast moment frame unit being erected.

Fig. 9b. Ala Moana Hotel, Honolulu, Hawaii. Precast moment frame unit being erected.

Fig. 9c. Details for splicing upper and lower columns used in building Ala Moana Hotel, Honolulu, Hawaii.
Fig. 9d. Ala Moana Hotel, Honolulu, Hawaii. Solid precast concrete walls were applied to transverse and longitudinal facade to minimize drift between floors.
sisting element in the longitudinal direction of the building while the shear core primarily resisted the lateral forces in the transverse direction. This structure, designed for Seismic Zone II, experienced seismic forces of magnitude 6.0 on the Richter Scale in April of 1973 without any visible damage.

The precast concrete column tree frames were spliced at midheight with grouted steel sleeves (see Figs. 9b and 9c). To minimize the drift and torsional effects in the longitudinal and transverse directions, solid precast concrete panels were inserted at the end walls and extreme ends of the longitudinal facade (see Fig. 9d).

In most high rise buildings, shear walls combined with moment frames are generally used to resist lateral forces. As shown in Fig. 10, a pure shear wall is stiffer at the lower levels while a moment frame is stiffer at the upper levels; the combination of these two elements can result in minimizing large deflections in both the upper and lower floor levels.

Fig. 11 illustrates that a soft floor can be particularly hazardous if it is located in the lowest floor of a building where maximum lateral loads could occur under seismic conditions. An actual example of this type of failure is shown in Fig. 8. However, soft floors have been known to fail under seismic conditions even if located on upper story levels.

If only the lower floors were to be stiffened with shear walls and then not continued at the upper levels, abrupt change in stiffness could result in the failure of the soft floor at the upper level where a sudden change of stiffness occurs. Therefore, where practical, both shear walls and moment frames should run continuously throughout the entire building height without allowing such soft floors to occur.

A structural arrangement utilizing shear walls connected by deep girders and columns (where they are permitted architecturally) can create significant additional stiffness in the building frame.

In Figs. 12a and 12b, a giant moment frame system can be created by utilizing shear walls in combination with wall girders that are floor-to-floor in height. These wall girders could be used wherever permissible, such as at
mechanical room and elevator room floor levels. As shown in Fig. 13, when shear walls are coupled together with deep wall girders the resulting deflections are minimized considerably.

Fig. 14 shows a coupled shear wall system utilized in the building shown in Fig. 6. The vertical shear walls of the elevator cores are joined together at the roof penthouse level to enhance the stiffness of the building.

In Fig. 15, shear walls located at the end walls of the building are joined together with deep wall girders at the roof level to form a giant moment frame. The deep wall girders at the roof level are also used to support mechanical equipment rooms.

An effective and cost beneficial lateral shear resisting framing system could be attained by the use of super diagonals as shown in Fig. 16. These super diagonals couple the internal shear walls with the exterior column frame to generate maximum resistance and stiffness against lateral and overturning forces. The super diagonal can extend several floors vertically from each level in order to generate the most efficient height to span ratio. Super diagonals can be used effectively where deep wall girders are inappropriate because of occupant access problems.

Architectural and functional considerations will determine the most ideal locations of these super diagonal elements. Generally, the roof area is free from any functional restraints and super diagonal frames easily could be placed in this location to enhance the lateral stiffness of the building.

Eccentricities in the framing system should be avoided wherever possible regardless of justification by structural calculations and detailing. Eccentricities generally produce exaggerated movements that can be damaging. Indeed, many buildings with even moderate eccentricities have suffered total collapse during earthquakes. Deflections, drift and rotations should be minimized in every instance since they are the prime cause of damage and failure in building frames and non-structural elements. Furthermore, a symmetrical building would favor economical precast concrete construction due to the uniformity in production and erection of the precast concrete components.

Seismic joints (Fig. 17a) between building units should be avoided wherever possible since separate building units have different periods of vibration and under seismic activity tend to pound against each other at the joint areas. Considerable damage in seismic joint areas due to pounding of separate building units was witnessed in the 1985 Mexico City earthquake (Fig. 18). In many cases, total building collapse was directly caused by this pounding.

By eliminating seismic joints and developing full continuity between building frame units, pounding can be prevented. However, in eliminating seismic joints, floor areas now become relatively large and the question of shrinkage effects and floor slab restraints causing problems of slab and wall cracking is of major concern. In most instances, these problems can be solved by the addition of continuous reinforcing steel in both directions to handle the shrinkage effects.

To reduce shrinkage stresses, these large floor areas can also be cast in smaller sections to establish temporary

Fig. 12. Schematic elevations of giant moment frame system.

Fig. 13. Relative stiffness of shear walls connected with deep wall girders (giant moment frames).
shrinkage control joints which are then connected structurally after a long period of curing and drying time during construction. One method of connecting the smaller cast sections at the shrinkage control joints is shown in Fig. 19.

In other instances, however, the floor area may be so large that seismic (expansion) joints become absolutely necessary. In this instance, the width of the joint should be sufficient to prevent contact between the separate framing units. However, in the case of an extended period of seismic activity, resonance may build up considerable movement beyond the magnitude of the joint separation provided and severe damage may result. Installation of a tension tie system in combination with a suitable cushioning material between building units could be considered a partial solution in preventing potential damage from pounding action (Fig. 20).

In this regard, it is again important to emphasize the need for structural stiffness in the separate framing units since higher stiffness coefficients will reduce the degree of lateral movement and thus reduce the difficulty in developing a manageable seismic joint. The stiffness can be increased substantially by using shear walls, giant moment frames or super diagonal systems.

CONCLUDING REMARKS

For precast concrete construction in seismic areas, all joints between precast concrete units should develop full continuity and toughness. Floor slab and beam units utilizing composite in-place concrete topping and jointing can develop horizontal diaphragm action and fail-safe connections to resist seismic forces.

With the availability of reliable mechanical splicing devices, precast concrete shear wall, slab, column, beam and girder components can be installed at any convenient location in a structure to develop full continuity and monolithic frame action. The structural analysis for precast concrete construction would be identical to that of a cast-in-place concrete structure.

Stiffness in the overall framing system is necessary to insure against structural and nonstructural building damage by minimizing drift, deflection and rotation. To develop framing stiffness, shear wall elements are extremely important. When these shear wall panels are integrated with deep wall girders to produce giant moment frames, the stiffness in the structure is greatly enhanced. Super diagonal frames can also be an effective method to develop stiffness in the building frame. These frames can be used where access or functional requirements require more open space in a floor plan.

In seismic areas, the existence of soft floors in the framing system can lead to severe damage or collapse. Eccentricities (non-symmetry) in the framing system should be avoided, regardless of mathematical justifications and structural detailing.

Seismic joints between building units should be avoided wherever possible in order to prevent pounding damage. If such joints become necessary due to the overall size of the structure, tension ties in combination with a cushioning material can be included in the seismic joint design to reduce possible damage due to pounding action.
Giant moment frame elevations utilized in the Municipal Office Building, City and County of Honolulu, Hawaii.

Fig. 16. Super diagonal frame system.

Fig. 17. Building with seismic joints (top) and building without seismic joints (bottom).
Fig. 18. Damage and collapse of buildings due to pounding action at seismic joints during Mexico City earthquake of September 19, 1985.
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7. Fintel, Mark, “Performance of Precast and Prestressed Concrete in Mexico Earthquake,” PCI JOURNAL, V. 31, No. 1, January-February 1986, pp. 18-42. (See also Discussion in November-December 1986 PCI JOURNAL, pp. 143-144.)


